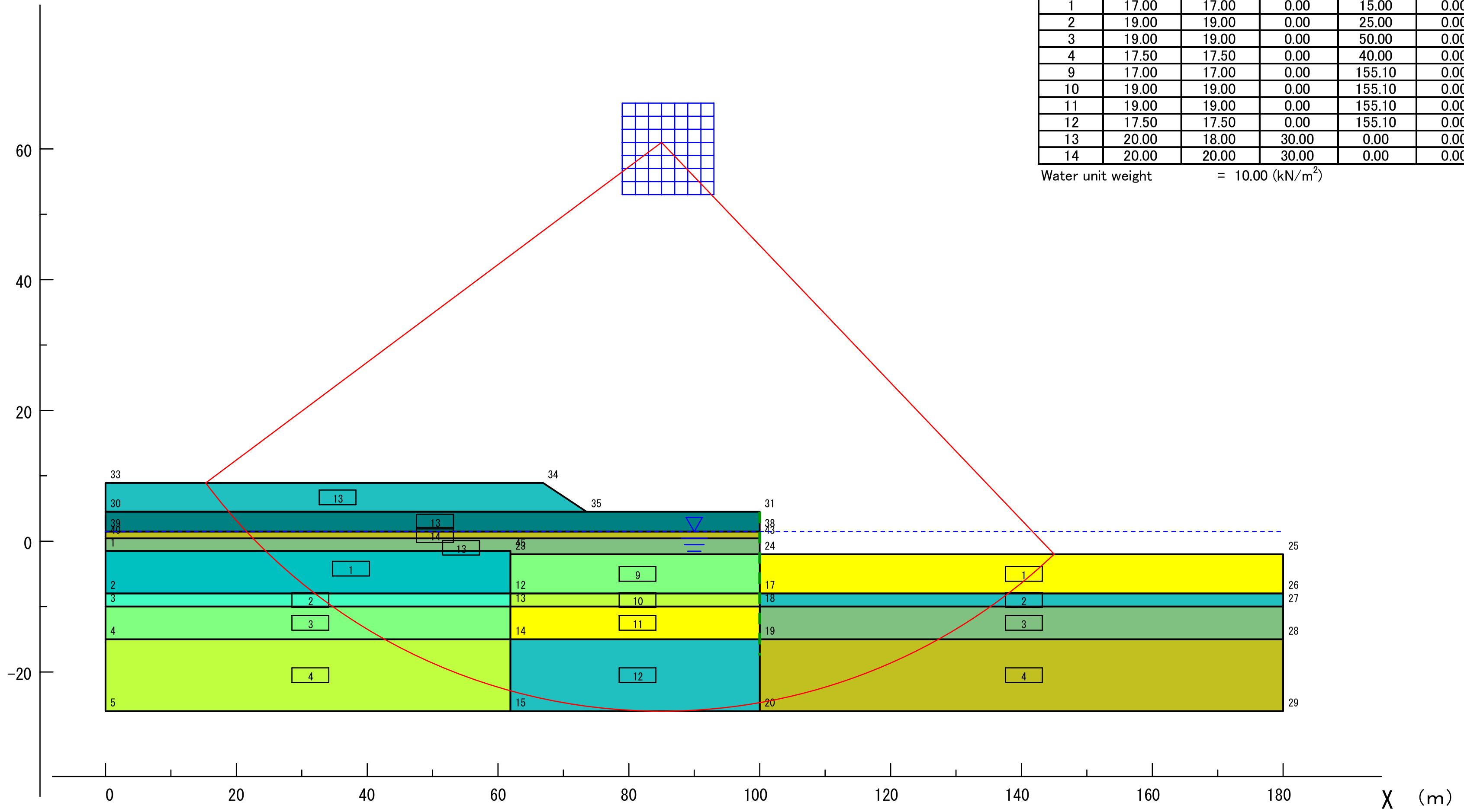
Appendix 7-4

Slope Stability Calculation Result

Section-① Preload



Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	2.464	
Center of arc	X	=	85.00	(m)
	Y	=	61.00	(m)
Radius	R	=	87.00	(m)
Resisting moment	M_R	=	843803.6	(kN・m)
Sliding moment	M_D	=	342498.8	(kN・m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

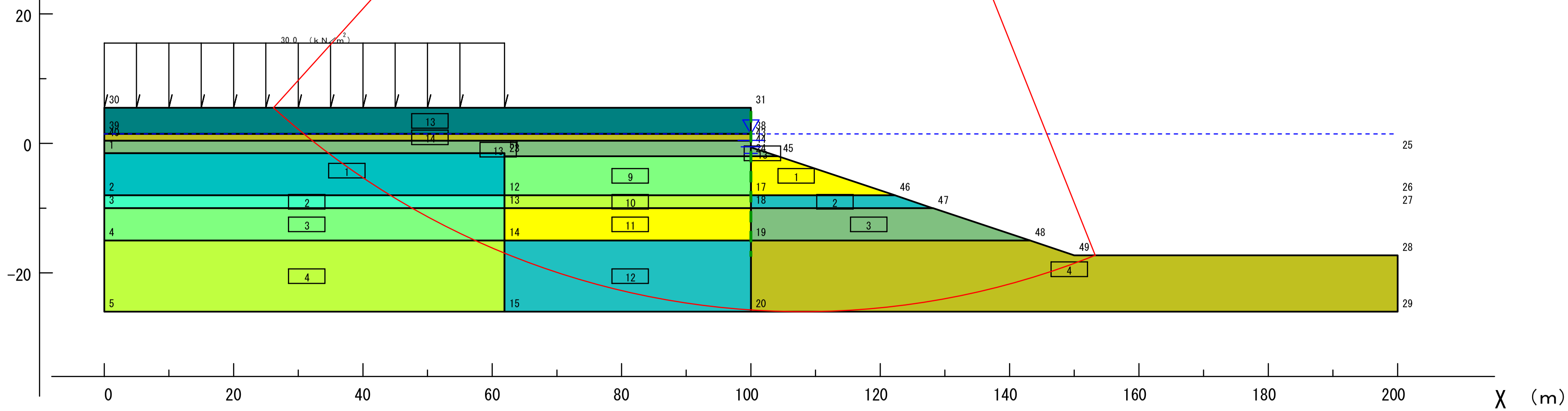
Section-① After Completion of Retaining Wall

Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	2.031	
Center of arc	X	=	108.00	(m)
	Y	=	96.00	(m)
Radius	R	=	122.00	(m)
Resisting moment	M_R	=	1174027.3	(kN・m)
Sliding moment	M_D	=	578133.6	(kN・m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)



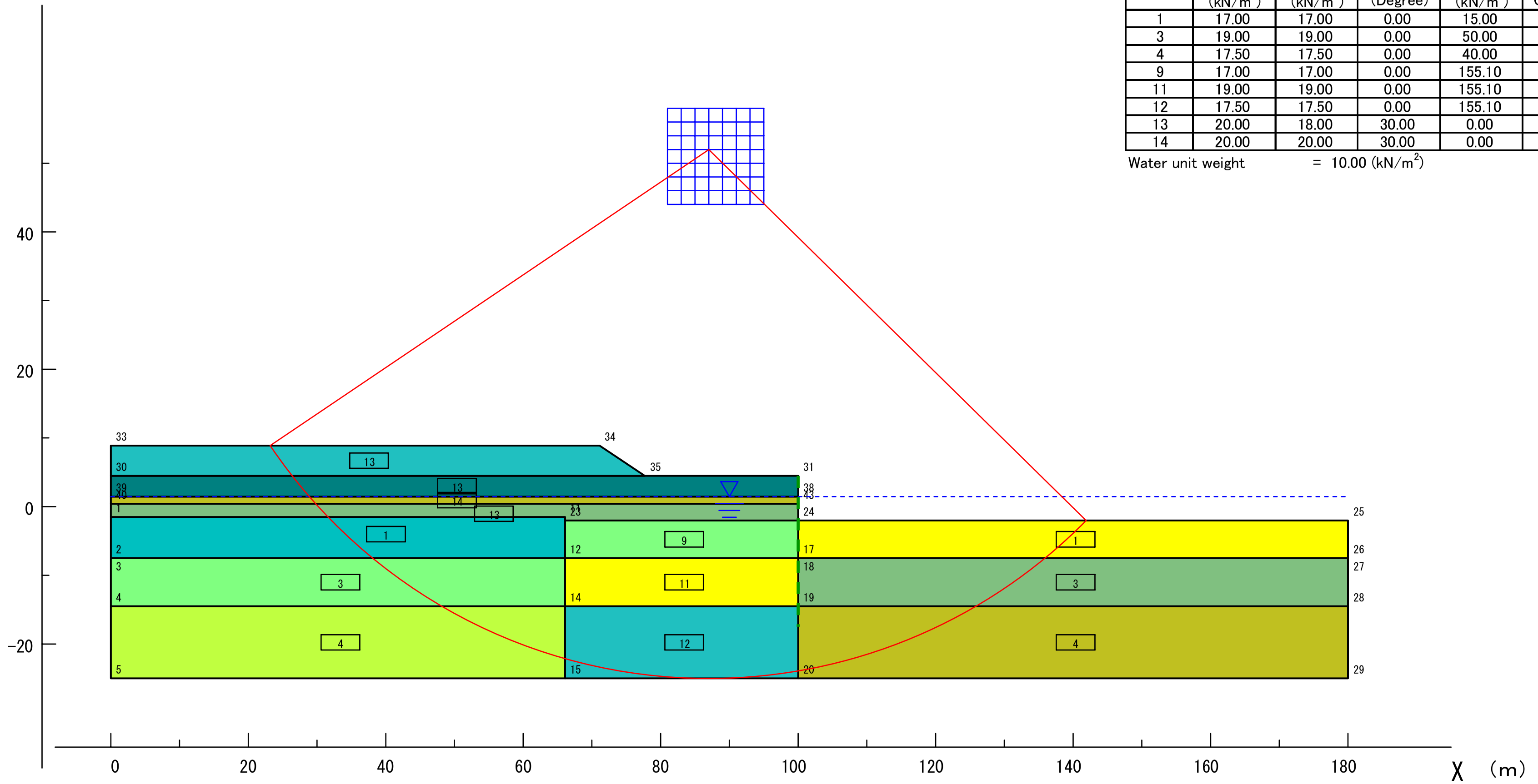
Section-a Preload

Scale ; 1/ 1000

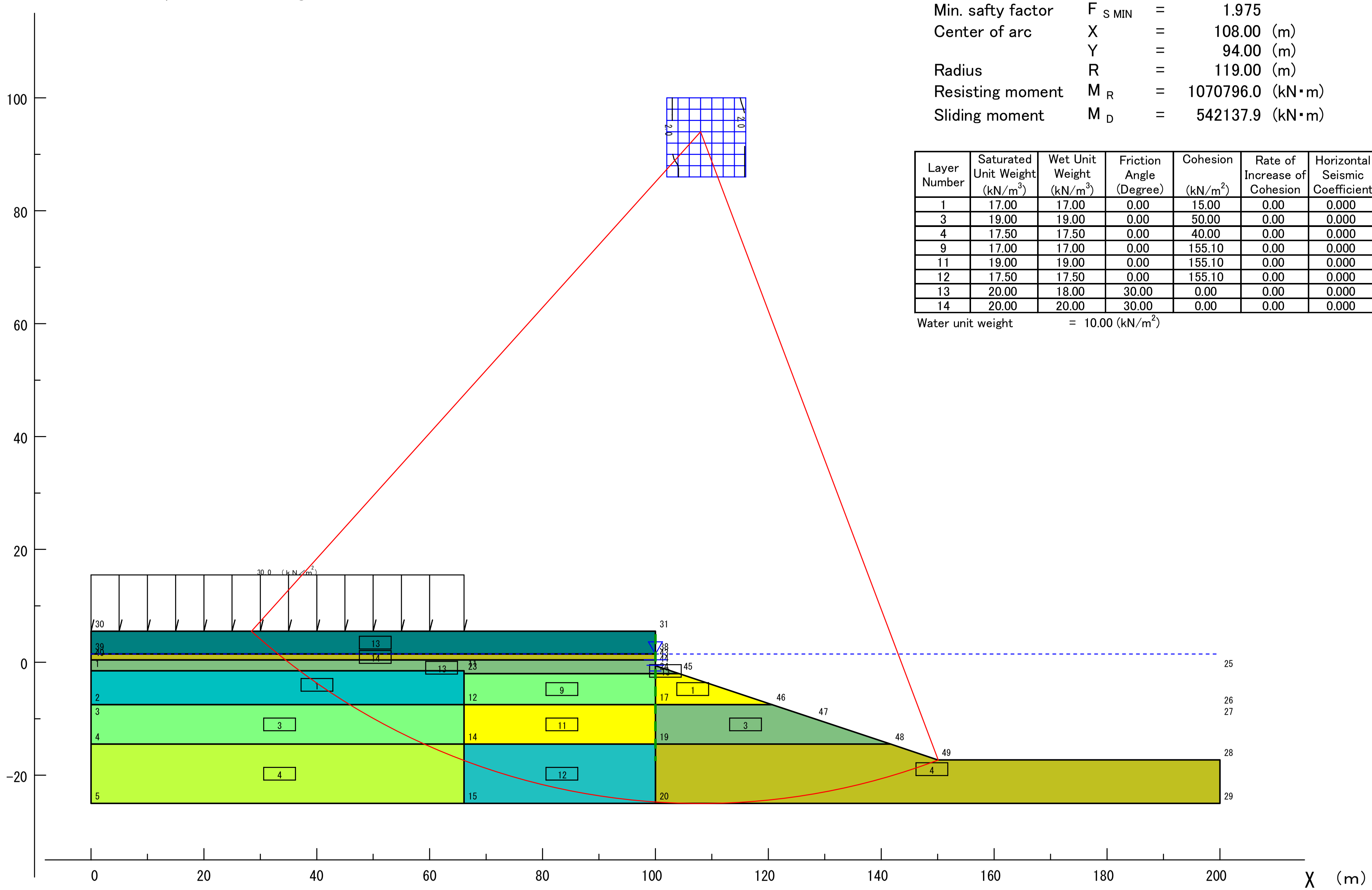
Min. safty factor $F_{S\,MIN}$ = 2.391
Center of arc X = 87.00 (m)
Y = 52.00 (m)
Radius R = 77.00 (m)
Resisting moment M_R = 691289.3 (kN・m)
Sliding moment M_D = 289129.8 (kN・m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)



Section-a After Completion of Retaining Wall



Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	1.975
Center of arc	X	=	108.00 (m)
	Y	=	94.00 (m)
Radius	R	=	119.00 (m)
Resisting moment	M_R	=	1070796.0 (kN･m)
Sliding moment	M_D	=	542137.9 (kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

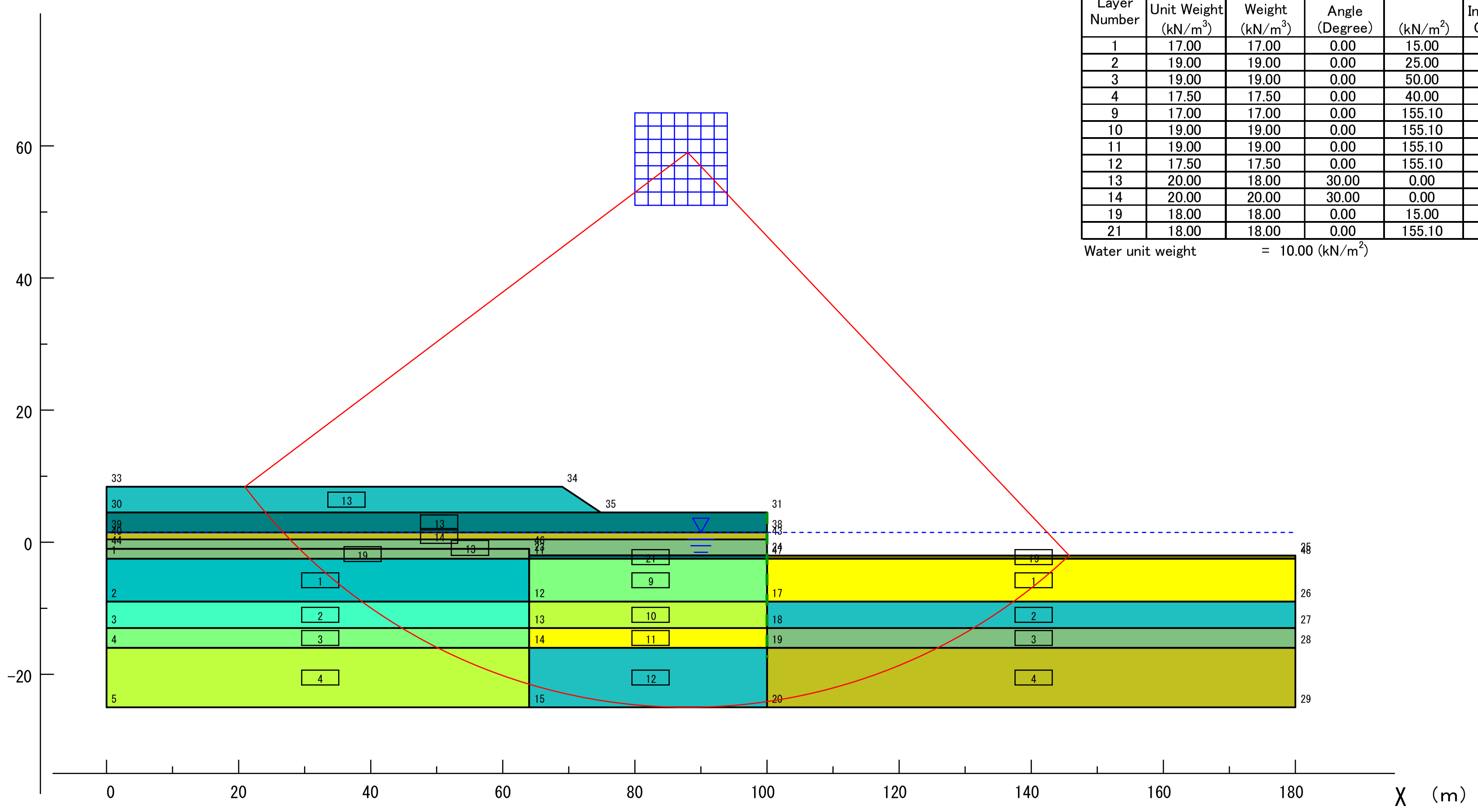
Section-b Preload

Scale ; 1/ 1000

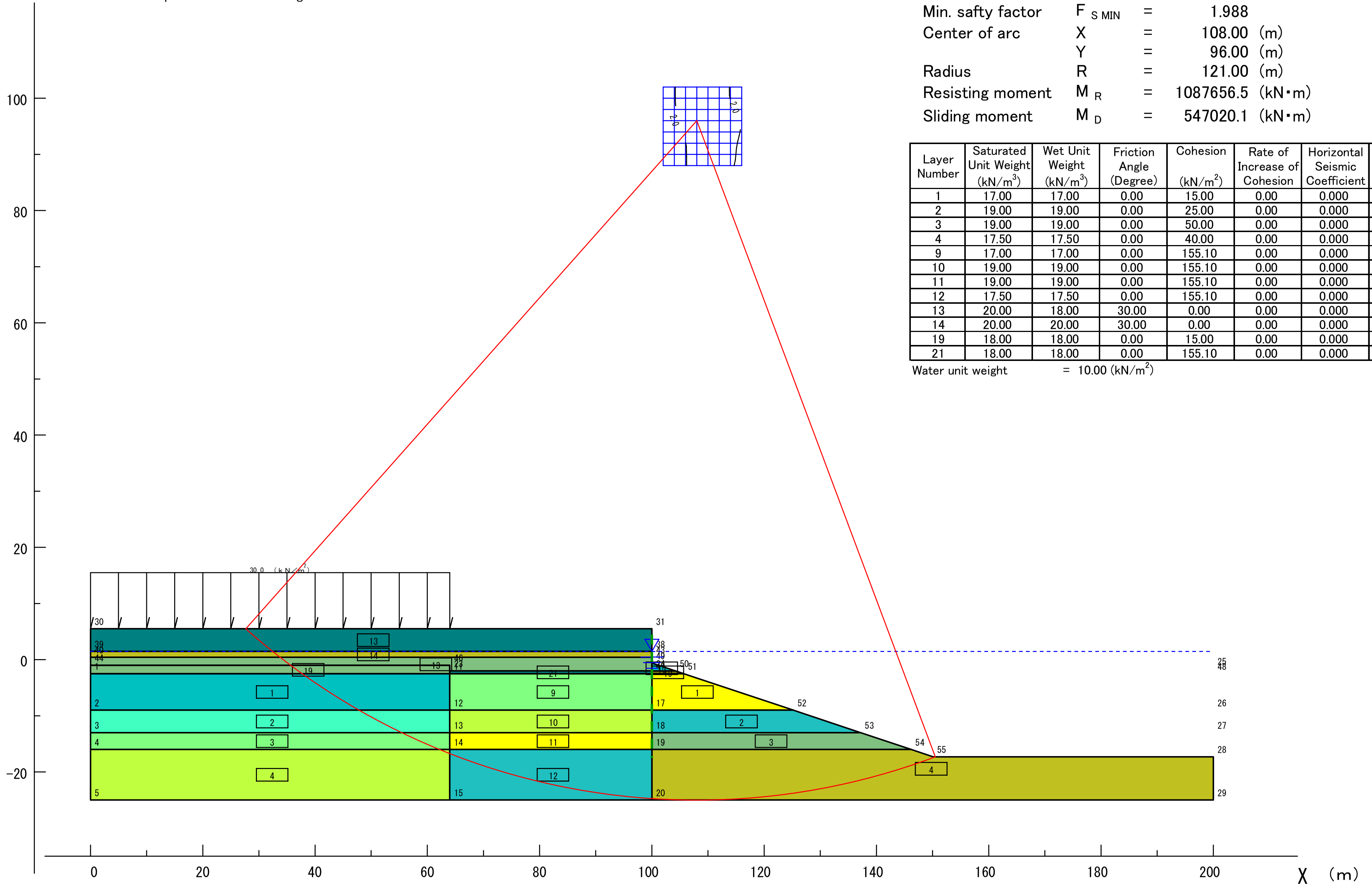
Min. safty factor	$F_{S\,MIN}$	=	2.498
Center of arc	X	=	88.00 (m)
	Y	=	59.00 (m)
Radius	R	=	84.00 (m)
Resisting moment	M_R	=	750854.9 (kN･m)
Sliding moment	M_D	=	300560.7 (kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000
19	18.00	18.00	0.00	15.00	0.00	0.000	0.000
21	18.00	18.00	0.00	155.10	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)



Section-b After Completion of Retaining Wall



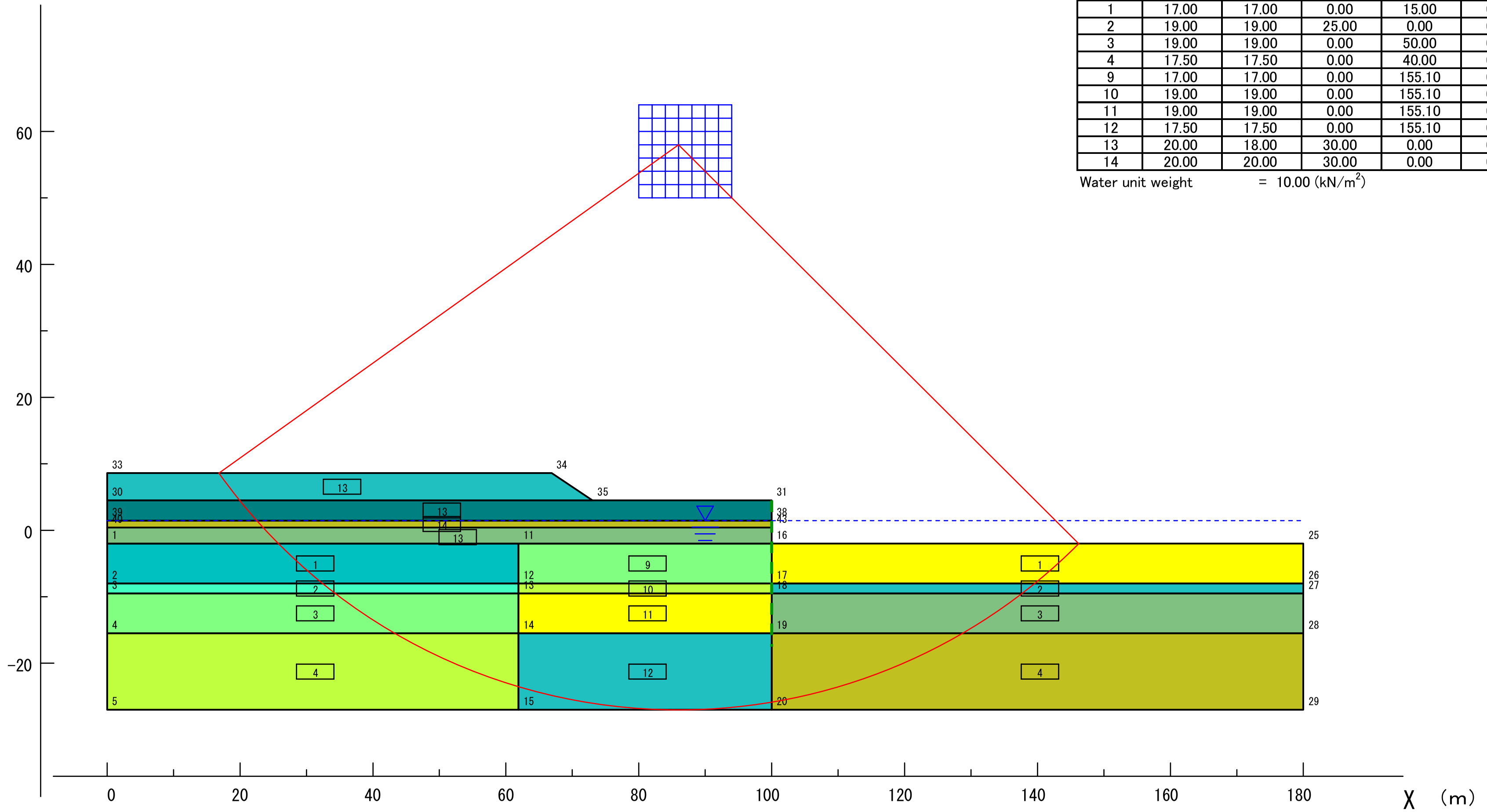
Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	1.988	
Center of arc	X	=	108.00	(m)
	Y	=	96.00	(m)
Radius	R	=	121.00	(m)
Resisting moment	M_R	=	1087656.5	(kN・m)
Sliding moment	M_D	=	547020.1	(kN・m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000
19	18.00	18.00	0.00	15.00	0.00	0.000	0.000
21	18.00	18.00	0.00	155.10	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

Section-c Preload



Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	2.531	
Center of arc	X	=	86.00	(m)
	Y	=	58.00	(m)
Radius	R	=	85.00	(m)
Resisting moment	M_R	=	838798.0	(kN･m)
Sliding moment	M_D	=	331425.4	(kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	25.00	0.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

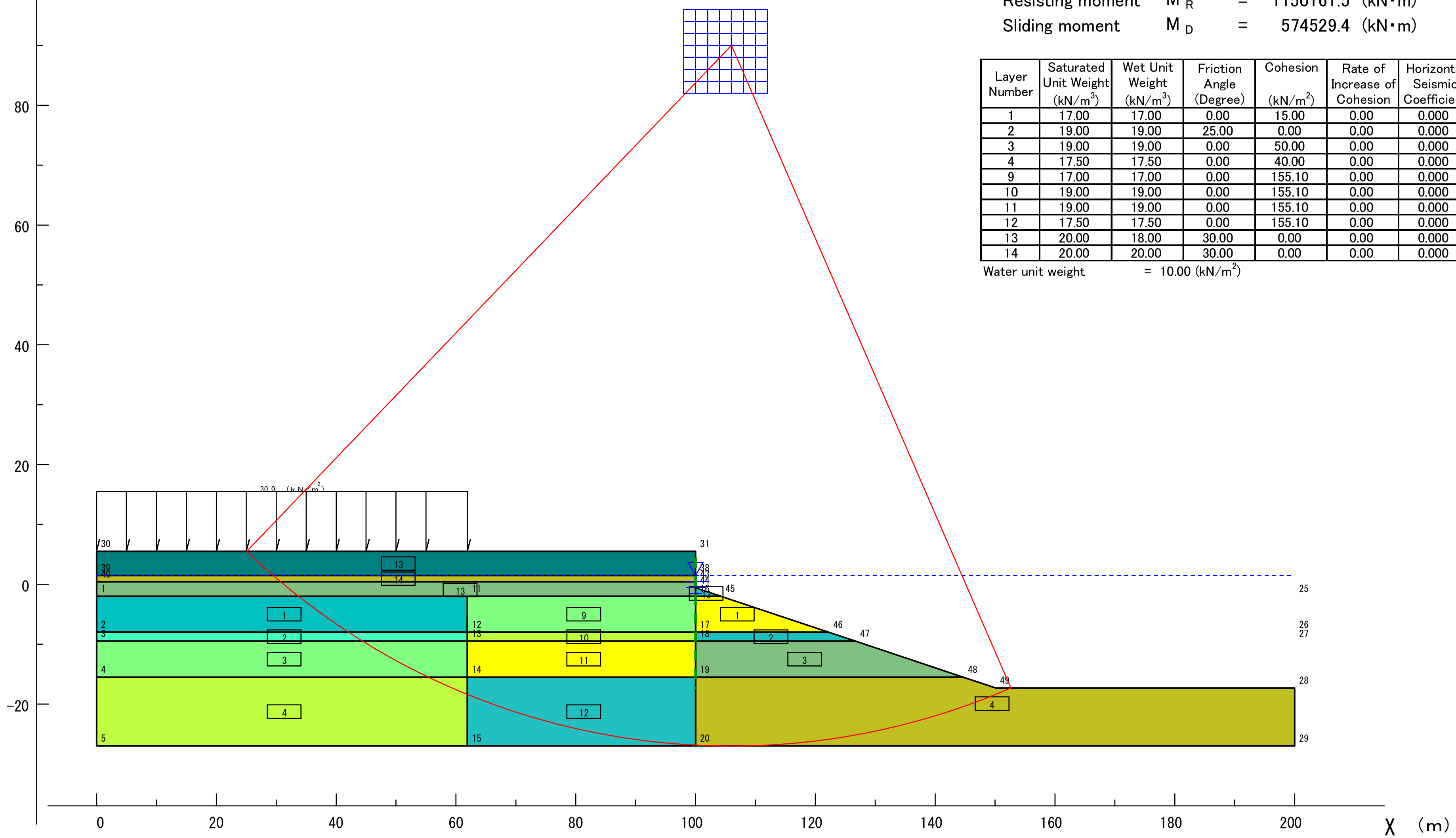
Section-c After Completion of Retaining Wall

Scale ; 1/ 1000

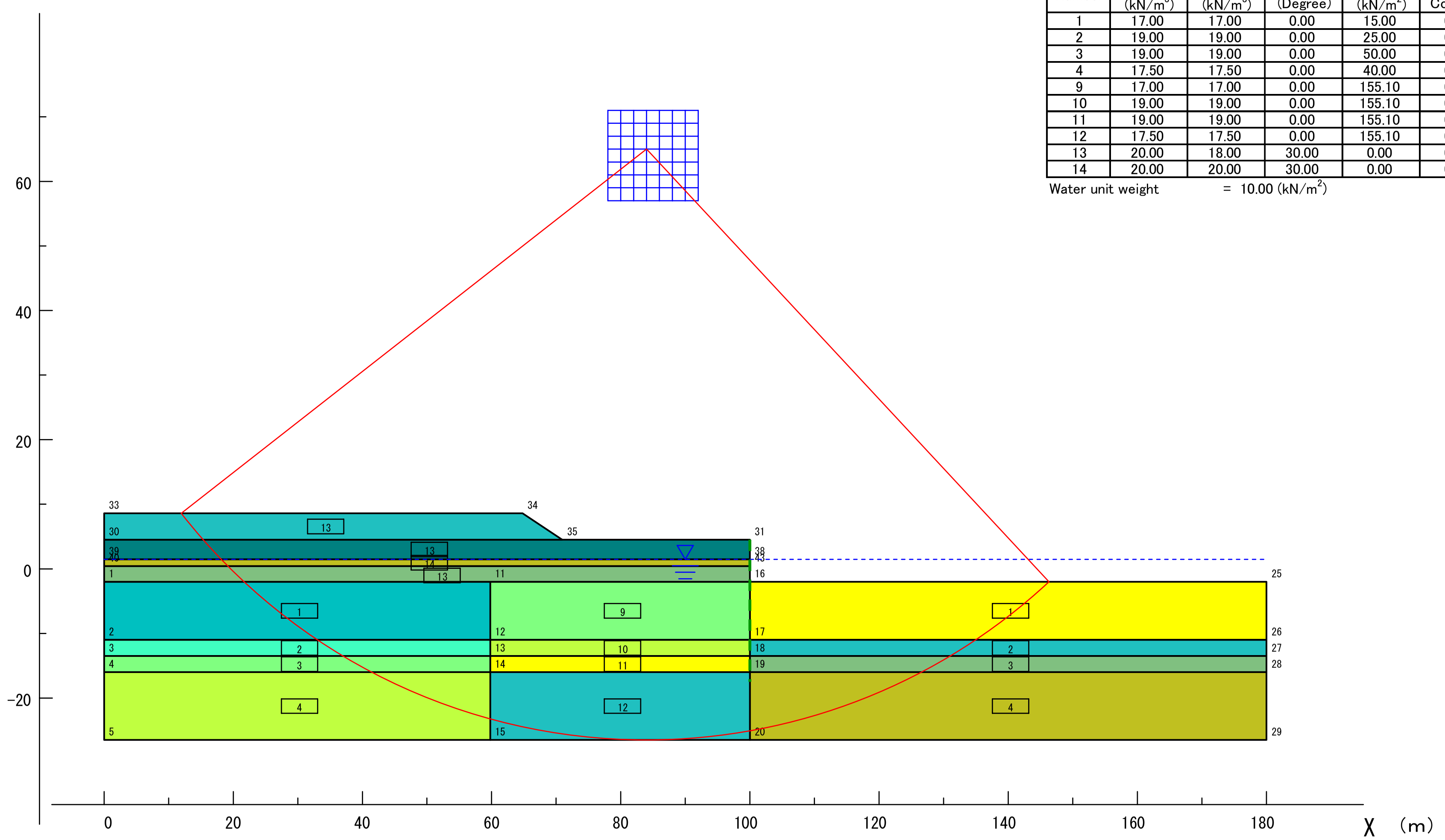
Min. safty factor	$F_{S\ MIN}$	=	2.002	
Center of arc	X	=	106.00	(m)
	Y	=	90.00	(m)
Radius	R	=	117.00	(m)
Resisting moment	M_R	=	1150161.5	(kN･m)
Sliding moment	M_D	=	574529.4	(kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	25.00	0.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)



Section-d Preload



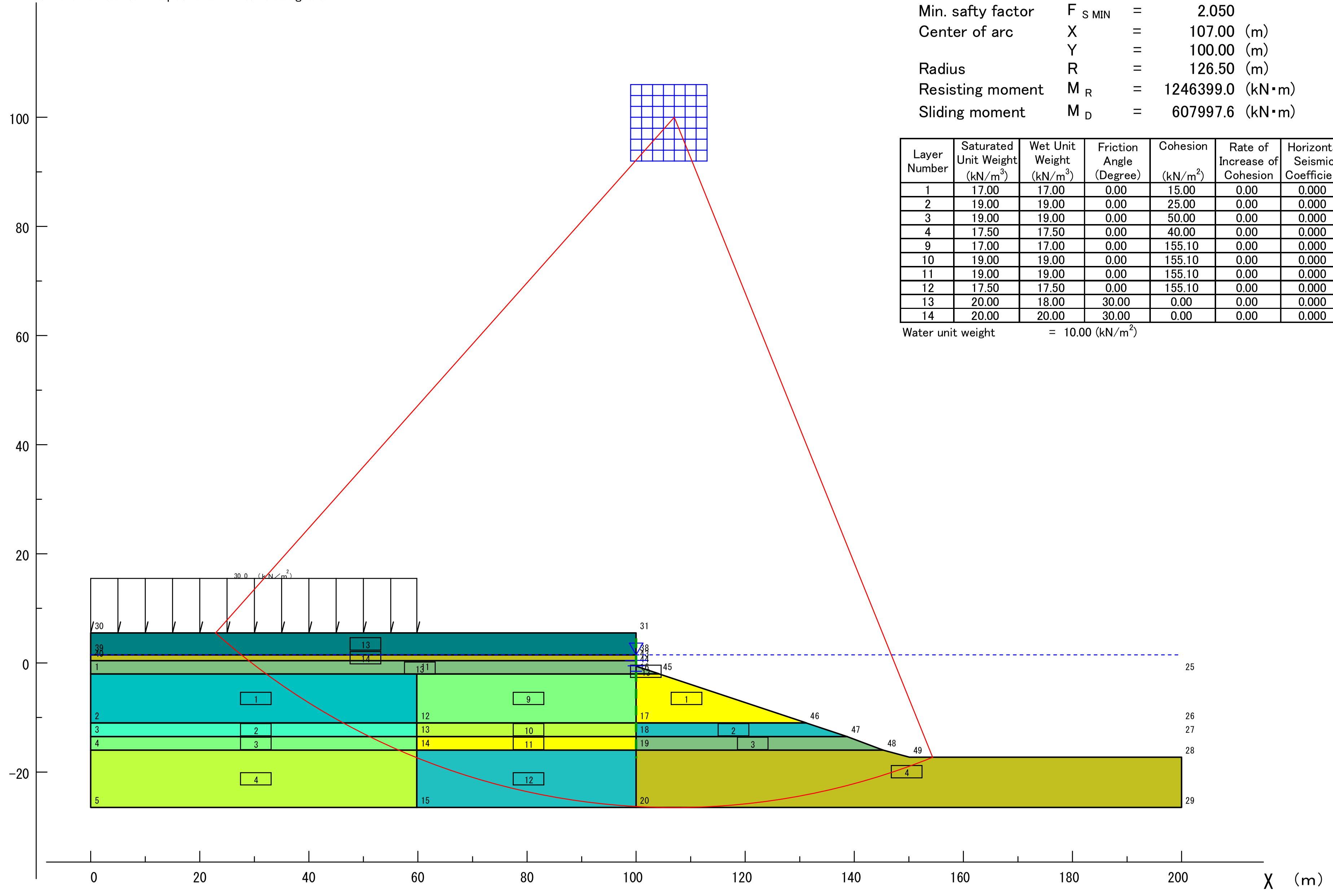
Scale ; 1/ 1000

Min. safty factor	$F_{S\ MIN}$	=	2.506	
Center of arc	X	=	84.00	(m)
	Y	=	65.00	(m)
Radius	R	=	91.50	(m)
Resisting moment	M_R	=	893255.7	(kN･m)
Sliding moment	M_D	=	356423.0	(kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

Section-d After Completion of Retaining Wall



Scale ; 1/ 1000

Min. safty factor	$F_{S\,MIN}$	=	2.050	
Center of arc	X	=	107.00	(m)
	Y	=	100.00	(m)
Radius	R	=	126.50	(m)
Resisting moment	M_R	=	1246399.0	(kN･m)
Sliding moment	M_D	=	607997.6	(kN･m)

Layer Number	Saturated Unit Weight (kN/m^3)	Wet Unit Weight (kN/m^3)	Friction Angle (Degree)	Cohesion (kN/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	17.00	17.00	0.00	15.00	0.00	0.000	0.000
2	19.00	19.00	0.00	25.00	0.00	0.000	0.000
3	19.00	19.00	0.00	50.00	0.00	0.000	0.000
4	17.50	17.50	0.00	40.00	0.00	0.000	0.000
9	17.00	17.00	0.00	155.10	0.00	0.000	0.000
10	19.00	19.00	0.00	155.10	0.00	0.000	0.000
11	19.00	19.00	0.00	155.10	0.00	0.000	0.000
12	17.50	17.50	0.00	155.10	0.00	0.000	0.000
13	20.00	18.00	30.00	0.00	0.00	0.000	0.000
14	20.00	20.00	30.00	0.00	0.00	0.000	0.000

Water unit weight = 10.00 (kN/m^2)

Appendix 7-5

Active Earth Pressure on Steel Sheet Pipe Pile

Determination of Active Earth Pressure on SPSP

1. Method of Active Earth Pressure Calculation

The active earth pressure on the steel pipe sheet pile (SPSP) will be calculated based on the "Technical Standards and Commentaries for Port and Harbour Facilities in Japan (TSCPHFJ)".

[For sandy soil – ordinary conditions]

14.2 Earth Pressure under Ordinary Conditions

14.2.1 Earth Pressure of Sandy Soil under Ordinary Conditions (Notification Article 11, Clause 1, Number 1)

The earth pressure of sandy soil acting on the backface wall of structure and the angle of sliding surface shall be calculated by the following equations:

(1) Active Earth Pressure and the Angle of Failure Surface.

$$p_{ai} = K_{ai}\left[\Sigma\gamma_i h_i + \frac{w\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (14.2.1)$$

$$\cot(\zeta_i-\beta) = -\tan(\phi_i+\delta+\psi-\beta)+\sec(\phi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\psi+\delta)\sin(\phi_i+\delta)}{\cos(\psi-\beta)\sin(\phi_i-\beta)}} \qquad (14.2.2)$$

where

$$K_{ai} = \frac{\cos^2(\phi_i-\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1+\sqrt{\frac{\sin(\phi_i+\delta)\sin(\phi_i-\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}\right]^2}$$

with

p_{ai}, p_{pi} : active and passive earth pressure, respectively, acting on the bottom level of the i-th soil layer (kN/m^2)
ϕ_i : angle of internal friction of the i-th soil layer (°)
γ_i : unit weight of the i-th soil layer (kN/m^3)
h_i : thickness of the i-th soil layer (m)
K_{ai}, K_{pi} : coefficients of active and passive earth pressures, respectively, in the i-th soil layer
ψ : angle of batter of backface wall from vertical line (°)
β : angle of backfill ground surface from horizontal line (°)
δ : angle of friction between backfilling material and backface wall (°)
ζ_i : angle of failure surface of the i-th soil layer (°)
w : uniformly distributed surcharge (kN/m^2)

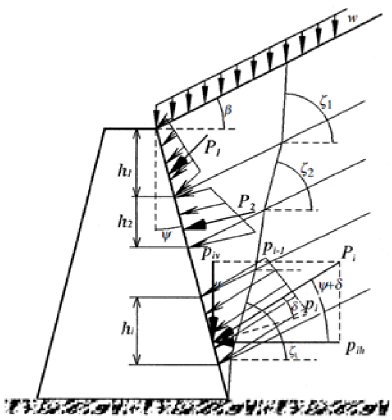


Fig. 14.2.1 Schematic Diagram of Earth Pressure Acting on Retaining Wall

1

[For cohesive soil – ordinary conditions]

14.2.2 Earth Pressure of Cohesive Soil under Ordinary Conditions (Notification Article 11, Clause 1, Number 2)

The earth pressure of cohesive soil acting on the backface wall of structure and the angle of failure surface shall be calculated by following equations:

(1) Active Earth Pressure

$$p_a = \Sigma \gamma_i h_i + w - 2c \qquad (14.2.5)$$

(2) Passive Earth Pressure

$$p_p = \Sigma \gamma_i h_i + w + 2c \qquad (14.2.6)$$

where

p_a : active earth pressure acting on the bottom level of the i-th soil layer (kN/m^2)
p_p : passive earth pressure acts on the bottom level of the i-th soil layer (kN/m^2)
γ_i : unit weight of the i-th soil layer (kN/m^3)
h_i : thickness of the i-th soil layer (m)
w : uniformly distributed surcharge (kN/m^2)
c : cohesion of soil in the i-th layer (kN/m^2)

[For sandy soil – seismic conditions]

14.3 Earth Pressure during Earthquake

14.3.1 Earth Pressure of Sandy Soil during Earthquake (Notification Article 18, Clause 1, Number 1)

The earth pressure of sandy soil acting on a backface wall of structure during an earthquake and the angle of failure surface shall be calculated by following equations:

(1) Active Earth Pressure and the Angle of Failure Surface.

$$p_{ai} = K_{ai}\left[\Sigma \gamma_i h_i + \frac{w\cos\psi}{\cos(\psi - \beta)}\right]\cos\psi \qquad (14.3.1)$$

$$\cot(\zeta_i - \beta) = -\tan(\phi_i + \delta + \psi - \beta) + \sec(\phi_i + \delta + \psi - \beta)\sqrt{\frac{\cos(\psi + \delta + \theta)\sin(\phi_i + \delta)}{\cos(\psi - \beta)\sin(\phi_i - \beta - \theta)}} \qquad (14.3.2)$$

where

$$K_{ai} = \frac{\cos^2(\phi_i - \psi - \theta)}{\cos\theta\cos^2\psi\cos(\delta + \psi + \theta)\left[1 + \sqrt{\frac{\sin(\phi_i + \delta)\sin(\phi_i - \beta - \theta)}{\cos(\delta + \psi + \theta)\cos(\psi - \beta)}}\right]^2}$$

where

θ : composite seismic angle (°) given by the following equations:
$\theta = \tan^{-1}k$ (above water level)
$\theta = \tan^{-1}k'$ (below water level)
k : seismic coefficient
k' : apparent seismic coefficient

The notations other than θ, k, and k' are the same as those defined in **14.2.1 Earth Pressure of Sandy Soil under Ordinary Conditions**. The apparent seismic coefficient k' is described in **14.3.3 Apparent Seismic Coefficient**.

[For cohesive soil – seismic conditions]

14.3.2 Earth Pressure of Cohesive Soil during Earthquake (Notification Article 18, Clause 1, Number 2)

The earth pressure of cohesive soil acting on a backface wall of structure during an earthquake shall be calculated by the following:

(1) Active Earth Pressure
Active earth pressure shall be calculated using an appropriate earth pressure equation which takes the seismic coefficient into account so that the structural stability will be secured during an earthquake.

[Commentary]

(1) The active earth pressure acting on a backface wall of structure during an earthquake and the angle of sliding surface should be calculated by following equations:

$$p_a = \frac{(\Sigma\gamma_i h_i + w)\sin(\zeta_a + \theta)}{\cos\theta\sin\zeta_a} - \frac{c}{\cos\zeta_a\sin\zeta_a} \qquad (14.3.5)$$

$$\zeta_a = \tan^{-1}\sqrt{1 - \left(\frac{\Sigma\gamma_i h_i + 2w}{2c}\right)\tan\theta} \qquad (14.3.6)$$

where
p_a : active earth pressure, acting on the bottom level of the i-th soil layer (kN/m^2)
γ_i : unit weight of the i-th soil layer (kN/m^3)
h_i : thickness of the i-th soil layer (m)
ζ_a : angle of failure surface of the i-th soil layer (°)
w : uniformly distributed surcharge (kN/m^2)
c : cohesion of the soil (kN/m^2)
θ : composite seismic angle (°) ($\theta = \tan^{-1}k$ or $\theta = \tan^{-1}k'$)
k : seismic coefficient
k' : apparent seismic coefficient

(2) There are many unknown items concerning the method for determining the passive earth pressure of cohesive soil during an earthquake. From the practical point of view, the passive earth pressure in ordinary condition discribed in **14.2.2 Earth Pressure of Cohesive Soil under Ordinary Conditions** can be used as passive earth pressure during an earthquake.

(3) The apparent seismic coefficient should be used to calculate the earth pressure of cohesive soil down to the sea bottom during an earthquake. The apparent seismic coefficient may be set as zero when calculating the earth pressure at the depth of 10 m from the sea bottom or deeper. The earth pressure between these two depths is determined assuming that the earth pressure is linearly distributed between them. If the earth pressure at the depth of 10 m below the sea bottom becomes less than the earth pressure at the sea bottom, the latter should be applied down to the depth of 10 m.

[Apparent seismic coefficient]

14.3.3 Apparent Seismic Coefficient (Notification Article 19)

It shall be standard to calculate the earth pressure acting below the residual water level during an earthquake according to the procedures given in **14.3.1 Earth Pressure of Sandy Soil during Earthquake** and **14.3.2 Earth Pressure of Cohesive Soil during Earthquake**, by using the apparent seismic coefficient that is determined by equation (**14.3.7**).

$$k' = \frac{2(\Sigma\gamma_t h_i + \Sigma\gamma h_j + w) + \gamma h}{2[\Sigma\gamma_t h_i + \Sigma(\gamma - 10)h_j + w] + (\gamma - 10)h} k \qquad (14.3.7)$$

where

- k' : apparent seismic coefficient
- γ_t : unit weight of soil layer above the residual water level (kN/m³)
- γ : unit weight (in the air) of saturated soil layer below the residual water level (kN/m³)
- w : uniform external load at the ground surface (kN/m²)
- h_i : thickness of the i-th soil layer above the residual water level (m)
- h_j : thickness of the j-th soil layer below the residual water level (m)
- h : thickness of soil layer to calculate earth pressure below the residual watere level (m)
- k : seismic coefficient

[Commentary]

(1) In case of stability analysis of quaywall with use of equation (**14.3.7**), the dynamic water pressure during an earthquake should be applied to the wall in the seaward direction.

(2) The concept of the apparent seismic coefficient k' is expressed by the following equation:

$$\gamma_t \times k = (\gamma - 10) \times k'$$

(3) A product of unit weight of a soil layer (in the air) and seismic coefficient becomes equal to the product of submerged unit weight of a soil layer and the apparent coefficient for the soil below the water level.

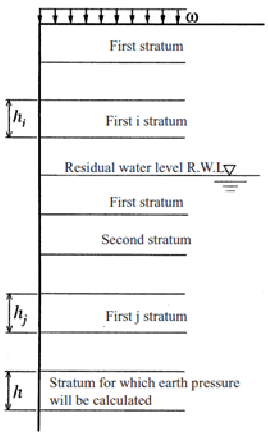


Fig. C- 14.3.2 Cross Section of Soil Layers and Symbols.

2. Special Consideration for Container Berth/Barge Berth

The active earth pressure on the steel pipe sheet pile (SPSP) needs special consideration due to the existence of the DMM block.

There will be a gap between the SPSP and the DMM as shown in Figure 1 to allow for possible construction deviations of the DMM installation.

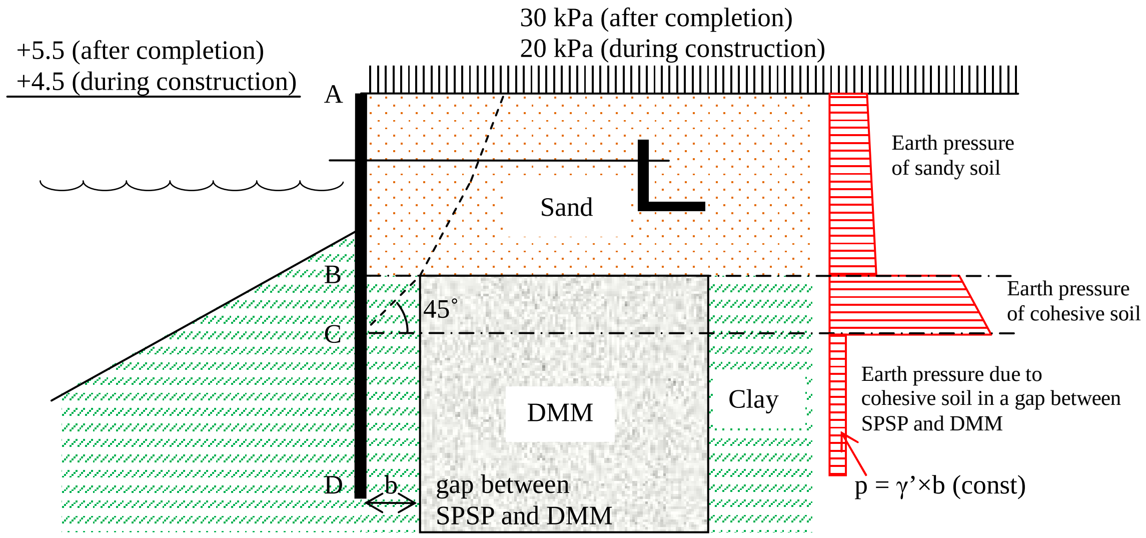
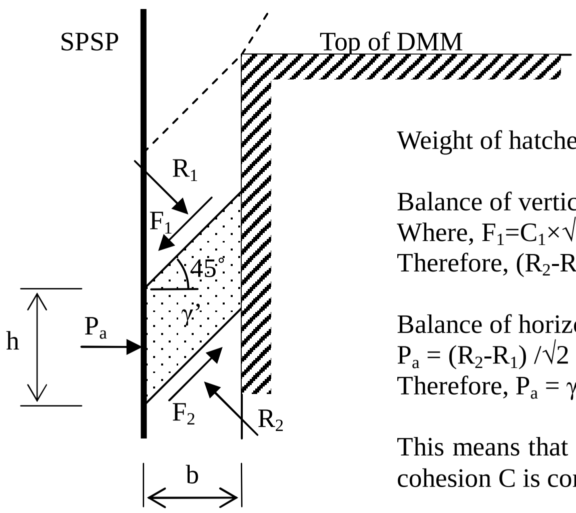


Figure 1 Active earth pressure on the SPSP

In this case, following active earth pressure shall be considered:

- Between points A and B : Earth pressure of sandy soil (not affected by DMM)
- Between points B and C : Earth pressure of cohesive soil (not affected by DMM)
- Between points C and D : Earth pressure of cohesive soil (reduced by DMM)

For the active earth pressure between points C and D is obtained as follows:



Weight of hatched area : $W = \gamma' bh$

Balance of vertical forces: $(R_1+F_1)/\sqrt{2} + W = (R_2+F_2)/\sqrt{2}$
Where, $F_1=C_1\times\sqrt{2}b$, $F_2=C_2\times\sqrt{2}b$; C_1 and C_2 are cohesion of soil
Therefore, $(R_2-R_1)/\sqrt{2} = \gamma' bh + (F_1-F_2)/\sqrt{2} = \gamma' bh + (C_1-C_2)b$

Balance of horizontal forces:
$P_a = (R_2-R_1)/\sqrt{2} + (F_1-F_2)/\sqrt{2}$
Therefore, $P_a = \gamma' bh + (C_1-C_2)b + (C_1-C_2)b = \gamma' bh + 2(C_1-C_2)b$

This means that the earth pressure p_a equals to $p_a=\gamma' b$ (const), when cohesion C is constant.

The gap between SPSP and DMM is set to be b=1.0m.

Active earth pressure below the point C is calculated in the following pages and summarized in Table 1.

Table 1 Summary of active earth pressure below point C

	Active earth pressure	Remarks
Layer 2	p_a = 7.0 kPa	
Layer 3	Generally in a range of 0 kPa ~ 9 kPa. However, averaged pressure is much smaller than that for Layer 2.	
Layer 4	Generally $p_a \approx 0$ kPa. For block a, c and 1, relatively larger values were obtained just below Layer 4 but are much smaller than 7.0kPa multiplied with the thickness of Layer 4.	
Layer 5	p_a = 7.5 kPa	

In conclusion, the design adopted the following values of earth pressure on the SPSP below the point C.

- **Design active earth pressure below point C : p_a = 7.00 kPa**

The above is considered to be conservative as explained below:

- As shown in Table 1, the values of earth pressure obtained for Layers 3 and 4 have generally smaller effect than p_a=7.0kPa (value of Layer-2).
- The values for Layer 5 is slightly larger than Layer 2 but has negligible effect to the SPSP design (i.e., penetration of SPSP into Layer 5 is very short). Also, such differences can be compensated by the conservative assumption made for Layers 3 and 4.

Block-1

Layer	Elev. Top (m)	Elev. Bottom (m)	h (m)	Cu (kPa)	γ' (kN/m3)	$\gamma'h$ (kPa)	$\Sigma\gamma'h$ (kPa)	P (total) (kN/m)	p {P(i) - P(i-1)}/h (kPa)
2	-2.6	-3.0	0.4	15.0	7.0	2.8	2.8	2.8	7.00
2	-3.0	-3.5	0.5	15.0	7.0	3.5	6.3	6.3	7.00
2	-3.5	-4.0	0.5	15.0	7.0	3.5	9.8	9.8	7.00
2	-4.0	-4.5	0.5	15.0	7.0	3.5	13.3	13.3	7.00
2	-4.5	-5.0	0.5	15.0	7.0	3.5	16.8	16.8	7.00
2	-5.0	-5.5	0.5	15.0	7.0	3.5	20.3	20.3	7.00
2	-5.5	-6.0	0.5	15.0	7.0	3.5	23.8	23.8	7.00
2	-6.0	-6.5	0.5	15.0	7.0	3.5	27.3	27.3	7.00
2	-6.5	-7.0	0.5	15.0	7.0	3.5	30.8	30.8	7.00
2	-7.0	-7.5	0.5	15.0	7.0	3.5	34.3	34.3	7.00
2	-7.5	-8.0	0.5	15.0	7.0	3.5	37.8	37.8	7.00
3	-8.0	-8.5	0.5	25.0	9.0	4.5	42.3	37.8	0.00
3	-8.5	-9.0	0.5	25.0	9.0	4.5	46.8	37.8	0.00
3	-9.0	-9.5	0.5	25.0	9.0	4.5	51.3	37.8	0.00
3	-9.5	-10.0	0.5	25.0	9.0	4.5	55.8	37.8	0.00
4	-10.0	-10.5	0.5	50.0	9.0	4.5	60.3	37.8	0.00
4	-10.5	-11.0	0.5	50.0	9.0	4.5	64.8	37.8	0.00
4	-11.0	-11.5	0.5	50.0	9.0	4.5	69.3	37.8	0.00
4	-11.5	-12.0	0.5	50.0	9.0	4.5	73.8	37.8	0.00
4	-12.0	-12.5	0.5	50.0	9.0	4.5	78.3	37.8	0.00
4	-12.5	-13.0	0.5	50.0	9.0	4.5	82.8	37.8	0.00
4	-13.0	-13.5	0.5	50.0	9.0	4.5	87.3	37.8	0.00
4	-13.5	-14.0	0.5	50.0	9.0	4.5	91.8	37.8	0.00
4	-14.0	-14.5	0.5	50.0	9.0	4.5	96.3	37.8	0.00
4	-14.5	-15.0	0.5	50.0	9.0	4.5	100.8	37.8	0.00
5	-15.0	-15.5	0.5	40.0	7.5	3.8	104.6	54.6	33.50
5	-15.5	-16.0	0.5	40.0	7.5	3.8	108.3	58.3	7.50
5	-16.0	-16.5	0.5	40.0	7.5	3.8	112.1	62.1	7.50
5	-16.5	-17.0	0.5	40.0	7.5	3.8	115.8	65.8	7.50
5	-17.0	-17.5	0.5	40.0	7.5	3.8	119.6	69.6	7.50
5	-17.5	-18.0	0.5	40.0	7.5	3.8	123.3	73.3	7.50
5	-18.0	-18.5	0.5	40.0	7.5	3.8	127.1	77.1	7.50
5	-18.5	-19.0	0.5	40.0	7.5	3.8	130.8	80.8	7.50
5	-19.0	-19.5	0.5	40.0	7.5	3.8	134.6	84.6	7.50
5	-19.5	-20.0	0.5	40.0	7.5	3.8	138.3	88.3	7.50
5	-20.0	-20.5	0.5	40.0	7.5	3.8	142.1	92.1	7.50
5	-20.5	-21.0	0.5	40.0	7.5	3.8	145.8	95.8	7.50
5	-21.0	-21.5	0.5	40.0	7.5	3.8	149.6	99.6	7.50
5	-21.5	-22.0	0.5	40.0	7.5	3.8	153.3	103.3	7.50
5	-22.0	-22.5	0.5	40.0	7.5	3.8	157.1	107.1	7.50
5	-22.5	-23.0	0.5	40.0	7.5	3.8	160.8	110.8	7.50
5	-23.0	-23.5	0.5	40.0	7.5	3.8	164.6	114.6	7.50
5	-23.5	-24.0	0.5	40.0	7.5	3.8	168.3	118.3	7.50
5	-24.0	-24.5	0.5	40.0	7.5	3.8	172.1	122.1	7.50
5	-24.5	-25.0	0.5	40.0	7.5	3.8	175.8	125.8	7.50
5	-25.0	-25.5	0.5	40.0	7.5	3.8	179.6	129.6	7.50
5	-25.5	-26.0	0.5	40.0	7.5	3.8	183.3	133.3	7.50
5	-26.0	-26.5	0.5	40.0	7.5	3.8	187.1	137.1	7.50
5	-26.5	-27.0	0.5	40.0	7.5	3.8	190.8	140.8	7.50

Block-a

Layer	Elev. Top (m)	Elev. Bottom (m)	h (m)	Cu (kPa)	γ' (kN/m3)	γ'h (kPa)	Σγ'h (kPa)	P (total) (kN/m)	p_normal {P(i) - P(i-1)}/h (kPa)
2	-2.6	-3.0	0.4	15.0	7.0	2.8	2.8	2.8	7.00
2	-3.0	-3.5	0.5	15.0	7.0	3.5	6.3	6.3	7.00
2	-3.5	-4.0	0.5	15.0	7.0	3.5	9.8	9.8	7.00
2	-4.0	-4.5	0.5	15.0	7.0	3.5	13.3	13.3	7.00
2	-4.5	-5.0	0.5	15.0	7.0	3.5	16.8	16.8	7.00
2	-5.0	-5.5	0.5	15.0	7.0	3.5	20.3	20.3	7.00
2	-5.5	-6.0	0.5	15.0	7.0	3.5	23.8	23.8	7.00
2	-6.0	-6.5	0.5	15.0	7.0	3.5	27.3	27.3	7.00
2	-6.5	-7.0	0.5	15.0	7.0	3.5	30.8	30.8	7.00
2	-7.0	-7.5	0.5	15.0	7.0	3.5	34.3	34.3	7.00
4	-7.5	-8.0	0.5	50.0	9.0	4.5	38.8	34.3	0.00
4	-8.0	-8.5	0.5	50.0	9.0	4.5	43.3	34.3	0.00
4	-8.5	-9.0	0.5	50.0	9.0	4.5	47.8	34.3	0.00
4	-9.0	-9.5	0.5	50.0	9.0	4.5	52.3	34.3	0.00
4	-9.5	-10.0	0.5	50.0	9.0	4.5	56.8	34.3	0.00
4	-10.0	-10.5	0.5	50.0	9.0	4.5	61.3	34.3	0.00
4	-10.5	-11.0	0.5	50.0	9.0	4.5	65.8	34.3	0.00
4	-11.0	-11.5	0.5	50.0	9.0	4.5	70.3	34.3	0.00
4	-11.5	-12.0	0.5	50.0	9.0	4.5	74.8	34.3	0.00
4	-12.0	-12.5	0.5	50.0	9.0	4.5	79.3	34.3	0.00
4	-12.5	-13.0	0.5	50.0	9.0	4.5	83.8	34.3	0.00
4	-13.0	-13.5	0.5	50.0	9.0	4.5	88.3	34.3	0.00
4	-13.5	-14.0	0.5	50.0	9.0	4.5	92.8	34.3	0.00
4	-14.0	-14.5	0.5	50.0	9.0	4.5	97.3	34.3	0.00
5	-14.5	-15.0	0.5	40.0	7.5	3.8	101.1	51.1	33.50
5	-15.0	-15.5	0.5	40.0	7.5	3.8	104.8	54.8	7.50
5	-15.5	-16.0	0.5	40.0	7.5	3.8	108.6	58.6	7.50
5	-16.0	-16.5	0.5	40.0	7.5	3.8	112.3	62.3	7.50
5	-16.5	-17.0	0.5	40.0	7.5	3.8	116.1	66.1	7.50
5	-17.0	-17.5	0.5	40.0	7.5	3.8	119.8	69.8	7.50
5	-17.5	-18.0	0.5	40.0	7.5	3.8	123.6	73.6	7.50
5	-18.0	-18.5	0.5	40.0	7.5	3.8	127.3	77.3	7.50
5	-18.5	-19.0	0.5	40.0	7.5	3.8	131.1	81.1	7.50
5	-19.0	-19.5	0.5	40.0	7.5	3.8	134.8	84.8	7.50
5	-19.5	-20.0	0.5	40.0	7.5	3.8	138.6	88.6	7.50
5	-20.0	-20.5	0.5	40.0	7.5	3.8	142.3	92.3	7.50
5	-20.5	-21.0	0.5	40.0	7.5	3.8	146.1	96.1	7.50
5	-21.0	-21.5	0.5	40.0	7.5	3.8	149.8	99.8	7.50
5	-21.5	-22.0	0.5	40.0	7.5	3.8	153.6	103.6	7.50
5	-22.0	-22.5	0.5	40.0	7.5	3.8	157.3	107.3	7.50
5	-22.5	-23.0	0.5	40.0	7.5	3.8	161.1	111.1	7.50
5	-23.0	-23.5	0.5	40.0	7.5	3.8	164.8	114.8	7.50
5	-23.5	-24.0	0.5	40.0	7.5	3.8	168.6	118.6	7.50
5	-24.0	-24.5	0.5	40.0	7.5	3.8	172.3	122.3	7.50
5	-24.5	-25.0	0.5	40.0	7.5	3.8	176.1	126.1	7.50
5	-25.0	-25.5	0.5	40.0	7.5	3.8	179.8	129.8	7.50
5	-25.5	-26.0	0.5	40.0	7.5	3.8	183.6	133.6	7.50
5	-26.0	-26.5	0.5	40.0	7.5	3.8	187.3	137.3	7.50
5	-26.5	-27.0	0.5	40.0	7.5	3.8	191.1	141.1	7.50

Block-b

Layer	Elev. Top (m)	Elev. Bottom (m)	h (m)	Cu (kPa)	γ' (kN/m3)	γ'h (kPa)	Σγ'h (kPa)	P (total) (kN/m)	p {P(i) - P(i-1)}/h (kPa)
2	-2.6	-3.0	0.4	15.0	7.0	2.8	2.8	2.8	7.00
2	-3.0	-3.5	0.5	15.0	7.0	3.5	6.3	6.3	7.00
2	-3.5	-4.0	0.5	15.0	7.0	3.5	9.8	9.8	7.00
2	-4.0	-4.5	0.5	15.0	7.0	3.5	13.3	13.3	7.00
2	-4.5	-5.0	0.5	15.0	7.0	3.5	16.8	16.8	7.00
2	-5.0	-5.5	0.5	15.0	7.0	3.5	20.3	20.3	7.00
2	-5.5	-6.0	0.5	15.0	7.0	3.5	23.8	23.8	7.00
2	-6.0	-6.5	0.5	15.0	7.0	3.5	27.3	27.3	7.00
2	-6.5	-7.0	0.5	15.0	7.0	3.5	30.8	30.8	7.00
2	-7.0	-7.5	0.5	15.0	7.0	3.5	34.3	34.3	7.00
2	-7.5	-8.0	0.5	15.0	7.0	3.5	37.8	37.8	7.00
2	-8.0	-8.5	0.5	15.0	7.0	3.5	41.3	41.3	7.00
2	-8.5	-9.0	0.5	15.0	7.0	3.5	44.8	44.8	7.00
3	-9.0	-9.5	0.5	25.0	9.0	4.5	49.3	44.8	0.00
3	-9.5	-10.0	0.5	25.0	9.0	4.5	53.8	44.8	0.00
3	-10.0	-10.5	0.5	25.0	9.0	4.5	58.3	44.8	0.00
3	-10.5	-11.0	0.5	25.0	9.0	4.5	62.8	44.8	0.00
3	-11.0	-11.5	0.5	25.0	9.0	4.5	67.3	47.3	5.00
3	-11.5	-12.0	0.5	25.0	9.0	4.5	71.8	51.8	9.00
3	-12.0	-12.5	0.5	25.0	9.0	4.5	76.3	56.3	9.00
3	-12.5	-13.0	0.5	25.0	9.0	4.5	80.8	60.8	9.00
4	-13.0	-13.5	0.5	50.0	9.0	4.5	85.3	60.8	0.00
4	-13.5	-14.0	0.5	50.0	9.0	4.5	89.8	60.8	0.00
4	-14.0	-14.5	0.5	50.0	9.0	4.5	94.3	60.8	0.00
4	-14.5	-15.0	0.5	50.0	9.0	4.5	98.8	60.8	0.00
4	-15.0	-15.5	0.5	50.0	9.0	4.5	103.3	60.8	0.00
4	-15.5	-16.0	0.5	50.0	9.0	4.5	107.8	60.8	0.00
5	-16.0	-16.5	0.5	40.0	7.5	3.8	111.6	61.6	1.50
5	-16.5	-17.0	0.5	40.0	7.5	3.8	115.3	65.3	7.50
5	-17.0	-17.5	0.5	40.0	7.5	3.8	119.1	69.1	7.50
5	-17.5	-18.0	0.5	40.0	7.5	3.8	122.8	72.8	7.50
5	-18.0	-18.5	0.5	40.0	7.5	3.8	126.6	76.6	7.50
5	-18.5	-19.0	0.5	40.0	7.5	3.8	130.3	80.3	7.50
5	-19.0	-19.5	0.5	40.0	7.5	3.8	134.1	84.1	7.50
5	-19.5	-20.0	0.5	40.0	7.5	3.8	137.8	87.8	7.50
5	-20.0	-20.5	0.5	40.0	7.5	3.8	141.6	91.6	7.50
5	-20.5	-21.0	0.5	40.0	7.5	3.8	145.3	95.3	7.50
5	-21.0	-21.5	0.5	40.0	7.5	3.8	149.1	99.1	7.50
5	-21.5	-22.0	0.5	40.0	7.5	3.8	152.8	102.8	7.50
5	-22.0	-22.5	0.5	40.0	7.5	3.8	156.6	106.6	7.50
5	-22.5	-23.0	0.5	40.0	7.5	3.8	160.3	110.3	7.50
5	-23.0	-23.5	0.5	40.0	7.5	3.8	164.1	114.1	7.50
5	-23.5	-24.0	0.5	40.0	7.5	3.8	167.8	117.8	7.50
5	-24.0	-24.5	0.5	40.0	7.5	3.8	171.6	121.6	7.50
5	-24.5	-25.0	0.5	40.0	7.5	3.8	175.3	125.3	7.50
5	-25.0	-25.5	0.5	40.0	7.5	3.8	179.1	129.1	7.50
5	-25.5	-26.0	0.5	40.0	7.5	3.8	182.8	132.8	7.50
5	-26.0	-26.5	0.5	40.0	7.5	3.8	186.6	136.6	7.50
5	-26.5	-27.0	0.5	40.0	7.5	3.8	190.3	140.3	7.50

Block-c

Layer	Elev. Top (m)	Elev. Bottom (m)	h (m)	Cu (kPa)	γ' (kN/m3)	γ'h (kPa)	Σγ'h (kPa)	P (total) (kN/m)	p {P(i) - P(i-1)}/h (kPa)
2	-3.0	-3.5	0.5	15.0	7.0	3.5	3.5	3.5	7.00
2	-3.5	-4.0	0.5	15.0	7.0	3.5	7.0	7.0	7.00
2	-4.0	-4.5	0.5	15.0	7.0	3.5	10.5	10.5	7.00
2	-4.5	-5.0	0.5	15.0	7.0	3.5	14.0	14.0	7.00
2	-5.0	-5.5	0.5	15.0	7.0	3.5	17.5	17.5	7.00
2	-5.5	-6.0	0.5	15.0	7.0	3.5	21.0	21.0	7.00
2	-6.0	-6.5	0.5	15.0	7.0	3.5	24.5	24.5	7.00
2	-6.5	-7.0	0.5	15.0	7.0	3.5	28.0	28.0	7.00
2	-7.0	-7.5	0.5	15.0	7.0	3.5	31.5	31.5	7.00
2	-7.5	-8.0	0.5	15.0	7.0	3.5	35.0	35.0	7.00
2	-8.0	-8.5	0.5	15.0	7.0	3.5	38.5	38.5	7.00
2	-8.5	-9.0	0.5	15.0	7.0	3.5	42.0	42.0	7.00
4	-9.0	-9.5	0.5	50.0	9.0	4.5	46.5	42.0	0.00
4	-9.5	-10.0	0.5	50.0	9.0	4.5	51.0	42.0	0.00
4	-10.0	-10.5	0.5	50.0	9.0	4.5	55.5	42.0	0.00
4	-10.5	-11.0	0.5	50.0	9.0	4.5	60.0	42.0	0.00
4	-11.0	-11.5	0.5	50.0	9.0	4.5	64.5	42.0	0.00
4	-11.5	-12.0	0.5	50.0	9.0	4.5	69.0	42.0	0.00
4	-12.0	-12.5	0.5	50.0	9.0	4.5	73.5	42.0	0.00
4	-12.5	-13.0	0.5	50.0	9.0	4.5	78.0	42.0	0.00
4	-13.0	-13.5	0.5	50.0	9.0	4.5	82.5	42.0	0.00
4	-13.5	-14.0	0.5	50.0	9.0	4.5	87.0	42.0	0.00
4	-14.0	-14.5	0.5	50.0	9.0	4.5	91.5	42.0	0.00
4	-14.5	-15.0	0.5	50.0	9.0	4.5	96.0	42.0	0.00
4	-15.0	-15.5	0.5	50.0	9.0	4.5	100.5	42.0	0.00
5	-15.5	-16.0	0.5	40.0	7.5	3.8	104.3	54.3	24.50
5	-16.0	-16.5	0.5	40.0	7.5	3.8	108.0	58.0	7.50
5	-16.5	-17.0	0.5	40.0	7.5	3.8	111.8	61.8	7.50
5	-17.0	-17.5	0.5	40.0	7.5	3.8	115.5	65.5	7.50
5	-17.5	-18.0	0.5	40.0	7.5	3.8	119.3	69.3	7.50
5	-18.0	-18.5	0.5	40.0	7.5	3.8	123.0	73.0	7.50
5	-18.5	-19.0	0.5	40.0	7.5	3.8	126.8	76.8	7.50
5	-19.0	-19.5	0.5	40.0	7.5	3.8	130.5	80.5	7.50
5	-19.5	-20.0	0.5	40.0	7.5	3.8	134.3	84.3	7.50
5	-20.0	-20.5	0.5	40.0	7.5	3.8	138.0	88.0	7.50
5	-20.5	-21.0	0.5	40.0	7.5	3.8	141.8	91.8	7.50
5	-21.0	-21.5	0.5	40.0	7.5	3.8	145.5	95.5	7.50
5	-21.5	-22.0	0.5	40.0	7.5	3.8	149.3	99.3	7.50
5	-22.0	-22.5	0.5	40.0	7.5	3.8	153.0	103.0	7.50
5	-22.5	-23.0	0.5	40.0	7.5	3.8	156.8	106.8	7.50
5	-23.0	-23.5	0.5	40.0	7.5	3.8	160.5	110.5	7.50
5	-23.5	-24.0	0.5	40.0	7.5	3.8	164.3	114.3	7.50
5	-24.0	-24.5	0.5	40.0	7.5	3.8	168.0	118.0	7.50
5	-24.5	-25.0	0.5	40.0	7.5	3.8	171.8	121.8	7.50
5	-25.0	-25.5	0.5	40.0	7.5	3.8	175.5	125.5	7.50
5	-25.5	-26.0	0.5	40.0	7.5	3.8	179.3	129.3	7.50
5	-26.0	-26.5	0.5	40.0	7.5	3.8	183.0	133.0	7.50
5	-26.5	-27.0	0.5	40.0	7.5	3.8	186.8	136.8	7.50
5	-27.0	-27.5	0.5	40.0	7.5	3.8	190.5	140.5	7.50

Block-d

Layer	Elev. Top (m)	Elev. Bottom (m)	h (m)	Cu (kPa)	γ' (kN/m3)	γ'h (kPa)	Σγ'h (kPa)	P (total) (kN/m)	p {P(i) - P(i-1)}/h (kPa)
2	-2.0	-2.5	0.5	15.0	7.0	3.5	3.5	3.5	7.00
2	-2.5	-3.0	0.5	15.0	7.0	3.5	7.0	7.0	7.00
2	-3.0	-3.5	0.5	15.0	7.0	3.5	10.5	10.5	7.00
2	-3.5	-4.0	0.5	15.0	7.0	3.5	14.0	14.0	7.00
2	-4.0	-4.5	0.5	15.0	7.0	3.5	17.5	17.5	7.00
2	-4.5	-5.0	0.5	15.0	7.0	3.5	21.0	21.0	7.00
2	-5.0	-5.5	0.5	15.0	7.0	3.5	24.5	24.5	7.00
2	-5.5	-6.0	0.5	15.0	7.0	3.5	28.0	28.0	7.00
2	-6.0	-6.5	0.5	15.0	7.0	3.5	31.5	31.5	7.00
2	-6.5	-7.0	0.5	15.0	7.0	3.5	35.0	35.0	7.00
2	-7.0	-7.5	0.5	15.0	7.0	3.5	38.5	38.5	7.00
2	-7.5	-8.0	0.5	15.0	7.0	3.5	42.0	42.0	7.00
2	-8.0	-8.5	0.5	15.0	7.0	3.5	45.5	45.5	7.00
2	-8.5	-9.0	0.5	15.0	7.0	3.5	49.0	49.0	7.00
2	-9.0	-9.5	0.5	15.0	7.0	3.5	52.5	52.5	7.00
2	-9.5	-10.0	0.5	15.0	7.0	3.5	56.0	56.0	7.00
2	-10.0	-10.5	0.5	15.0	7.0	3.5	59.5	59.5	7.00
2	-10.5	-11.0	0.5	15.0	7.0	3.5	63.0	63.0	7.00
3	-11.0	-11.5	0.5	25.0	9.0	4.5	67.5	63.0	0.00
3	-11.5	-12.0	0.5	25.0	9.0	4.5	72.0	63.0	0.00
3	-12.0	-12.5	0.5	25.0	9.0	4.5	76.5	63.0	0.00
3	-12.5	-13.0	0.5	25.0	9.0	4.5	81.0	63.0	0.00
3	-13.0	-13.5	0.5	25.0	9.0	4.5	85.5	65.5	5.00
4	-13.5	-14.0	0.5	50.0	9.0	4.5	90.0	65.5	0.00
4	-14.0	-14.5	0.5	50.0	9.0	4.5	94.5	65.5	0.00
4	-14.5	-15.0	0.5	50.0	9.0	4.5	99.0	65.5	0.00
4	-15.0	-15.5	0.5	50.0	9.0	4.5	103.5	65.5	0.00
4	-15.5	-16.0	0.5	50.0	9.0	4.5	108.0	65.5	0.00
5	-16.0	-16.5	0.5	40.0	7.5	3.8	111.8	65.5	0.00
5	-16.5	-17.0	0.5	40.0	7.5	3.8	115.5	65.5	0.00
5	-17.0	-17.5	0.5	40.0	7.5	3.8	119.3	69.3	7.50
5	-17.5	-18.0	0.5	40.0	7.5	3.8	123.0	73.0	7.50
5	-18.0	-18.5	0.5	40.0	7.5	3.8	126.8	76.8	7.50
5	-18.5	-19.0	0.5	40.0	7.5	3.8	130.5	80.5	7.50
5	-19.0	-19.5	0.5	40.0	7.5	3.8	134.3	84.3	7.50
5	-19.5	-20.0	0.5	40.0	7.5	3.8	138.0	88.0	7.50
5	-20.0	-20.5	0.5	40.0	7.5	3.8	141.8	91.8	7.50
5	-20.5	-21.0	0.5	40.0	7.5	3.8	145.5	95.5	7.50
5	-21.0	-21.5	0.5	40.0	7.5	3.8	149.3	99.3	7.50
5	-21.5	-22.0	0.5	40.0	7.5	3.8	153.0	103.0	7.50
5	-22.0	-22.5	0.5	40.0	7.5	3.8	156.8	106.8	7.50
5	-22.5	-23.0	0.5	40.0	7.5	3.8	160.5	110.5	7.50
5	-23.0	-23.5	0.5	40.0	7.5	3.8	164.3	114.3	7.50
5	-23.5	-24.0	0.5	40.0	7.5	3.8	168.0	118.0	7.50
5	-24.0	-24.5	0.5	40.0	7.5	3.8	171.8	121.8	7.50
5	-24.5	-25.0	0.5	40.0	7.5	3.8	175.5	125.5	7.50
5	-25.0	-25.5	0.5	40.0	7.5	3.8	179.3	129.3	7.50
5	-25.5	-26.0	0.5	40.0	7.5	3.8	183.0	133.0	7.50
5	-26.0	-26.5	0.5	40.0	7.5	3.8	186.8	136.8	7.50

3. Active Earth Pressure

Active earth pressure

1) After completion

Surcharge load w = 30.00 kPa

	Above W or Below W	Sand or Clay	h (m)	γ (kN/m3)	γsat (kN/m3)	φ (deg)	δ (deg)	c (kPa)	γ' h+w (kPa)	γsat h+w	For Sand Kai x cos φ	For Sand ζa (rad)	For Sand pa (kPa)	For Clay ζa (rad)	For Clay pa (kPa)	Adopted pa (kPa)
5.50									30.0	30.0	0.291	0.992	8.73			8.730
	Above	Sand	1.95	18	20	30	15	0								
3.55									65.1	65.1	0.291	0.992	18.94			18.940
3.55									65.1	65.1	0.291	0.992	18.94			18.940
	Below	Sand	5.55	18	20	30	15	0								
-2.00									120.6	176.1	0.291	0.992	35.09			35.090
-2.00									120.6	176.1				0.785	90.60	90.600
	Below	Clay	1.00	17	17	0	0	15								
-3.00									127.6	193.1				0.785	97.60	97.600
-3.00																7.000
	Below	Clay	22.00	As explaind in section 2, pa = 7.0 kPa is used.												
-25.00																7.000

2) During construction

Surcharge load w = 20.00 kPa

	Above W or Below W	Sand or Clay	h (m)	γ (kN/m3)	γsat (kN/m3)	φ (deg)	δ (deg)	c (kPa)	γ' h+w (kPa)	γsat h+w	For Sand Kai x cos φ	For Sand ζa (rad)	For Sand pa (kPa)	For Clay ζa (rad)	For Clay pa (kPa)	Adopted pa (kPa)
4.50									20.0	20.0	0.291	0.992	5.82			5.820
	Above	Sand	0.95	18	20	30	15	0								
3.55									37.1	37.1	0.291	0.992	10.80			10.800
3.55									37.1	37.1	0.291	0.992	10.80			10.800
	Below	Sand	5.55	18	20	30	15	0								
-2.00									92.6	148.1	0.291	0.992	26.95			26.950
-2.00									92.6	148.1				0.785	62.60	62.600
	Below	Clay	1.00	17	17	0	0	15								
-3.00									99.6	165.1				0.785	69.60	69.600
-3.00																7.000
	Below	Clay	22.00	As explaind in section 2, pa = 7.0 kPa is used.												
-25.00																7.000

Appendix 7-6

Passive Earth Pressure on Steel Sheet Pipe Pile

Determination of Passive Earth Pressure on SPSP

1. Method of Passive Earth Pressure Calculation

A magnitude of the passive earth pressure acting on the steel pipe wall for the revetment of the Container Berth and the Barge Berth is the most important factor for the steel pipe wall design.

A calculation method of the passive earth pressure in the case of the sloped cohesive ground is not indicated in the "Technical Standards and Commentaries for Port and Harbour Facilities in Japan (TSCPHFJ)" but the following method is recommended in the "Guidelines for Port and Harbour Design 1989", which is a draft TSCPHFJ with more detailed commentaries than the published TSCPHFJ. This method is called as the "circular slip method" in this document.

(Extracted from 5.2.2 of Chapter 5, Part 8 of Guidelines for Port and Harbour Design)

$$P_{2h}=\cos\delta W_2' \cdot \frac{\cos(\phi-\theta-\varepsilon)}{\sin(\theta-\phi+\delta)} \qquad (5.5)$$

また，作用点は矢板根入れ長の 1/3 点とする簡便法を用いることがある。

粘性土地盤の場合には，図−5.17 のように，すべり面を矢板下端から発生する円弧とし，円弧の中心及び半径を変えてモーメントの釣り合いにより P を求め，その最小値を受働土圧合力とする。

ただし，P の作用点は矢板根入れ長の 1/3 点とする。

図−5.17 において

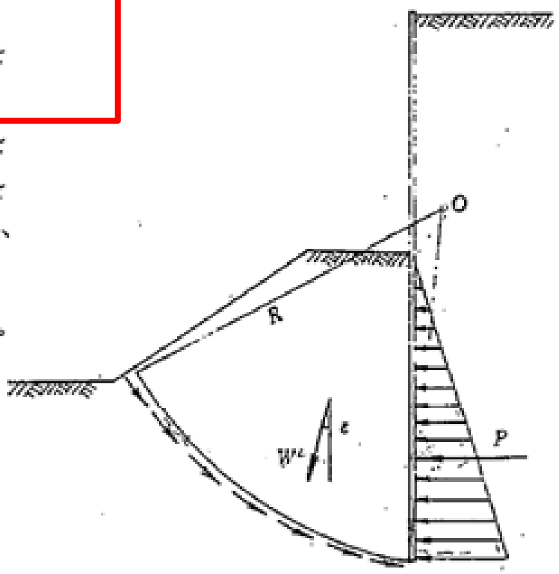
O ：円弧の中心点

R ：円弧の半径（m）

W'：地震時の土けいの重量（tf/m）

P ：受働土圧（tf/m）

c ：粘着力（tf/m²）



図−5.17 斜面をもった矢板根入れ部の受働土圧（粘性土地盤）

(4) クレーン等の荷役機械がある場合には，そのための基礎を設けることがのぞましいが，設けない場合には集中荷重あるいは線荷重に換算して，土圧の増加分を第 5 編 5.1 地中応力 により求める。

English translation:

In the case of the (sloped) ground of the cohesive soil, a circular passive failure surface starting from the bottom of the sheet pile as shown in Figure 5.17 is assumed. The force P is obtained for the assumed center and radius of the circle so that the moment about the center of the circle will be balance. The passive earth pressure is obtained as the minimum value of P, which is obtained by changing the center and radius of the circle. For this calculation, the acting point of P shall be one third of the embedded length of the sheet pile.

Where in Figure 5.17;

- O : the center of the circular slip surface
- R : the radius of the circular slip surface (m)
- W' : the weight of soil above the circular slip surface (tf/m)
- P : the passive earth pressure (tf/m)
- c : the shear strength of the soil (tf/m^2)

1

2. Verification of Circular Slip Method

At first, the passive earth pressure in the case of the horizontal ground was obtained by the following two methods for a purpose to ensure an appropriateness of the use of the circular slip method:

- Ordinary formula (i.e., $p_p = \sum\gamma h + 2c$)
- Circular slip method

The soil parameters of Block b were used for the comparison.

1) Ordinary formula

$p_p = \sum\gamma h + 2c$

where;

p_p : the passive earth pressure (kPa)
γ : the effective weight of the soil (kN/m3)
h : the thickness of the soil layer (m)
c : the shear strength of the soil (kPa)

Theoretical passive earth pressure by the above formula is obtained as shown in Figure 1.

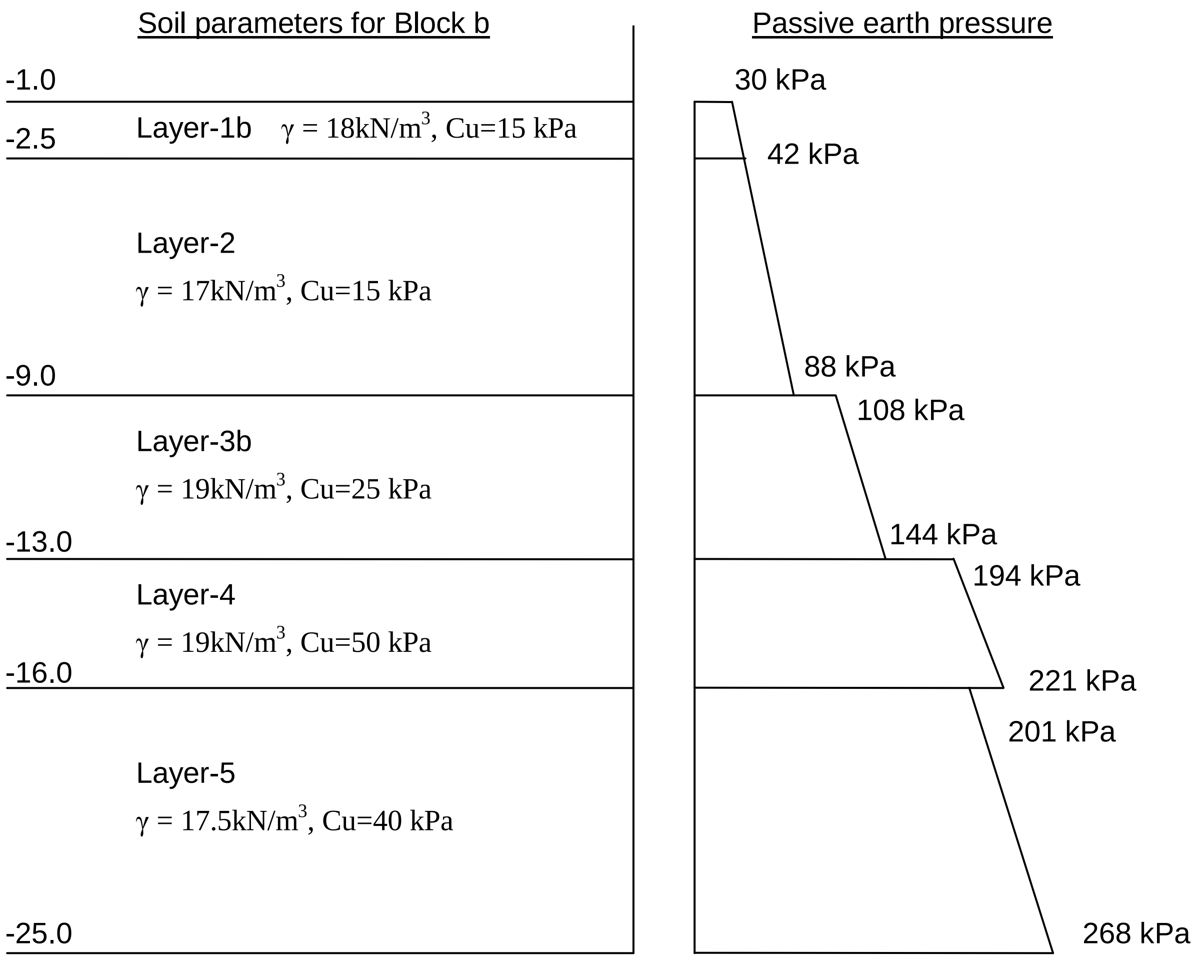


Figure 1 Passive earth pressure (ordinary formula)

2) Circular slip method

The spreadsheet calculation was carried out by using Microsoft Excel. Figure 2 presents a part of the spreadsheet for reference.

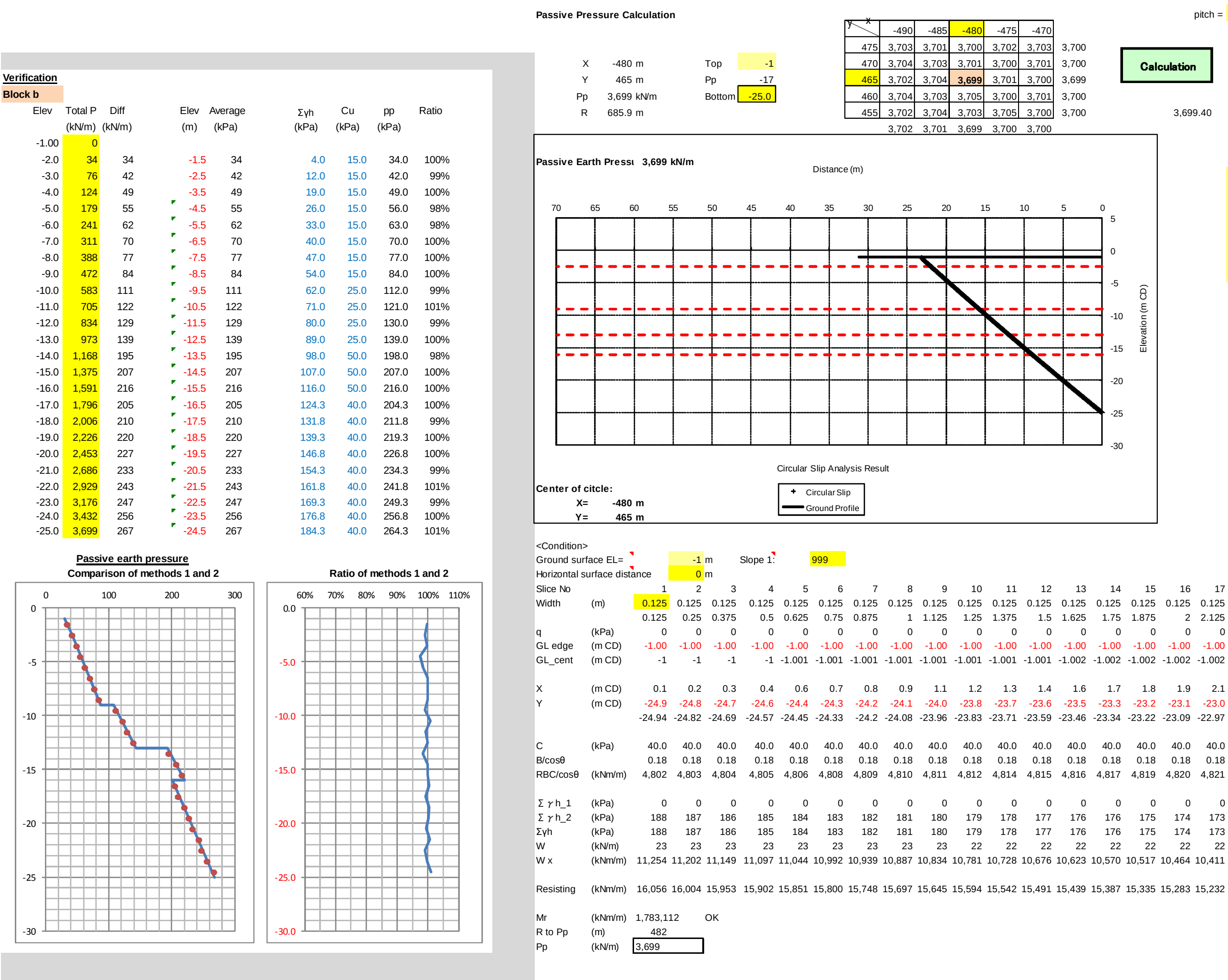


Figure 2 Part of spreadsheet for calculation of circular slip method

The slope of the ground is taken as 1 : 999 (≈ horizontal).

The starting point of the circular slip failure surface has been changed by every 1m. The passive earth pressure was obtained by the following formula.

$$p(z+\Delta z/2) = \{P(z+\Delta z) - P(z)\}/\Delta z$$

where; p (z) is the passive earth pressure at depth z (kPa)

P (z) is the sum of the passive earth pressure above the depth z (kN/m)

Δz is the change in depth from the previous calculation (= 1m)

As shown in Figure 2, the failure line obtained is nearly straight with 45 degrees to horizontal which is similar to the ordinary formula.

Figure 3 presents the comparison of the ordinary formula and the circular slip method. Although there are slight deviations of the circular slip method from the theoretical method (i.e. ordinary formula) which is considered due to the setting of slice width and/or decimal treatment, etc., the results of these two methods could be said nearly equal.

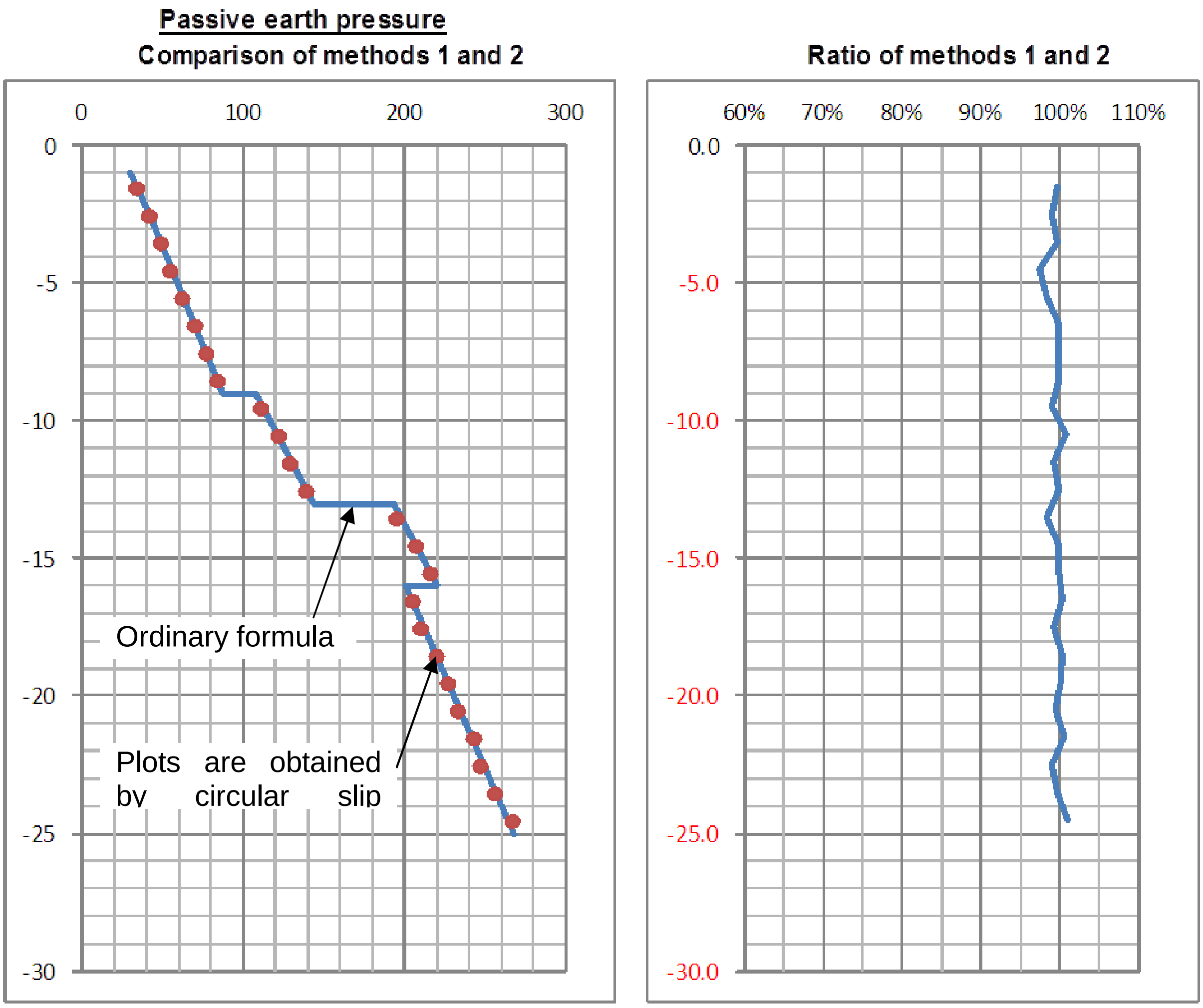


Figure 3 Comparison of ordinary formula and circular slip method

From the above, it is concluded that the circular slip method can be applied for this design and also appropriateness of the spreadsheet in Figure 2 was verified.

3. Passive Earth Pressure Calculation for Sloped Ground

The passive earth pressure of the slope ground is estimated in a similar manner to the chapter 2 with slope angle of 1 in 3. A part of the spreadsheet is presented in Figure 4.

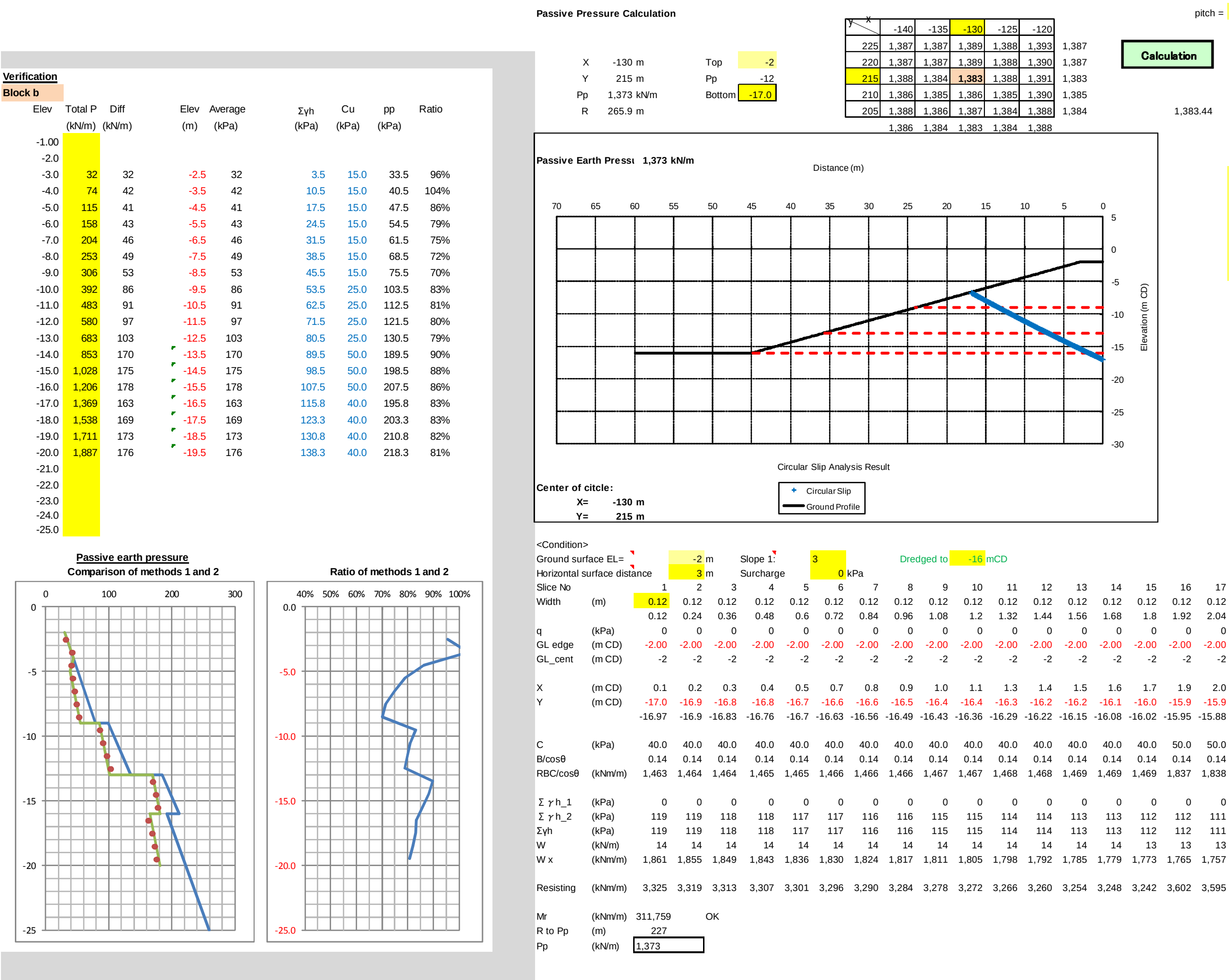


Figure 4 Part of spreadsheet for calculation of circular slip method (sloped ground)

As shown in Figure 4, the failure surface will be nearly straight but with the angle to horizontal rather gentle than 45˚.

Figure 5 presents the comparison of the ordinary formula and the circular slip method.

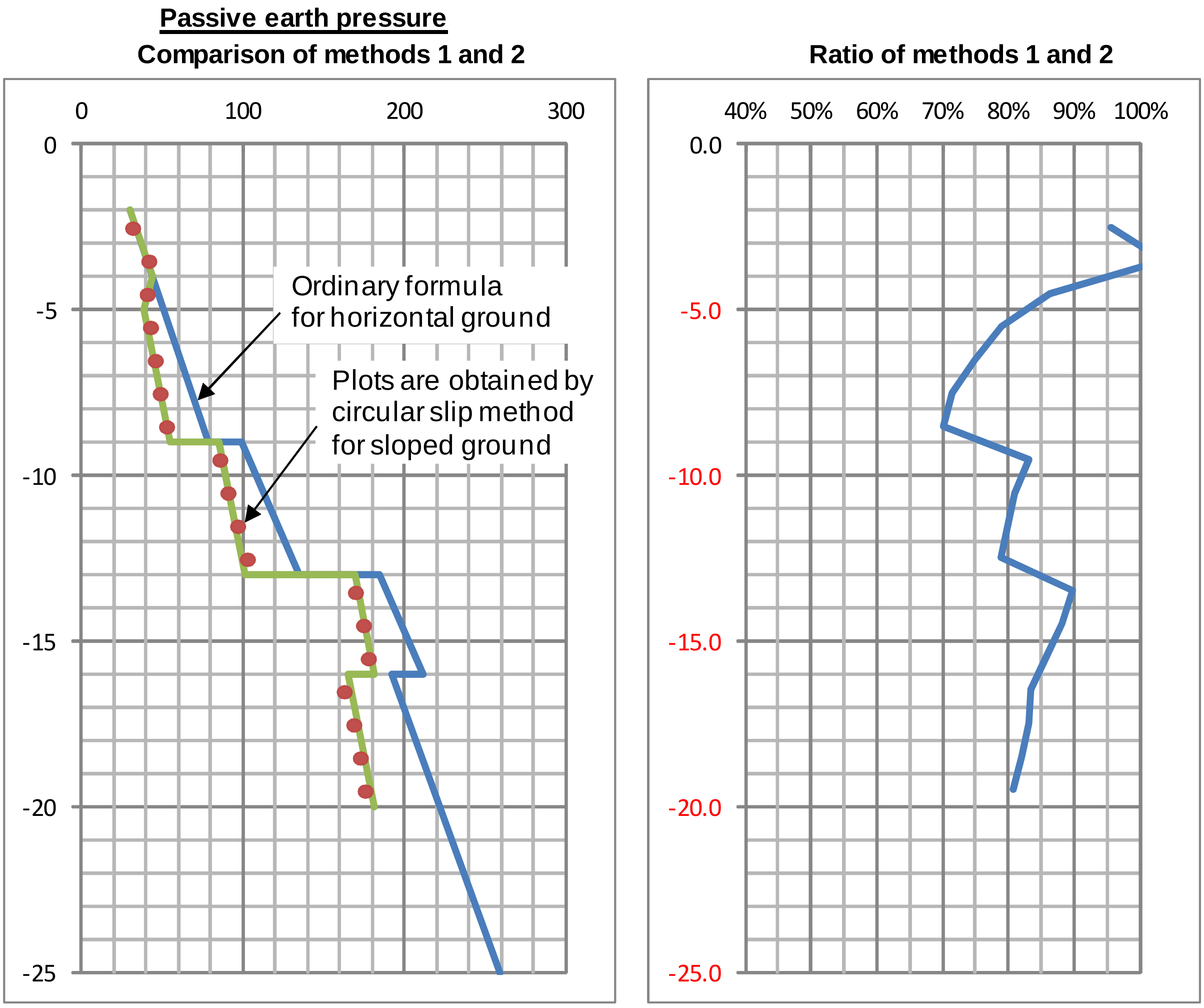


Figure 5 Comparison of ordinary formula and circular slip method (Sloped ground)

From Figure 5, it is obvious that the passive earth pressure of the sloped ground will be generally decreased from that of the horizontal ground.

4. Passive Earth Pressure of Sloped Ground for Design of Revetment

The seabed profile for the passive earth pressure calculation for each block is shown in Figure 6.

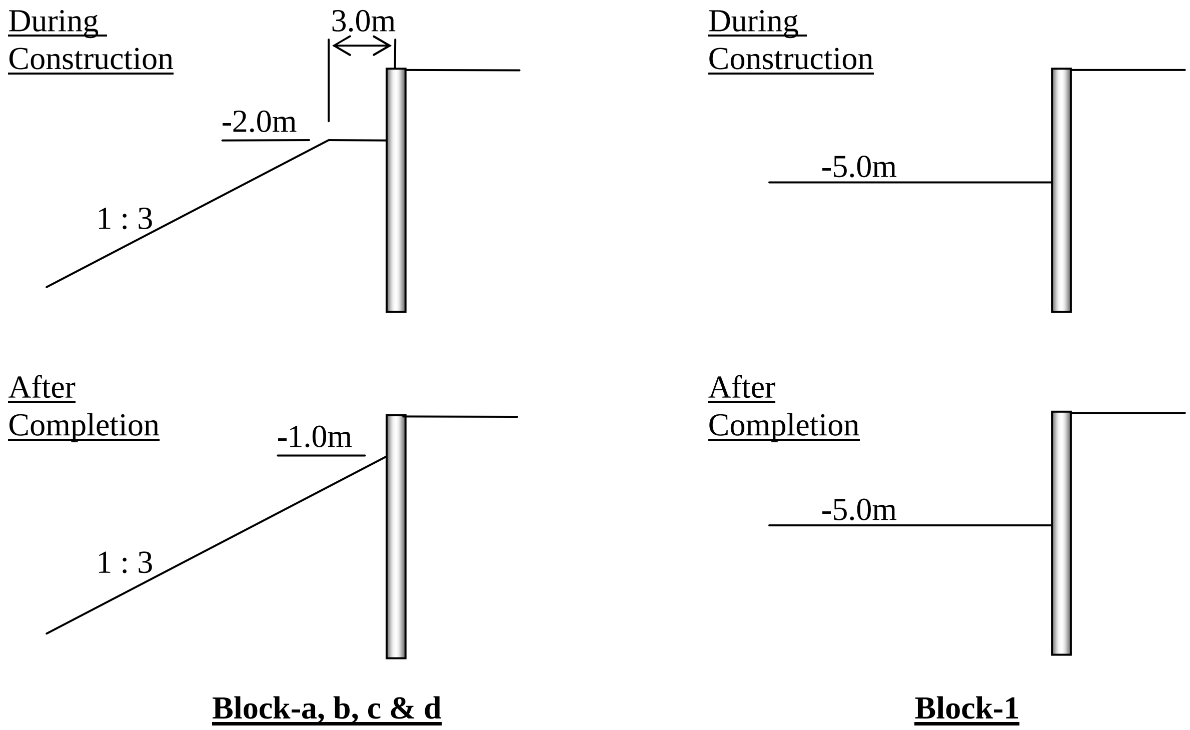


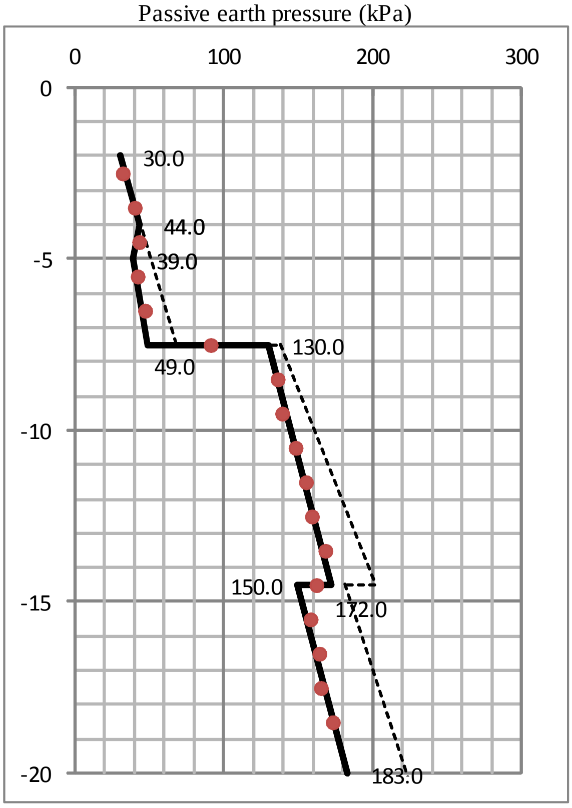
Figure 6 Seabed profiles for passive earth pressure calculation

In Block c, there is a 1.5m thk loose sand layer (Layer 3a) from -8.0mCD to -9.5mCD. However, in the calculation of the passive earth pressure, this layer was regarded as the soft clay layer (Layer 2) for simple and conservative calculation.

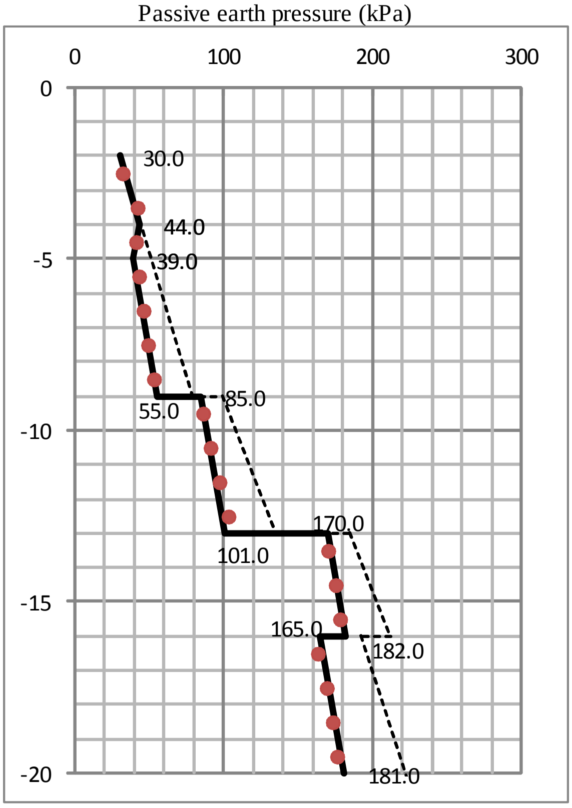
For Block-a, b, c and d, the passive earth pressures of sloped ground are obtained by the circular slip method as shown in Figures 7 and 8. These will be used for the revetment design.

For Block 1, the design will be done with the horizontal seabed level at -5.0m and therefore ordinary formula of the passive earth pressure (i.e. $p=\sum\gamma h+2C$) was used to obtain the passive earth pressure. The passive earth pressure for the Block 1 is shown in Table 1 and Figure 9.

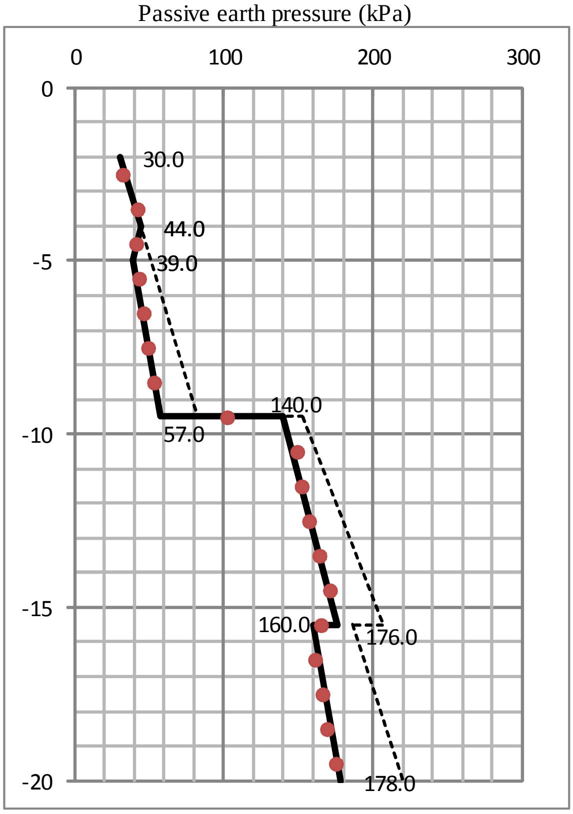
Block-a



Block-b



Block-c



Block-d

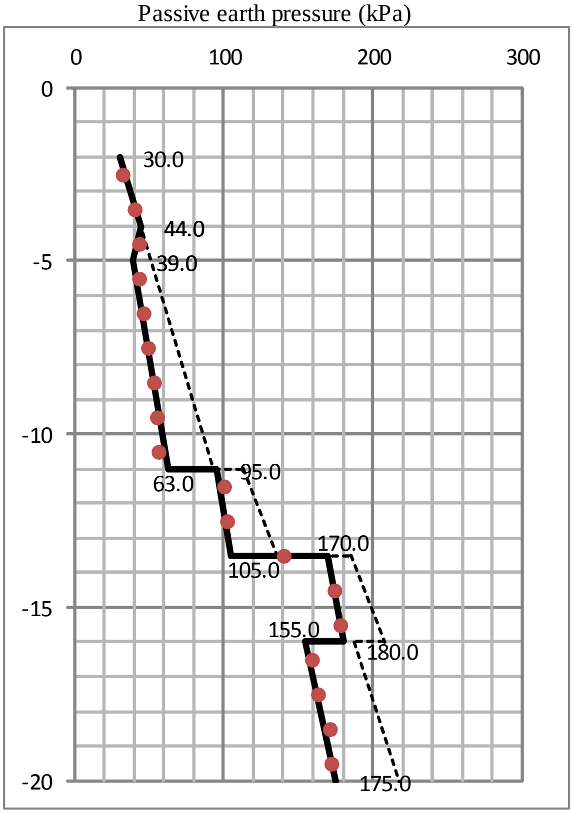
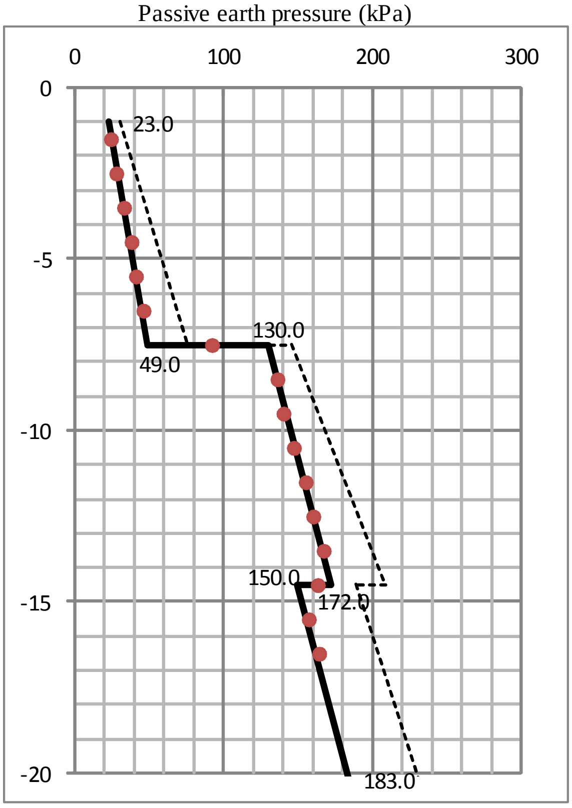
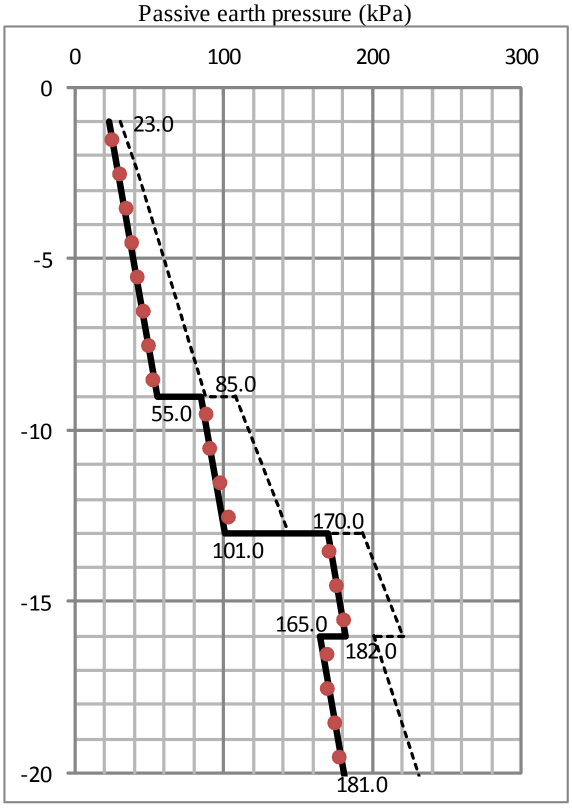


Figure 7 Passive earth pressures for revetment design (Block a, b, c & d) – During Construction

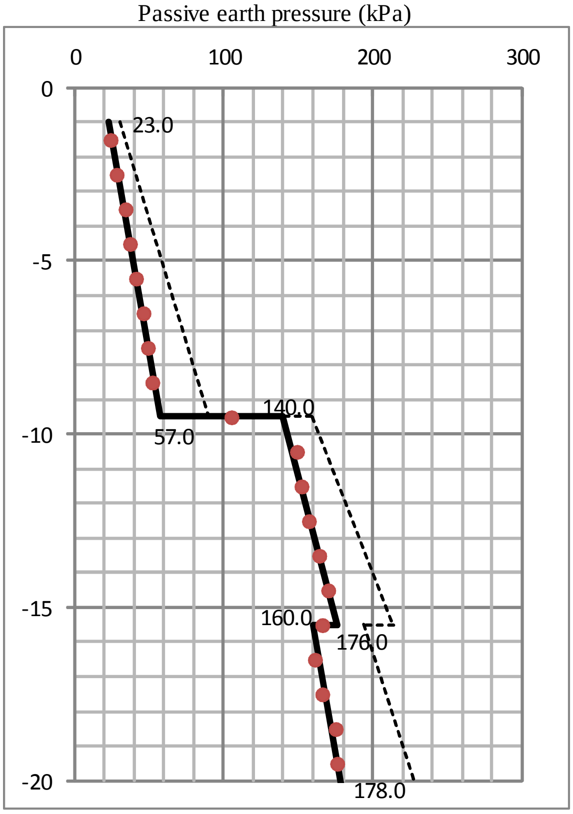
Block-a



Block-b



Block-c



Block-d

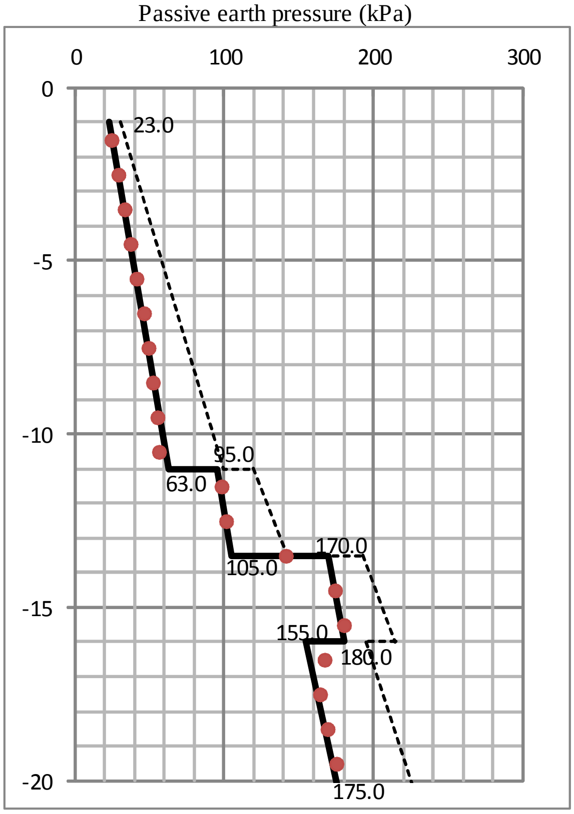


Figure 8 Passive earth pressures for revetment design (Block a, b, c & d) – After Completion

Table 1 Passive earth pressures for revetment design (Block 1)

Layer	Elev.	h (m)	C (kPa)	γ ' (kN/m^3)	∑γh (kPa)	Passive earth pressure p_p = ∑γh+2C (kPa)
2	-5.0 -8.0	3.0	15	7.0	0.0 21.0	0.0 + 2×15 = 30.0 21.0 + 2×15 = 51.0
3	-8.0 -10.0	2.0	25	9.0	21.0 39.0	21.0 + 2×25 = 71.0 39.0 + 2×25 = 89.0
4	-10.0 -15.0	5.0	50	9.0	39.0 84.0	39.0 + 2×50 = 139.0 84.0 + 2×50 = 184.0
5	-15.0 -26.0	11.0	40	7.5	84.0 166.5	84.0 + 2×40 = 164.0 166.5 + 2×40 = 246.5

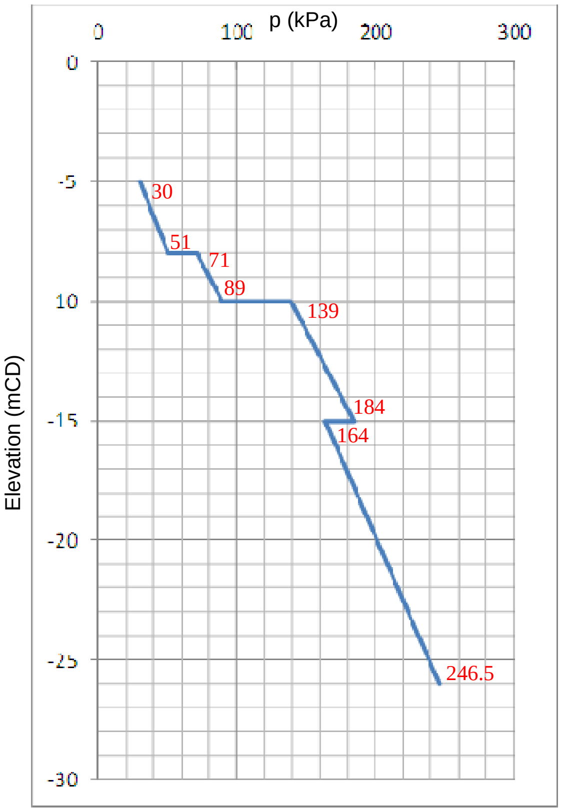


Figure 9 Passive earth pressures for revetment design (Block 1)
– both During Construction and After Completion

Appendix 7-7

Calculation Output for Steel Sheet Pipe Pile Wall (After Completion)

Output of Design Software for Anchor Wall (After Completion) – Junction to Service Berth

1. Design Conditions

1-1 Dimensions

Ground elevation	+5.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-5.00 m
Angle of seabed slope	0.0˚

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	30 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

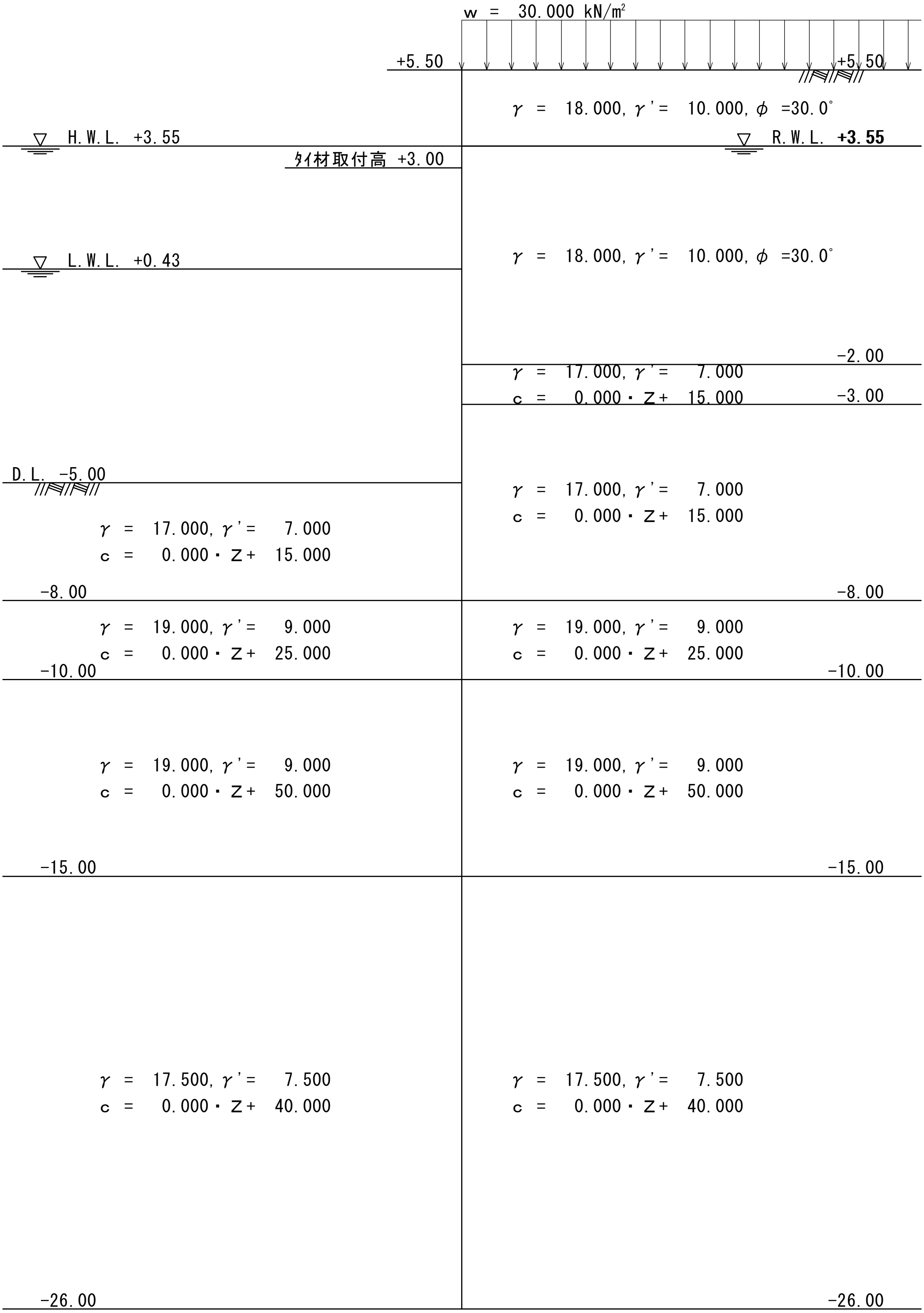
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
5.50 3.55	1.95	8.730 18.940
3.55 3.00	0.55	18.940 20.540
3.00 -2.00	5.00	20.540 35.090
-2.00 -3.00	1.00	90.600 97.600
-3.00 -8.00	5.00	7.000 7.000
-8.00 -10.00	2.00	7.000 7.000
-10.00 -15.00	5.00	7.000 7.000
-15.00 -26.00	11.00	7.000 7.000

(2) Residual water pressure

$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)

h_w is the height of the tidal lag (m)

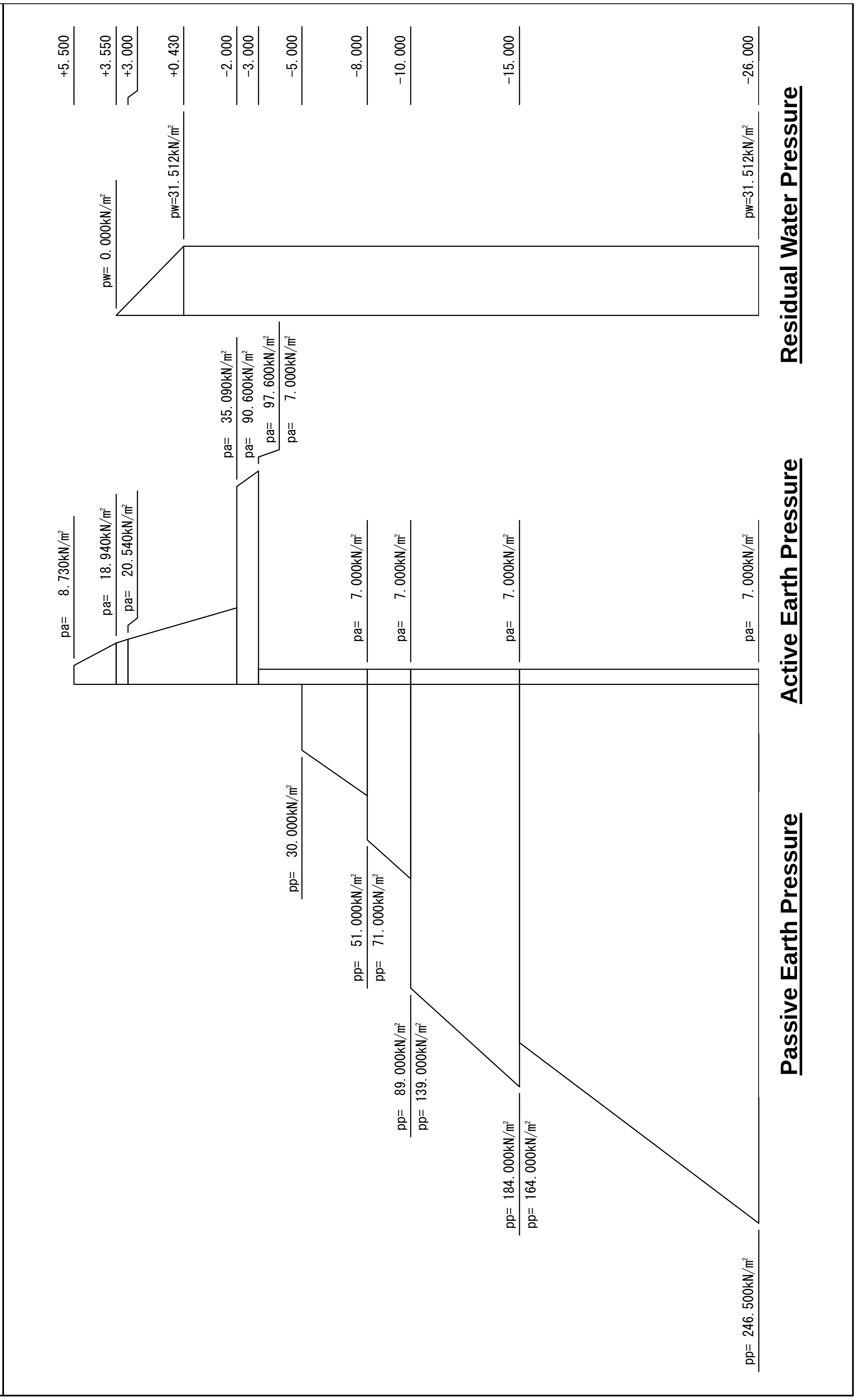
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-5.00 -8.00	3.00	30.000 51.000
-8.00 -10.00	2.00	71.000 89.000
-10.00 -15.00	5.00	139.000 184.000
-15.00 -26.00	11.00	164.000 246.500

Earth Pressure and Water Pressure



(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 3.55	8.730+ 0.000= 8.730 18.940+ 0.000= 18.940	——— ———
3.55 3.00	18.940+ 0.000= 18.940 20.540+ 5.555= 26.095	——— ———
3.00 0.43	20.540+ 5.555= 26.095 28.019+ 31.512= 59.531	——— ———
0.43 -2.00	28.019+ 31.512= 59.531 35.090+ 31.512= 66.602	——— ———
-2.00 -3.00	90.600+ 31.512= 122.112 97.600+ 31.512= 129.112	——— ———
-3.00 -5.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	——— ———
-5.00 -8.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	30.000 51.000
-8.00 -10.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	71.000 89.000
-10.00 -15.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	139.000 184.000
-15.00 -26.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	164.000 246.500

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -5.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-5.000	-1369.207	263.678	241.600
-6.000	-1833.725	290.866	219.424
-7.000	-2274.387	312.767	195.535
-8.000	-2676.627	330.188	169.126
-9.000	-2982.565	342.803	119.523
-10.000	-3123.607	350.285	66.053
-11.000	-2951.847	351.307	-39.957
-12.000	-2311.902	344.894	-147.532
-13.000	-1148.645	331.877	-257.503
-14.000	600.285	312.893	-370.507

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -13.00 and -14.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-13.695	0.000	319.295	-335.643

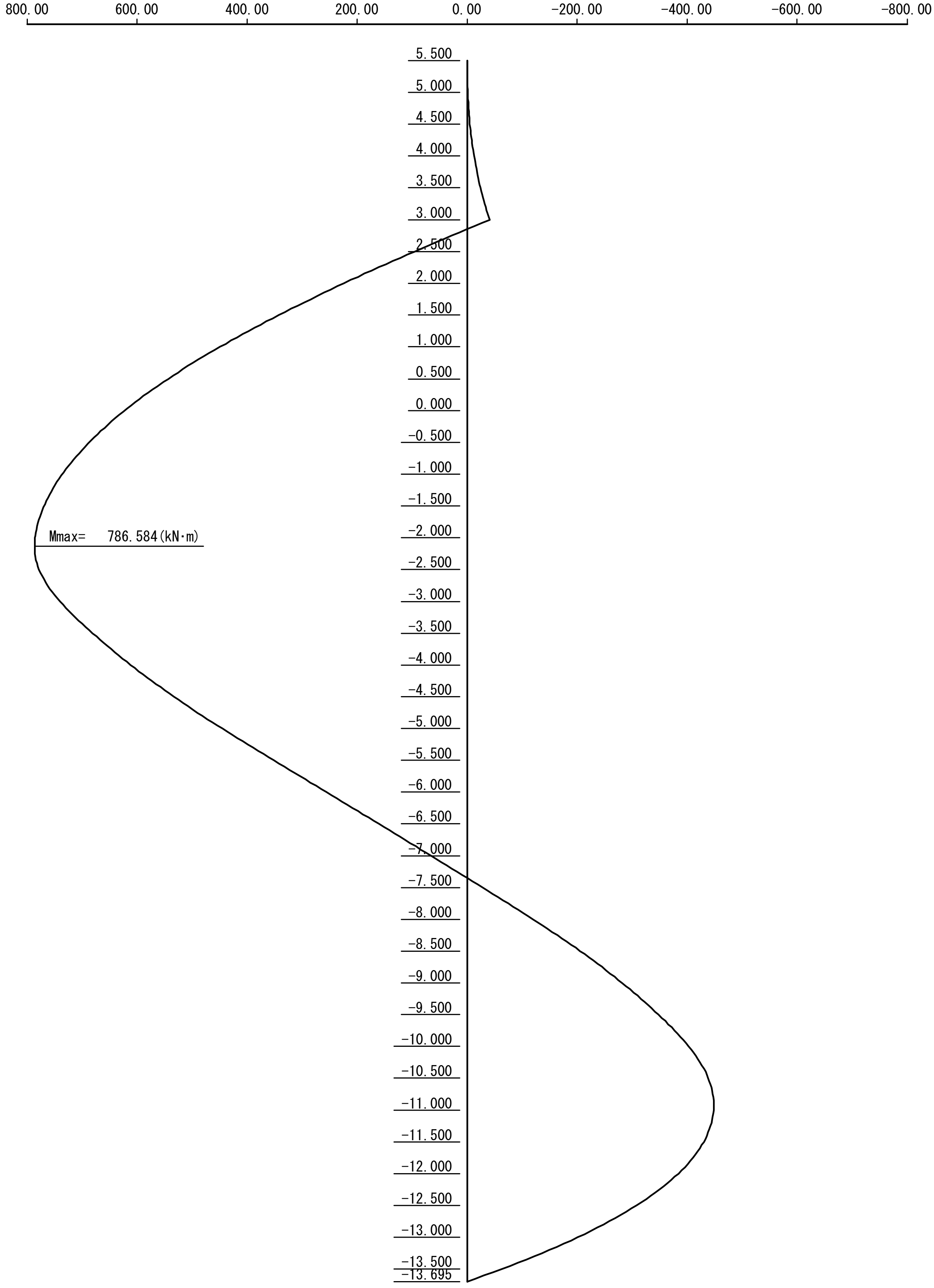
Embedded length of the pipe wall : L = 1.2 × (-5.000 + 13.695) = 10.434 m
Toe elevation : D = -5.000 – 10.434 = -15.434 m
Reaction force at tie setting point : 319.295 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	-1.200	-5.019
4.500	-5.238	-11.348
4.000	-12.766	-18.985
3.550	-23.069	-26.978
3.050	-39.196	-38.074
3.000	-41.132	-39.363
2.500	95.301	265.258
2.000	223.584	247.332
1.500	342.091	226.153
1.000	449.195	201.722
0.500	543.271	174.038
0.430	555.309	169.903
-0.070	632.759	139.773
-0.570	694.962	108.917
-1.070	741.554	77.333
-1.570	772.173	45.021
-2.000	785.452	16.651
-2.500	778.367	-45.280
-3.000	739.880	-108.961
-3.500	680.586	-128.217
-4.000	611.663	-147.473
-4.500	533.113	-166.729
-5.000	444.934	-185.985
-5.500	351.024	-189.366
-6.000	255.860	-190.997
-6.500	160.319	-190.878
-7.000	65.274	-189.009
-7.500	-28.399	-185.390
-8.000	-119.824	-180.021
-8.500	-205.586	-162.652
-9.000	-282.101	-143.033
-9.500	-348.244	-121.164
-10.000	-402.890	-97.045
-10.500	-438.664	-45.676
-11.000	-448.191	7.943
-11.500	-430.346	63.812
-12.000	-384.004	121.931
-12.500	-308.040	182.300
-13.000	-201.329	244.919
-13.500	-62.746	309.788
-13.695	0.000	335.643

Maximum bending moment : 786.584 kN-m/m
Elevation of maximum bending moment : -2.136 m
Depth of 1st steady point (moment=0) : -7.347 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
1	5.50	1/2x	8.730x	1.950	8.512	-1.850	-15.747	
2	3.55	1/2x	18.940x	1.950	18.466	-1.200	-22.159	-45.487
3	3.55	1/2x	18.940x	0.550	5.208	-0.367	-1.911	
4	3.00	1/2x	26.095x	0.550	7.176	-0.183	-1.313	-49.356
5	3.00	1/2x	26.095x	2.570	33.532	0.857	28.737	
6	0.43	1/2x	59.531x	2.570	76.497	1.713	131.039	142.375
7	0.43	1/2x	59.531x	2.430	72.330	3.380	244.475	
8	-2.00	1/2x	66.602x	2.430	80.921	4.190	339.059	842.616
9	-2.00	1/2x	122.112x	1.000	61.056	5.333	325.612	
10	-3.00	1/2x	129.112x	1.000	64.556	5.667	365.839	1672.357
11	-3.00	1/2x	38.512x	2.000	38.512	6.667	256.760	
12	-5.00	1/2x	38.512x	2.000	38.512	7.333	282.408	2319.359
13	-5.00	1/2x	38.512x	3.000	57.768	9.000	519.912	
14	-8.00	1/2x	38.512x	3.000	57.768	10.000	577.680	3636.469
15	-8.00	1/2x	38.512x	2.000	38.512	11.667	449.320	
16	-10.00	1/2x	38.512x	2.000	38.512	12.333	474.968	4745.615
17	-10.00	1/2x	38.512x	5.000	96.280	14.667	1412.139	
18	-15.00	1/2x	38.512x	5.000	96.280	16.333	1572.541	8327.231
19	-15.00	1/2x	38.512x	11.000	211.816	21.667	4589.417	
20	-26.00	1/2x	38.512x	11.000	211.816	25.333	5365.935	20273.653

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

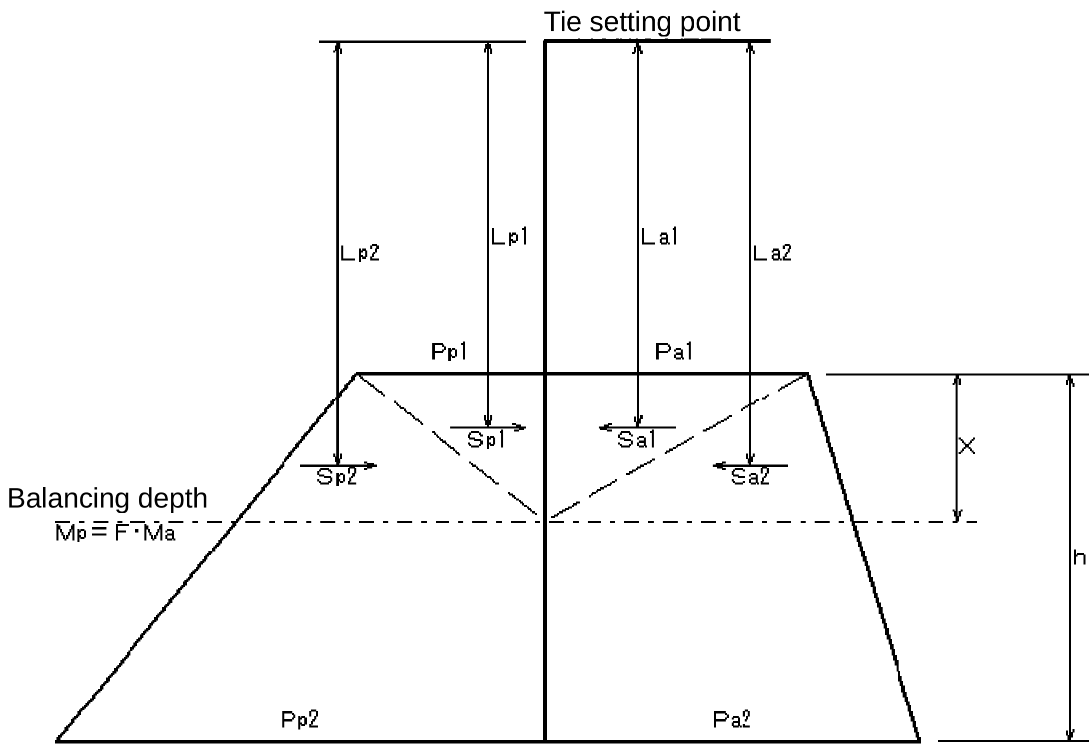
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-5.00	1/2x 30.000x 3.000	45.000	9.000	405.000	
2	-8.00	1/2x 51.000x 3.000	76.500	10.000	765.000	1170.000
3	-8.00	1/2x 71.000x 2.000	71.000	11.667	828.357	
4	-10.00	1/2x 89.000x 2.000	89.000	12.333	1097.637	3095.994
5	-10.00	1/2x 139.000x 5.000	347.500	14.667	5096.782	
6	-15.00	1/2x 184.000x 5.000	460.000	16.333	7513.180	15705.956
7	-15.00	1/2x 164.000x 11.000	902.000	21.667	19543.634	
8	-26.00	1/2x 246.500x 11.000	1355.750	25.333	34345.215	69594.805

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1} X}{2}, \quad L_{a(p)1} = L + \frac{1}{3} X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$	: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)
$M_{a(p)1}$	: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)
$M_{a(p)1'}$	: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$M_{a(p)2'}$	: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$S_{a(p)1}$	: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$S_{a(p)2}$	: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$L_{a(p)1}$	: the arm length of $S_{a(p)1}$ from the tie setting point (m)
$L_{a(p)2}$	: the arm length of $S_{a(p)2}$ from the tie setting point (m)
$P_{a(p)1}$	: the active (passive) side pressure at the top elevation of the soil layer (kPa)
$P_{a(p)1}$	: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)
h	: the thickness of the soil layer (m)
L	: the vertical distance from the tie setting point to the top elevation of the soil layer (m)
X	: the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -10.00 m FOS×M_{a1} = 4745.615 > M_{p1} = 3095.994
at -15.00 m FOS×M_{a2} = 8327.231 < M_{p2} = 15705.956

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \quad L_{a1} = 13.00 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 250.328X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{5.000}X\right]X}{2} = 0.000X^2 + 19.256X \ , \quad L_{a2} = 13.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 250.328X$$

$$M_a = 0.000X^3 + 19.256X^2 + 500.656X + 3954.679$$

[Moment on passive side]

$$S_{p1} = \frac{139.000X}{2} = 69.500X \ , \quad L_{p1} = 13.000 + \frac{1}{3}X$$

$$M_{p1'} = 23.167X^2 + 903.500X$$

$$S_{p2} = \frac{\left[139.000 + \frac{184.000 - 139.000}{5.000}X\right]X}{2} = 4.500X^2 + 69.500X \ , \quad L_{p2} = 13.000 + \frac{2}{3}X$$

$$M_{p2'} = 3.000X^3 + 104.833X^2 + 903.500X$$

$$M_p = 3.000X^3 + 128.000X^2 + 1807.000X + 3095.994$$

[Balancing depth]

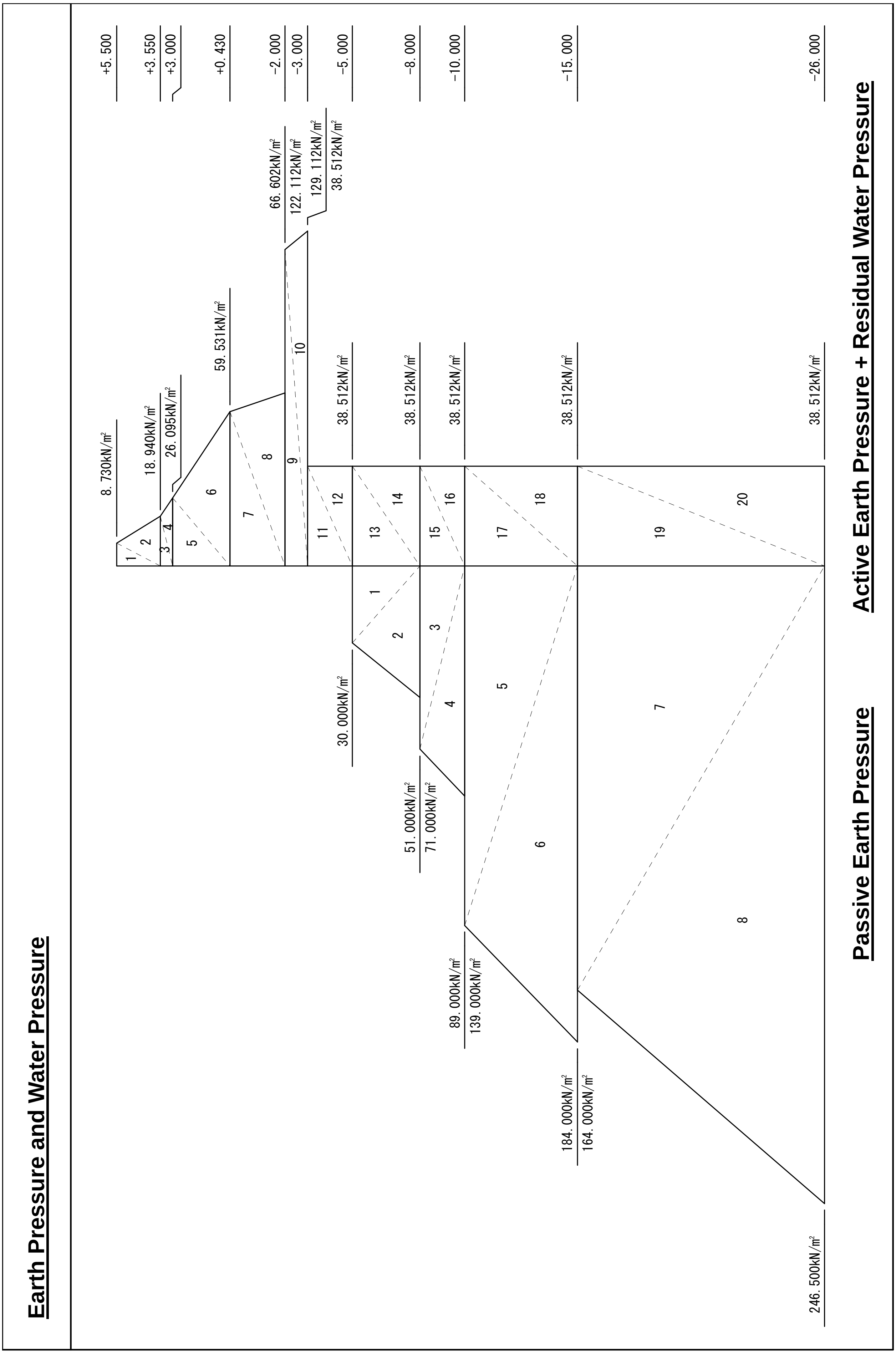
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 500.656X + 3954.679)$$

$$= 3.000X^3 + 128.000X^2 + 1807.000X + 3095.994$$

X= 1.231 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -10.000 – 1.231 = -11.231 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+3.550
+3.000
+0.430
-2.000
-3.000
-5.000
-8.000
-10.000
-15.000
-26.000
8.730kN/m²
18.940kN/m²
26.095kN/m²
59.531kN/m²
66.602kN/m²
122.112kN/m²
129.112kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
30.000kN/m²
51.000kN/m²
71.000kN/m²
89.000kN/m²
139.000kN/m²
184.000kN/m²
164.000kN/m²
246.500kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 3.55	8.730 18.940	——— ———	8.730 18.940
3.55 3.00	18.940 26.095	——— ———	18.940 26.095
3.00 0.43	26.095 59.531	——— ———	26.095 59.531
0.43 -2.00	59.531 66.602	——— ———	59.531 66.602
-2.00 -3.00	122.112 129.112	——— ———	122.112 129.112
-3.00 -5.00	38.512 38.512	——— ———	38.512 38.512
-5.00 -8.00	38.512 38.512	30.000 51.000	8.512 -12.488
-8.00 -10.00	38.512 38.512	71.000 89.000	-32.488 -50.488
-10.00 -15.00	38.512 38.512	139.000 184.000	-100.488 -145.488
-15.00 -26.00	38.512 38.512	164.000 246.500	-125.488 -207.988

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -5.00 m $P_{a1} = 38.512 > P_{p1} = 30.000$

at -8.00 m $P_{a2} = 38.512 < P_{p2} = 51.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -5.00 - \frac{3.00 \times (30.000 - 38.512)}{(38.512 - 38.512) - (51.000 - 30.000)} = -6.216m$$

Accordingly, the virtual seabed level is obtained as -6.216m.

3) Moment about tie setting point

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 8.730x 1.950	8.512	-1.850	-15.747
2	1/2x 18.940x 1.950	18.466	-1.200	-22.159
3	1/2x 18.940x 0.550	5.208	-0.367	-1.911
4	1/2x 26.095x 0.550	7.176	-0.183	-1.313
5	1/2x 26.095x 2.570	33.532	0.857	28.737
6	1/2x 59.531x 2.570	76.497	1.713	131.039
7	1/2x 59.531x 2.430	72.330	3.380	244.475
8	1/2x 66.602x 2.430	80.921	4.190	339.059
9	1/2x 122.112x 1.000	61.056	5.333	325.612
10	1/2x 129.112x 1.000	64.556	5.667	365.839
11	1/2x 38.512x 2.000	38.512	6.667	256.760
12	1/2x 38.512x 2.000	38.512	7.333	282.408
13	1/2x 8.512x 1.216	5.175	8.405	43.496
Total		510.453	———	1976.295

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

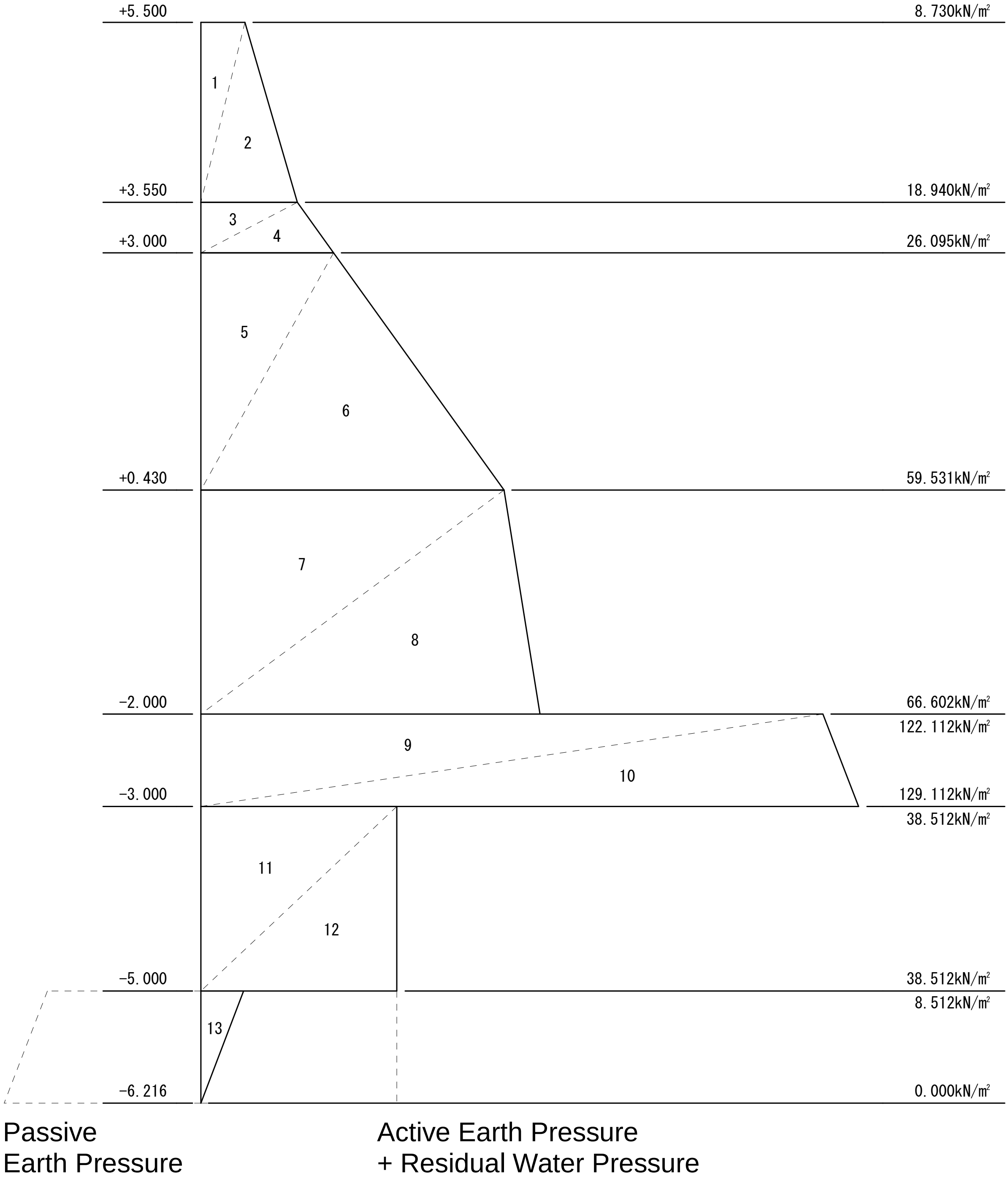
4) Reaction forces

Distance between supports : L_T = 3.000 – (-6.216) = 9.216 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1976.295}{9.216} = 214.442 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 510.453 - 214.442 = 296.011 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 3.55	8.512 18.466	26.978		-26.978
3.55 3.00	5.208 7.176	39.362	296.011	256.649
3.00 0.43	33.532 76.497	149.391	296.011	146.620
0.43 -2.00	72.330 80.921	302.642	296.011	-6.631
-2.00 -3.00	61.056 64.556	428.254	296.011	-132.243
-3.00 -5.00	38.512 38.512	505.278	296.011	-209.267
-5.00 -6.22	5.175 0.000	510.453	296.011	-214.442

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -2.000m.

$$Q = 146.620 - \frac{[59.531 + (59.531 + 2.910X)]X}{2} = 0$$

X = 2.330 m

Shear force zero point : DL = 0.430 – 2.330 = -1.900m

6) Calculation of moment

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	-8.512	6.750	-57.456
2	1/2x	18.940x	1.950	-18.466	6.100	-112.643
3	1/2x	18.940x	0.550	-5.208	5.267	-27.431
4	1/2x	26.095x	0.550	-7.176	5.083	-36.476
5	1/2x	26.095x	2.570	-33.532	4.043	-135.570
6	1/2x	59.531x	2.570	-76.497	3.187	-243.796
7	1/2x	59.531x	2.330	-69.354	1.553	-107.707
8	1/2x	66.311x	2.330	-77.252	0.777	-60.025
Total				―――	―――	-781.104

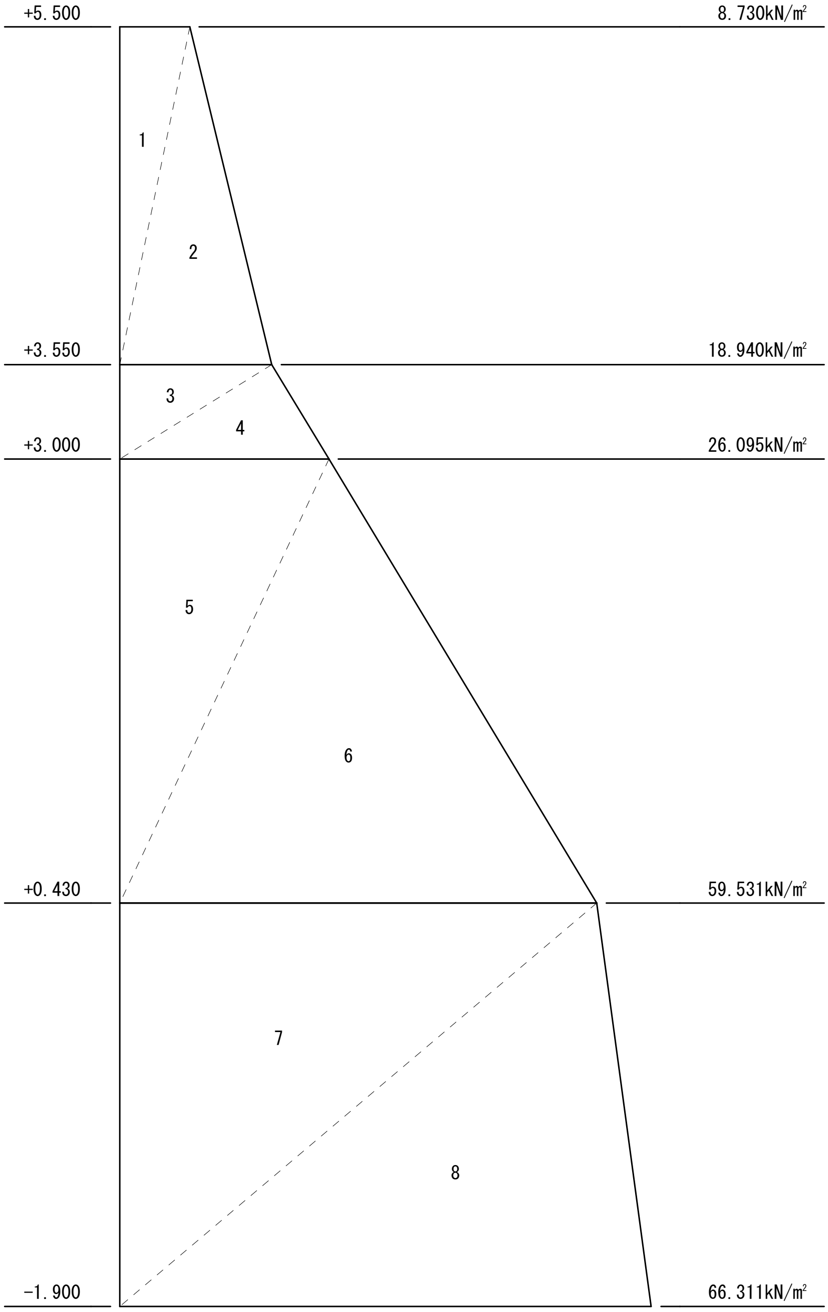
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-1.900) = 4.900 m

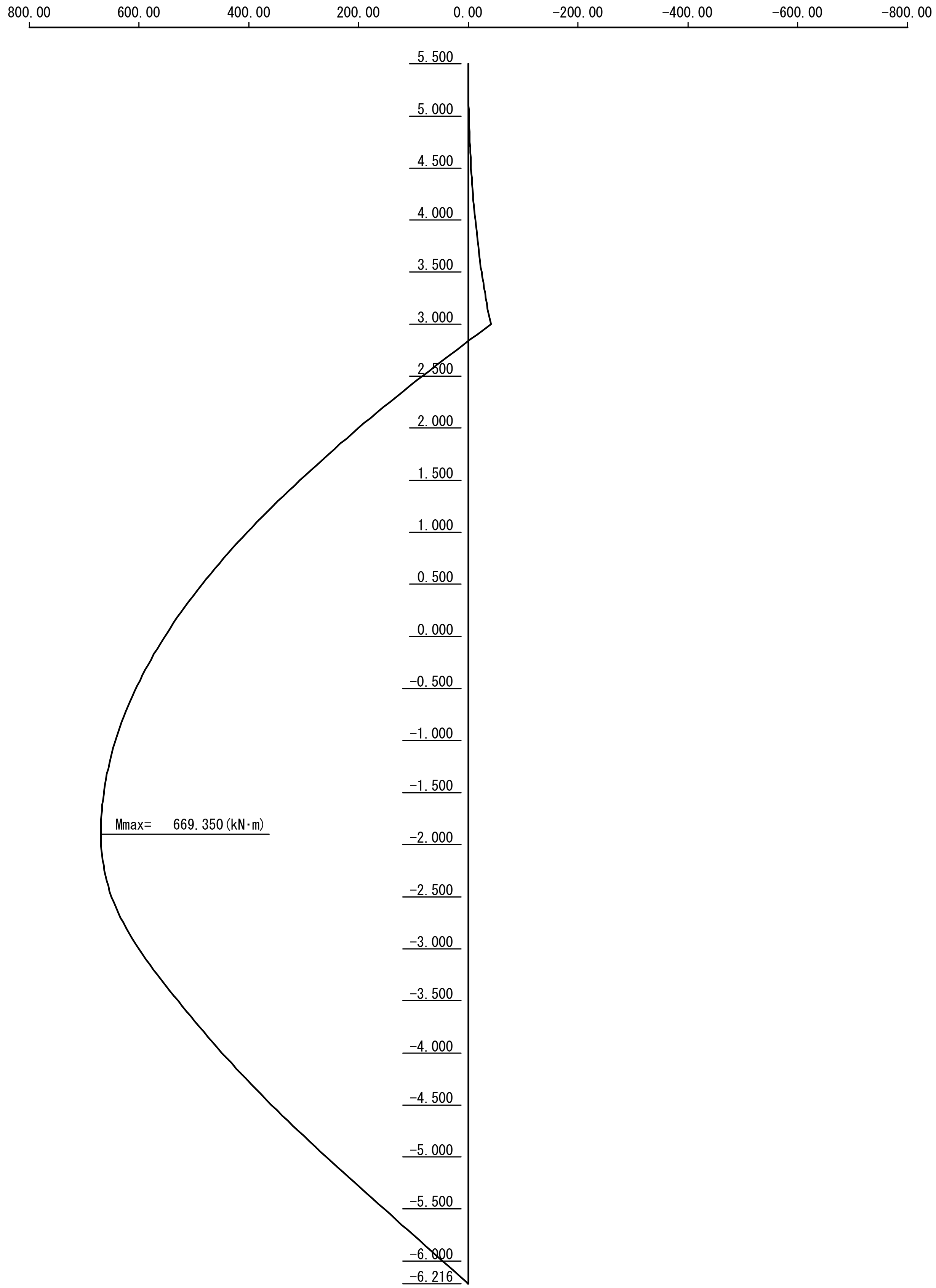
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 296.011 \times 4.900 - 781.104 = 669.350$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t14 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 267050 cm^4
Section modulus : Z = 6676 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 272500 cm^4/m
- Section modulus : Z = 6812 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 241898 cm^4/m
- Section modulus : Z = 6070 cm^3/m

(2) Embedded depth of pipe wall
Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -15.434 m --- adopted
- Free earth support method : -11.231 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-15.434) = 18.934m → 19.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 19.000 = -15.500 m

(3) Stress on pipe wall section
Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 786.584 kN-m/m --- adopted
- Free earth support method : 669.350 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{786.584 \times 10^6}{6070 \times 10^3} = 129.6 MPa \quad \leq \quad \sigma_a = 140 MPa \;\; .... \; OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 319.295 kN-m/m --- adopted
- Free earth support method : 296.011 kN-m/m

T = 319.295 × 1.960 × sec(0.000) × sec (0.000) = 625.818 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 625.818 × 3.8 = 2378.108 kN/wire

Tie wire shall have minimum tensile strength of 2379 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{625.818 \times 1.960}{10.0} = 122.660 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[300×90×10.0×15.5

Steel grade : SS400
Section modulus : Z = 494.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{122.660 \times 10^6}{2 \times 494.0 \times 10^3} = 124.1 MPa \quad \leq \quad \sigma_a = 140 MPa \;\; .... \; OK$$

Output of Design Software for Anchor Wall (After Completion) – Revetment Block a

1. Design Conditions

1-1 Dimensions

Ground elevation	+5.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-1.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	30 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

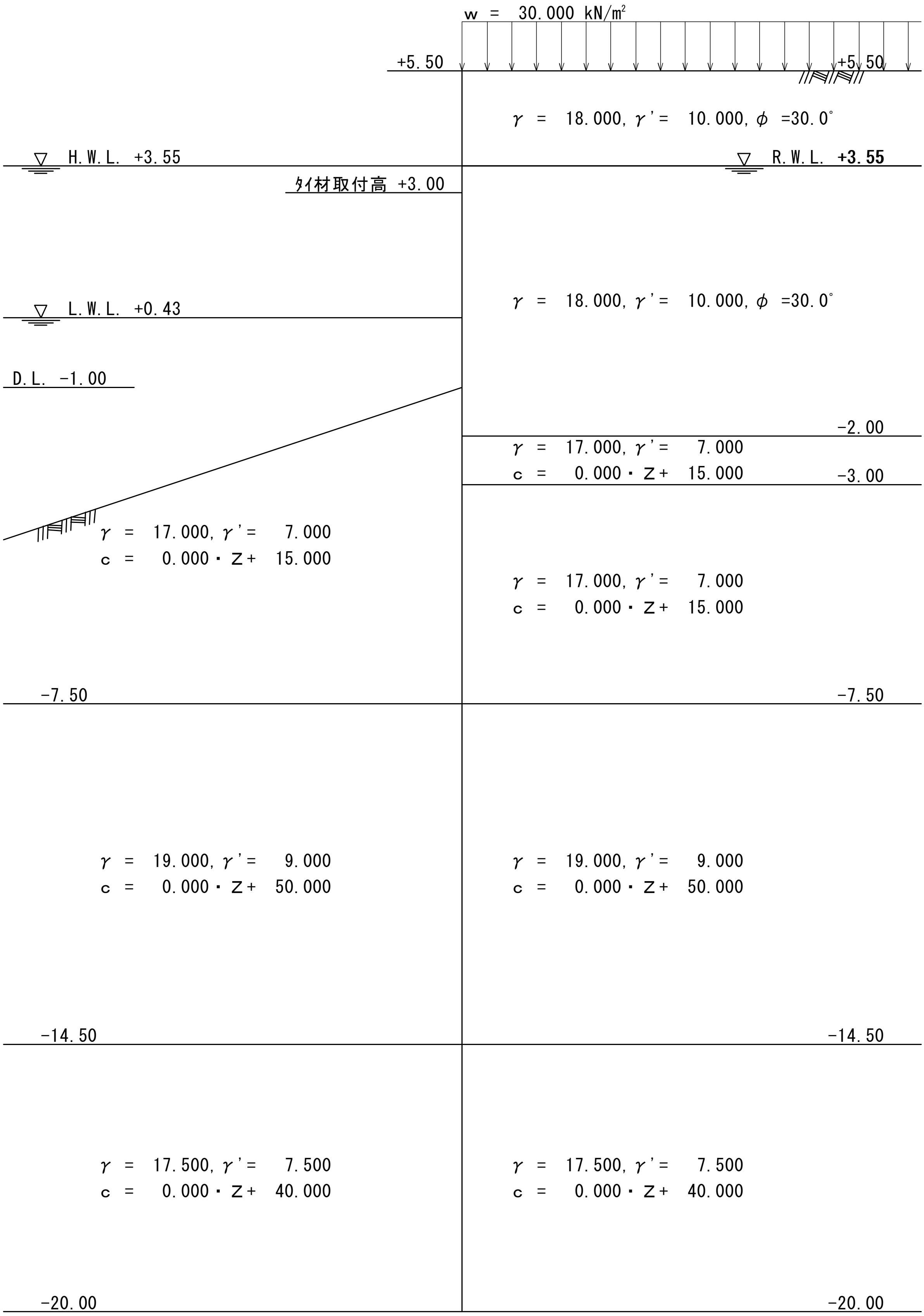
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
5.50 3.55	1.95	8.730 18.940
3.55 3.00	0.55	18.940 20.540
3.00 -2.00	5.00	20.540 35.090
-2.00 -3.00	1.00	90.600 97.600
-3.00 -7.50	4.50	7.000 7.000
-7.50 -11.00	3.50	7.000 7.000
-11.00 -14.50	3.50	7.000 7.000
-14.50 -25.00	10.50	7.000 7.000

(2) Residual water pressure

$p_w = \gamma_w \times h_w$

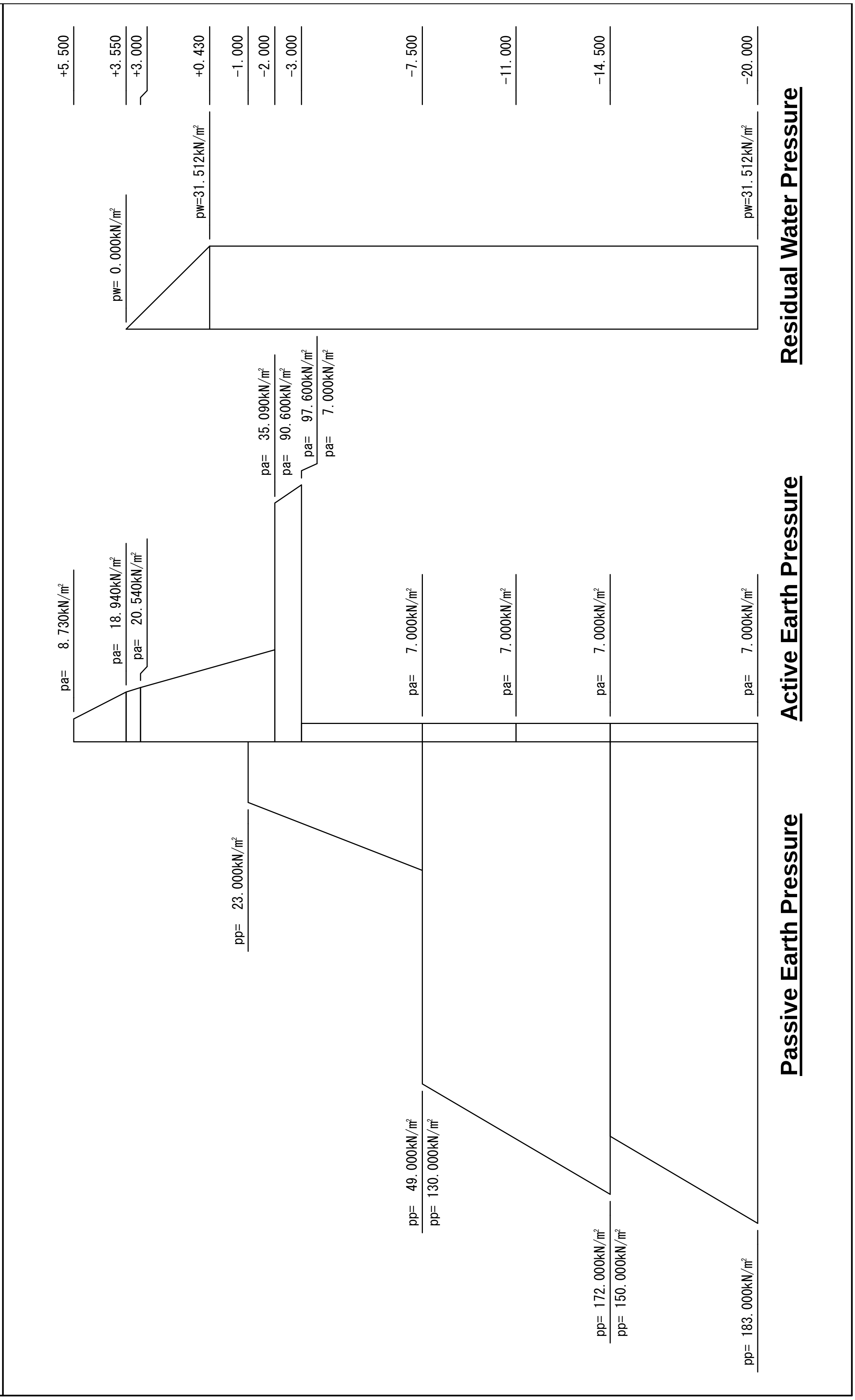
where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-1.00 -7.50	6.50	23.000 49.000
-7.50 -14.50	7.00	130.000 172.000
-14.50 -20.00	5.50	150.000 183.000

Earth Pressure and Water Pressure



(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 3.55	8.730+ 0.000= 8.730 18.940+ 0.000= 18.940	——— ———
3.55 3.00	18.940+ 0.000= 18.940 20.540+ 5.555= 26.095	——— ———
3.00 0.43	20.540+ 5.555= 26.095 28.019+ 31.512= 59.531	——— ———
0.43 -1.00	28.019+ 31.512= 59.531 32.180+ 31.512= 63.692	——— ———
-1.00 -2.00	32.180+ 31.512= 63.692 35.090+ 31.512= 66.602	23.000 27.000
-2.00 -3.00	90.600+ 31.512= 122.112 97.600+ 31.512= 129.112	27.000 31.000
-3.00 -7.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	31.000 49.000
-7.50 -11.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	130.000 151.000
-11.00 -14.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	151.000 172.000
-14.50 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	150.000 183.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -1.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-1.000	-111.249	135.301	102.195
-2.000	-238.845	159.773	117.870
-3.000	-459.920	187.428	186.827
-4.000	-747.349	214.559	165.208
-5.000	-1017.829	235.345	145.934
-6.000	-1269.787	251.459	127.332
-7.000	-1496.898	263.902	108.400
-8.000	-1653.670	272.385	48.680
-9.000	-1509.613	272.422	-48.845
-10.000	-1000.244	264.723	-144.634
-11.000	-88.979	250.516	-239.915
-12.000	1266.775	230.706	-335.593

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -11.00 and -12.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-11.078	0.000	249.166	-247.337

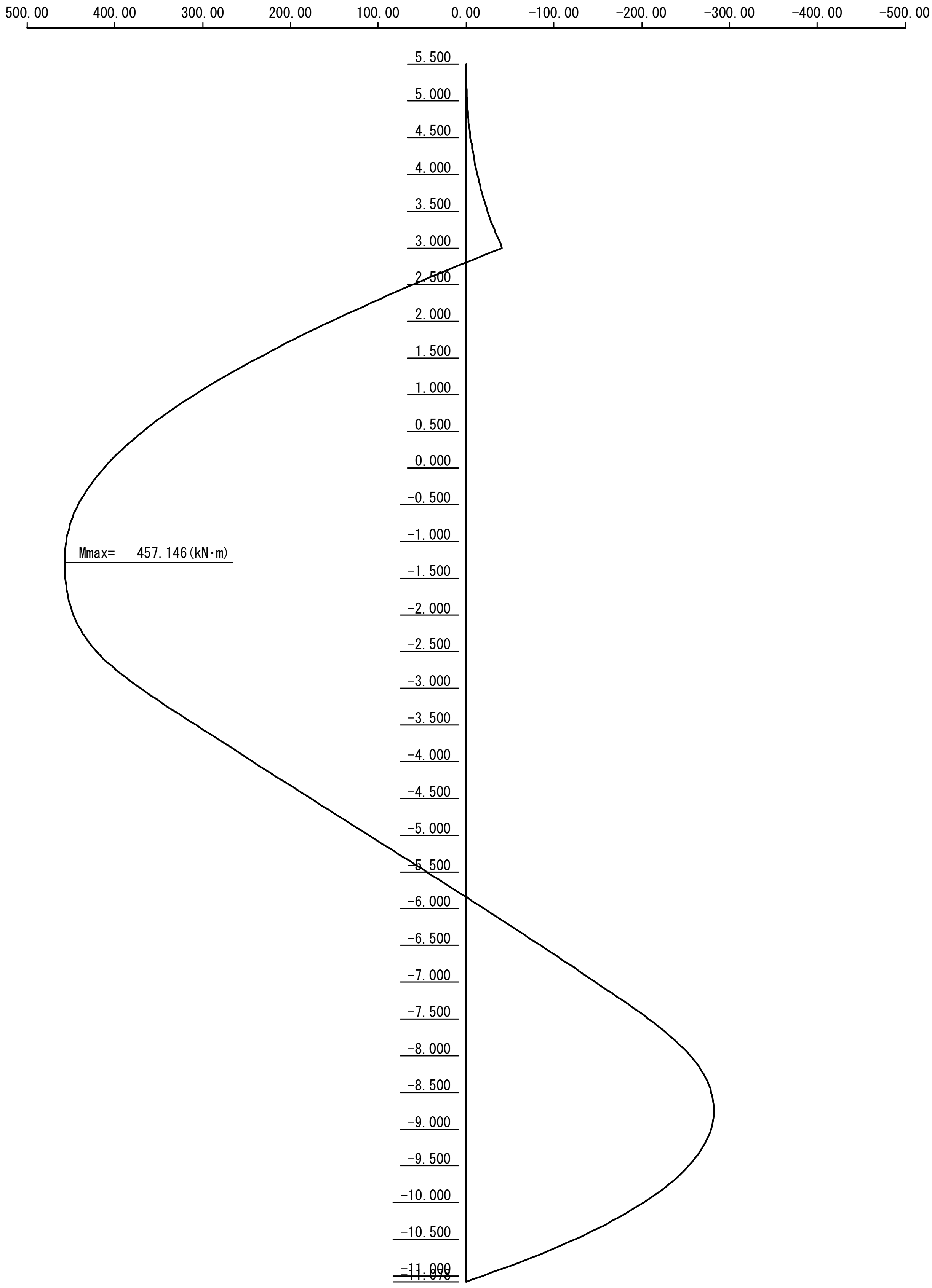
Embedded length of the pipe wall : L = 1.2 × (-1.000 + 11.078) = 12.094 m
Toe elevation : D = -1.000 – 12.094 = -13.094 m
Reaction force at tie setting point : 249.166 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	-1.200	-5.019
4.500	-5.238	-11.348
4.000	-12.766	-18.985
3.550	-23.069	-26.978
3.050	-39.196	-38.074
3.000	-41.132	-39.363
2.500	60.237	195.129
2.000	153.455	177.203
1.500	236.898	156.024
1.000	308.937	131.593
0.500	367.948	103.909
0.430	375.078	99.774
-0.070	417.463	69.644
-0.570	444.601	38.788
-1.000	455.469	11.669
-1.500	456.239	-8.540
-2.000	446.973	-28.478
-2.500	420.783	-76.409
-3.000	370.440	-125.090
-3.500	307.039	-128.346
-4.000	242.261	-130.602
-4.500	176.604	-131.858
-5.000	110.570	-132.114
-5.500	44.657	-131.370
-6.000	-20.633	-129.626
-6.500	-84.802	-126.882
-7.000	-147.348	-123.138
-7.500	-207.773	-118.394
-8.000	-255.409	-71.900
-8.500	-279.423	-23.906
-9.000	-279.065	25.588
-9.500	-253.584	76.582
-10.000	-202.232	129.076
-10.500	-124.258	183.070
-11.000	-18.912	238.564
-11.078	0.000	247.337

Maximum bending moment : 457.146 kN-m/m
Elevation of maximum bending moment : -1.288 m
Depth of 1st steady point (moment=0) : -5.841 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
1	5.50	1/2x	8.730x	1.950	8.512	-1.850	-15.747	
2	3.55	1/2x	18.940x	1.950	18.466	-1.200	-22.159	-45.487
3	3.55	1/2x	18.940x	0.550	5.208	-0.367	-1.911	
4	3.00	1/2x	26.095x	0.550	7.176	-0.183	-1.313	-49.356
5	3.00	1/2x	26.095x	2.570	33.532	0.857	28.737	
6	0.43	1/2x	59.531x	2.570	76.497	1.713	131.039	142.375
7	0.43	1/2x	59.531x	1.430	42.565	3.047	129.696	
8	-1.00	1/2x	63.692x	1.430	45.540	3.523	160.437	490.535
9	-1.00	1/2x	63.692x	1.000	31.846	4.333	137.989	
10	-2.00	1/2x	66.602x	1.000	33.301	4.667	155.416	842.621
11	-2.00	1/2x	122.112x	1.000	61.056	5.333	325.612	
12	-3.00	1/2x	129.112x	1.000	64.556	5.667	365.839	1672.362
13	-3.00	1/2x	38.512x	4.500	86.652	7.500	649.890	
14	-7.50	1/2x	38.512x	4.500	86.652	9.000	779.868	3388.072
15	-7.50	1/2x	38.512x	3.500	67.396	11.667	786.309	
16	-11.00	1/2x	38.512x	3.500	67.396	12.833	864.893	5369.514
17	-11.00	1/2x	38.512x	3.500	67.396	15.167	1022.195	
18	-14.50	1/2x	38.512x	3.500	67.396	16.333	1100.779	7917.083
19	-14.50	1/2x	38.512x	5.500	105.908	19.333	2047.519	
20	-20.00	1/2x	38.512x	5.500	105.908	21.167	2241.755	13064.212

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

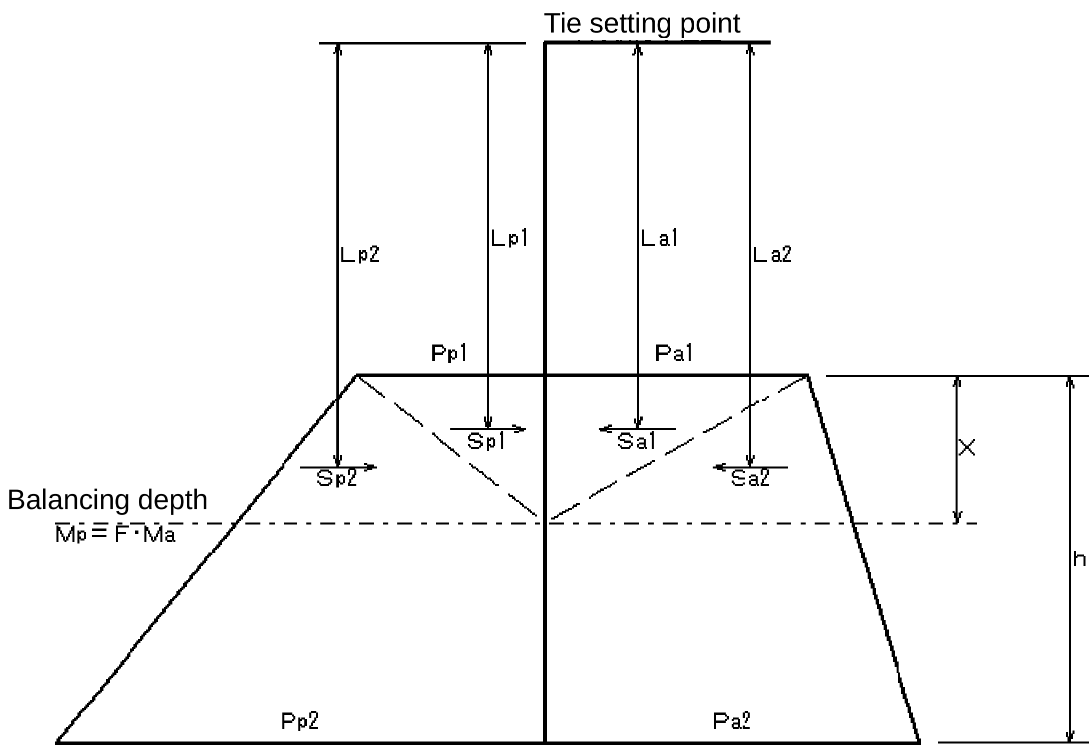
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-1.00	1/2x 23.000x 1.000	11.500	4.333	49.830	
2	-2.00	1/2x 27.000x 1.000	13.500	4.667	63.004	112.834
3	-2.00	1/2x 27.000x 1.000	13.500	5.333	71.996	
4	-3.00	1/2x 31.000x 1.000	15.500	5.667	87.838	272.668
5	-3.00	1/2x 31.000x 4.500	69.750	7.500	523.125	
6	-7.50	1/2x 49.000x 4.500	110.250	9.000	992.250	1788.043
7	-7.50	1/2x 130.000x 3.500	227.500	11.667	2654.242	
8	-11.00	1/2x 151.000x 3.500	264.250	12.833	3391.120	7833.405
9	-11.00	1/2x 151.000x 3.500	264.250	15.167	4007.880	
10	-14.50	1/2x 172.000x 3.500	301.000	16.333	4916.233	16757.518
11	-14.50	1/2x 150.000x 5.500	412.500	19.333	7974.862	
12	-20.00	1/2x 183.000x 5.500	503.250	21.167	10652.293	35384.673

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1} X}{2}, \quad L_{a(p)1} = L + \frac{1}{3} X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X\right] X}{2}, \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -7.50 m FOS×M_{a1} = 3388.072 > M_{p1} = 1788.043
at -11.00 m FOS×M_{a2} = 5369.514 < M_{p2} = 7833.405

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \text{ , } L_{a1} = 10.50 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 202.188X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{3.500}X\right]X}{2} = 0.000X^2 + 19.256X \text{ , } L_{a2} = 10.500 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 202.188X$$

$$M_a = 0.000X^3 + 19.256X^2 + 404.376X + 2823.393$$

[Moment on passive side]

$$S_{p1} = \frac{130.000X}{2} = 65.000X \text{ , } L_{p1} = 10.500 + \frac{1}{3}X$$

$$M_{p1'} = 21.667X^2 + 682.500X$$

$$S_{p2} = \frac{\left[130.000 + \frac{151.000 - 130.000}{3.500}X\right]X}{2} = 3.000X^2 + 65.000X \text{ , } L_{p2} = 10.500 + \frac{2}{3}X$$

$$M_{p2'} = 2.000X^3 + 74.833X^2 + 682.500X$$

$$M_p = 2.000X^3 + 96.500X^2 + 1365.000X + 1788.043$$

[Balancing depth]

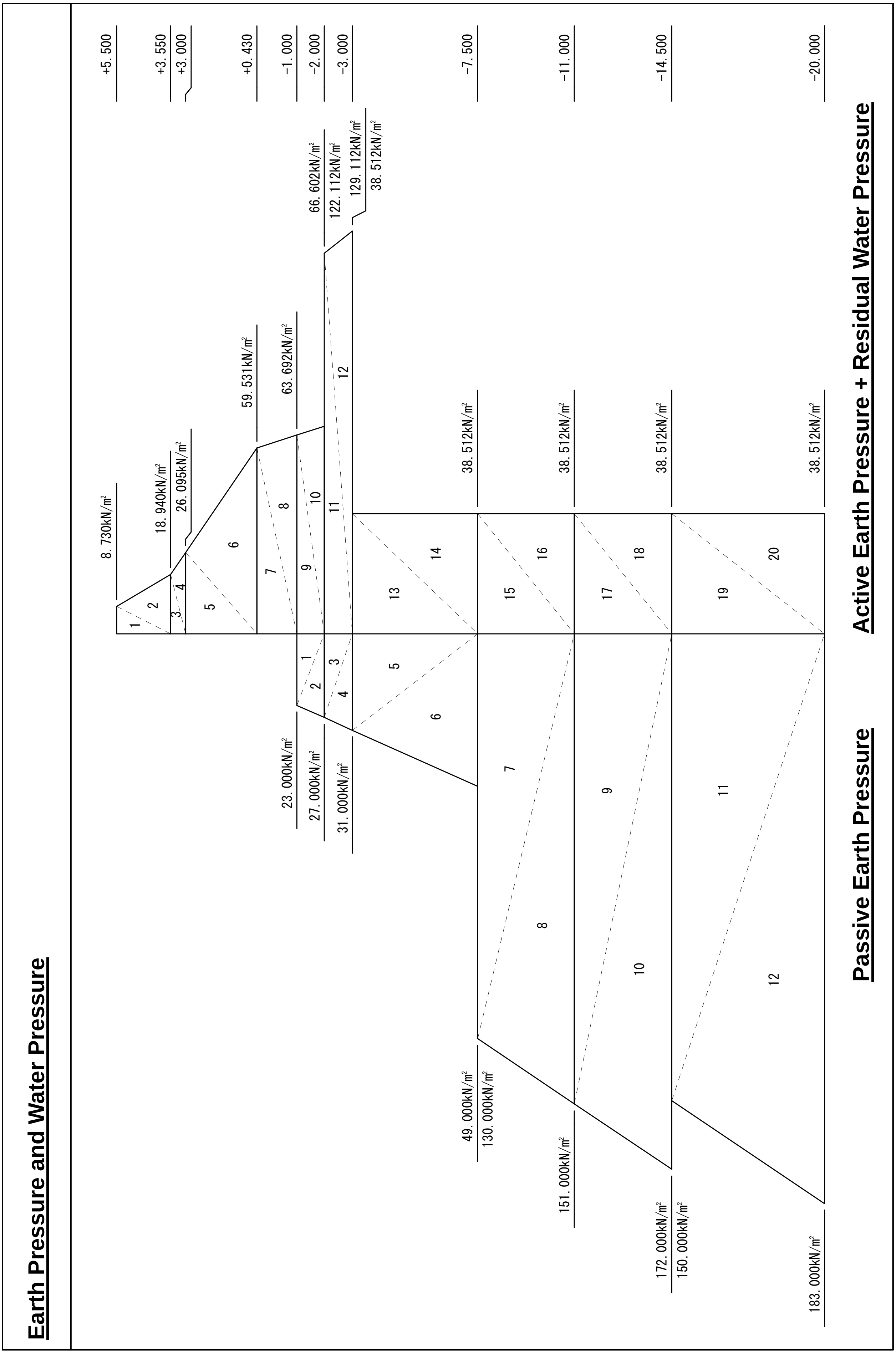
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 404.376X + 2823.393)$$

$$= 2.000X^3 + 96.500X^2 + 1365.000X + 1788.043$$

X= 1.597 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -7.500 – 1.597 = -9.097 m


Earth Pressure and Water Pressure
Passive Earth Pressure
Active Earth Pressure + Residual Water Pressure
+5.500
+3.550
+3.000
+0.430
-1.000
-2.000
-3.000
-7.500
-11.000
-14.500
-20.000
8.730kN/m²
18.940kN/m²
26.095kN/m²
59.531kN/m²
63.692kN/m²
66.602kN/m²
122.112kN/m²
129.112kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
23.000kN/m²
27.000kN/m²
31.000kN/m²
49.000kN/m²
130.000kN/m²
151.000kN/m²
172.000kN/m²
150.000kN/m²
183.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 3.55	8.730 18.940	——— ———	8.730 18.940
3.55 3.00	18.940 26.095	——— ———	18.940 26.095
3.00 0.43	26.095 59.531	——— ———	26.095 59.531
0.43 -1.00	59.531 63.692	——— ———	59.531 63.692
-1.00 -2.00	63.692 66.602	23.000 27.000	40.692 39.602
-2.00 -3.00	122.112 129.112	27.000 31.000	95.112 98.112
-3.00 -7.50	38.512 38.512	31.000 49.000	7.512 -10.488
-7.50 -11.00	38.512 38.512	130.000 151.000	-91.488 -112.488
-11.00 -14.50	38.512 38.512	151.000 172.000	-112.488 -133.488
-14.50 -20.00	38.512 38.512	150.000 183.000	-111.488 -144.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $\quad P_{a1} = 38.512 \quad > \quad P_{p1} = 31.000$
at -7.50 m $\quad P_{a2} = 38.512 \quad < \quad P_{p2} = 49.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

- EL_{top} : the top elevation of the soil layer in question (m)
- P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)
- P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)
- P_{p1} : the passive earth pressure at the top of the soil layer (kPa)
- P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)
- h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{4.50 \times (31.000 - 38.512)}{(38.512 - 38.512) - (49.000 - 31.000)} = -4.878m$$

Accordingly, the virtual seabed level is obtained as -4.878m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	8.512	-1.850	-15.747
2	1/2x	18.940x	1.950	18.466	-1.200	-22.159
3	1/2x	18.940x	0.550	5.208	-0.367	-1.911
4	1/2x	26.095x	0.550	7.176	-0.183	-1.313
5	1/2x	26.095x	2.570	33.532	0.857	28.737
6	1/2x	59.531x	2.570	76.497	1.713	131.039
7	1/2x	59.531x	1.430	42.565	3.047	129.696
8	1/2x	63.692x	1.430	45.540	3.523	160.437
9	1/2x	40.692x	1.000	20.346	4.333	88.159
10	1/2x	39.602x	1.000	19.801	4.667	92.411
11	1/2x	95.112x	1.000	47.556	5.333	253.616
12	1/2x	98.112x	1.000	49.056	5.667	278.000
13	1/2x	7.512x	1.878	7.054	6.626	46.740
Total				381.309	——	1167.705

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

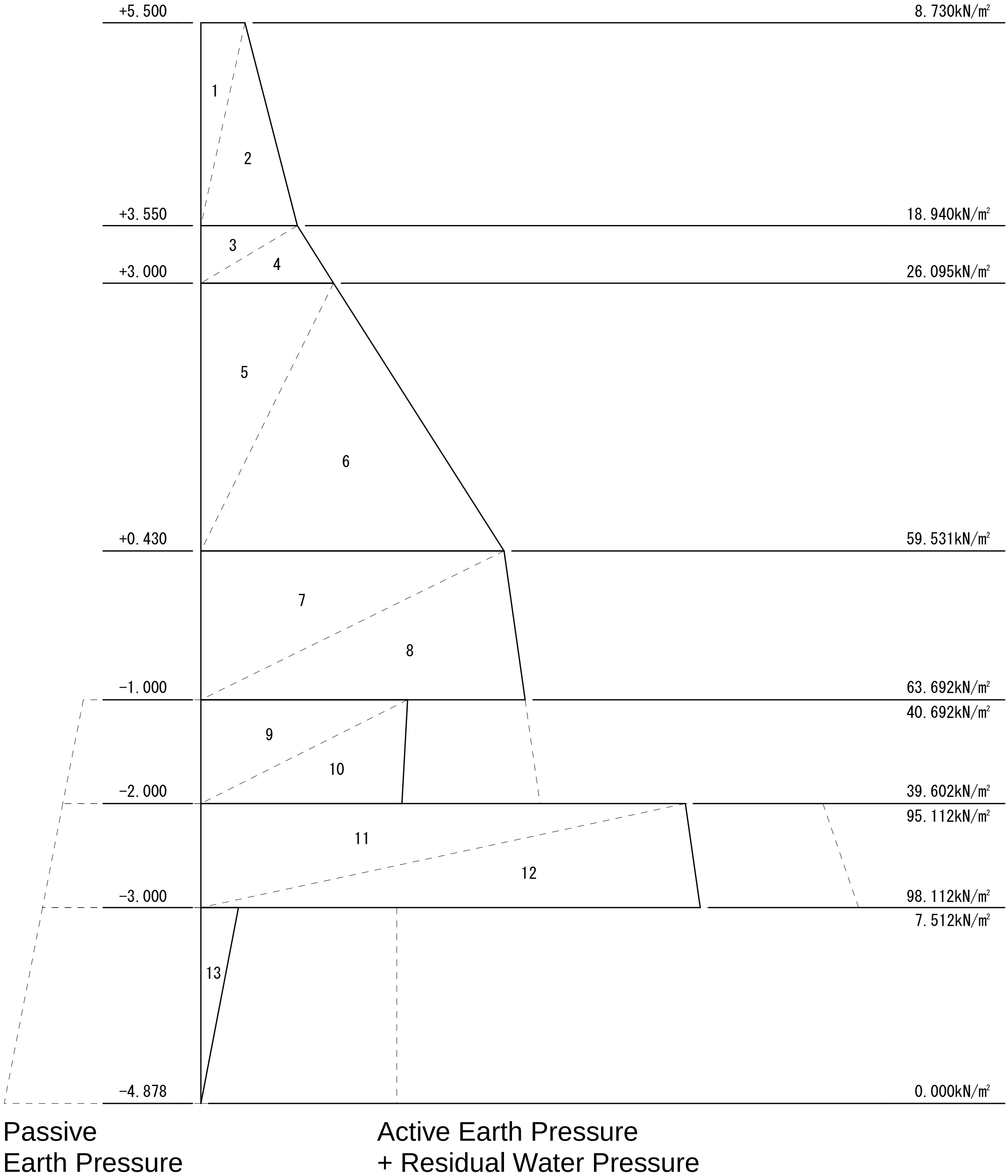
4) Reaction forces

Distance between supports : $L_T = 3.000 - (-4.878) = 7.878$ m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1167.705}{7.878} = 148.224 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 381.309 - 148.224 = 233.085 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 3.55	8.512 18.466	26.978		-26.978
3.55 3.00	5.208 7.176	39.362	233.085	193.723
3.00 0.43	33.532 76.497	149.391	233.085	83.694
0.43 -1.00	42.565 45.540	237.496	233.085	-4.411
-1.00 -2.00	20.346 19.801	277.643	233.085	-44.558
-2.00 -3.00	47.556 49.056	374.255	233.085	-141.170
-3.00 -4.88	7.054 0.000	381.309	233.085	-148.224

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -1.000m.

$$Q = 83.694 - \frac{[59.531 + (59.531 + 2.910X)]X}{2} = 0$$

X = 1.361 m

Shear force zero point : DL = +0.43 – 1.361 = -0.931m

6) Calculation of moment

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	-8.512	5.781	-49.208
2	1/2x	18.940x	1.950	-18.466	5.131	-94.749
3	1/2x	18.940x	0.550	-5.208	4.298	-22.384
4	1/2x	26.095x	0.550	-7.176	4.114	-29.522
5	1/2x	26.095x	2.570	-33.532	3.074	-103.077
6	1/2x	59.531x	2.570	-76.497	2.218	-169.670
7	1/2x	59.531x	1.361	-40.511	0.907	-36.743
8	1/2x	63.491x	1.361	-43.206	0.454	-19.616
Total				———	———	-524.969

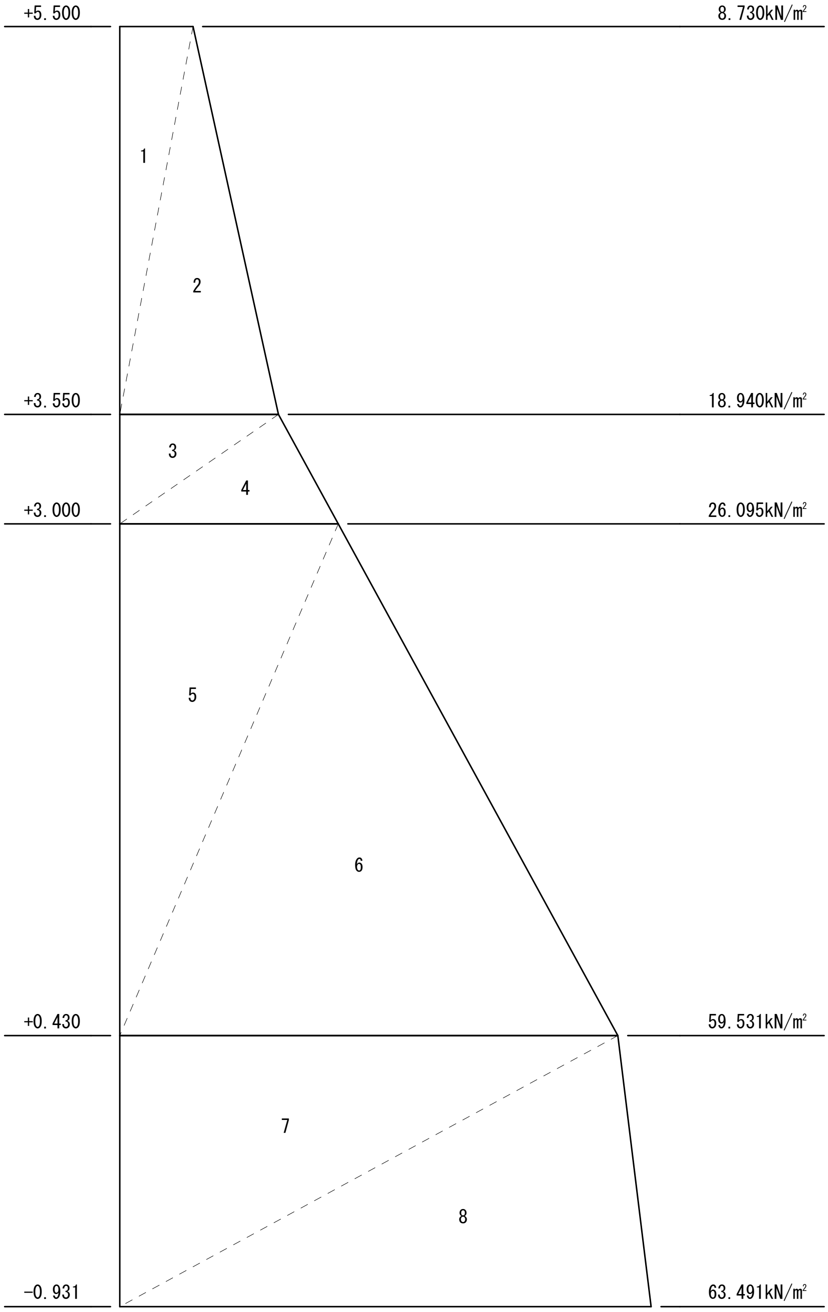
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.931) = 3.931 m

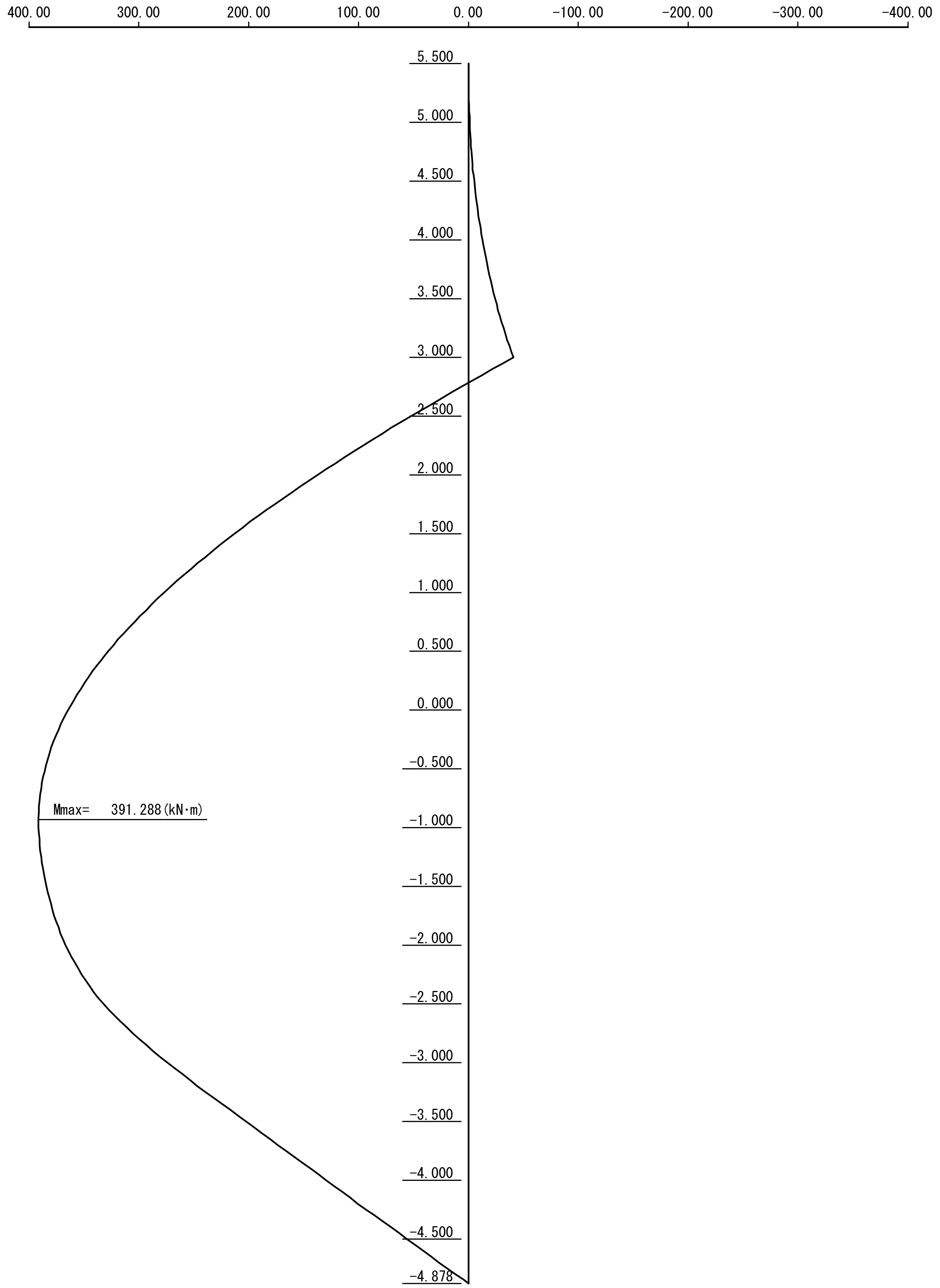
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 233.085 \times 3.931 - 524.969 = 391.288$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -13.094 m --- adopted
- Free earth support method : -9.097 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-13.094) = 16.594m → 17.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 17.000 = -13.500 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 457.146 kN-m/m --- adopted
- Free earth support method : 391.288 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{457.146 \times 10^6}{4191 \times 10^3} = 109.1 MPa \quad \leq \quad \sigma_a = 140 MPa \quad .... OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

- T is the tension force on tie wire (kN/wire)
- A_p is the reaction force of the pipe wall at tie setting point (kN/m)
- L is the spacing of the tie wire (m)
- θ_1 is the vertical angle of the tie wire to horizontal (deg.)
- θ_2 is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 249.166 kN-m/m --- adopted
- Free earth support method : 233.085 kN-m/m

T = 249.166 × 1.960 × sec(0.000) × sec (0.000) = 488.365 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 488.365 × 3.8 = 1855.787 kN/wire

Tie wire shall have minimum tensile strength of 1856 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max}	is the maximum bending moment on the waling beam (kN-m)
T	is the tension force on tie wire (kN/wire)
L	is the spacing of the tie wire (m)

$$M_{maz} = \frac{488.365 \times 1.960}{10.0} = 95.720 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{95.720 \times 10^6}{2 \times 374.0 \times 10^3} = 128.0 MPa \quad \leq \quad \sigma_a = 140 MPa \quad .... \ OK$$

Output of Design Software for Anchor Wall (After Completion)
– Revetment Block b

1. Design Conditions

1-1 Dimensions

Ground elevation	+5.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-1.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment　　+0.00 m (not used)

1-5 Surcharge Load	(Active side)	30 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water　　10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

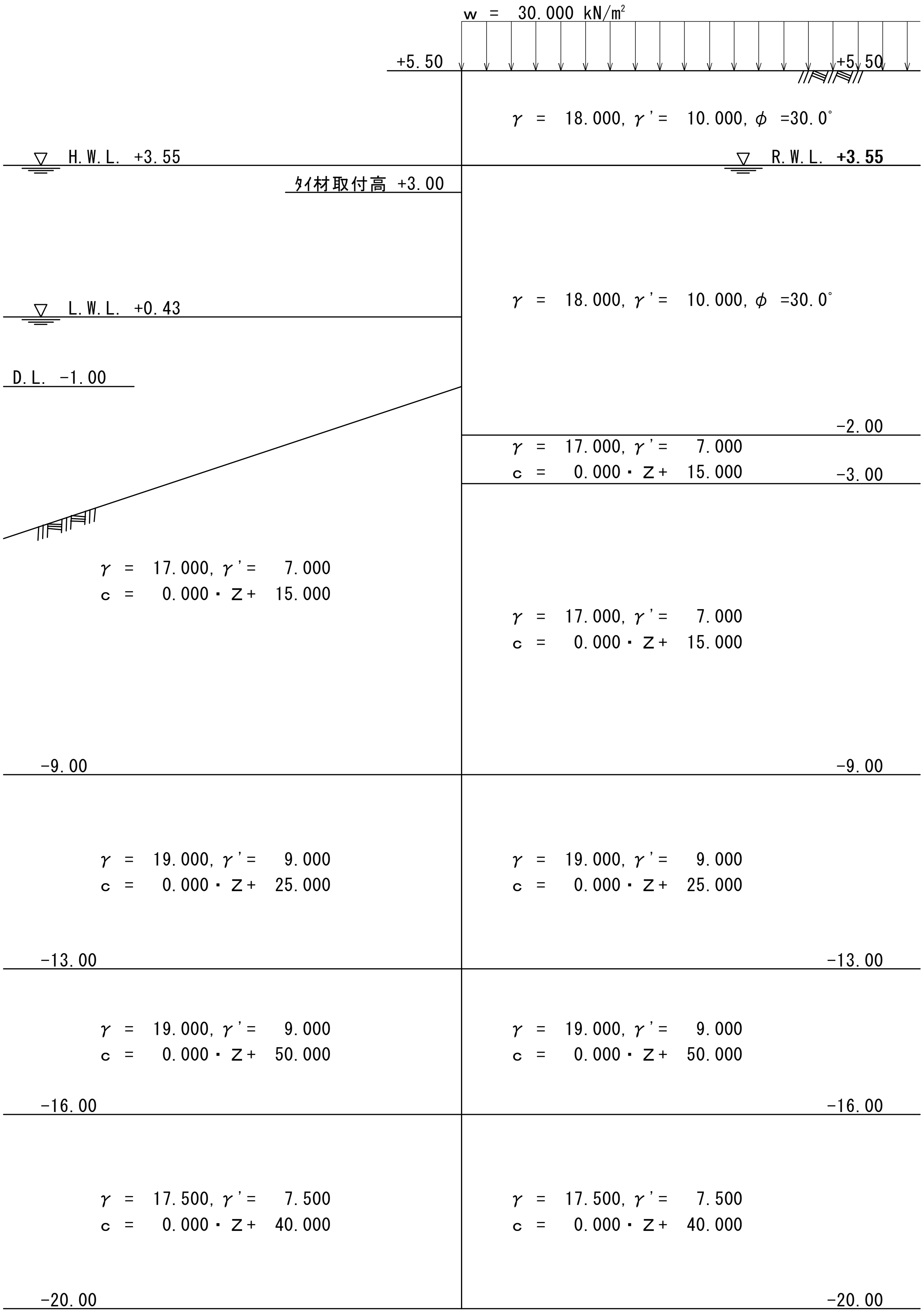
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
5.50 3.55	1.95	8.730 18.940
3.55 3.00	0.55	18.940 20.540
3.00 -2.00	5.00	20.540 35.090
-2.00 -3.00	1.00	90.600 97.600
-3.00 -9.00	6.00	7.000 7.000
-9.00 -11.00	2.00	7.000 7.000
-11.00 -13.00	2.00	7.000 7.000
-13.00 -16.00	3.00	7.000 7.000
-16.00 -20.00	4.00	7.000 7.000

(2) Residual water pressure

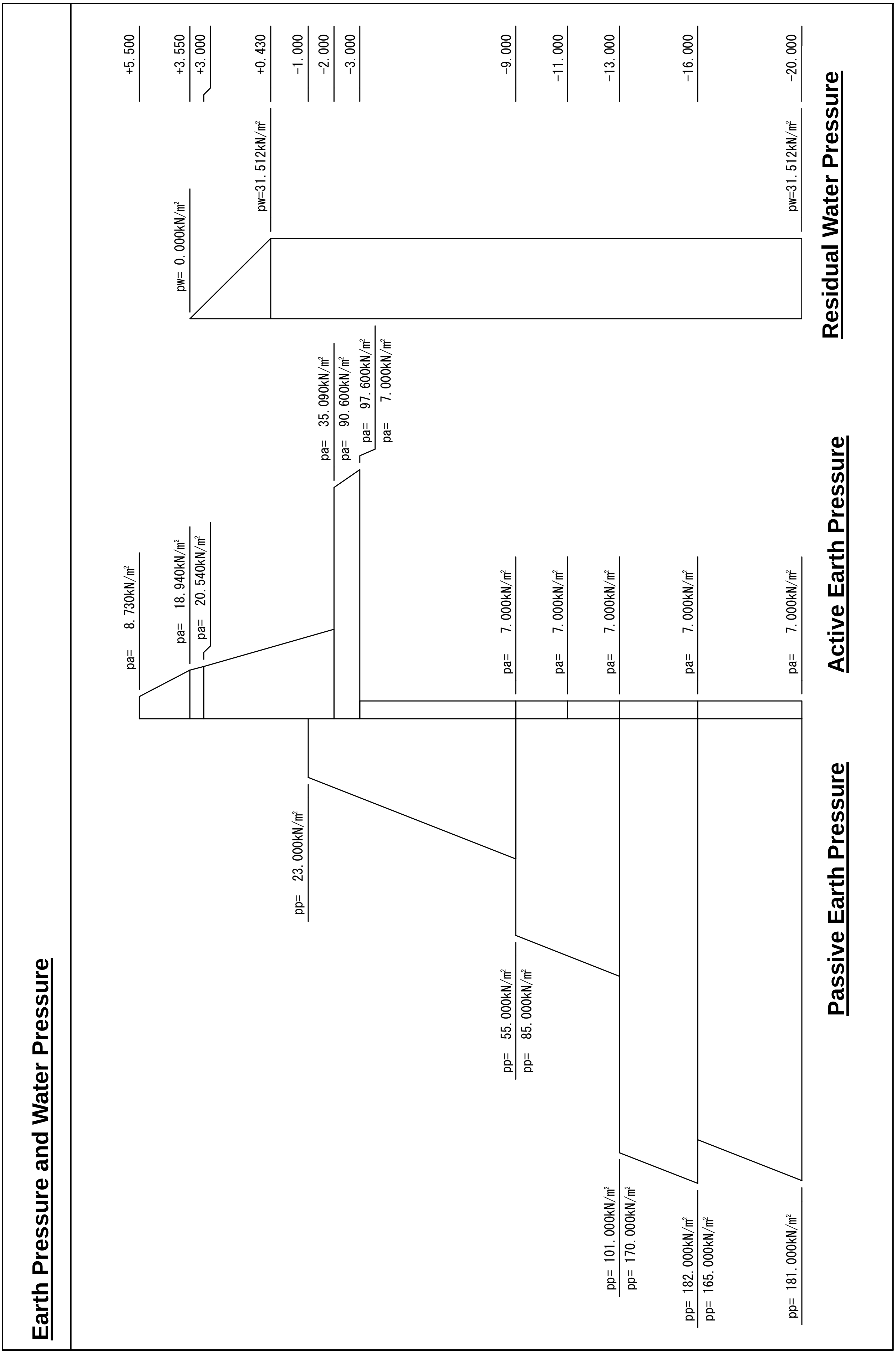
$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-1.00 -9.00	8.00	23.000 55.000
-9.00 -13.00	4.00	85.000 101.000
-13.00 -16.00	3.00	170.000 182.000
-16.00 -20.00	4.00	165.000 181.000


Earth Pressure and Water Pressure
+5.500
+3.550
+3.000
+0.430
-1.000
-2.000
-3.000
-9.000
-11.000
-13.000
-16.000
-20.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
Residual Water Pressure
pa= 8.730kN/m²
pa= 18.940kN/m²
pa= 20.540kN/m²
pa= 35.090kN/m²
pa= 90.600kN/m²
pa= 97.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
Active Earth Pressure
pp= 23.000kN/m²
pp= 55.000kN/m²
pp= 85.000kN/m²
pp= 101.000kN/m²
pp= 170.000kN/m²
pp= 182.000kN/m²
pp= 165.000kN/m²
pp= 181.000kN/m²
Passive Earth Pressure

(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 3.55	8.730+ 0.000= 8.730 18.940+ 0.000= 18.940	―――― ――――
3.55 3.00	18.940+ 0.000= 18.940 20.540+ 5.555= 26.095	―――― ――――
3.00 0.43	20.540+ 5.555= 26.095 28.019+ 31.512= 59.531	―――― ――――
0.43 -1.00	28.019+ 31.512= 59.531 32.180+ 31.512= 63.692	―――― ――――
-1.00 -2.00	32.180+ 31.512= 63.692 35.090+ 31.512= 66.602	23.000 27.000
-2.00 -3.00	90.600+ 31.512= 122.112 97.600+ 31.512= 129.112	27.000 31.000
-3.00 -9.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	31.000 55.000
-9.00 -11.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	85.000 93.000
-11.00 -13.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	93.000 101.000
-13.00 -16.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	170.000 182.000
-16.00 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	165.000 181.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -1.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-1.000	-111.249	135.301	102.195
-2.000	-238.845	159.773	117.870
-3.000	-459.920	187.427	186.828
-4.000	-747.349	214.558	165.209
-5.000	-1017.829	235.345	145.934
-6.000	-1269.787	251.459	127.332
-7.000	-1496.898	263.902	108.401
-8.000	-1689.275	273.308	88.507
-9.000	-1834.063	280.109	67.218
-10.000	-1855.666	283.441	15.398
-11.000	-1674.978	282.689	-36.338
-12.000	-1268.930	278.406	-88.543
-13.000	-610.981	271.003	-141.628
-14.000	512.658	258.765	-262.878

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -13.00 and -14.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-13.614	0.000	264.266	-216.393

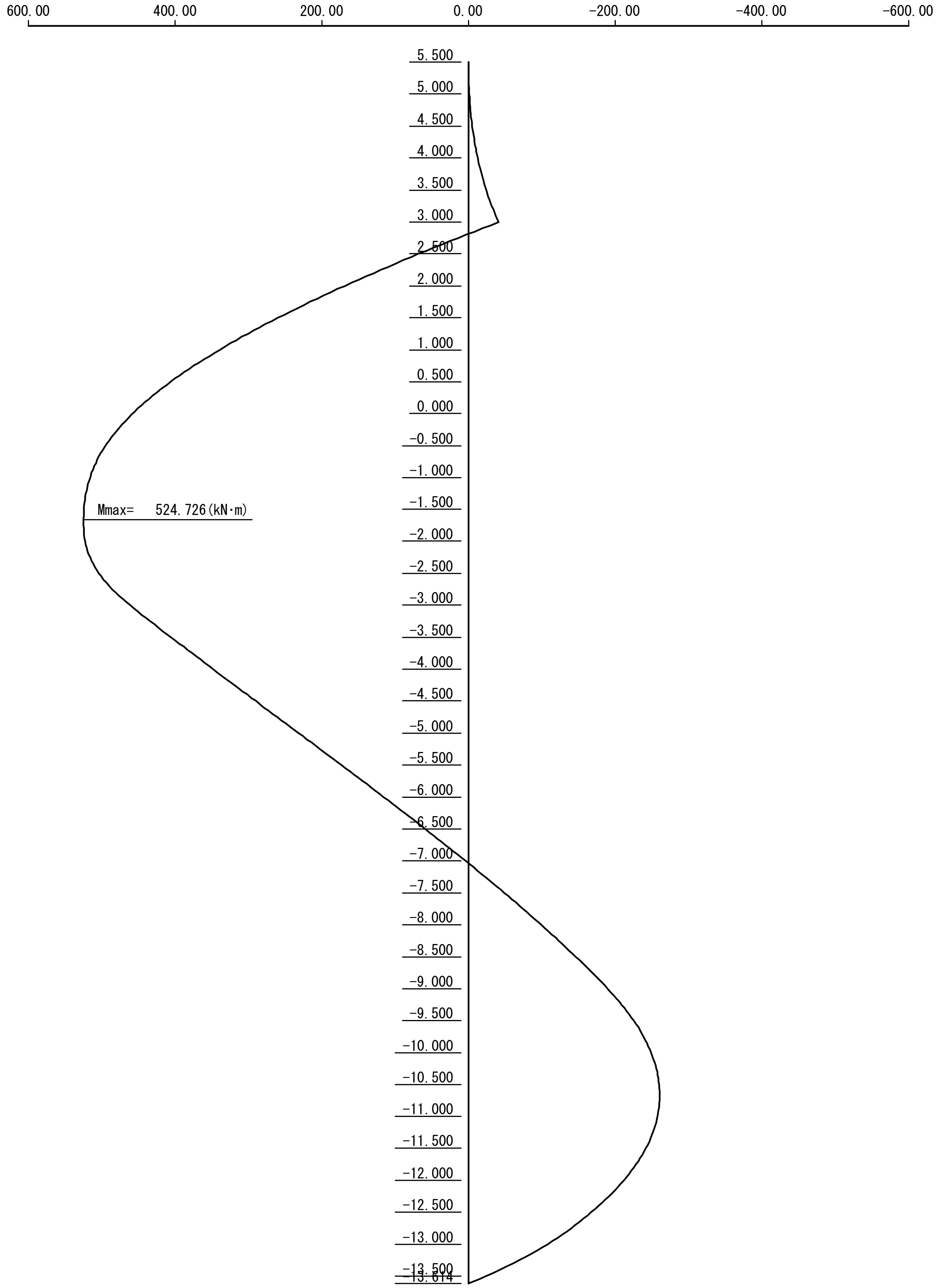
Embedded length of the pipe wall : $L = 1.2 \times (-1.000 + 13.614) = 15.137$ m
Toe elevation : $D = -1.000 – 15.137 = -16.137$ m
Reaction force at tie setting point : 264.266 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	-1.200	-5.019
4.500	-5.238	-11.348
4.000	-12.766	-18.985
3.550	-23.069	-26.978
3.050	-39.196	-38.074
3.000	-41.132	-39.363
2.500	67.787	210.229
2.000	168.555	192.303
1.500	259.548	171.124
1.000	339.137	146.693
0.500	405.698	119.009
0.430	413.885	114.874
-0.070	463.820	84.744
-0.570	498.508	53.888
-1.000	515.869	26.769
-1.500	524.189	6.560
-1.600	524.645	2.550
-2.000	522.473	-13.378
-2.500	503.833	-61.309
-2.600	497.219	-70.985
-3.000	461.040	-109.990
-3.500	405.189	-113.246
-4.000	347.961	-115.502
-4.500	289.854	-116.758
-5.000	231.370	-117.014
-5.500	173.007	-116.270
-6.000	115.267	-114.526
-6.500	58.648	-111.782
-7.000	3.652	-108.038
-7.500	-49.223	-103.294
-8.000	-99.476	-97.550
-8.500	-146.606	-90.806
-9.000	-190.115	-83.062
-9.500	-225.751	-59.318
-10.000	-249.266	-34.574
-10.500	-260.158	-8.830
-11.000	-257.929	17.914
-11.500	-242.077	45.658
-12.000	-212.104	74.402
-12.500	-167.508	104.146
-13.000	-107.791	134.890
-13.500	-23.826	201.134
-13.614	0.000	216.393

Maximum bending moment : 524.726 kN-m/m
Elevation of maximum bending moment : -1.664 m
Depth of 1st steady point (moment=0) : -7.034 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

M_p = FOS×M_a

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
1	5.50	1/2x	8.730x	1.950	8.512	-1.850	-15.747	
2	3.55	1/2x	18.940x	1.950	18.466	-1.200	-22.159	-45.487
3	3.55	1/2x	18.940x	0.550	5.208	-0.367	-1.911	
4	3.00	1/2x	26.095x	0.550	7.176	-0.183	-1.313	-49.356
5	3.00	1/2x	26.095x	2.570	33.532	0.857	28.737	
6	0.43	1/2x	59.531x	2.570	76.497	1.713	131.039	142.375
7	0.43	1/2x	59.531x	1.430	42.565	3.047	129.696	
8	-1.00	1/2x	63.692x	1.430	45.540	3.523	160.437	490.535
9	-1.00	1/2x	63.692x	1.000	31.846	4.333	137.989	
10	-2.00	1/2x	66.602x	1.000	33.301	4.667	155.416	842.621
11	-2.00	1/2x	122.112x	1.000	61.056	5.333	325.612	
12	-3.00	1/2x	129.112x	1.000	64.556	5.667	365.839	1672.362
13	-3.00	1/2x	38.512x	6.000	115.536	8.000	924.288	
14	-9.00	1/2x	38.512x	6.000	115.536	10.000	1155.360	4167.940
15	-9.00	1/2x	38.512x	2.000	38.512	12.667	487.832	
16	-11.00	1/2x	38.512x	2.000	38.512	13.333	513.480	5369.514
17	-11.00	1/2x	38.512x	2.000	38.512	14.667	564.856	
18	-13.00	1/2x	38.512x	2.000	38.512	15.333	590.504	6755.946
19	-13.00	1/2x	38.512x	3.000	57.768	17.000	982.056	
20	-16.00	1/2x	38.512x	3.000	57.768	18.000	1039.824	9182.202
21	-16.00	1/2x	38.512x	4.000	77.024	20.333	1566.129	
22	-20.00	1/2x	38.512x	4.000	77.024	21.667	1668.879	13064.212

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

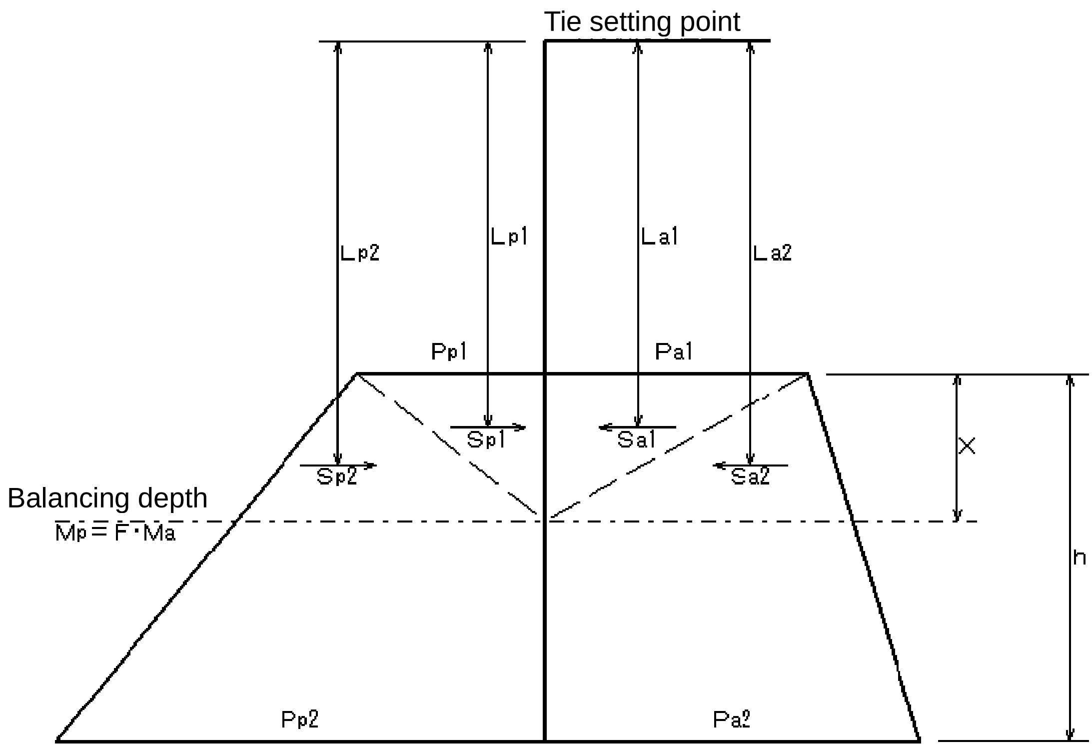
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-1.00	1/2x 23.000x 1.000	11.500	4.333	49.830	
2	-2.00	1/2x 27.000x 1.000	13.500	4.667	63.004	112.834
3	-2.00	1/2x 27.000x 1.000	13.500	5.333	71.996	
4	-3.00	1/2x 31.000x 1.000	15.500	5.667	87.838	272.668
5	-3.00	1/2x 31.000x 6.000	93.000	8.000	744.000	
6	-9.00	1/2x 55.000x 6.000	165.000	10.000	1650.000	2666.668
7	-9.00	1/2x 85.000x 2.000	85.000	12.667	1076.695	
8	-11.00	1/2x 93.000x 2.000	93.000	13.333	1239.969	4983.332
9	-11.00	1/2x 93.000x 2.000	93.000	14.667	1364.031	
10	-13.00	1/2x 101.000x 2.000	101.000	15.333	1548.633	7895.996
11	-13.00	1/2x 170.000x 3.000	255.000	17.000	4335.000	
12	-16.00	1/2x 182.000x 3.000	273.000	18.000	4914.000	17144.996
13	-16.00	1/2x 165.000x 4.000	330.000	20.333	6709.890	
14	-20.00	1/2x 181.000x 4.000	362.000	21.667	7843.454	31698.340

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \text{ , } \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$	: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)
$M_{a(p)1}$	: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)
$M_{a(p)1'}$	: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$M_{a(p)2'}$	: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$S_{a(p)1}$	: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$S_{a(p)2}$	: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$L_{a(p)1}$	: the arm length of $S_{a(p)1}$ from the tie setting point (m)
$L_{a(p)2}$	: the arm length of $S_{a(p)2}$ from the tie setting point (m)
$P_{a(p)1}$	: the active (passive) side pressure at the top elevation of the soil layer (kPa)
$P_{a(p)1}$	: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)
h	: the thickness of the soil layer (m)
L	: the vertical distance from the tie setting point to the top elevation of the soil layer (m)
X	: the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -11.00 m	FOS×M_{a1} = 5369.514	> M_{p1} = 4983.332
at -13.00 m	FOS×M_{a2} = 6755.946	< M_{p2} = 7895.996

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \quad L_{a1} = 14.000 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 269.584X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{2.000}X\right]X}{2} = 0.000X^2 + 19.256X \ , \quad L_{a2} = 14.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 269.584X$$

$$M_a = 0.000X^3 + 19.256X^2 + 539.168X + 4474.595$$

[Moment on passive side]

$$S_{p1} = \frac{93.000X}{2} = 46.500X \ , \quad L_{p1} = 14.000 + \frac{1}{3}X$$

$$M_{p1'} = 15.500X^2 + 651.000X$$

$$S_{p2} = \frac{\left[93.000 + \frac{101.000 - 93.000}{2.000}X\right]X}{2} = 2.000X^2 + 46.500X \ , \quad L_{p2} = 14.000 + \frac{2}{3}X$$

$$M_{p2'} = 1.333X^3 + 59.000X^2 + 651.000X$$

$$M_p = 1.333X^3 + 74.500X^2 + 1302.000X + 4983.332$$

[Balancing depth]

$$FOS \cdot M_a = M_p$$

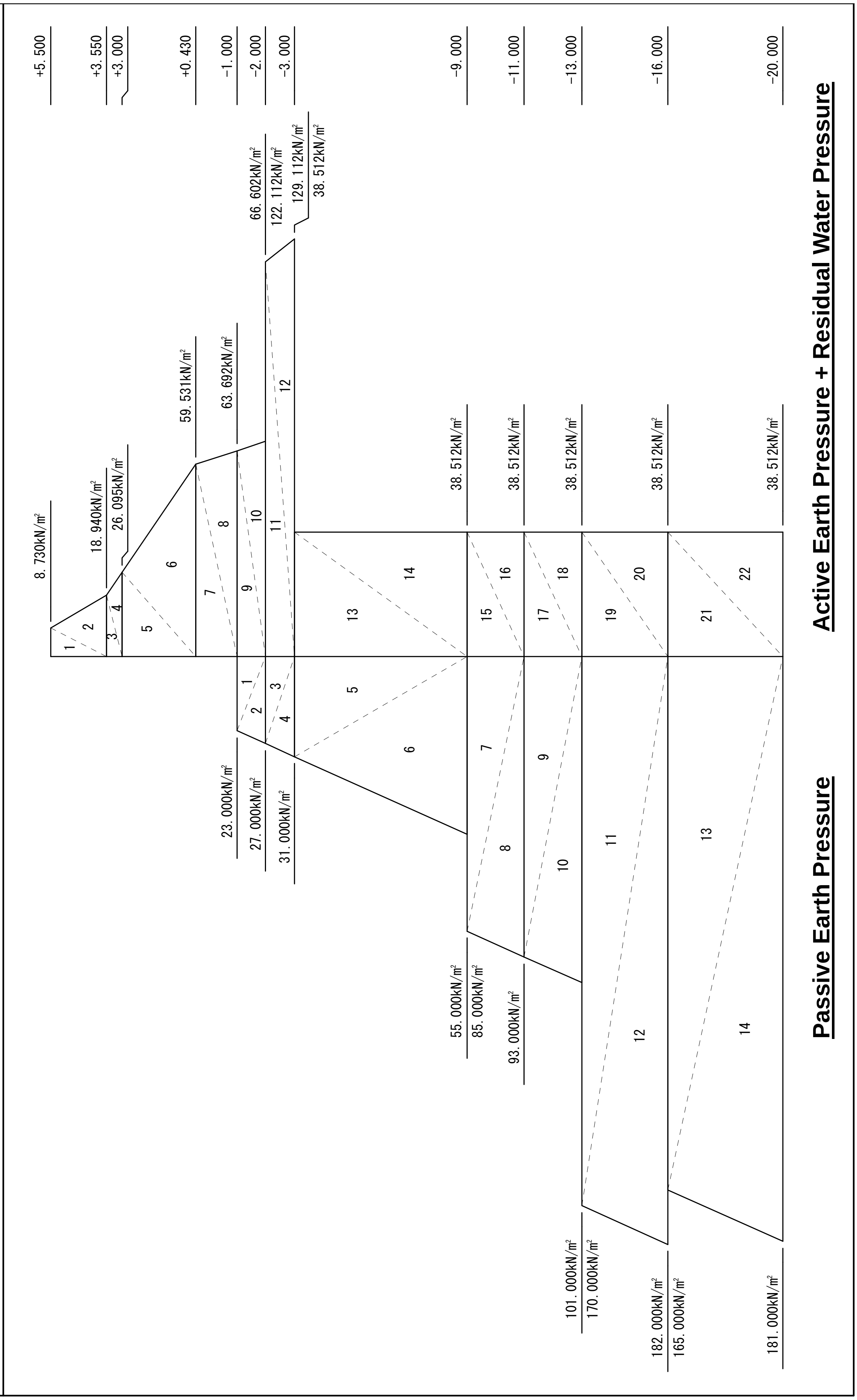
$$1.2 \times (0.000X^3 + 19.256X^2 + 539.168X + 4474.595)$$

$$= 1.333X^3 + 74.500X^2 + 1302.000X + 4983.332$$

X= 0.564 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -11.000 – 0.564 = -11.564 m

Earth Pressure and Water Pressure



(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 3.55	8.730 18.940	——— ———	8.730 18.940
3.55 3.00	18.940 26.095	——— ———	18.940 26.095
3.00 0.43	26.095 59.531	——— ———	26.095 59.531
0.43 -1.00	59.531 63.692	——— ———	59.531 63.692
-1.00 -2.00	63.692 66.602	23.000 27.000	40.692 39.602
-2.00 -3.00	122.112 129.112	27.000 31.000	95.112 98.112
-3.00 -9.00	38.512 38.512	31.000 55.000	7.512 -16.488
-9.00 -11.00	38.512 38.512	85.000 93.000	-46.488 -54.488
-11.00 -13.00	38.512 38.512	93.000 101.000	-54.488 -62.488
-13.00 -16.00	38.512 38.512	170.000 182.000	-131.488 -143.488
-16.00 -20.00	38.512 38.512	165.000 181.000	-126.488 -142.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $P_{a1} = 38.512$ > $P_{p1} = 31.000$
at -9.00 m $P_{a2} = 38.512$ < $P_{p2} = 55.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{6.00 \times (31.000 - 38.512)}{(38.512 - 38.512) - (55.000 - 31.000)} = -4.878m$$

Accordingly, the virtual seabed level is obtained as -4.878m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	8.512	-1.850	-15.747
2	1/2x	18.940x	1.950	18.466	-1.200	-22.159
3	1/2x	18.940x	0.550	5.208	-0.367	-1.911
4	1/2x	26.095x	0.550	7.176	-0.183	-1.313
5	1/2x	26.095x	2.570	33.532	0.857	28.737
6	1/2x	59.531x	2.570	76.497	1.713	131.039
7	1/2x	59.531x	1.430	42.565	3.047	129.696
8	1/2x	63.692x	1.430	45.540	3.523	160.437
9	1/2x	40.692x	1.000	20.346	4.333	88.159
10	1/2x	39.602x	1.000	19.801	4.667	92.411
11	1/2x	95.112x	1.000	47.556	5.333	253.616
12	1/2x	98.112x	1.000	49.056	5.667	278.000
13	1/2x	7.512x	1.878	7.054	6.626	46.740
Total				381.309	———	1167.705

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

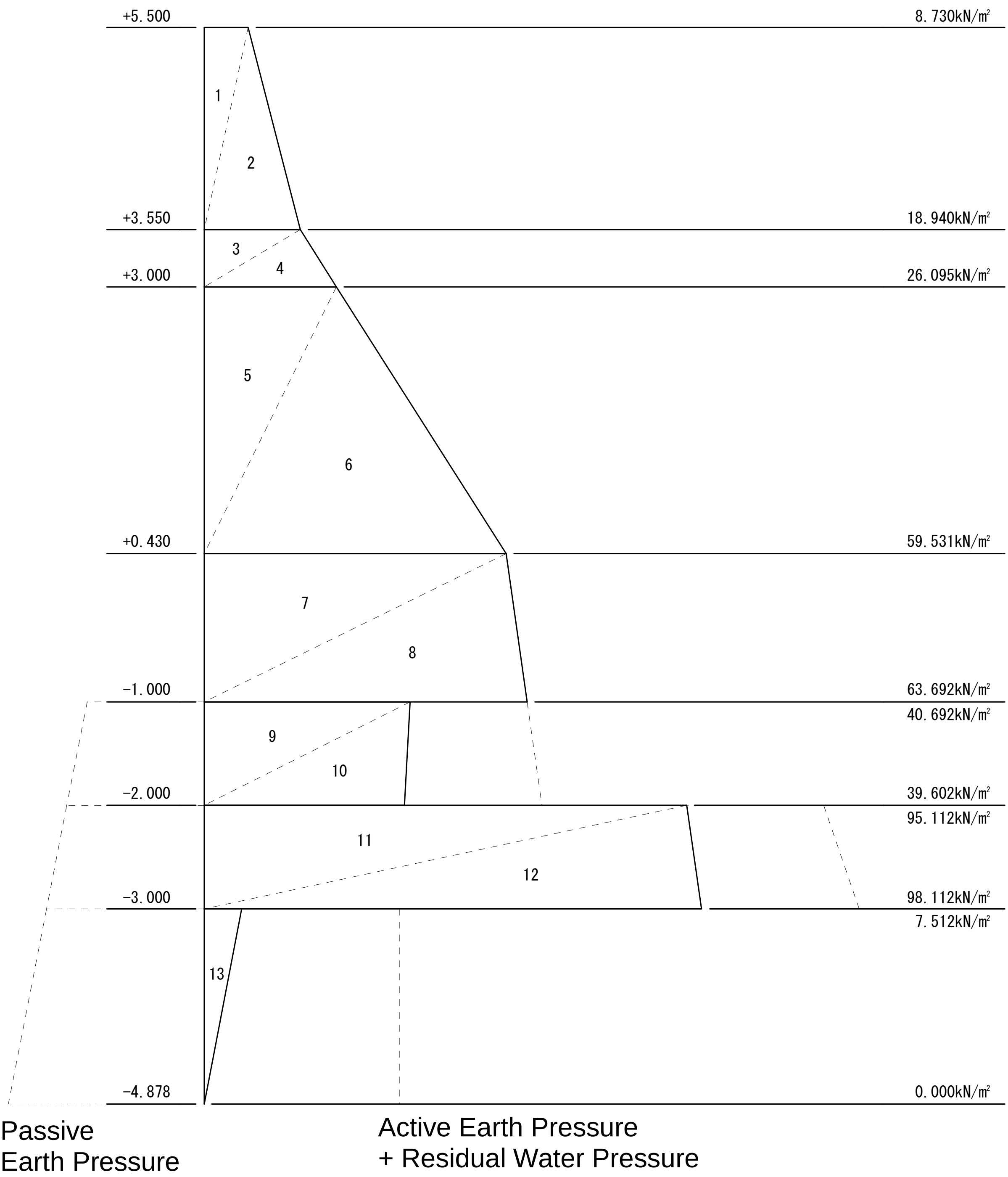
4) Reaction forces

Distance between supports : L_T = 3.000 – (-4.878) = 7.878 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1167.706}{7.878} = 148.224 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 381.309 - 148.224 = 233.085 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 3.55	8.512 18.466	26.978		-26.978
3.55 3.00	5.208 7.176	39.362	233.085	193.723
3.00 0.43	33.532 76.497	149.391	233.085	83.694
0.43 -1.00	42.565 45.540	237.496	233.085	-4.411
-1.00 -2.00	20.346 19.801	277.643	233.085	-44.558
-2.00 -3.00	47.556 49.056	374.255	233.085	-141.170
-3.00 -4.88	7.054 0.000	381.309	233.085	-148.224

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -1.000m.

$$Q = 83.694 - \frac{[59.531 + (59.531 + 2.910X)]X}{2} = 0$$

X = 1.361 m

Shear force zero point : DL = +0.43 – 1.361 = -0.931m

6) Calculation of moment

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 8.730x 1.950	-8.512	5.781	-49.208
2	1/2x 18.940x 1.950	-18.466	5.131	-94.749
3	1/2x 18.940x 0.550	-5.208	4.298	-22.384
4	1/2x 26.095x 0.550	-7.176	4.114	-29.522
5	1/2x 26.095x 2.570	-33.532	3.074	-103.077
6	1/2x 59.531x 2.570	-76.497	2.218	-169.670
7	1/2x 59.531x 1.361	-40.511	0.907	-36.743
8	1/2x 63.491x 1.361	-43.206	0.454	-19.616
Total		———	———	-524.969

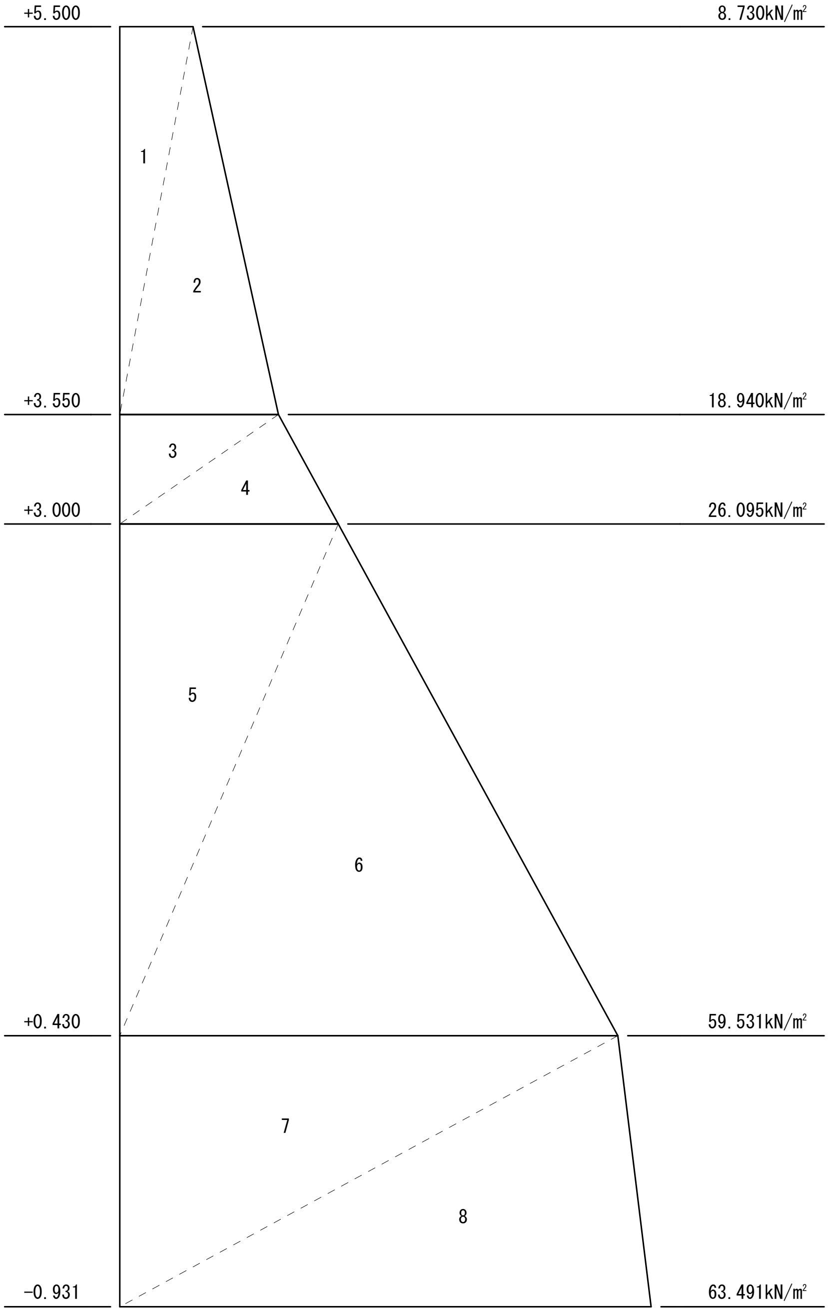
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.931) = 3.931 m

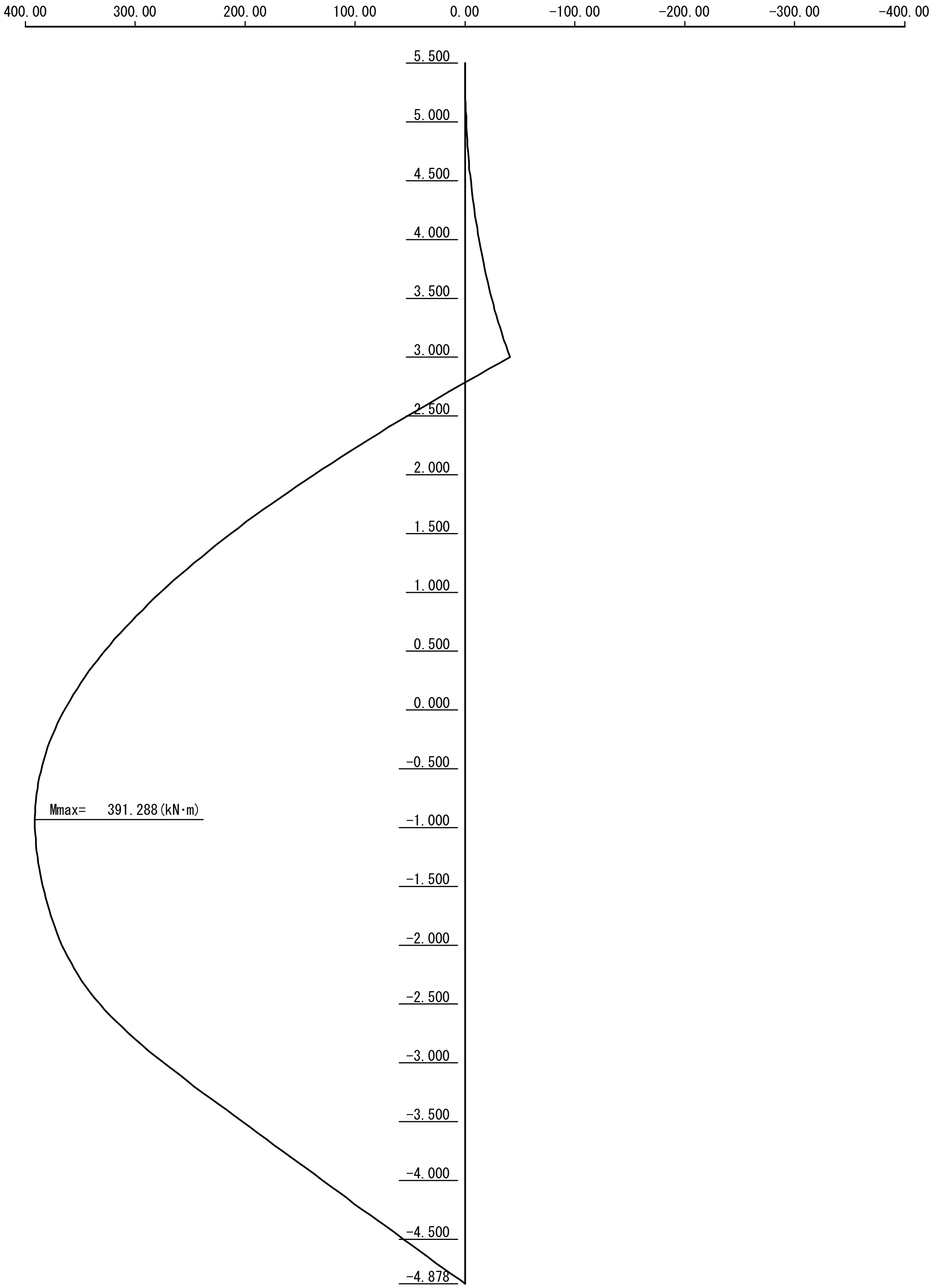
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 233.085 \times 3.931 - 524.969 = 391.288$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall
Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -16.137 m --- adopted
- Free earth support method : -11.564 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-16.137) = 19.637m → 20.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 20.000 = -16.500 m

(3) Stress on pipe wall section
Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 524.726 kN-m/m --- adopted
- Free earth support method : 391.288 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{524.726\times10^6}{4191\times10^3} = 125.2MPa \quad \leq \quad \sigma_a = 140MPa \quad .... \ OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 264.266 kN-m/m --- adopted
- Free earth support method : 233.085 kN-m/m

T = 264.266 × 1.960 × sec(0.000) × sec (0.000) = 517.961 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 517.961 × 3.8 = 1968.252 kN/wire

Tie wire shall have minimum tensile strength of 1969 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{517.961 \times 1.960}{10.0} = 101.520 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{101.520 \times 10^6}{2 \times 374.0 \times 10^3} = 135.7 MPa \quad \leq \quad \sigma_a = 140 MPa \ \ .... \ OK$$

Output of Design Software for Anchor Wall (After Completion)
– Revetment Block c

1. Design Conditions

1-1 Dimensions

Ground elevation	+5.50 m
Top of cope concrete	+5.50 m
Top of pile wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-1.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	30 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

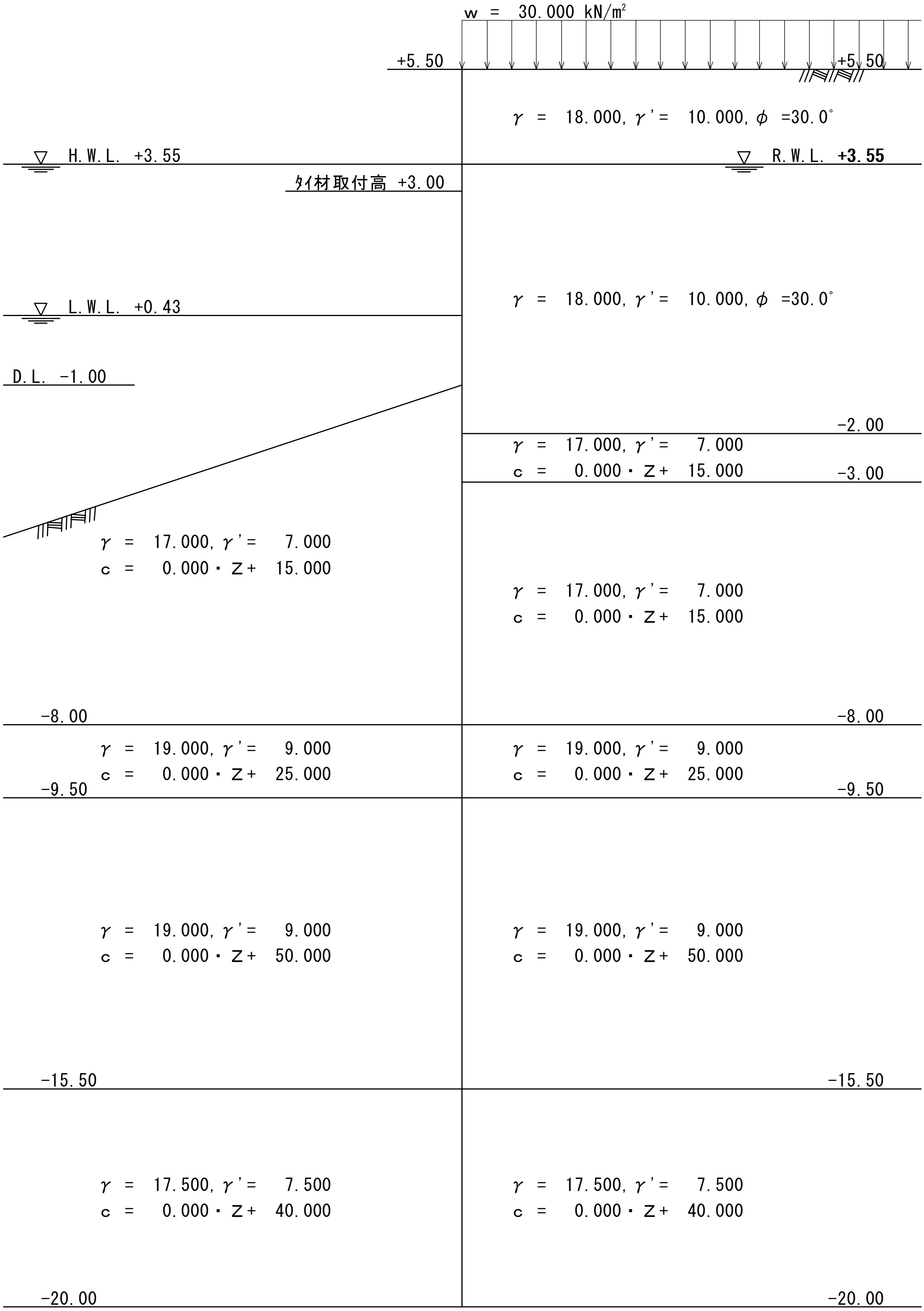
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
5.50 3.55	1.95	8.730 18.940
3.55 3.00	0.55	18.940 20.540
3.00 -2.00	5.00	20.540 35.090
-2.00 -3.00	1.00	90.600 97.600
-3.00 -8.00	5.00	7.000 7.000
-8.00 -9.50	1.50	7.000 7.000
-9.50 -11.00	1.50	7.000 7.000
-11.00 -15.50	4.50	7.000 7.000
-15.50 -20.00	4.50	7.000 7.000

(2) Residual water pressure

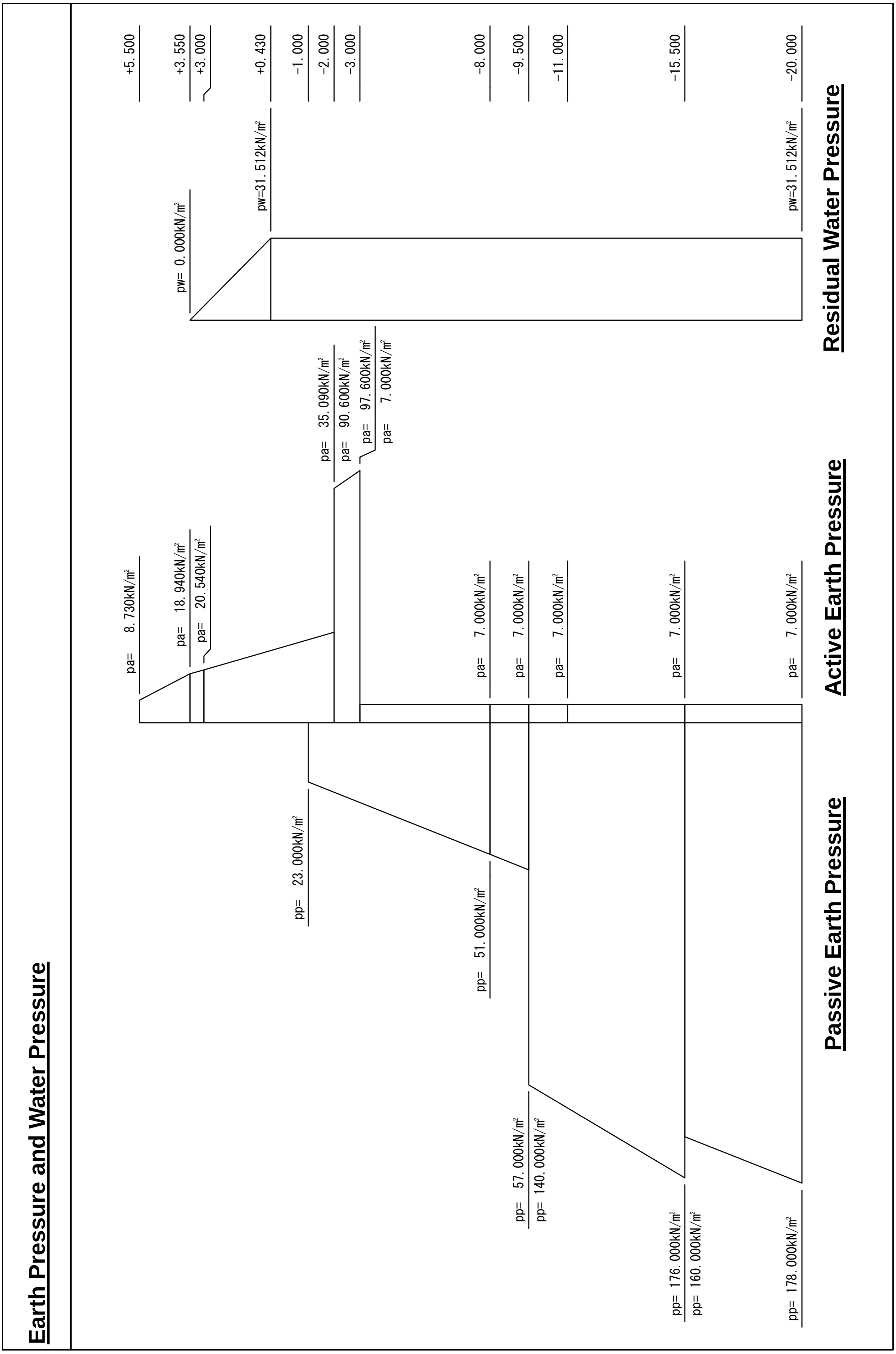
$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-1.00 -8.00	7.00	23.000 51.000
-8.00 -9.50	1.50	51.000 57.000
-9.50 -15.50	6.00	140.000 176.000
-15.50 -20.00	4.50	160.000 178.000


Earth Pressure and Water Pressure
Passive Earth Pressure
Active Earth Pressure
Residual Water Pressure
+5.500
+3.550
+3.000
+0.430
-1.000
-2.000
-3.000
-8.000
-9.500
-11.000
-15.500
-20.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
pa= 8.730kN/m²
pa= 18.940kN/m²
pa= 20.540kN/m²
pa= 35.090kN/m²
pa= 90.600kN/m²
pa= 97.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pp= 23.000kN/m²
pp= 51.000kN/m²
pp= 57.000kN/m²
pp= 140.000kN/m²
pp= 176.000kN/m²
pp= 160.000kN/m²
pp= 178.000kN/m²

(4) Summary of Forces on the Wall

Elev	Active side (kPa)	Passive side(kPa)
(m)	$P_a + P_w$	P_p
5.50 3.55	8.730+ 0.000= 8.730 18.940+ 0.000= 18.940	—— ——
3.55 3.00	18.940+ 0.000= 18.940 20.540+ 5.555= 26.095	—— ——
3.00 0.43	20.540+ 5.555= 26.095 28.019+ 31.512= 59.531	—— ——
0.43 -1.00	28.019+ 31.512= 59.531 32.180+ 31.512= 63.692	—— ——
-1.00 -2.00	32.180+ 31.512= 63.692 35.090+ 31.512= 66.602	23.000 27.000
-2.00 -3.00	90.600+ 31.512= 122.112 97.600+ 31.512= 129.112	27.000 31.000
-3.00 -8.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	31.000 51.000
-8.00 -9.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	51.000 57.000
-9.50 -11.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	140.000 149.000
-11.00 -15.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	149.000 176.000
-15.50 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	160.000 178.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -1.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-1.000	-111.249	135.301	102.195
-2.000	-238.845	159.773	117.870
-3.000	-459.920	187.428	186.827
-4.000	-747.349	214.559	165.208
-5.000	-1017.829	235.345	145.934
-6.000	-1269.787	251.459	127.332
-7.000	-1496.898	263.902	108.400
-8.000	-1689.275	273.308	88.507
-9.000	-1834.063	280.108	67.219
-10.000	-1872.340	283.792	3.297
-11.000	-1521.256	280.224	-100.623
-12.000	-703.050	269.766	-203.653
-13.000	623.948	253.335	-306.710

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -12.00 and -13.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-12.579	0.000	260.932	-263.241

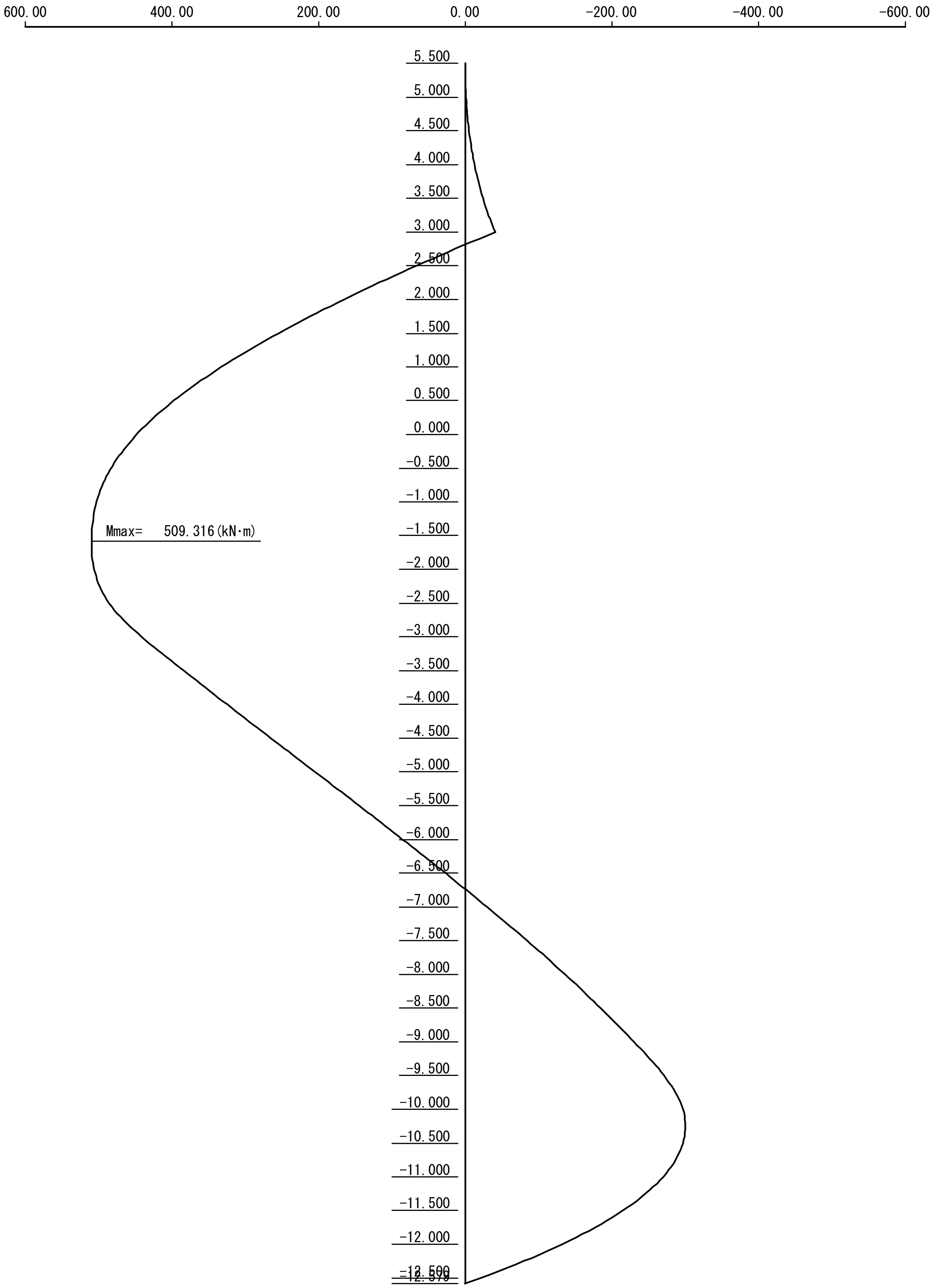
Embedded length of the pipe wall : L = 1.2 × (-1.000 + 12.579) = 13.895 m
Toe elevation : D = -1.000 – 13.895 = -14.895 m
Reaction force at tie setting point : 260.931 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	-1.200	-5.019
4.500	-5.238	-11.348
4.000	-12.766	-18.985
3.550	-23.069	-26.978
3.050	-39.196	-38.074
3.000	-41.132	-39.363
2.500	66.120	206.895
2.000	165.221	188.969
1.500	254.547	167.790
1.000	332.469	143.359
0.500	397.363	115.675
0.430	405.316	111.540
-0.070	453.584	81.410
-0.570	486.606	50.554
-1.000	502.533	23.435
-1.500	509.186	3.226
-2.000	505.803	-16.712
-2.500	485.496	-64.643
-3.000	441.036	-113.324
-3.500	383.518	-116.580
-4.000	324.623	-118.836
-4.500	264.849	-120.092
-5.000	204.698	-120.348
-5.500	144.668	-119.604
-6.000	85.261	-117.860
-6.500	26.975	-115.116
-7.000	-29.688	-111.372
-7.500	-84.230	-106.628
-8.000	-136.150	-100.884
-8.500	-184.947	-94.140
-9.000	-230.123	-86.396
-9.500	-271.176	-77.652
-10.000	-297.191	-26.158
-10.500	-297.084	26.836
-11.000	-270.105	81.330
-11.500	-215.504	137.324
-12.000	-132.530	194.818
-12.500	-20.435	253.812
-12.579	0.000	263.241

Maximum bending moment : 509.316 kN-m/m
Elevation of maximum bending moment : -1.580 m
Depth of 1st steady point (moment=0) : -6.736 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
1	5.50	1/2x	8.730x	1.950	8.512	-1.850	-15.747	
2	3.55	1/2x	18.940x	1.950	18.466	-1.200	-22.159	-45.487
3	3.55	1/2x	18.940x	0.550	5.208	-0.367	-1.911	
4	3.00	1/2x	26.095x	0.550	7.176	-0.183	-1.313	-49.356
5	3.00	1/2x	26.095x	2.570	33.532	0.857	28.737	
6	0.43	1/2x	59.531x	2.570	76.497	1.713	131.039	142.375
7	0.43	1/2x	59.531x	1.430	42.565	3.047	129.696	
8	-1.00	1/2x	63.692x	1.430	45.540	3.523	160.437	490.535
9	-1.00	1/2x	63.692x	1.000	31.846	4.333	137.989	
10	-2.00	1/2x	66.602x	1.000	33.301	4.667	155.416	842.621
11	-2.00	1/2x	122.112x	1.000	61.056	5.333	325.612	
12	-3.00	1/2x	129.112x	1.000	64.556	5.667	365.839	1672.362
13	-3.00	1/2x	38.512x	5.000	96.280	7.667	738.179	
14	-8.00	1/2x	38.512x	5.000	96.280	9.333	898.581	3636.474
15	-8.00	1/2x	38.512x	1.500	28.884	11.500	332.166	
16	-9.50	1/2x	38.512x	1.500	28.884	12.000	346.608	4451.003
17	-9.50	1/2x	38.512x	1.500	28.884	13.000	375.492	
18	-11.00	1/2x	38.512x	1.500	28.884	13.500	389.934	5369.514
19	-11.00	1/2x	38.512x	4.500	86.652	15.500	1343.106	
20	-15.50	1/2x	38.512x	4.500	86.652	17.000	1473.084	8748.942
21	-15.50	1/2x	38.512x	4.500	86.652	20.000	1733.040	
22	-20.00	1/2x	38.512x	4.500	86.652	21.500	1863.018	13064.212

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

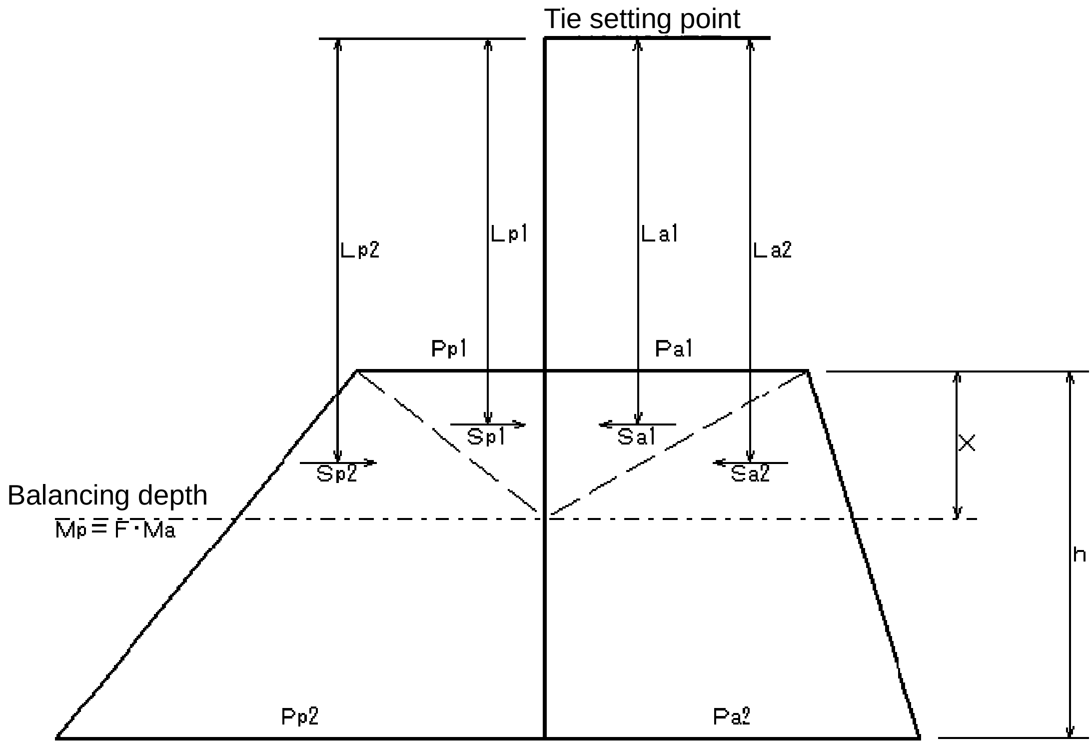
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-1.00	1/2x	23.000x	1.000	11.500	4.333	49.830	
2	-2.00	1/2x	27.000x	1.000	13.500	4.667	63.004	112.834
3	-2.00	1/2x	27.000x	1.000	13.500	5.333	71.996	
4	-3.00	1/2x	31.000x	1.000	15.500	5.667	87.838	272.668
5	-3.00	1/2x	31.000x	5.000	77.500	7.667	594.192	
6	-8.00	1/2x	51.000x	5.000	127.500	9.333	1189.958	2056.818
7	-8.00	1/2x	51.000x	1.500	38.250	11.500	439.875	
8	-9.50	1/2x	57.000x	1.500	42.750	12.000	513.000	3009.693
9	-9.50	1/2x	140.000x	1.500	105.000	13.000	1365.000	
10	-11.00	1/2x	149.000x	1.500	111.750	13.500	1508.625	5883.318
11	-11.00	1/2x	149.000x	4.500	335.250	15.500	5196.375	
12	-15.50	1/2x	176.000x	4.500	396.000	17.000	6732.000	17811.693
13	-15.50	1/2x	160.000x	4.500	360.000	20.000	7200.000	
14	-20.00	1/2x	178.000x	4.500	400.500	21.500	8610.750	33622.443

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}\ ,\quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -9.50 m FOS×M_{a1} = 4451.003 > M_{p1} = 3009.693
at -11.00 m FOS×M_{a2} = 5369.514 < M_{p2} = 5883.318

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \;,\;\; L_{a1} = 12.500 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 240.700X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{1.500}X\right]X}{2} = 0.000X^2 + 19.256X \;,\;\; L_{a2} = 12.500 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 240.700X$$

$$M_a = 0.000X^3 + 19.256X^2 + 481.400X + 3709.169$$

[Moment on passive side]

$$S_{p1} = \frac{140.000X}{2} = 70.000X \;,\;\; L_{p1} = 12.500 + \frac{1}{3}X$$

$$M_{p1'} = 23.333X^2 + 875.000X$$

$$S_{p2} = \frac{\left[140.000 + \frac{149.000 - 140.000}{1.500}X\right]X}{2} = 3.000X^2 + 70.000X \;,\;\; L_{p2} = 12.500 + \frac{2}{3}X$$

$$M_{p2'} = 2.000X^3 + 84.167X^2 + 875.000X$$

$$M_p = 2.000X^3 + 107.500X^2 + 1750.000X + 3009.693$$

[Balancing depth]

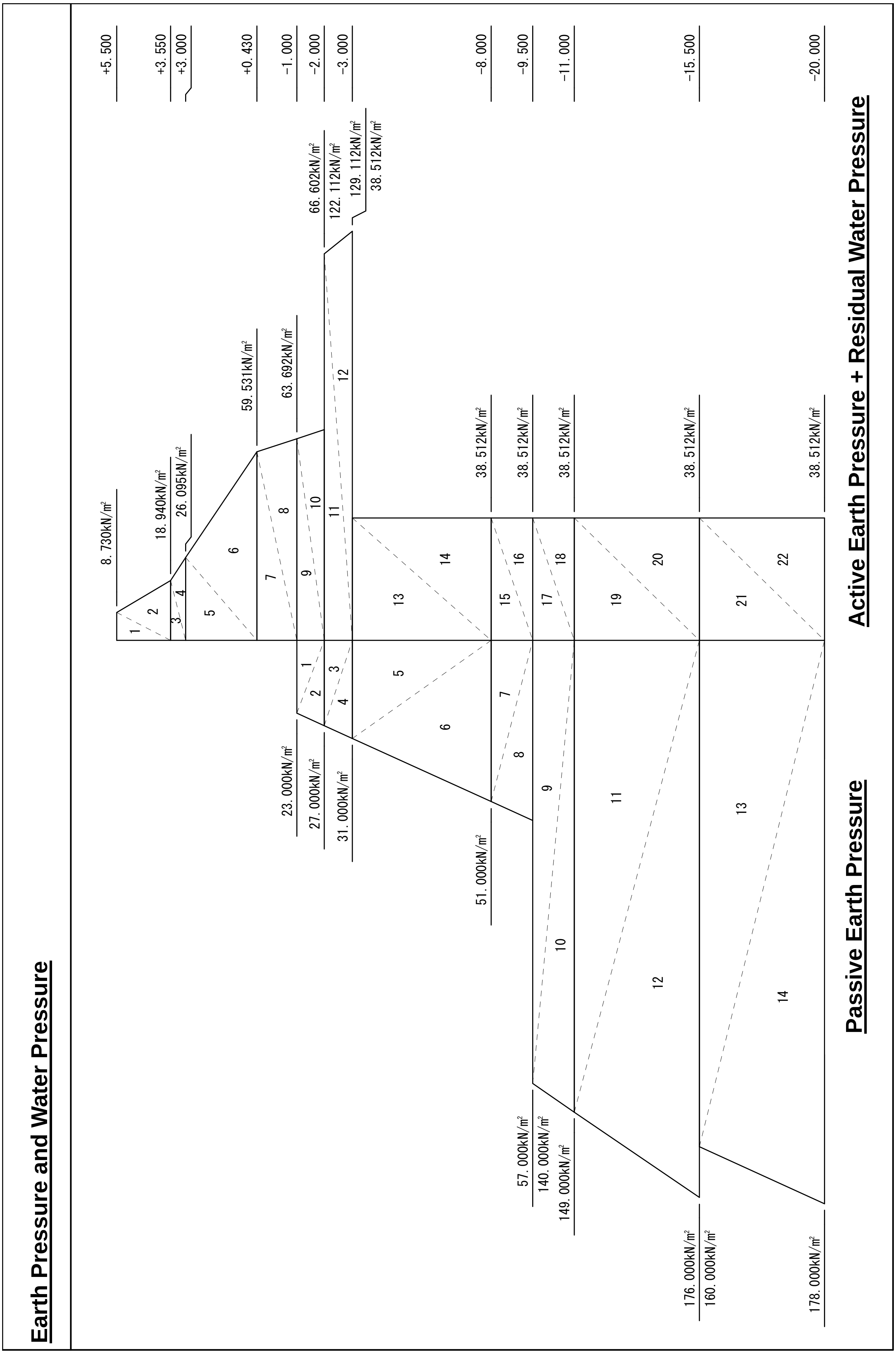
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 481.400X + 3709.169)$$

$$= 2.000X^3 + 107.500X^2 + 1750.000X + 3009.693$$

X= 1.134 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -9.500 – 1.134 = -10.634 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+3.550
+3.000
+0.430
-1.000
-2.000
-3.000
-8.000
-9.500
-11.000
-15.500
-20.000
8.730kN/m²
18.940kN/m²
26.095kN/m²
59.531kN/m²
63.692kN/m²
66.602kN/m²
122.112kN/m²
129.112kN/m²
38.512kN/m²
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38.512kN/m²
38.512kN/m²
38.512kN/m²
23.000kN/m²
27.000kN/m²
31.000kN/m²
51.000kN/m²
57.000kN/m²
140.000kN/m²
149.000kN/m²
176.000kN/m²
160.000kN/m²
178.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 3.55	8.730 18.940	——— ———	8.730 18.940
3.55 3.00	18.940 26.095	——— ———	18.940 26.095
3.00 0.43	26.095 59.531	——— ———	26.095 59.531
0.43 -1.00	59.531 63.692	——— ———	59.531 63.692
-1.00 -2.00	63.692 66.602	23.000 27.000	40.692 39.602
-2.00 -3.00	122.112 129.112	27.000 31.000	95.112 98.112
-3.00 -8.00	38.512 38.512	31.000 51.000	7.512 -12.488
-8.00 -9.50	38.512 38.512	51.000 57.000	-12.488 -18.488
-9.50 -11.00	38.512 38.512	140.000 149.000	-101.488 -110.488
-11.00 -15.50	38.512 38.512	149.000 176.000	-110.488 -137.488
-15.50 -20.00	38.512 38.512	160.000 178.000	-121.488 -139.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $\quad P_{a1} = 38.512 \quad > \quad P_{p1} = 31.000$
at -8.00 m $\quad P_{a2} = 38.512 \quad < \quad P_{p2} = 51.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{5.00 \times (31.000 - 38.512)}{(38.512 - 38.512) - (51.000 - 31.000)} = -4.878m$$

Accordingly, the virtual seabed level is obtained as -4.878m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	8.512	-1.850	-15.747
2	1/2x	18.940x	1.950	18.466	-1.200	-22.159
3	1/2x	18.940x	0.550	5.208	-0.367	-1.911
4	1/2x	26.095x	0.550	7.176	-0.183	-1.313
5	1/2x	26.095x	2.570	33.532	0.857	28.737
6	1/2x	59.531x	2.570	76.497	1.713	131.039
7	1/2x	59.531x	1.430	42.565	3.047	129.696
8	1/2x	63.692x	1.430	45.540	3.523	160.437
9	1/2x	40.692x	1.000	20.346	4.333	88.159
10	1/2x	39.602x	1.000	19.801	4.667	92.411
11	1/2x	95.112x	1.000	47.556	5.333	253.616
12	1/2x	98.112x	1.000	49.056	5.667	278.000
13	1/2x	7.512x	1.878	7.054	6.626	46.740
Total				381.309	——	1167.705

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

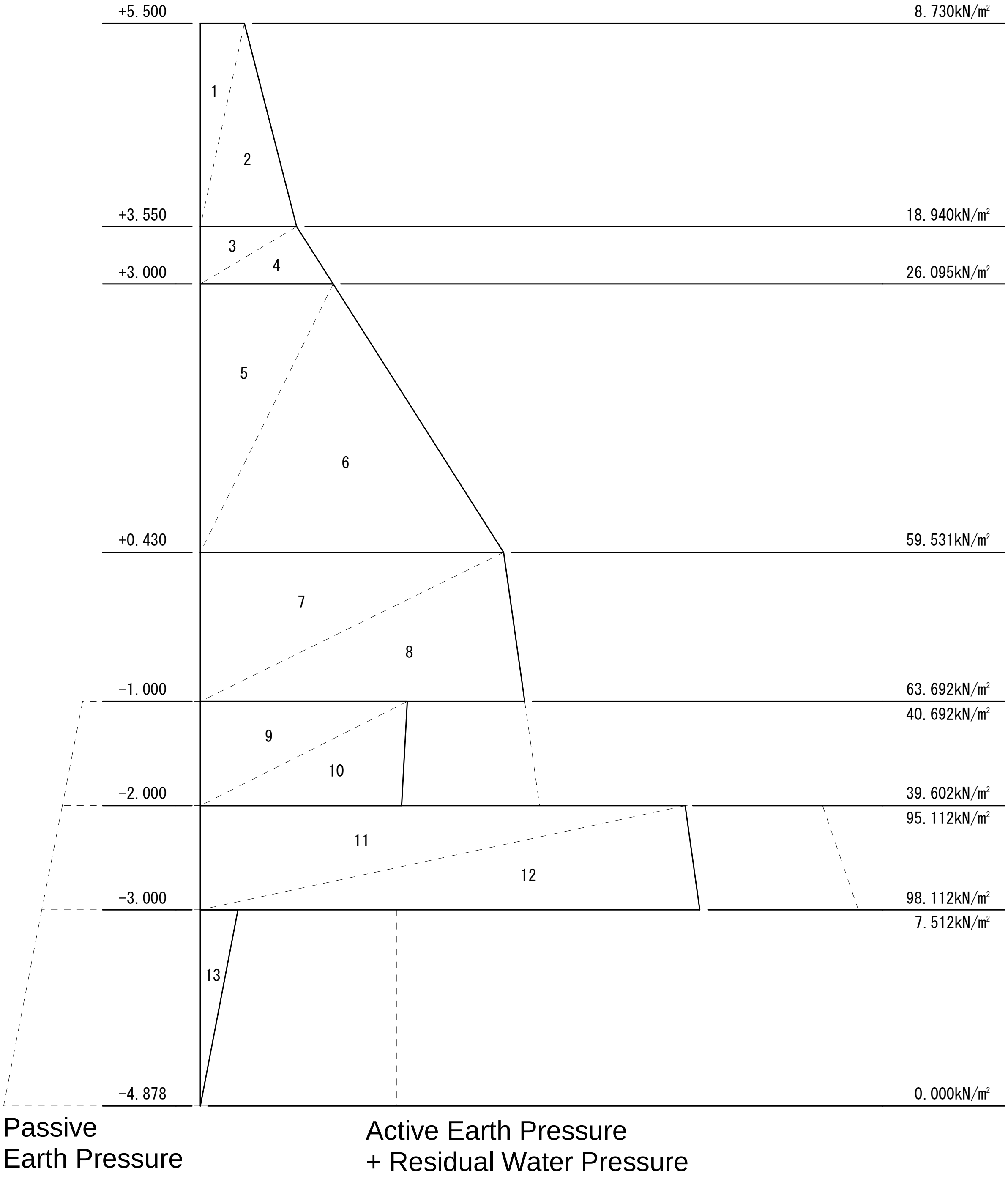
4) Reaction forces

Distance between supports : L_T = 3.000 – (-4.878) = 7.878 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1167.705}{7.878} = 148.224kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 381.309 - 148.224 = 233.085kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 3.55	8.512 18.466	26.978		-26.978
3.55 3.00	5.208 7.176	39.362	233.085	193.723
3.00 0.43	33.532 76.497	149.391	233.085	83.694
0.43 -1.00	42.565 45.540	237.496	233.085	-4.411
-1.00 -2.00	20.346 19.801	277.643	233.085	-44.558
-2.00 -3.00	47.556 49.056	374.255	233.085	-141.170
-3.00 -4.88	7.054 0.000	381.309	233.085	-148.224

Shear force $Q = A_p - \sum S$

The above table suggests that the shear force zero point exists in between 0.430m and -1.000m.

$$Q = 83.694 - \frac{[59.531 + (59.531 + 2.910X)]X}{2} = 0$$

X = 1.361 m

Shear force zero point : DL = +0.43 – 1.361 = -0.931m

6) Calculation of moment

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	-8.512	5.781	-49.208
2	1/2x	18.940x	1.950	-18.466	5.131	-94.749
3	1/2x	18.940x	0.550	-5.208	4.298	-22.384
4	1/2x	26.095x	0.550	-7.176	4.114	-29.522
5	1/2x	26.095x	2.570	-33.532	3.074	-103.077
6	1/2x	59.531x	2.570	-76.497	2.218	-169.670
7	1/2x	59.531x	1.361	-40.511	0.907	-36.743
8	1/2x	63.491x	1.361	-43.206	0.454	-19.616
Total				———	———	-524.969

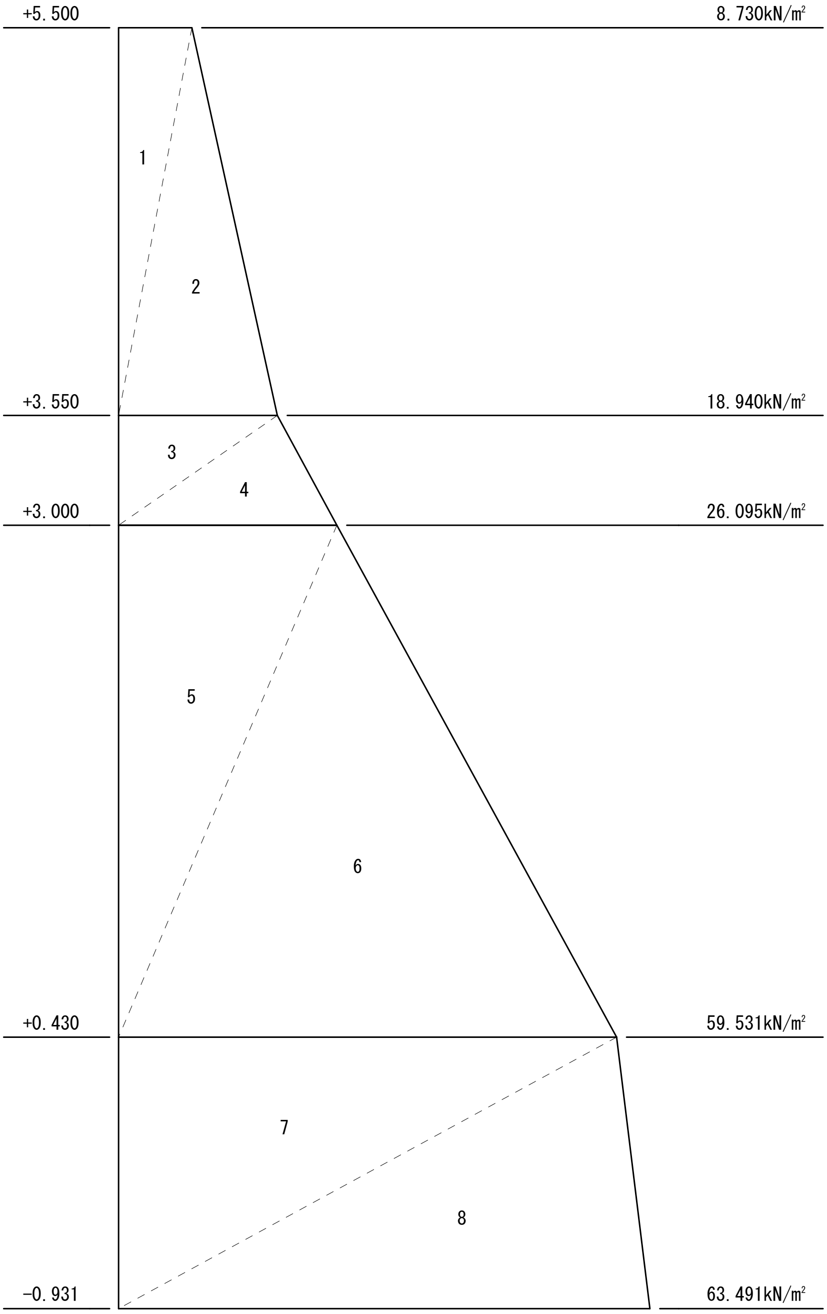
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.931) = 3.931 m

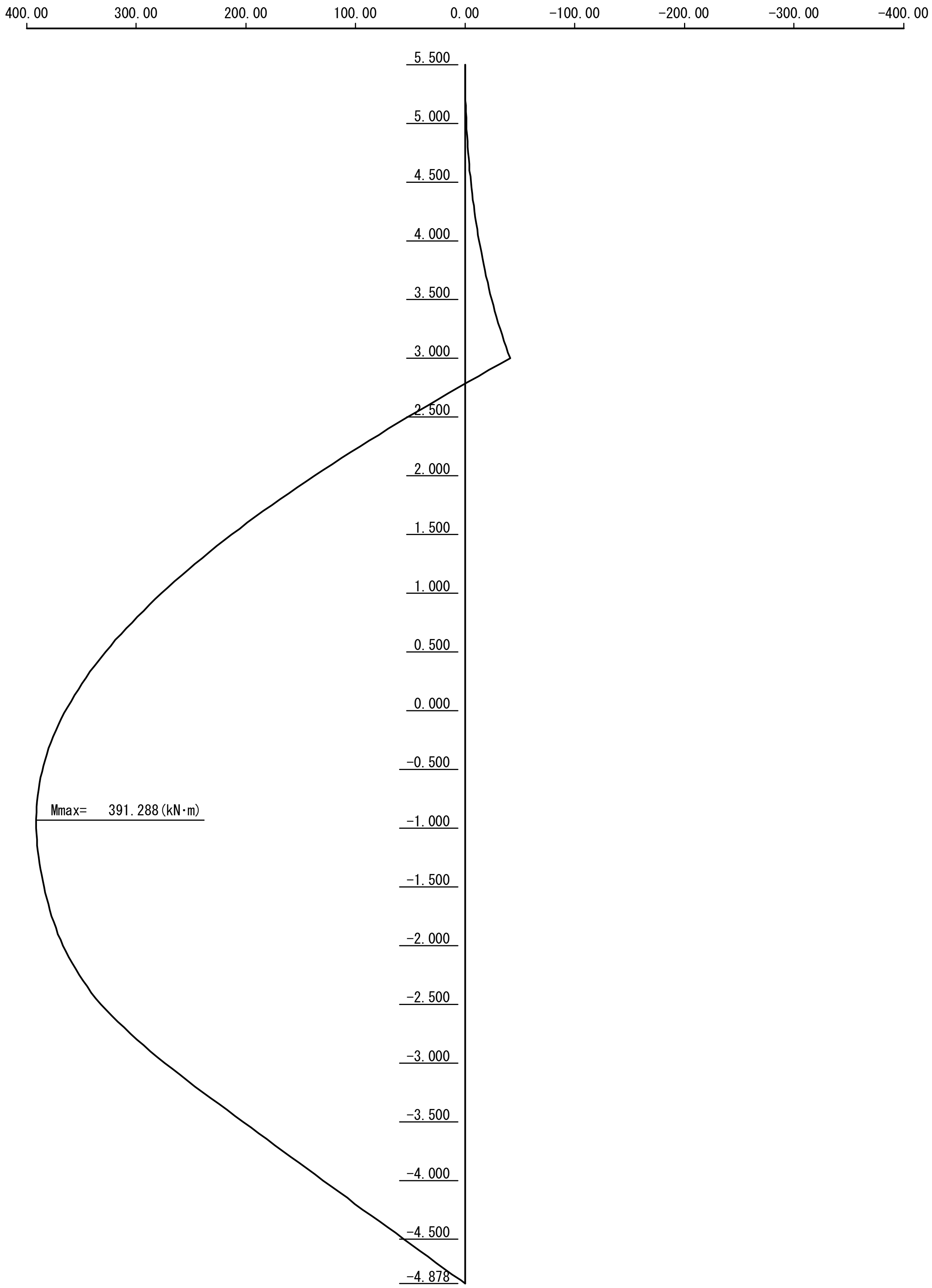
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 233.085 \times 3.931 - 524.969 = 391.288$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -14.895 m --- adopted
- Free earth support method : -10.634 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-14.895) = 18.395m → 18.500 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 18.500 = -15.000 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 509.316 kN-m/m --- adopted
- Free earth support method : 391.288 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{509.311 \times 10^6}{4191 \times 10^3} = 121.5 MPa \quad \leq \quad \sigma_a = 140 MPa \quad .... OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 260.931 kN-m/m --- adopted
- Free earth support method : 233.085 kN-m/m

T = 260.931 × 1.960 × sec(0.000) × sec (0.000) = 511.425 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 511.425 × 3.8 = 1943.415 kN/wire

Tie wire shall have minimum tensile strength of 1944 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{511.425 \times 1.960}{10.0} = 100.239 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{100.239 \times 10^6}{2 \times 374.0 \times 10^3} = 134.0 MPa \quad \leq \quad \sigma_a = 140 MPa \text{ .... } OK$$

<u>Output of Design Software for Anchor Wall (After Completion)</u>
– Revetment Block d

1. Design Conditions

1-1 Dimensions

Ground elevation	+5.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-1.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	30 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water $10.10\ kN/m^3$

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

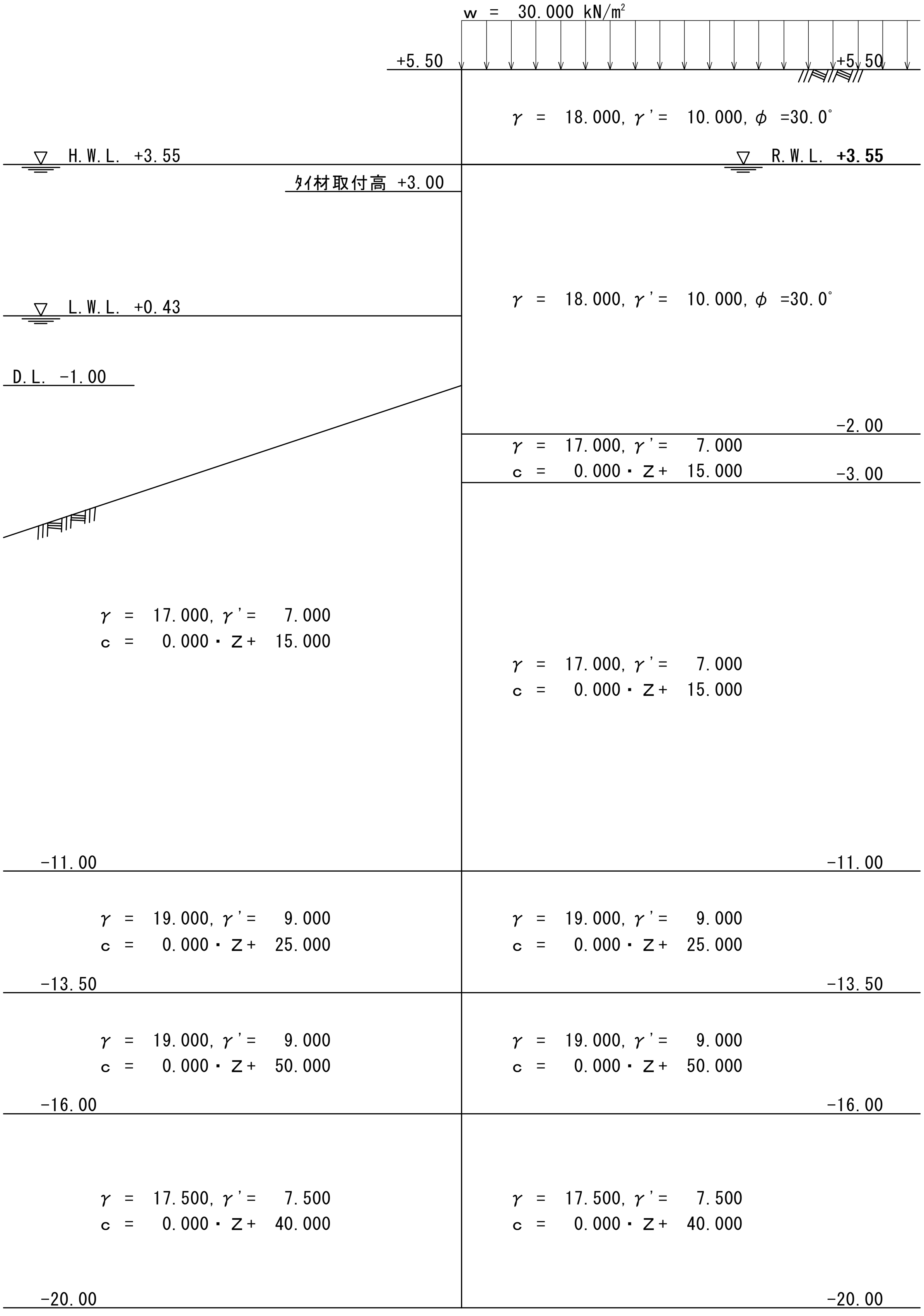
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
5.50 3.55	1.95	8.730 18.940
3.55 3.00	0.55	18.940 20.540
3.00 -2.00	5.00	20.540 35.090
-2.00 -3.00	1.00	90.600 97.600
-3.00 -11.00	8.00	7.000 7.000
-11.00 -13.50	2.50	7.000 7.000
-13.50 -16.00	2.50	7.000 7.000
-16.00 -20.00	4.00	7.000 7.000

(2) Residual water pressure

$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)

h_w is the height of the tidal lag (m)

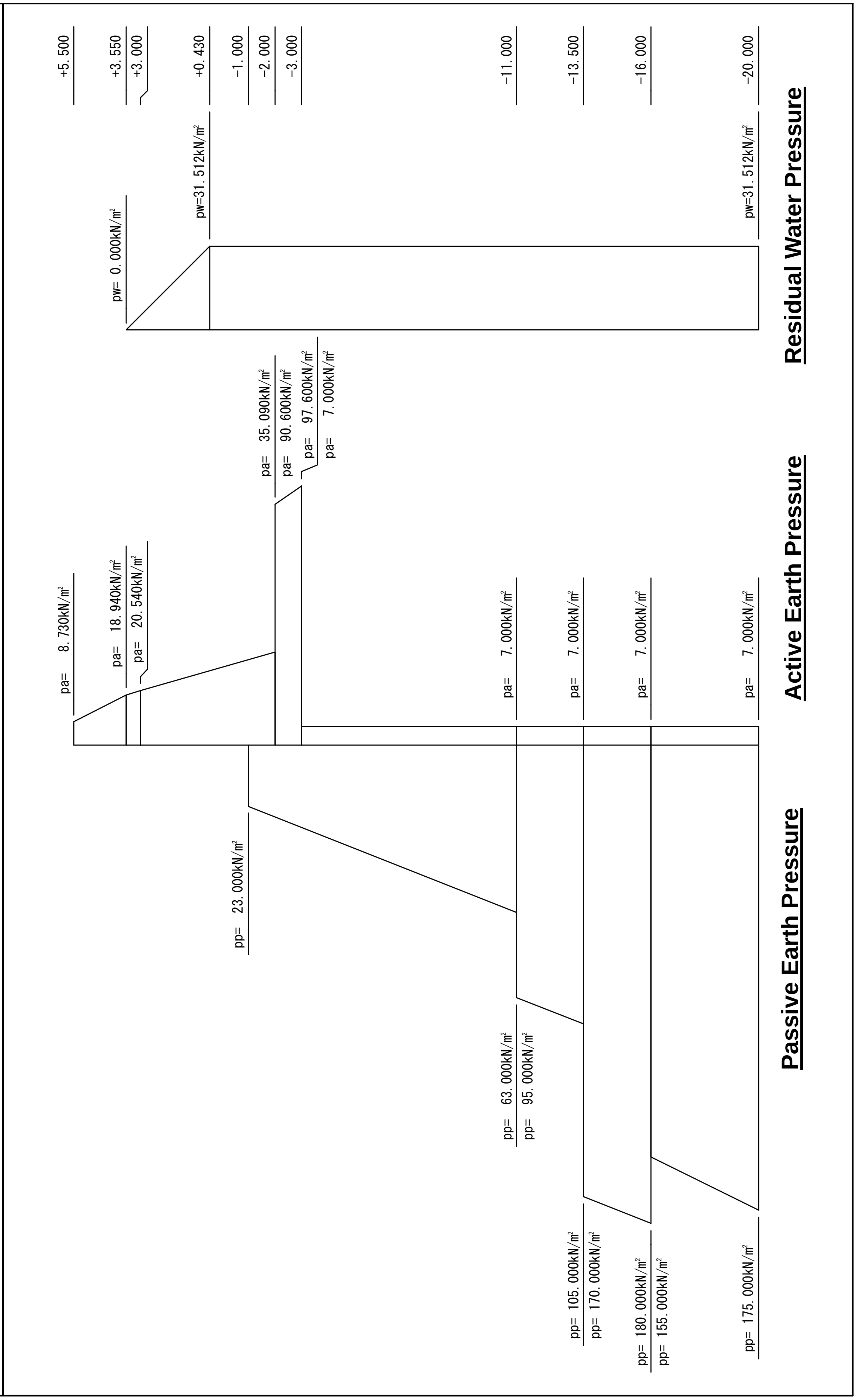
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-1.00	10.00	23.000
-11.00		63.000
-11.00	2.50	95.000
-13.50		105.000
-13.50	2.50	170.000
-16.00		180.000
-16.00	4.00	155.000
-20.00		175.000

Earth Pressure and Water Pressure



(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 3.55	8.730+ 0.000= 8.730 18.940+ 0.000= 18.940	——— ———
3.55 3.00	18.940+ 0.000= 18.940 20.540+ 5.555= 26.095	——— ———
3.00 0.43	20.540+ 5.555= 26.095 28.019+ 31.512= 59.531	——— ———
0.43 -1.00	28.019+ 31.512= 59.531 32.180+ 31.512= 63.692	——— ———
-1.00 -2.00	32.180+ 31.512= 63.692 35.090+ 31.512= 66.602	23.000 27.000
-2.00 -3.00	90.600+ 31.512= 122.112 97.600+ 31.512= 129.112	27.000 31.000
-3.00 -11.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	31.000 63.000
-11.00 -13.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	95.000 105.000
-13.50 -16.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	170.000 180.000
-16.00 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	155.000 175.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -1.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-1.000	-111.249	135.301	102.195
-2.000	-238.845	159.773	117.870
-3.000	-459.920	187.428	186.827
-4.000	-747.349	214.559	165.208
-5.000	-1017.829	235.345	145.934
-6.000	-1269.787	251.459	127.332
-7.000	-1496.898	263.902	108.400
-8.000	-1689.275	273.308	88.507
-9.000	-1834.063	280.109	67.218
-10.000	-1915.762	284.597	44.242
-11.000	-1916.406	286.972	19.379
-12.000	-1740.925	286.337	-38.474
-13.000	-1290.981	282.000	-96.625
-14.000	-494.416	273.903	-187.516
-15.000	948.493	259.740	-308.841

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -14.00 and -15.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-14.402	0.000	268.956	-236.494

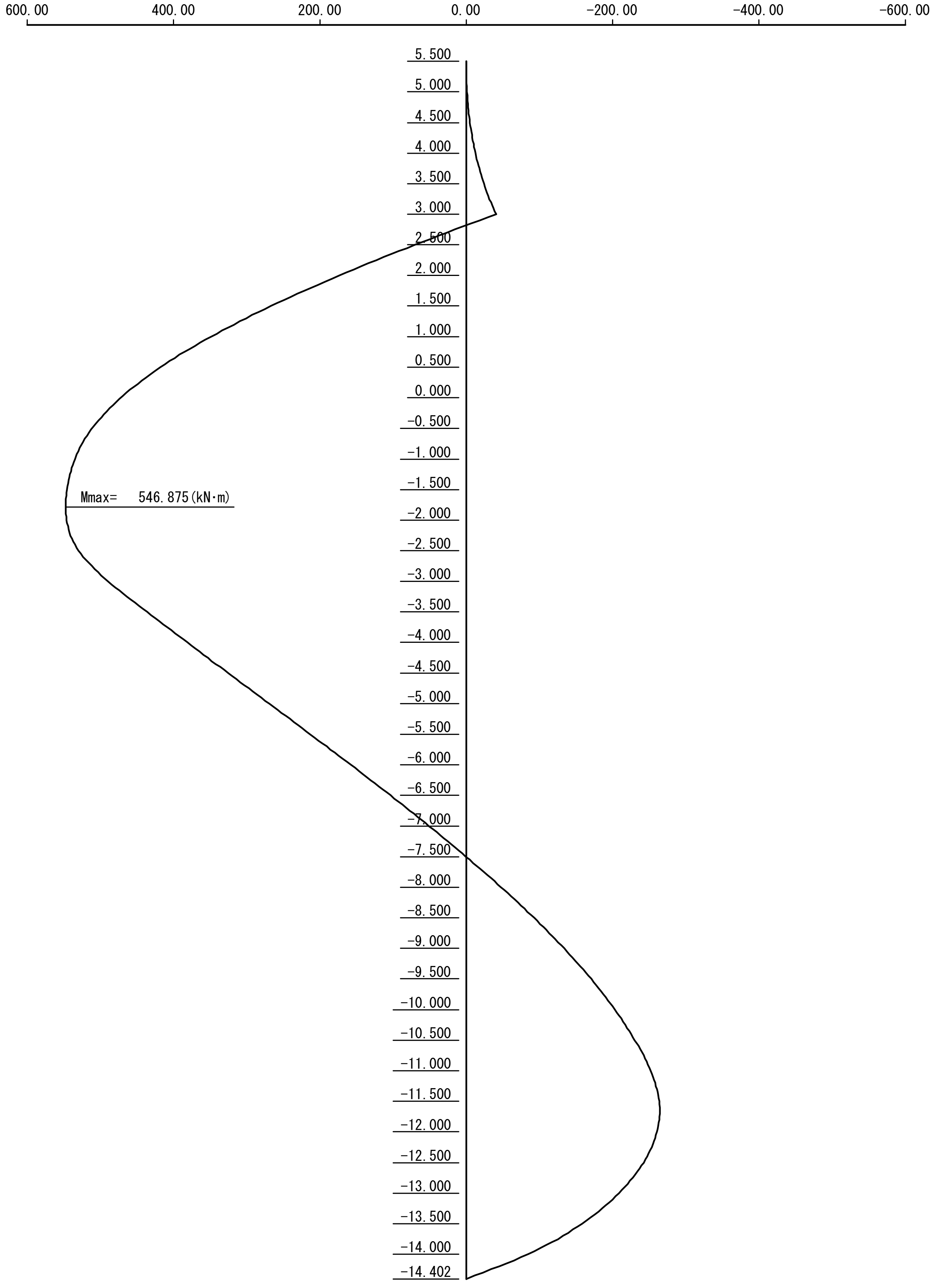
Embedded length of the pipe wall : L = 1.2 × (-1.000 + 14.402) = 16.082 m
Toe elevation : D = -1.000 – 16.082 = -17.082 m
Reaction force at tie setting point : 268.956 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	-1.200	-5.019
4.500	-5.238	-11.348
4.000	-12.766	-18.985
3.550	-23.069	-26.978
3.050	-39.196	-38.074
3.000	-41.132	-39.363
2.500	70.132	214.919
2.000	173.245	196.993
1.500	266.583	175.814
1.000	348.517	151.383
0.500	417.423	123.699
0.430	425.938	119.564
-0.070	478.218	89.434
-0.570	515.251	58.578
-1.000	534.629	31.459
-1.500	545.294	11.250
-2.000	545.923	-8.688
-2.500	529.628	-56.619
-3.000	489.180	-105.300
-3.500	435.674	-108.556
-4.000	380.791	-110.812
-4.500	325.029	-112.068
-5.000	268.890	-112.324
-5.500	212.872	-111.580
-6.000	157.477	-109.836
-6.500	103.203	-107.092
-7.000	50.552	-103.348
-7.500	0.022	-98.604
-8.000	-47.886	-92.860
-8.500	-92.671	-86.116
-9.000	-133.835	-78.372
-9.500	-170.876	-69.628
-10.000	-203.296	-59.884
-10.500	-230.593	-49.140
-11.000	-252.269	-37.396
-11.500	-263.822	-8.652
-12.000	-260.754	21.092
-12.500	-242.563	51.836
-13.000	-208.751	83.580
-13.500	-158.816	116.324
-14.000	-84.135	182.568
-14.402	0.000	236.494

Maximum bending moment : 546.875 kN-m/m
Elevation of maximum bending moment : -1.781 m
Depth of 1st steady point (moment=0) : -7.500 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
1	5.50	1/2x	8.730x	1.950	8.512	-1.850	-15.747	
2	3.55	1/2x	18.940x	1.950	18.466	-1.200	-22.159	-45.487
3	3.55	1/2x	18.940x	0.550	5.208	-0.367	-1.911	
4	3.00	1/2x	26.095x	0.550	7.176	-0.183	-1.313	-49.356
5	3.00	1/2x	26.095x	2.570	33.532	0.857	28.737	
6	0.43	1/2x	59.531x	2.570	76.497	1.713	131.039	142.375
7	0.43	1/2x	59.531x	1.430	42.565	3.047	129.696	
8	-1.00	1/2x	63.692x	1.430	45.540	3.523	160.437	490.535
9	-1.00	1/2x	63.692x	1.000	31.846	4.333	137.989	
10	-2.00	1/2x	66.602x	1.000	33.301	4.667	155.416	842.621
11	-2.00	1/2x	122.112x	1.000	61.056	5.333	325.612	
12	-3.00	1/2x	129.112x	1.000	64.556	5.667	365.839	1672.362
13	-3.00	1/2x	38.512x	8.000	154.048	8.667	1335.134	
14	-11.00	1/2x	38.512x	8.000	154.048	11.333	1745.826	5369.514
15	-11.00	1/2x	38.512x	2.500	48.140	14.833	714.061	
16	-13.50	1/2x	38.512x	2.500	48.140	15.667	754.209	7131.438
17	-13.50	1/2x	38.512x	2.500	48.140	17.333	834.411	
18	-16.00	1/2x	38.512x	2.500	48.140	18.167	874.559	9182.202
19	-16.00	1/2x	38.512x	4.000	77.024	20.333	1566.129	
20	-20.00	1/2x	38.512x	4.000	77.024	21.667	1668.879	13064.212

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

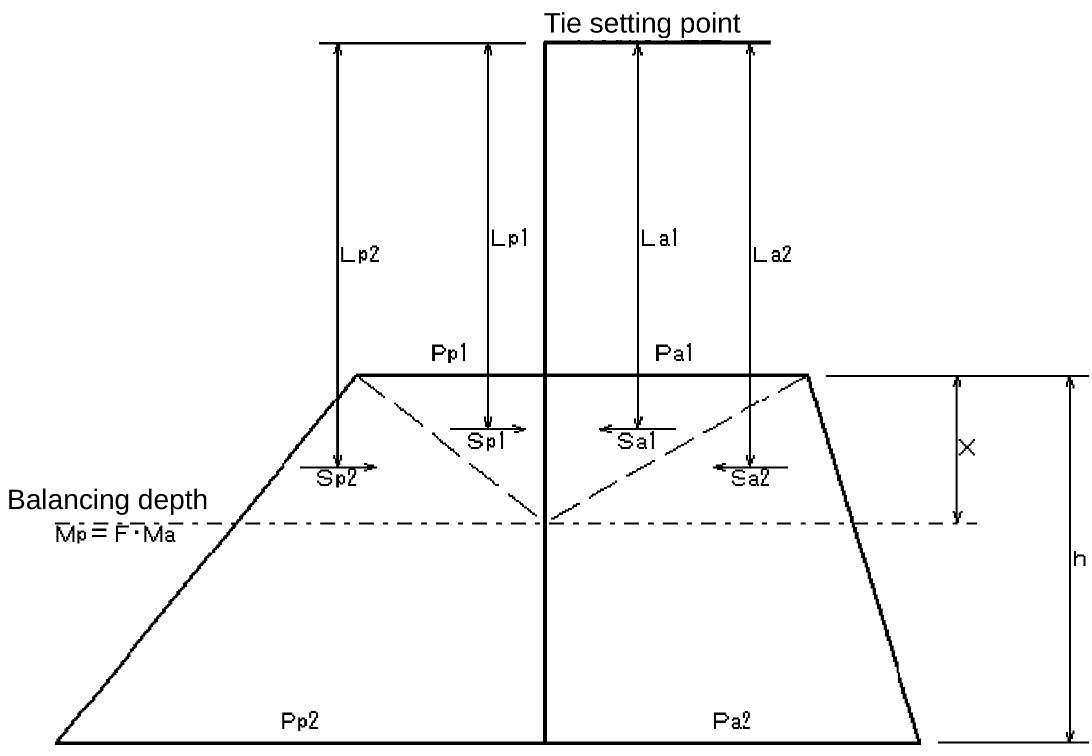
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-1.00	1/2x 23.000x 1.000	11.500	4.333	49.830	
2	-2.00	1/2x 27.000x 1.000	13.500	4.667	63.004	112.834
3	-2.00	1/2x 27.000x 1.000	13.500	5.333	71.996	
4	-3.00	1/2x 31.000x 1.000	15.500	5.667	87.838	272.668
5	-3.00	1/2x 31.000x 8.000	124.000	8.667	1074.708	
6	-11.00	1/2x 63.000x 8.000	252.000	11.333	2855.916	4203.292
7	-11.00	1/2x 95.000x 2.500	118.750	14.833	1761.419	
8	-13.50	1/2x 105.000x 2.500	131.250	15.667	2056.294	8021.005
9	-13.50	1/2x 170.000x 2.500	212.500	17.333	3683.262	
10	-16.00	1/2x 180.000x 2.500	225.000	18.167	4087.575	15791.842
11	-16.00	1/2x 155.000x 4.000	310.000	20.333	6303.230	
12	-20.00	1/2x 175.000x 4.000	350.000	21.667	7583.450	29678.522

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -11.00 m FOS×M_{a1} = 5369.514 > M_{p1} = 4203.292
at -13.50 m FOS×M_{a2} = 7131.438 > M_{p2} = 8021.005

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \quad L_{a1} = 14.000 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 269.584X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{2.500}X\right]X}{2} = 0.000X^2 + 19.256X \ , \quad L_{a2} = 14.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 269.584X$$

$$M_a = 0.000X^3 + 19.256X^2 + 539.168X + 4474.595$$

[Moment on passive side]

$$S_{p1} = \frac{95.000X}{2} = 47.500X \ , \quad L_{p1} = 14.000 + \frac{1}{3}X$$

$$M_{p1'} = 15.833X^2 + 665.000X$$

$$S_{p2} = \frac{\left[95.000 + \frac{105.000 - 95.000}{2.500}X\right]X}{2} = 2.000X^2 + 47.500X \ , \quad L_{p2} = 14.000 + \frac{2}{3}X$$

$$M_{p2'} = 1.333X^3 + 59.667X^2 + 665.000X$$

$$M_p = 1.333X^3 + 75.500X^2 + 1330.000X + 4203.292$$

[Balancing depth]

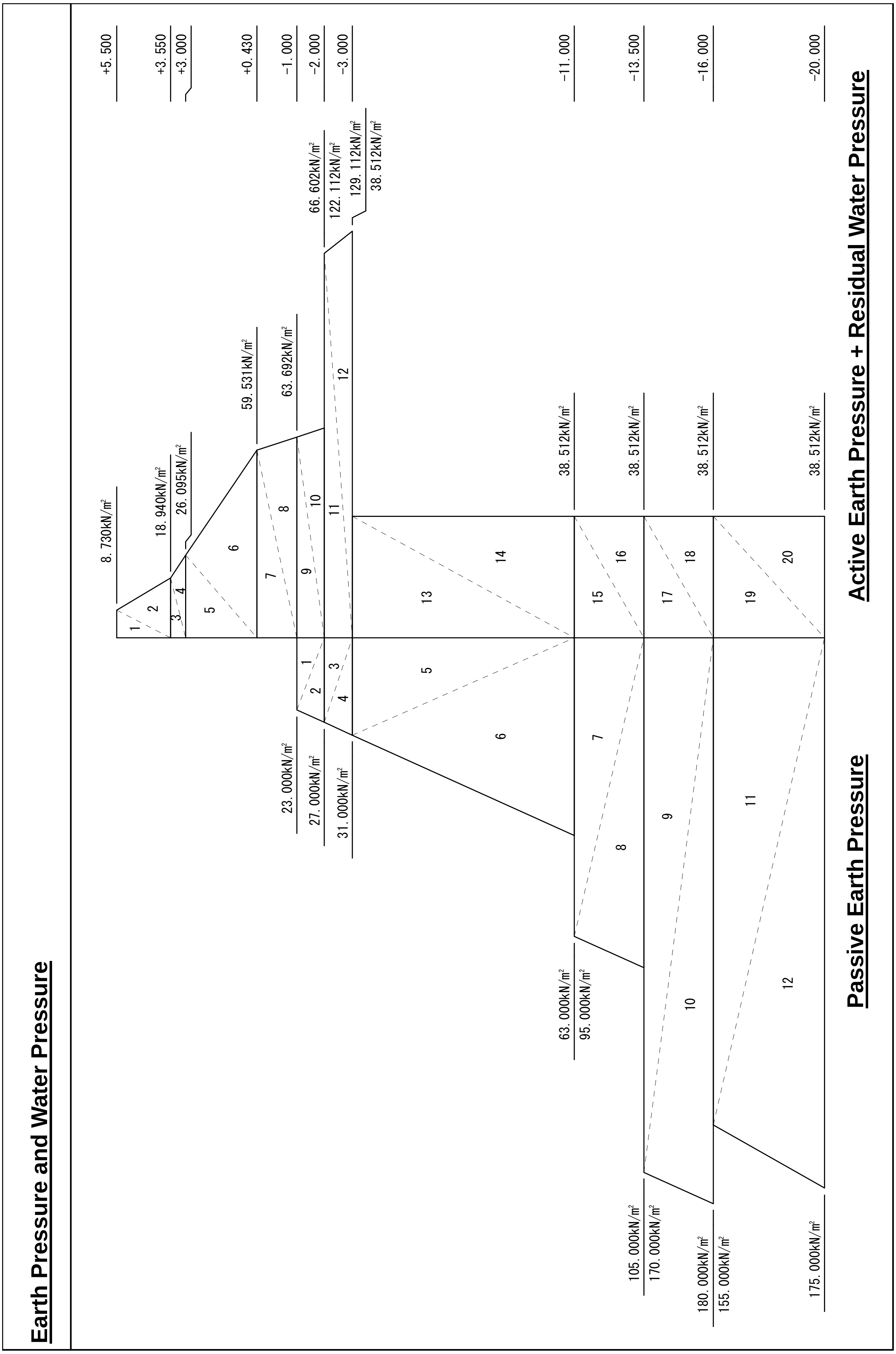
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 539.168X + 4474.595)$$

$$= 1.333X^3 + 75.500X^2 + 1330.000X + 4203.292$$

X= 1.523 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -11.000 – 1.523 = -12.523 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+3.550
+3.000
+0.430
-1.000
-2.000
-3.000
-11.000
-13.500
-16.000
-20.000
8.730kN/m²
18.940kN/m²
26.095kN/m²
59.531kN/m²
63.692kN/m²
66.602kN/m²
122.112kN/m²
129.112kN/m²
38.512kN/m²
23.000kN/m²
27.000kN/m²
31.000kN/m²
63.000kN/m²
95.000kN/m²
105.000kN/m²
170.000kN/m²
180.000kN/m²
155.000kN/m²
175.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 3.55	8.730 18.940	——— ———	8.730 18.940
3.55 3.00	18.940 26.095	——— ———	18.940 26.095
3.00 0.43	26.095 59.531	——— ———	26.095 59.531
0.43 -1.00	59.531 63.692	——— ———	59.531 63.692
-1.00 -2.00	63.692 66.602	23.000 27.000	40.692 39.602
-2.00 -3.00	122.112 129.112	27.000 31.000	95.112 98.112
-3.00 -11.00	38.512 38.512	31.000 63.000	7.512 -24.488
-11.00 -13.50	38.512 38.512	95.000 105.000	-56.488 -66.488
-13.50 -16.00	38.512 38.512	170.000 180.000	-131.488 -141.488
-16.00 -20.00	38.512 38.512	155.000 175.000	-116.488 -136.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $\quad P_{a1} = 38.512 \quad > \quad P_{p1} = 31.000$
at -11.00 m $\quad P_{a2} = 38.512 \quad < \quad P_{p2} = 63.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

- EL_{top} : the top elevation of the soil layer in question (m)
- P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)
- P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)
- P_{p1} : the passive earth pressure at the top of the soil layer (kPa)
- P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)
- h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{8.00 \times (31.000 - 38.512)}{(38.512 - 38.512) - (63.000 - 31.000)} = -4.878m$$

Accordingly, the virtual seabed level is obtained as -4.878m.

3) Moment about tie setting point

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 8.730x 1.950	8.512	-1.850	-15.747
2	1/2x 18.940x 1.950	18.466	-1.200	-22.159
3	1/2x 18.940x 0.550	5.208	-0.367	-1.911
4	1/2x 26.095x 0.550	7.176	-0.183	-1.313
5	1/2x 26.095x 2.570	33.532	0.857	28.737
6	1/2x 59.531x 2.570	76.497	1.713	131.039
7	1/2x 59.531x 1.430	42.565	3.047	129.696
8	1/2x 63.692x 1.430	45.540	3.523	160.437
9	1/2x 40.692x 1.000	20.346	4.333	88.159
10	1/2x 39.602x 1.000	19.801	4.667	92.411
11	1/2x 95.112x 1.000	47.556	5.333	253.616
12	1/2x 98.112x 1.000	49.056	5.667	278.000
13	1/2x 7.512x 1.878	7.054	6.626	46.740
Total		381.309	———	1167.705

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

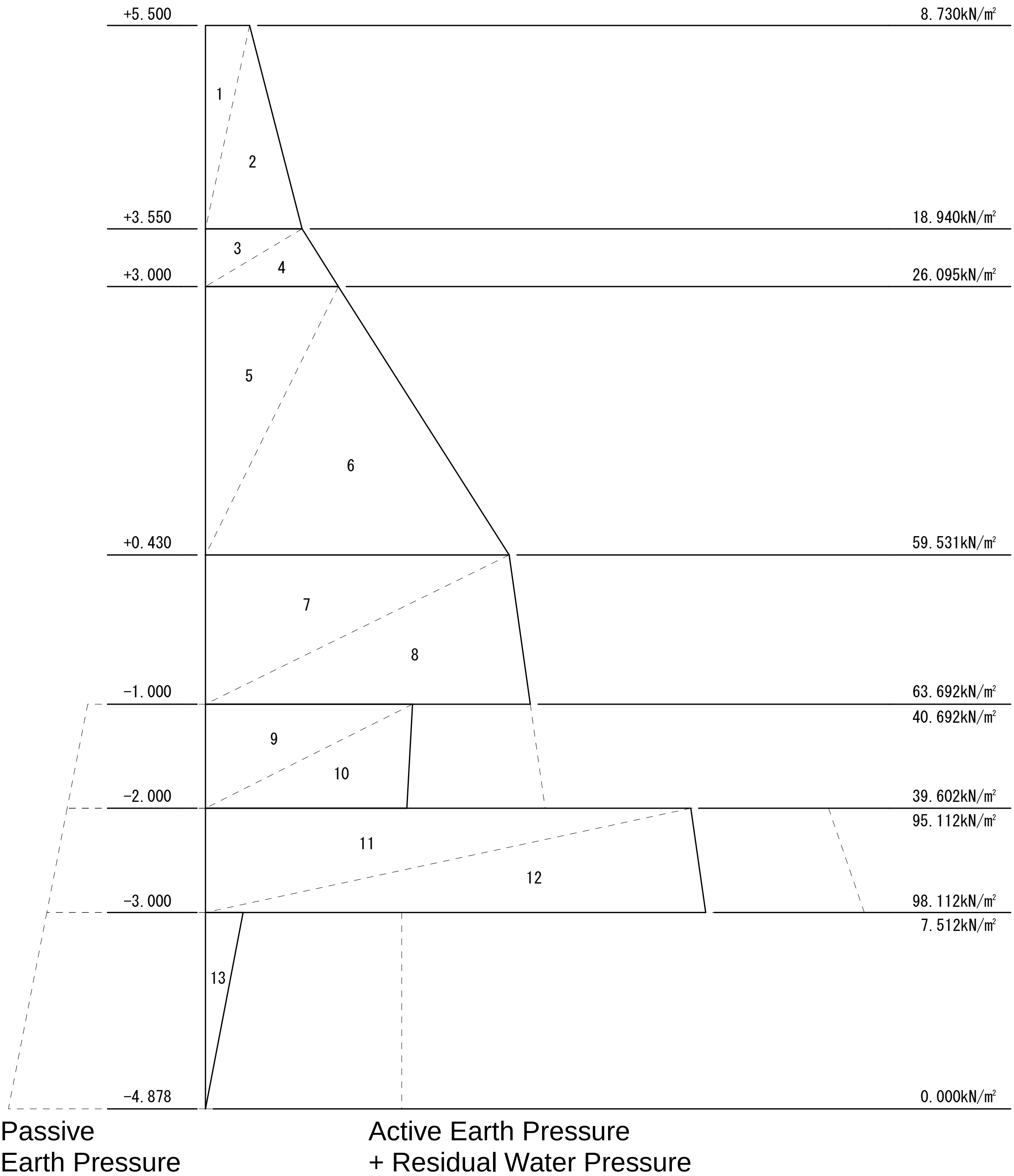
4) Reaction forces

Distance between supports : L_T = 3.000 – (-4.878) = 7.878 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1167.705}{7.878} = 148.224 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 381.309 - 148.224 = 233.085 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 3.55	8.512 18.466	26.978		-26.978
3.55 3.00	5.208 7.176	39.362	233.085	193.723
3.00 0.43	33.532 76.497	149.391	233.085	83.694
0.43 -1.00	42.565 45.540	237.496	233.085	-4.411
-1.00 -2.00	20.346 19.801	277.643	233.085	-44.558
-2.00 -3.00	47.556 49.056	374.255	233.085	-141.170
-3.00 -4.88	7.054 0.000	381.309	233.085	-148.224

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -1.000m.

$$Q = 83.694 - \frac{[59.531 + (59.531 + 2.910X)]X}{2} = 0$$

X = 1.361 m

Shear force zero point : DL = 0.430 – 1.361 = -0.931m

6) Calculation of moment

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	8.730x	1.950	-8.512	5.781	-49.208
2	1/2x	18.940x	1.950	-18.466	5.131	-94.749
3	1/2x	18.940x	0.550	-5.208	4.298	-22.384
4	1/2x	26.095x	0.550	-7.176	4.114	-29.522
5	1/2x	26.095x	2.570	-33.532	3.074	-103.077
6	1/2x	59.531x	2.570	-76.497	2.218	-169.670
7	1/2x	59.531x	1.361	-40.511	0.907	-36.743
8	1/2x	63.491x	1.361	-43.206	0.454	-19.616
Total				———	———	-524.969

where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

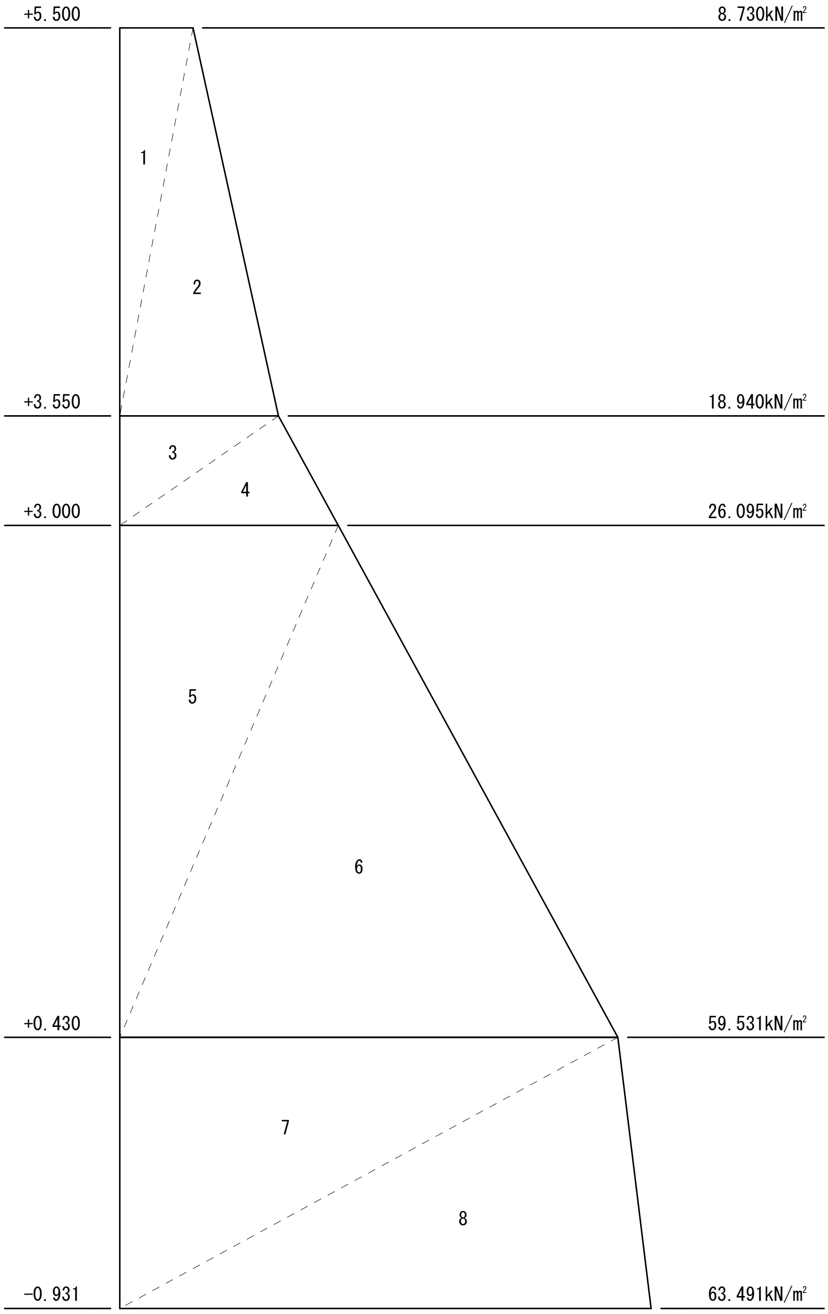
- Distance of the shear zero point to the tie setting point

  h = 3.000 – (-0.931) = 3.931 m

- Maximum bending moment

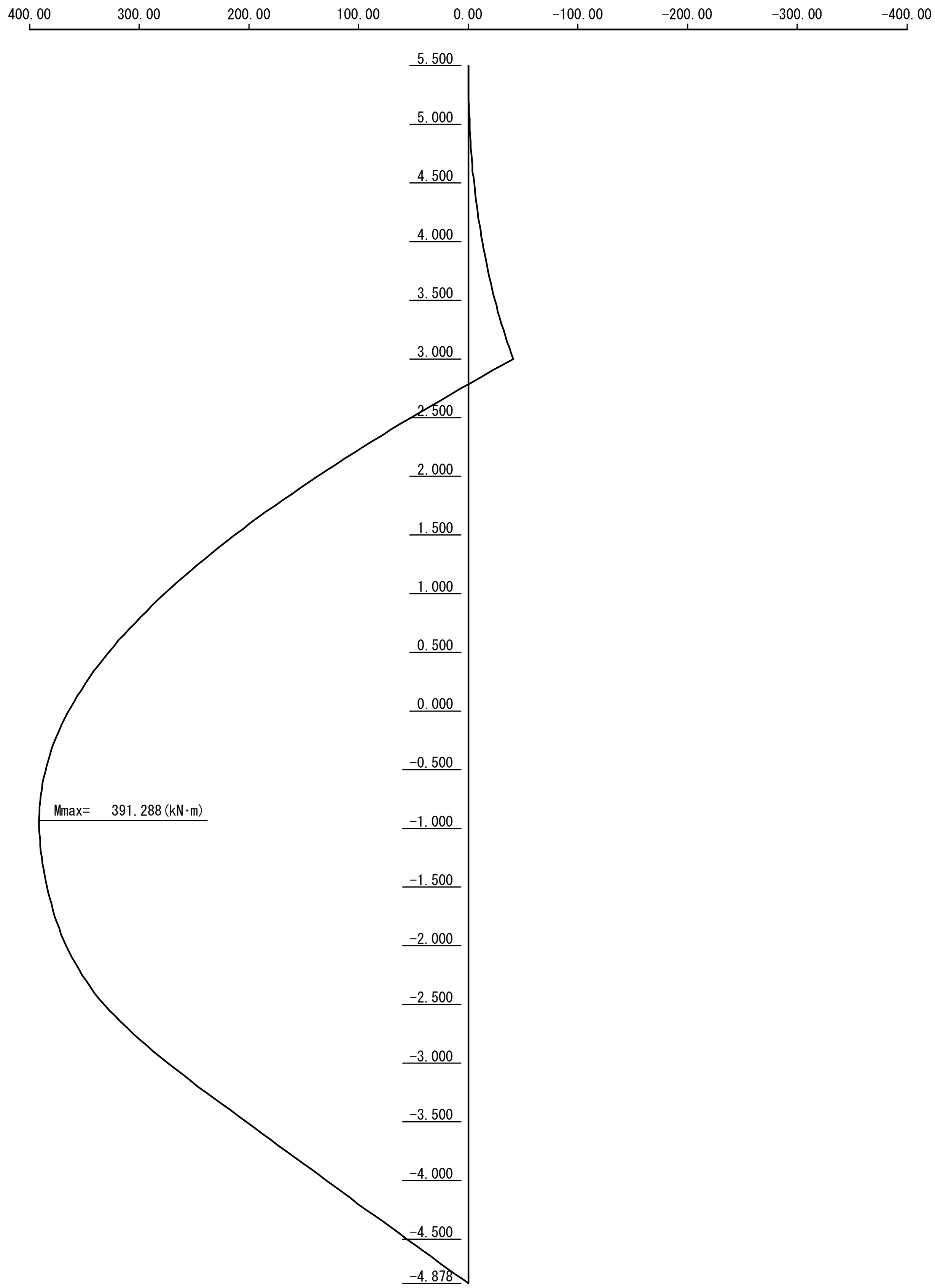
  $M_{max} = A_p \times h + \sum M$ = 233.085 ×3.931 – 524.969 = 391.288 kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -17.082 m --- adopted
- Free earth support method : -12.523 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-17.082) = 20.582m → 21.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 21.000 = -17.500 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 546.875 kN-m/m --- adopted
- Free earth support method : 391.288 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{546.875 \times 10^6}{4191 \times 10^3} = 130.5 MPa \quad \leq \quad \sigma_a = 140 MPa \quad .... \ OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 268.956 kN-m/m --- adopted
- Free earth support method : 233.085 kN-m/m

T = 268.956 × 1.96 × sec(0.000) × sec (0.000) = 527.154 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 527.154 × 3.8 = 2003.185 kN/wire

Tie wire shall have minimum tensile strength of 2004 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{527.154 \times 1.960}{10.0} = 103.322 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \le \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{103.322 \times 10^6}{2 \times 374.0 \times 10^3} = 138.1 MPa \quad \le \quad \sigma_a = 140 MPa \;\; .... \; OK$$

Appendix 7-8

Calculation Output for Steel Sheet Pipe Pile Wall (During Construction)

Output of Design Software for Anchor Wall (During Construction)
– Junction to Service Berth

1. Design Conditions

1-1 Dimensions

Ground elevation	+4.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-5.00 m
Angle of seabed slope	0.0˚

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	20 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

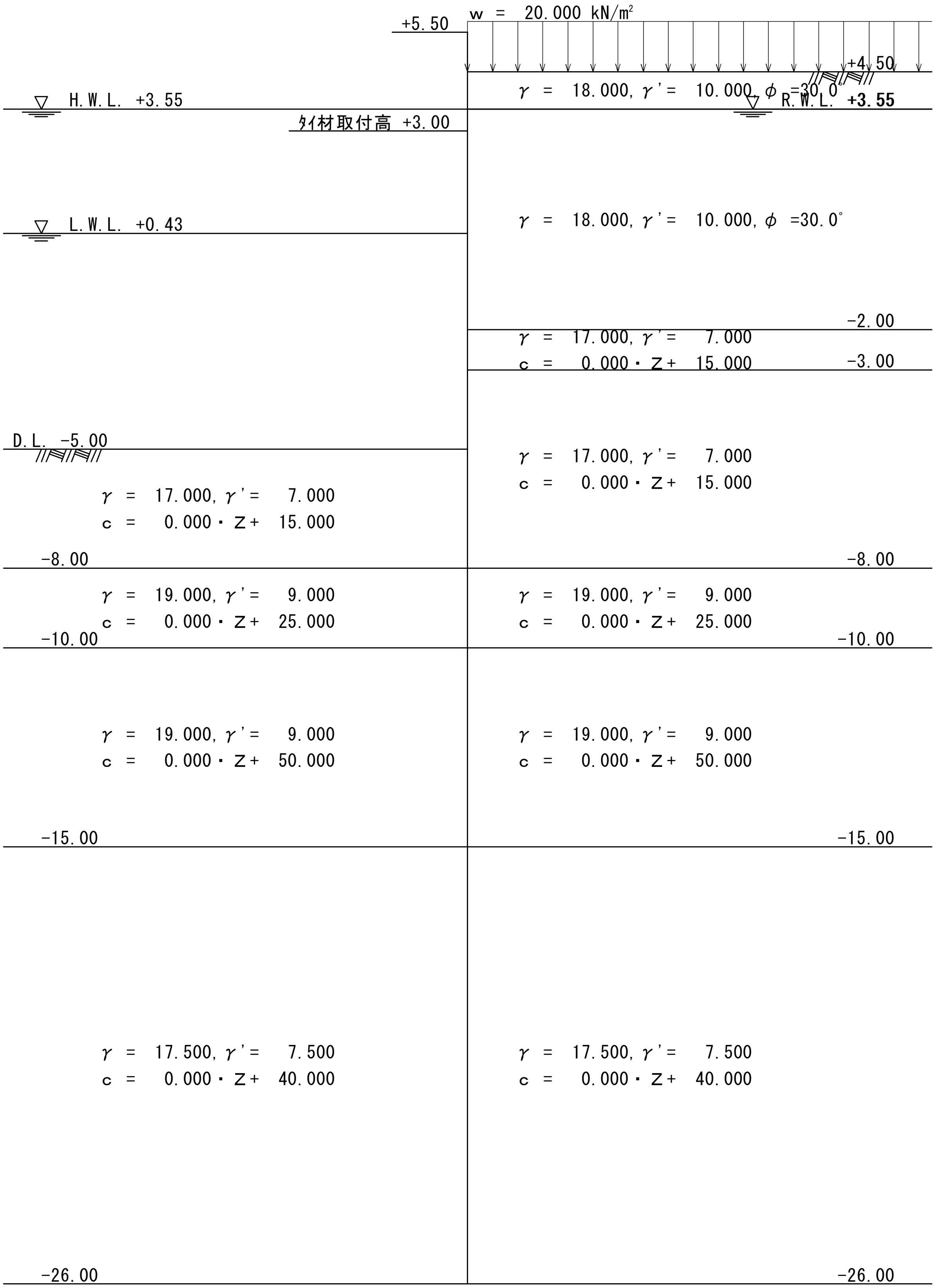
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
4.50 3.55	0.95	5.820 10.800
3.55 3.00	0.55	10.800 12.400
3.00 -2.00	5.00	12.400 26.950
-2.00 -3.00	1.00	62.600 69.600
-3.00 -8.00	5.00	7.000 7.000
-8.00 -10.00	2.00	7.000 7.000
-10.00 -15.00	5.00	7.000 7.000
-15.00 -26.00	11.00	7.000 7.000

(2) Residual water pressure

$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)

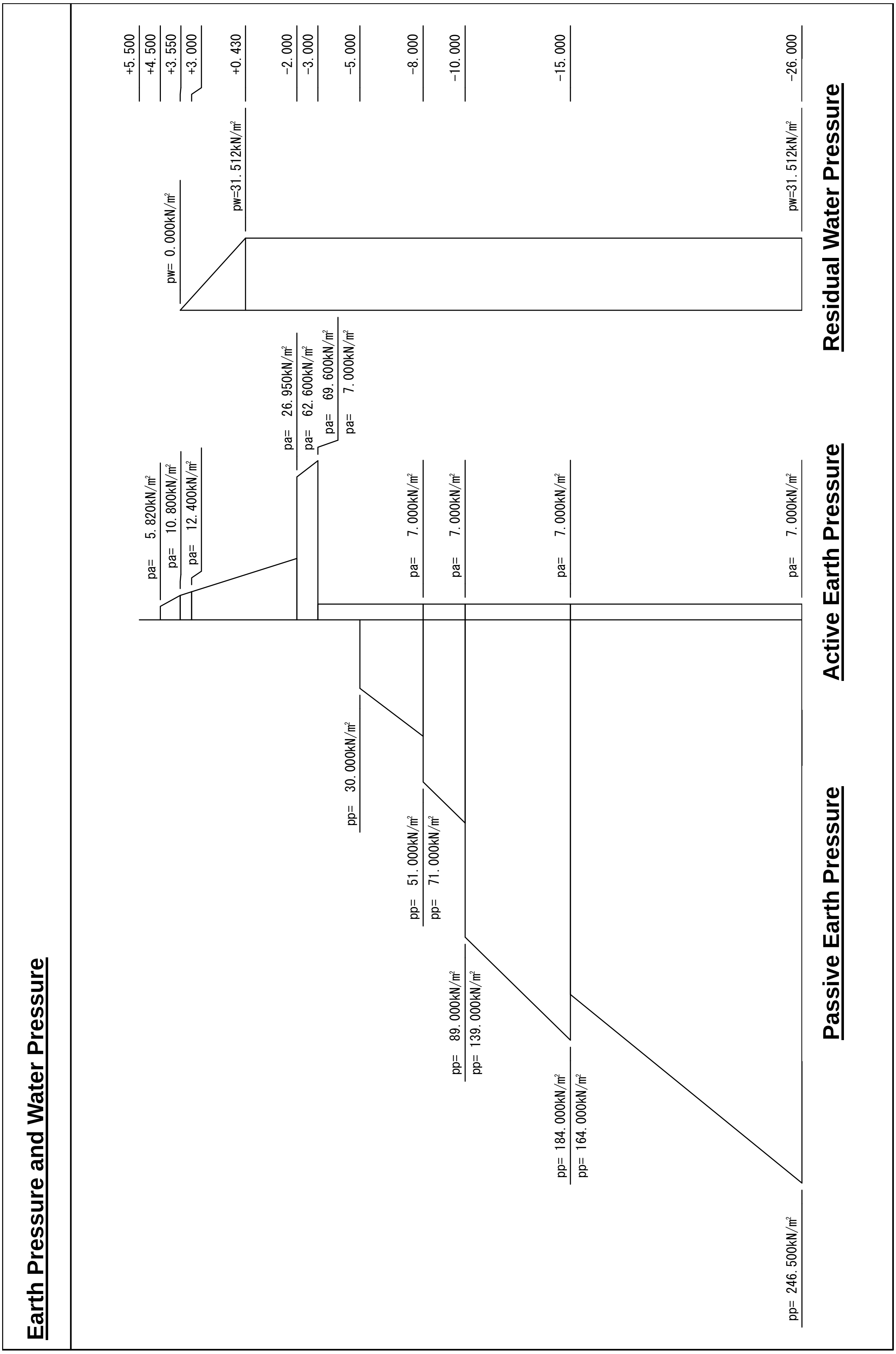
h_w is the height of the tidal lag (m)

γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-5.00 -8.00	3.00	30.000 51.000
-8.00 -10.00	2.00	71.000 89.000
-10.00 -15.00	5.00	139.000 184.000
-15.00 -26.00	11.00	164.000 246.500


Earth Pressure and Water Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-5.000
-8.000
-10.000
-15.000
-26.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
Residual Water Pressure
pa= 5.820kN/m²
pa= 10.800kN/m²
pa= 12.400kN/m²
pa= 26.950kN/m²
pa= 62.600kN/m²
pa= 69.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
Active Earth Pressure
pp= 30.000kN/m²
pp= 51.000kN/m²
pp= 71.000kN/m²
pp= 89.000kN/m²
pp= 139.000kN/m²
pp= 184.000kN/m²
pp= 164.000kN/m²
pp= 246.500kN/m²
Passive Earth Pressure

(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 4.50	0.000+ 0.000= 0.000 0.000+ 0.000= 0.000	―――― ――――
4.50 3.55	5.820+ 0.000= 5.820 10.800+ 0.000= 10.800	―――― ――――
3.55 3.00	10.800+ 0.000= 10.800 12.400+ 5.555= 17.955	―――― ――――
3.00 0.43	12.400+ 5.555= 17.955 19.879+ 31.512= 51.391	―――― ――――
0.43 -2.00	19.879+ 31.512= 51.391 26.950+ 31.512= 58.462	―――― ――――
-2.00 -3.00	62.600+ 31.512= 94.112 69.600+ 31.512= 101.112	―――― ――――
-3.00 -5.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	―――― ――――
-5.00 -8.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	30.000 51.000
-8.00 -10.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	71.000 89.000
-10.00 -15.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	139.000 184.000
-15.00 -26.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	164.000 246.500

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -5.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
−5.000	−1194.453	199.459	213.559
−6.000	−1607.766	223.532	194.498
−7.000	−1999.986	242.940	173.102
−8.000	−2355.793	258.321	148.733
−9.000	−2616.807	269.237	100.829
−10.000	−2714.084	275.282	48.796
−11.000	−2499.470	275.071	−55.981
−12.000	−1817.399	267.589	−162.487
−13.000	−612.608	253.637	−271.523
−14.000	1177.367	233.829	−383.703

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -13.00 and -14.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-13.383	0.000	246.716	-314.076

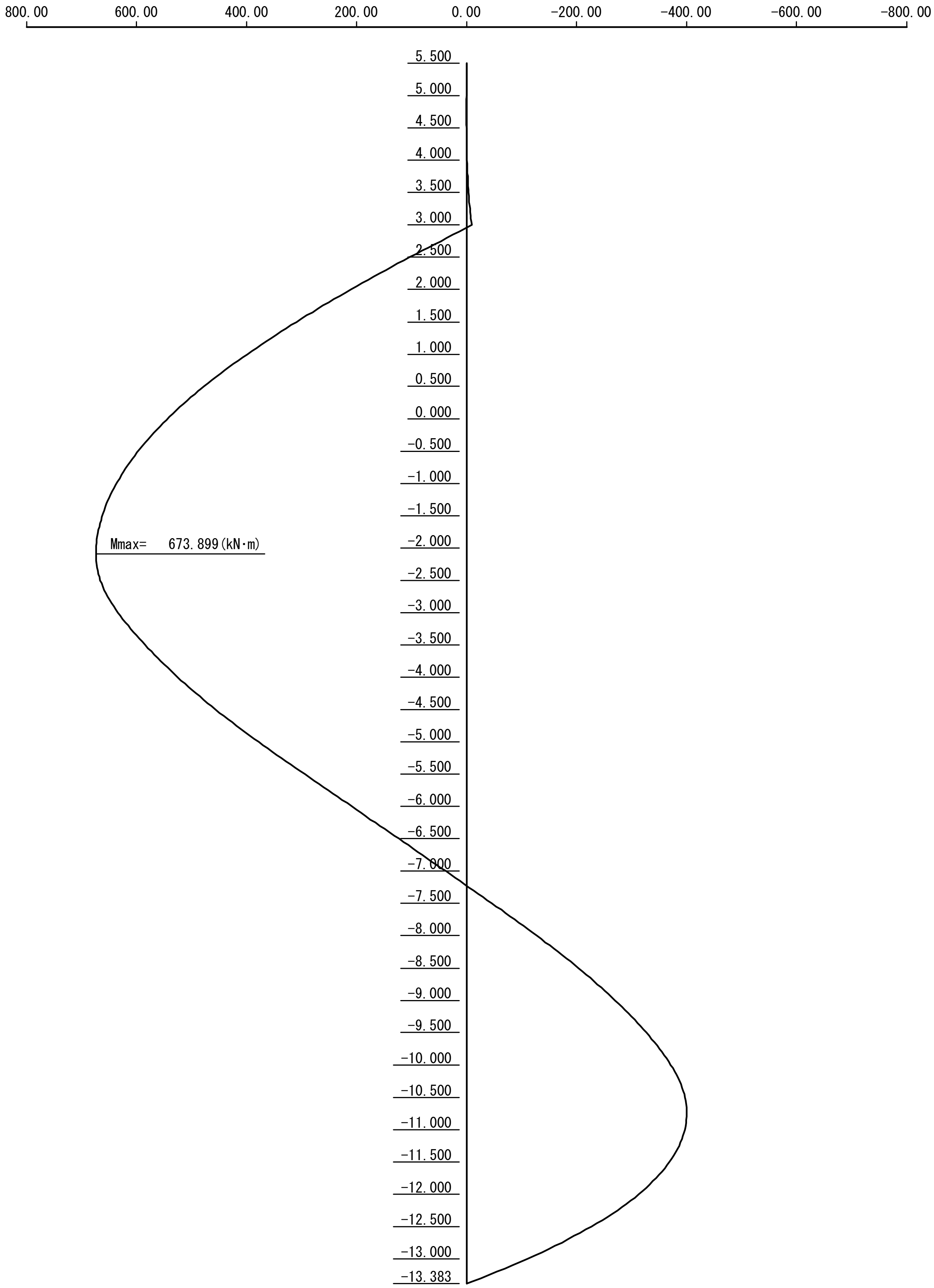
Embedded length of the pipe wall : L = 1.2 × (-5.000 + 13.383) = 10.060 m
Toe elevation : D = -5.000 – 10.060 = -15.060 m
Reaction force at tie setting point : 246.716 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	0.000	0.000
4.500	0.000	0.000
4.000	-0.837	-3.565
3.550	-3.375	-7.894
3.050	-8.944	-14.921
3.000	-9.712	-15.802
2.500	103.230	220.310
2.000	210.056	206.454
1.500	309.142	189.345
1.000	398.859	168.984
0.500	477.583	145.370
0.430	487.635	141.804
-0.070	552.052	115.745
-0.570	603.258	88.958
-1.070	640.889	61.444
-1.570	664.581	33.203
-2.000	673.531	8.333
-2.500	665.787	-39.598
-3.000	633.641	-89.279
-3.500	584.187	-108.535
-4.000	525.106	-127.791
-4.500	456.396	-147.047
-5.000	378.059	-166.303
-5.500	293.989	-169.684
-6.000	208.666	-171.315
-6.500	122.966	-171.196
-7.000	37.762	-169.327
-7.500	-46.070	-165.708
-8.000	-127.655	-160.339
-8.500	-203.576	-142.970
-9.000	-270.250	-123.351
-9.500	-326.552	-101.482
-10.000	-371.357	-77.363
-10.500	-397.290	-25.994
-11.000	-396.976	27.625
-11.500	-369.290	83.494
-12.000	-313.107	141.613
-12.500	-227.302	201.982
-13.000	-110.750	264.601
-13.383	0.000	314.076

Maximum bending moment : 673.899 kN-m/m
Elevation of maximum bending moment : -2.088 m
Depth of 1st steady point (moment=0) : -7.224 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

M_p = FOS×M_a

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
–	5.50	1/2x	0.000x	1.000	0.000	-2.167	0.000	
–	4.50	1/2x	0.000x	1.000	0.000	-1.833	0.000	0.000
1	4.50	1/2x	5.820x	0.950	2.764	-1.183	-3.270	
2	3.55	1/2x	10.800x	0.950	5.130	-0.867	-4.448	-9.262
3	3.55	1/2x	10.800x	0.550	2.970	-0.367	-1.090	
4	3.00	1/2x	17.955x	0.550	4.938	-0.183	-0.904	-11.655
5	3.00	1/2x	17.955x	2.570	23.072	0.857	19.773	
6	0.43	1/2x	51.391x	2.570	66.037	1.713	113.121	147.818
7	0.43	1/2x	51.391x	2.430	62.440	3.380	211.047	
8	-2.00	1/2x	58.462x	2.430	71.031	4.190	297.620	758.218
9	-2.00	1/2x	94.112x	1.000	47.056	5.333	250.950	
10	-3.00	1/2x	101.112x	1.000	50.556	5.667	286.501	1403.159
11	-3.00	1/2x	38.512x	2.000	38.512	6.667	256.760	
12	-5.00	1/2x	38.512x	2.000	38.512	7.333	282.408	2050.161
13	-5.00	1/2x	38.512x	3.000	57.768	9.000	519.912	
14	-8.00	1/2x	38.512x	3.000	57.768	10.000	577.680	3367.271
15	-8.00	1/2x	38.512x	2.000	38.512	11.667	449.320	
16	-10.00	1/2x	38.512x	2.000	38.512	12.333	474.968	4476.417
17	-10.00	1/2x	38.512x	5.000	96.280	14.667	1412.139	
18	-15.00	1/2x	38.512x	5.000	96.280	16.333	1572.541	8058.033
19	-15.00	1/2x	38.512x	11.000	211.816	21.667	4589.417	
20	-26.00	1/2x	38.512x	11.000	211.816	25.333	5365.935	20004.455

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

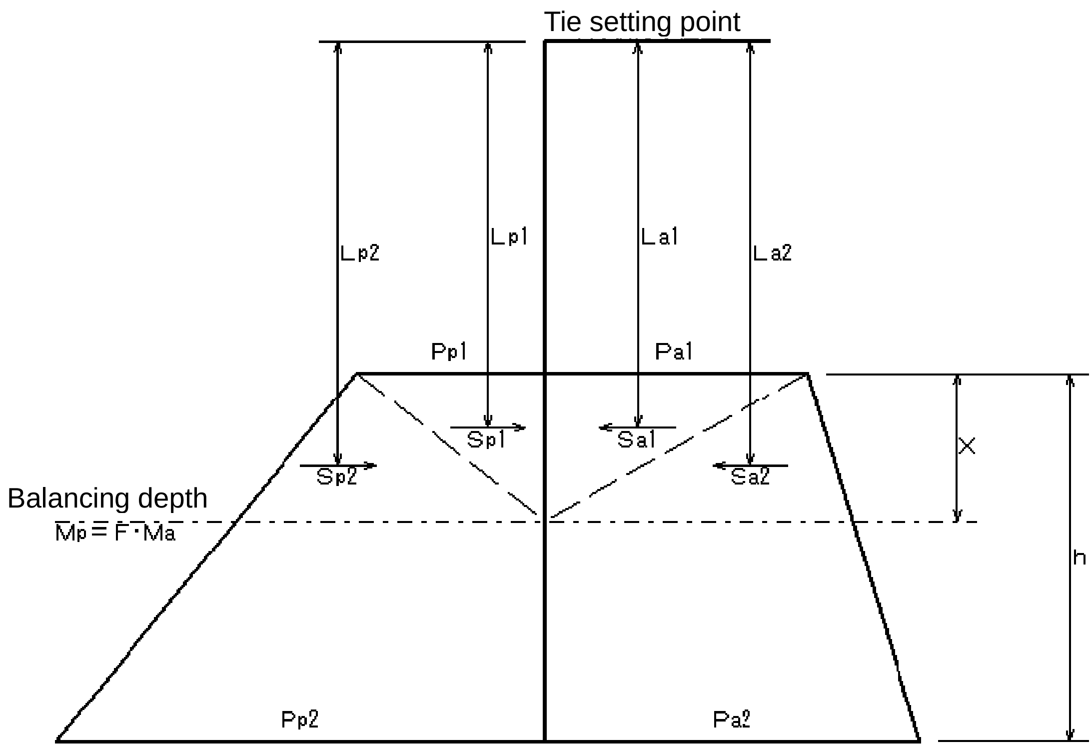
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-5.00	1/2x	30.000x	3.000	45.000	9.000	405.000	
2	-8.00	1/2x	51.000x	3.000	76.500	10.000	765.000	1170.000
3	-8.00	1/2x	71.000x	2.000	71.000	11.667	828.357	
4	-10.00	1/2x	89.000x	2.000	89.000	12.333	1097.637	3095.994
5	-10.00	1/2x	139.000x	5.000	347.500	14.667	5096.782	
6	-15.00	1/2x	184.000x	5.000	460.000	16.333	7513.180	15705.956
7	-15.00	1/2x	164.000x	11.000	902.000	21.667	19543.634	
8	-26.00	1/2x	246.500x	11.000	1355.750	25.333	34345.215	69594.805

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1} X}{2}, \quad L_{a(p)1} = L + \frac{1}{3} X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -10.00 m FOS×M_{a1} = 4476.417 > M_{p1} = 3095.994
at -15.00 m FOS×M_{a2} = 8058.033 < M_{p2} = 15705.956

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \text{ , } L_{a1} = 13.00 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 250.328X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{5.000}X\right]X}{2} = 0.000X^2 + 19.256X \text{ , } L_{a2} = 13.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 250.328X$$

$$M_a = 0.000X^3 + 19.256X^2 + 500.656X + 3730.348$$

[Moment on passive side]

$$S_{p1} = \frac{139.000X}{2} = 69.500X \text{ , } L_{p1} = 13.000 + \frac{1}{3}X$$

$$M_{p1'} = 23.167X^2 + 903.500X$$

$$S_{p2} = \frac{\left[139.000 + \frac{184.000 - 139.000}{5.000}X\right]X}{2} = 4.500X^2 + 69.500X \text{ , } L_{p2} = 13.000 + \frac{2}{3}X$$

$$M_{p2'} = 3.000X^3 + 104.833X^2 + 903.500X$$

$$M_p = 3.000X^3 + 128.000X^2 + 1807.000X + 3095.994$$

[Balancing depth]

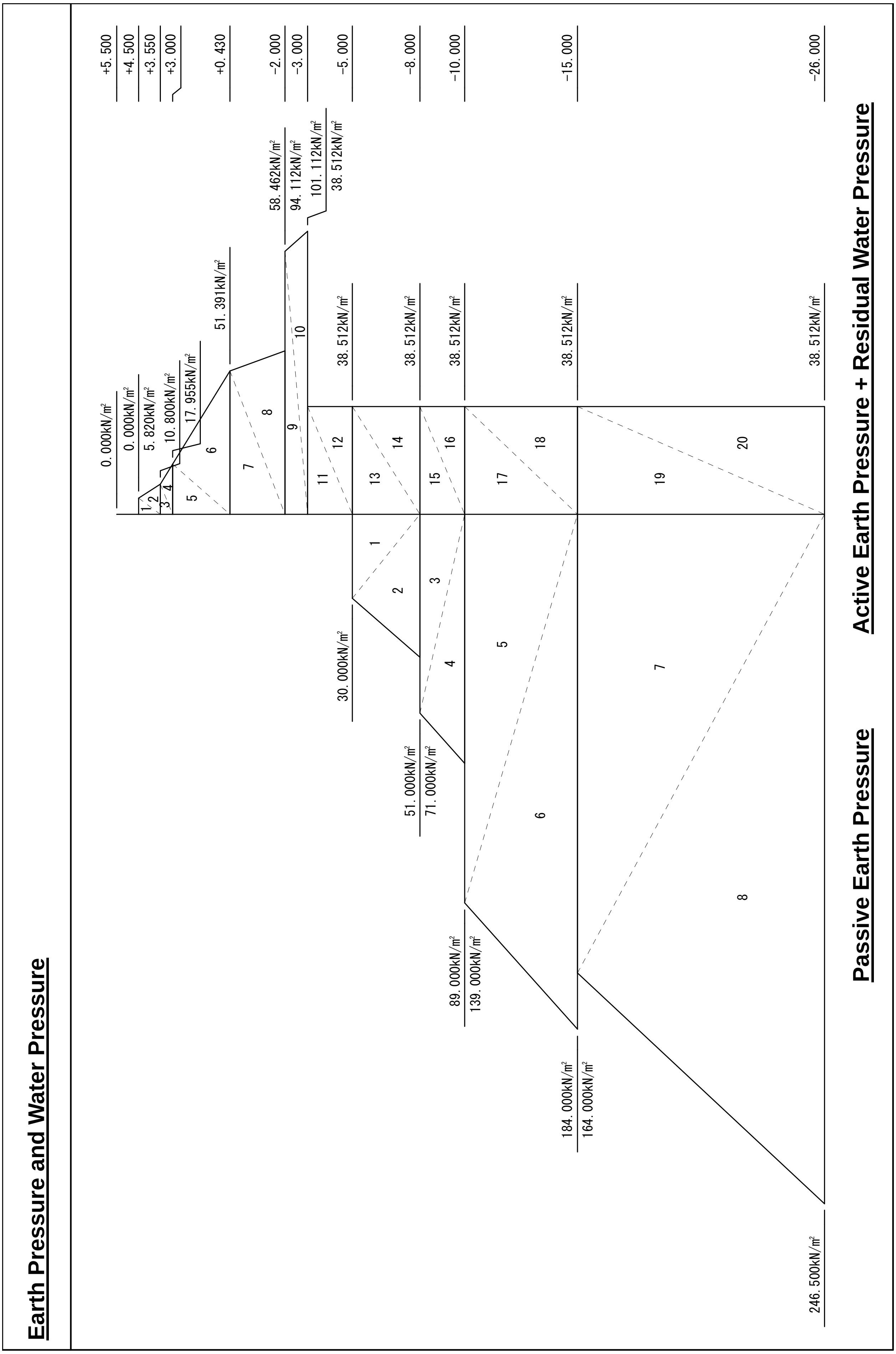
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 500.656X + 3730.348)$$

$$= 3.000X^3 + 128.000X^2 + 1807.000X + 3095.994$$

X= 1.046 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -10.000 – 1.046 = -11.046 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-5.000
-8.000
-10.000
-15.000
-26.000
0.000kN/m²
0.000kN/m²
5.820kN/m²
10.800kN/m²
17.955kN/m²
51.391kN/m²
58.462kN/m²
94.112kN/m²
101.112kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
30.000kN/m²
51.000kN/m²
71.000kN/m²
89.000kN/m²
139.000kN/m²
184.000kN/m²
164.000kN/m²
246.500kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 4.50	0.000 0.000	——— ———	0.000 0.000
4.50 3.55	5.820 10.800	——— ———	5.820 10.800
3.55 3.00	10.800 17.955	——— ———	10.800 17.955
3.00 0.43	17.955 51.391	——— ———	17.955 51.391
0.43 -2.00	51.391 58.462	——— ———	51.391 58.462
-2.00 -3.00	94.112 101.112	——— ———	94.112 101.112
-3.00 -5.00	38.512 38.512	——— ———	38.512 38.512
-5.00 -8.00	38.512 38.512	30.000 51.000	8.512 -12.488
-8.00 -10.00	38.512 38.512	71.000 89.000	-32.488 -50.488
-10.00 -15.00	38.512 38.512	139.000 184.000	-100.488 -145.488
-15.00 -26.00	38.512 38.512	164.000 246.500	-125.488 -207.988

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -5.00 m $P_{a1} = 38.512$ > $P_{p1} = 30.000$
at -8.00 m $P_{a2} = 38.512$ < $P_{p2} = 51.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

- EL_{top} : the top elevation of the soil layer in question (m)
- P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)
- P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)
- P_{p1} : the passive earth pressure at the top of the soil layer (kPa)
- P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)
- h : the thickness of the soil layer (m)

$$X = -5.00 - \frac{3.00 \times (30.000 - 38.512)}{(38.512 - 38.512) - (51.000 - 30.000)} = -6.216m$$

Accordingly, the virtual seabed level is obtained as -6.216m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	5.820x	0.950	2.764	-1.183	-3.270
2	1/2x	10.800x	0.950	5.130	-0.867	-4.448
3	1/2x	10.800x	0.550	2.970	-0.367	-1.090
4	1/2x	17.955x	0.550	4.938	-0.183	-0.904
5	1/2x	17.955x	2.570	23.072	0.857	19.773
6	1/2x	51.391x	2.570	66.037	1.713	113.121
7	1/2x	51.391x	2.430	62.440	3.380	211.047
8	1/2x	58.462x	2.430	71.031	4.190	297.620
9	1/2x	94.112x	1.000	47.056	5.333	250.950
10	1/2x	101.112x	1.000	50.556	5.667	286.501
11	1/2x	38.512x	2.000	38.512	6.667	256.760
12	1/2x	38.512x	2.000	38.512	7.333	282.408
13	1/2x	8.512x	1.216	5.175	8.405	43.496
Total				418.193	———	1751.964

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

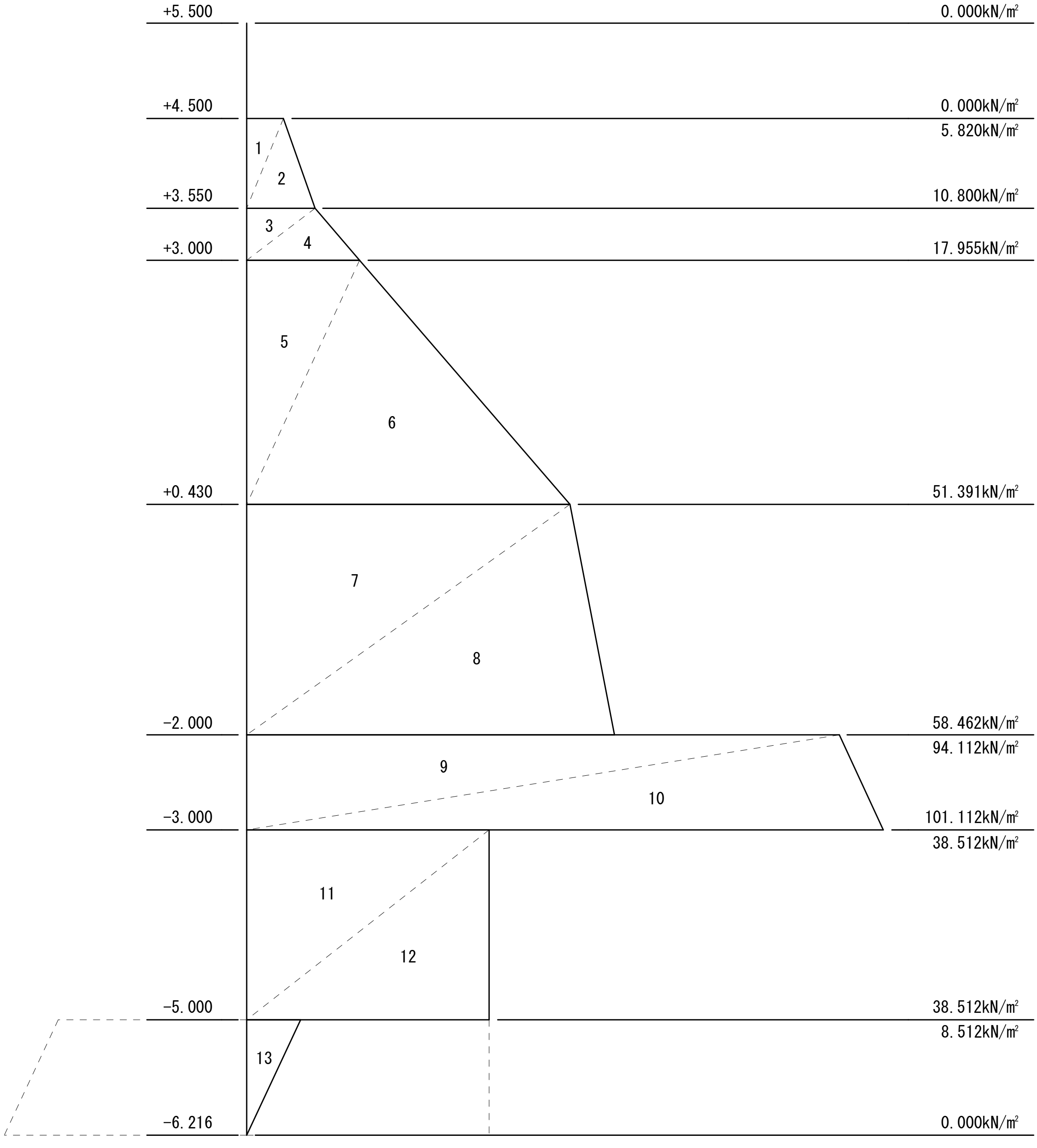
4) Reaction forces

Distance between supports : $L_T = 3.000 - (-6.216) = 9.216$ m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{1751.964}{9.216} = 190.100 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 418.193 - 190.100 = 228.093 kN/m$

External Force Diagram



Passive
Earth Pressure

Active Earth Pressure
+ Residual Water Pressure

5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 4.50	0.000 0.000	0.000		0.000
4.50 3.55	2.764 5.130	7.894		-7.894
3.55 3.00	2.970 4.938	15.802	228.093	212.291
3.00 0.43	23.072 66.037	104.911	228.093	123.182
0.43 -2.00	62.440 71.031	238.382	228.093	-10.289
-2.00 -3.00	47.056 50.556	335.994	228.093	-107.901
-3.00 -5.00	38.512 38.512	413.018	228.093	-184.925
-5.00 -6.22	5.175 0.000	418.193	228.093	-190.100

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -2.000m.

$$Q = 123.182 - \frac{[51.391 + (51.391 + 2.910X)]X}{2} = 0$$

X = 2.253 m

Shear force zero point : DL = 0.430 – 2.253 = -1.823m

6) Calculation of moment

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 5.820x 0.950	-2.764	6.006	-16.601
2	1/2x 10.800x 0.950	-5.130	5.690	-29.190
3	1/2x 10.800x 0.550	-2.970	5.190	-15.414
4	1/2x 17.955x 0.550	-4.938	5.006	-24.720
5	1/2x 17.955x 2.570	-23.072	3.966	-91.504
6	1/2x 51.391x 2.570	-66.037	3.110	-205.375
7	1/2x 51.391x 2.253	-57.892	1.502	-86.954
8	1/2x 57.947x 2.253	-65.277	0.751	-49.023
Total		———	———	-518.781

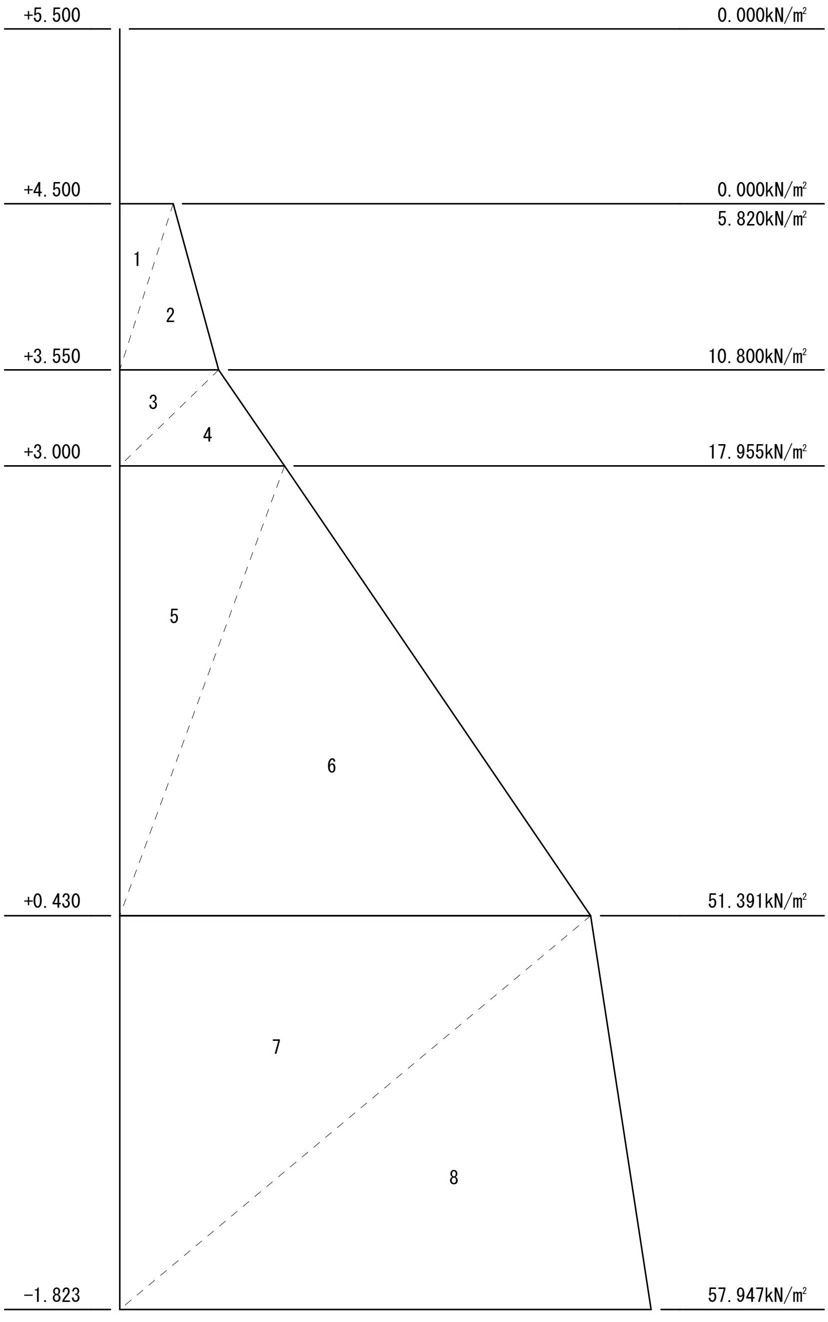
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-1.823) = 4.823 m

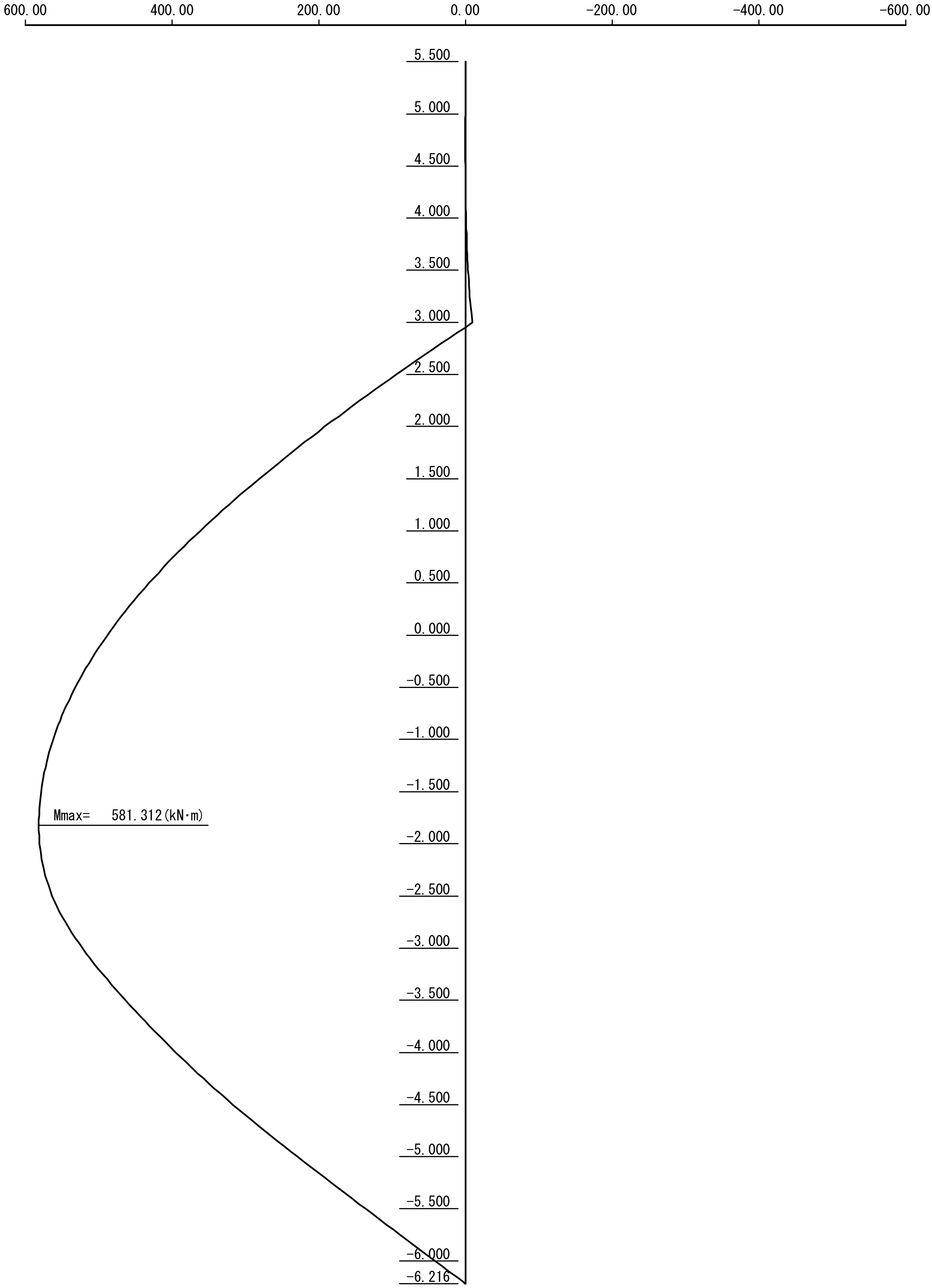
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 228.093 \times 4.823 - 518.781 = 581.312$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t14 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 267050 cm^4
Section modulus : Z = 6676 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 272500 cm^4/m
- Section modulus : Z = 6812 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 241898 cm^4/m
- Section modulus : Z = 6070 cm^3/m

(2) Embedded depth of pipe wall
Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -15.060 m --- adopted
- Free earth support method : -11.046 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-15.060) = 18.560m → 19.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 19.000 = -15.500 m

(3) Stress on pipe wall section
Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 673.899 kN-m/m --- adopted
- Free earth support method : 581.312 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{673.899 \times 10^6}{6070 \times 10^3} = 111.0 MPa \quad \leq \quad \sigma_a = 140 MPa \ \ .... \ OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 246.716 kN-m/m --- adopted
- Free earth support method : 228.093 kN-m/m

T = 246.716 × 1.960 × sec(0.000) × sec (0.000) = 483.563 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 362.673 × 3.8 = 1837.539 kN/wire

Tie wire shall have minimum tensile strength of <u>1838 kN</u>.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{483.563 \times 1.960}{10.0} = 94.778 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[300×90×10.0×15.5

Steel grade : SS400
Section modulus : Z = 494.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{94.778 \times 10^6}{2 \times 494.0 \times 10^3} = 95.9 MPa \quad \leq \quad \sigma_a = 140 MPa \ \ .... \ OK$$

Output of Design Software for Anchor Wall (During Construction) – Revetment Block a

1. Design Conditions

1-1 Dimensions

Ground elevation	+4.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-2.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	20 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

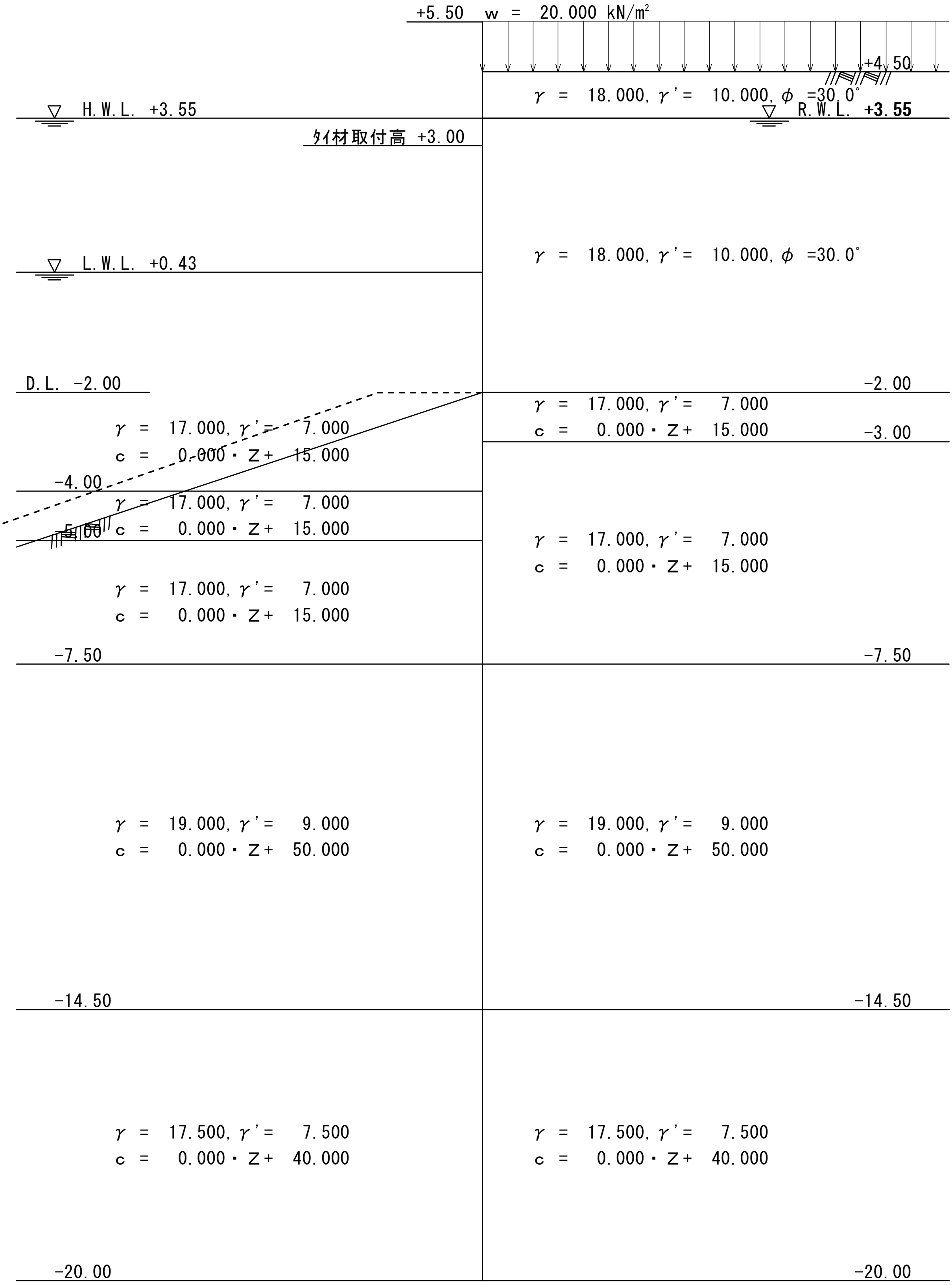
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
4.50 3.55	0.95	5.820 10.800
3.55 3.00	0.55	10.800 12.400
3.00 -2.00	5.00	12.400 26.950
-2.00 -3.00	1.00	62.600 69.600
-3.00 -7.50	4.50	7.000 7.000
-7.50 -12.00	4.50	7.000 7.000
-12.00 -14.50	2.50	7.000 7.000
-14.50 -20.00	5.50	7.000 7.000

(2) Residual water pressure

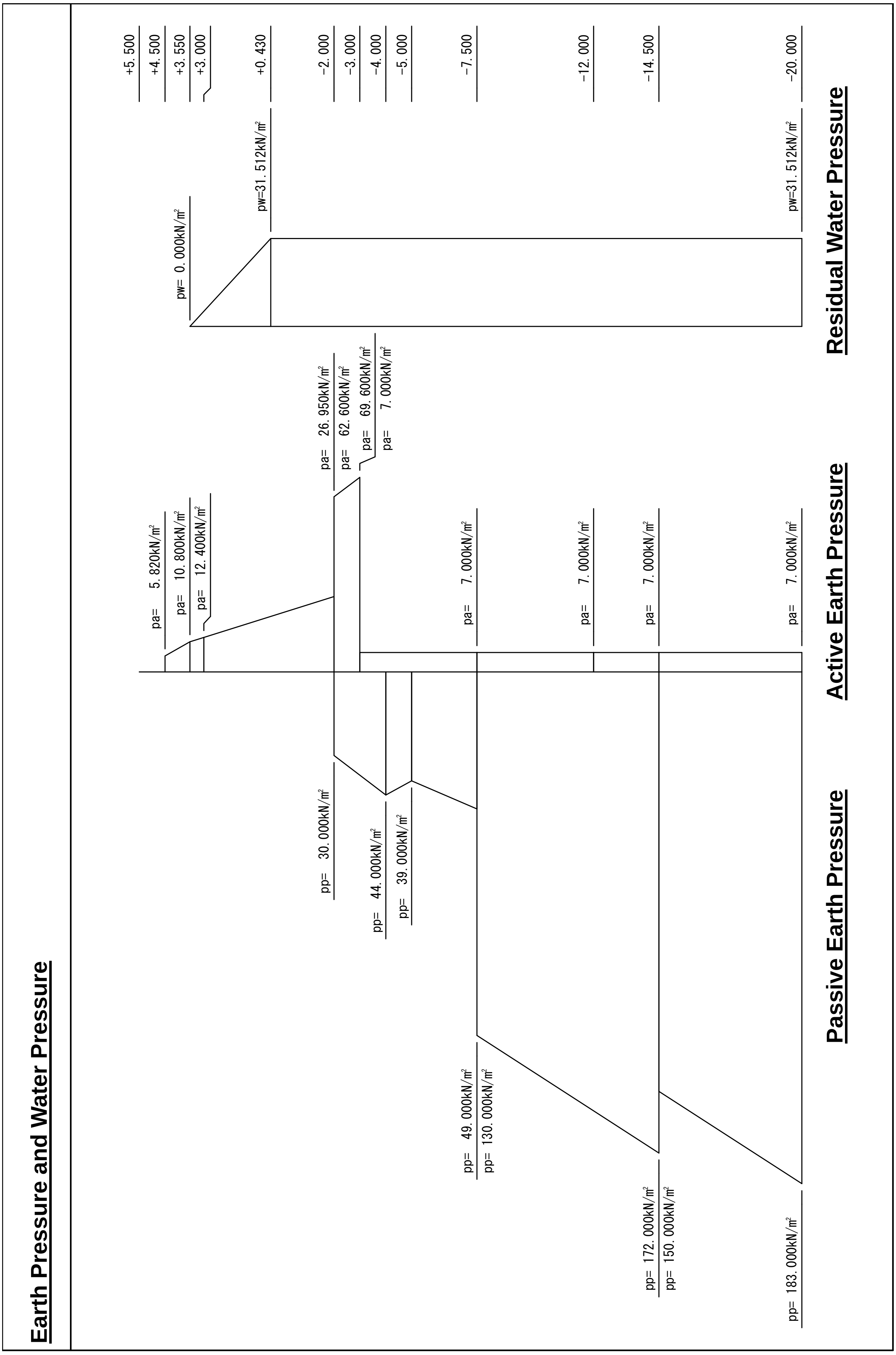
$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-2.00 -4.00	2.00	30.000 44.000
-4.00 -5.00	1.00	44.000 39.000
-5.00 -7.50	2.50	39.000 49.000
-7.50 -14.50	7.00	130.000 172.000
-14.50 -20.00	5.50	150.000 183.000


Earth Pressure and Water Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-7.500
-12.000
-14.500
-20.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
pa= 5.820kN/m²
pa= 10.800kN/m²
pa= 12.400kN/m²
pa= 26.950kN/m²
pa= 62.600kN/m²
pa= 69.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pp= 30.000kN/m²
pp= 44.000kN/m²
pp= 39.000kN/m²
pp= 49.000kN/m²
pp= 130.000kN/m²
pp= 172.000kN/m²
pp= 150.000kN/m²
pp= 183.000kN/m²
Passive Earth Pressure
Active Earth Pressure
Residual Water Pressure

(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 4.50	0.000+ 0.000= 0.000 0.000+ 0.000= 0.000	——— ———
4.50 3.55	5.820+ 0.000= 5.820 10.800+ 0.000= 10.800	——— ———
3.55 3.00	10.800+ 0.000= 10.800 12.400+ 5.555= 17.955	——— ———
3.00 0.43	12.400+ 5.555= 17.955 19.879+ 31.512= 51.391	——— ———
0.43 -2.00	19.879+ 31.512= 51.391 26.950+ 31.512= 58.462	——— ———
-2.00 -3.00	62.600+ 31.512= 94.112 69.600+ 31.512= 101.112	30.000 37.000
-3.00 -4.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	37.000 44.000
-4.00 -5.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	44.000 39.000
-5.00 -7.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	39.000 49.000
-7.50 -12.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	130.000 157.000
-12.00 -14.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	157.000 172.000
-14.50 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	150.000 183.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -2.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-2.000	-239.109	112.012	126.370
-3.000	-446.199	138.417	164.077
-4.000	-686.260	161.798	138.708
-5.000	-899.495	178.897	118.621
-6.000	-1095.816	191.976	103.054
-7.000	-1271.134	201.990	86.552
-8.000	-1378.909	208.488	28.816
-9.000	-1187.951	206.869	-67.053
-10.000	-633.294	197.770	-161.442
-11.000	321.991	182.363	-255.523

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -10.00 and -11.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-10.709	0.000	187.448	-228.100

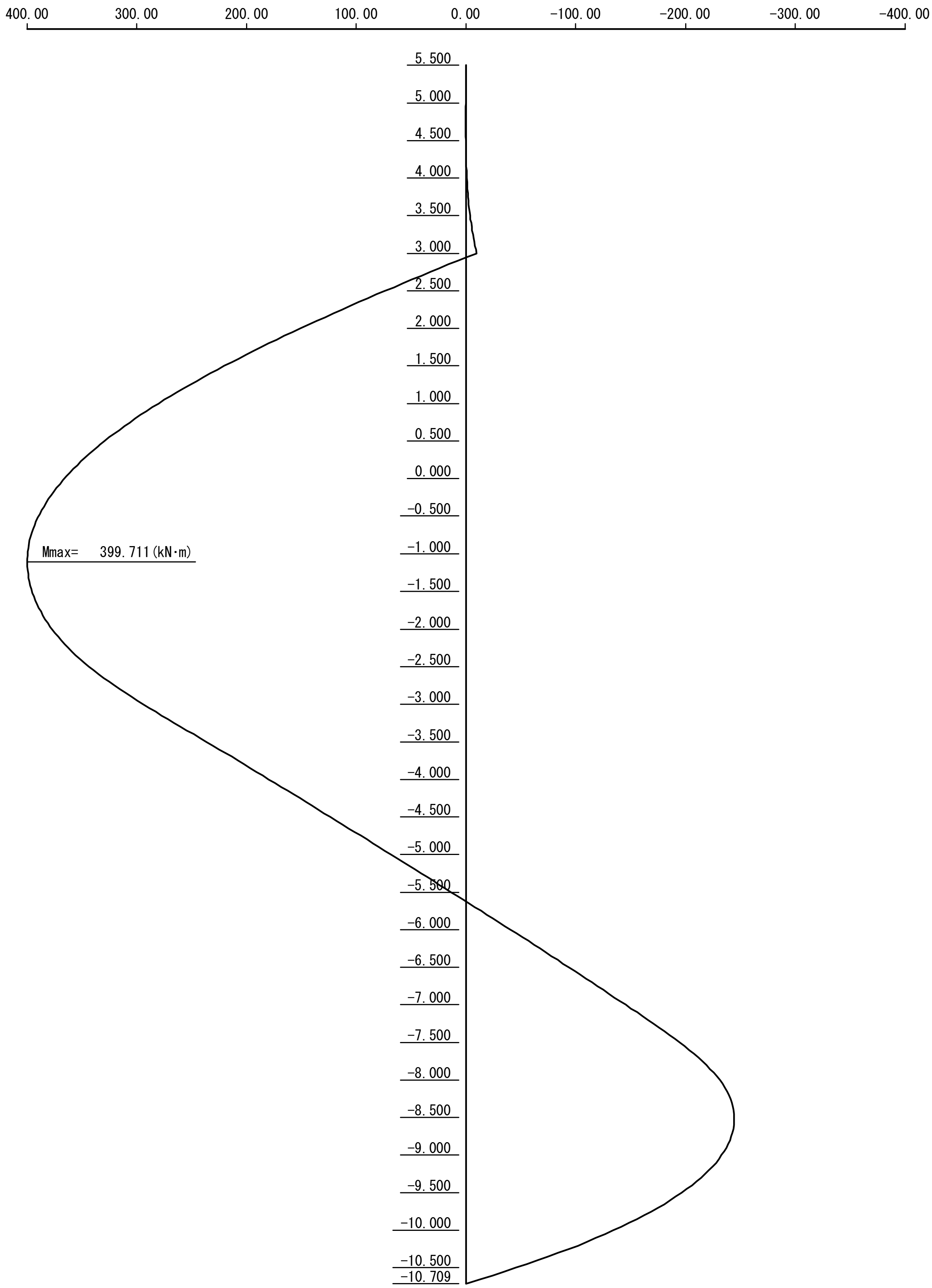
Embedded length of the pipe wall : L = 1.2 × (-2.000 + 10.709) = 10.451 m
Toe elevation : D = -2.000 – 10.451 = -12.451 m
Reaction force at tie setting point : 187.448 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	0.000	0.000
4.500	0.000	0.000
4.000	-0.837	-3.565
3.550	-3.375	-7.894
3.050	-8.944	-14.921
3.000	-9.712	-15.802
2.500	73.596	161.042
2.000	150.788	147.186
1.500	220.240	130.077
1.000	280.323	109.716
0.500	329.413	86.102
0.430	335.316	82.536
-0.070	370.100	56.477
-0.570	391.672	29.690
-1.070	399.669	2.176
-1.570	393.727	-26.065
-2.000	377.191	-50.935
-2.500	343.709	-82.991
-3.000	294.200	-115.047
-3.500	236.633	-114.928
-4.000	179.563	-113.059
-4.500	123.615	-110.940
-5.000	68.415	-110.071
-5.500	13.524	-109.327
-6.000	-40.746	-107.583
-6.500	-93.893	-104.839
-7.000	-145.418	-101.095
-7.500	-194.821	-96.351
-8.000	-231.436	-49.857
-8.500	-244.429	-1.863
-9.000	-233.049	47.631
-9.500	-196.548	98.625
-10.000	-134.174	151.119
-10.500	-45.179	205.113
-10.709	0.000	228.100

Maximum bending moment : 399.711 kN-m/m
Elevation of maximum bending moment : -1.109 m
Depth of 1st steady point (moment=0) : -5.624 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
―	5.50	1/2x	0.000x	1.000	0.000	-2.167	0.000	
―	4.50	1/2x	0.000x	1.000	0.000	-1.833	0.000	0.000
1	4.50	1/2x	5.820x	0.950	2.764	-1.183	-3.270	
2	3.55	1/2x	10.800x	0.950	5.130	-0.867	-4.448	-9.262
3	3.55	1/2x	10.800x	0.550	2.970	-0.367	-1.090	
4	3.00	1/2x	17.955x	0.550	4.938	-0.183	-0.904	-11.655
5	3.00	1/2x	17.955x	2.570	23.072	0.857	19.773	
6	0.43	1/2x	51.391x	2.570	66.037	1.713	113.121	147.818
7	0.43	1/2x	51.391x	2.430	62.440	3.380	211.047	
8	-2.00	1/2x	58.462x	2.430	71.031	4.190	297.620	758.218
9	-2.00	1/2x	94.112x	1.000	47.056	5.333	250.950	
10	-3.00	1/2x	101.112x	1.000	50.556	5.667	286.501	1403.159
11	-3.00	1/2x	38.512x	1.000	19.256	6.333	121.948	
12	-4.00	1/2x	38.512x	1.000	19.256	6.667	128.380	1703.553
13	-4.00	1/2x	38.512x	1.000	19.256	7.333	141.204	
14	-5.00	1/2x	38.512x	1.000	19.256	7.667	147.636	2050.161
15	-5.00	1/2x	38.512x	2.500	48.140	8.833	425.221	
16	-7.50	1/2x	38.512x	2.500	48.140	9.667	465.369	3118.869
17	-7.50	1/2x	38.512x	4.500	86.652	12.000	1039.824	
18	-12.00	1/2x	38.512x	4.500	86.652	13.500	1169.802	5770.420
19	-12.00	1/2x	38.512x	2.500	48.140	15.833	762.201	
20	-14.50	1/2x	38.512x	2.500	48.140	16.667	802.349	7647.880
21	-14.50	1/2x	38.512x	5.500	105.908	19.333	2047.519	
22	-20.00	1/2x	38.512x	5.500	105.908	21.167	2241.755	12795.009

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

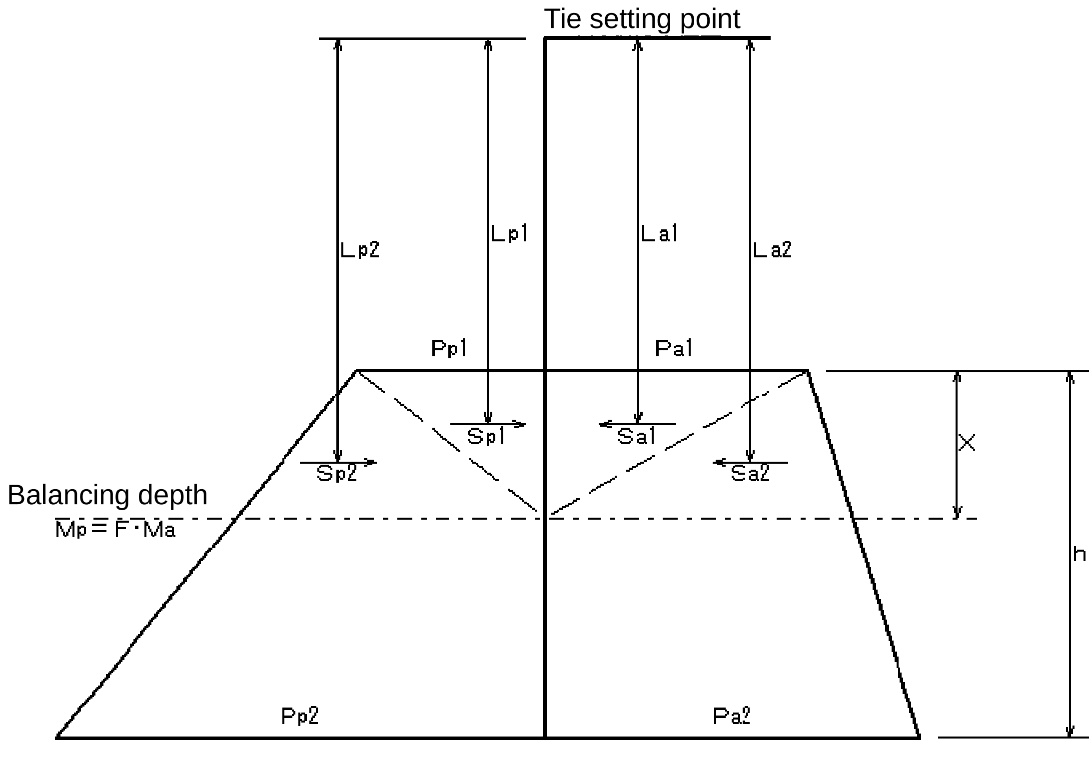
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-2.00	1/2x 30.000x 1.000	15.000	5.333	79.995	
2	-3.00	1/2x 37.000x 1.000	18.500	5.667	104.840	184.835
3	-3.00	1/2x 37.000x 1.000	18.500	6.333	117.160	
4	-4.00	1/2x 44.000x 1.000	22.000	6.667	146.674	448.669
5	-4.00	1/2x 44.000x 1.000	22.000	7.333	161.326	
6	-5.00	1/2x 39.000x 1.000	19.500	7.667	149.506	759.501
7	-5.00	1/2x 39.000x 2.500	48.750	8.833	430.609	
8	-7.50	1/2x 49.000x 2.500	61.250	9.667	592.104	1782.214
9	-7.50	1/2x 130.000x 4.500	292.500	12.000	3510.000	
10	-12.00	1/2x 157.000x 4.500	353.250	13.500	4768.875	10061.089
11	-12.00	1/2x 157.000x 2.500	196.250	15.833	3107.226	
12	-14.50	1/2x 172.000x 2.500	215.000	16.667	3583.405	16751.720
13	-14.50	1/2x 150.000x 5.500	412.500	19.333	7974.862	
14	-20.00	1/2x 183.000x 5.500	503.250	21.167	10652.293	35378.875

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$	: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)
$M_{a(p)1}$	: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)
$M_{a(p)1'}$	: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$M_{a(p)2'}$	: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$S_{a(p)1}$	: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$S_{a(p)2}$	: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$L_{a(p)1}$	: the arm length of $S_{a(p)1}$ from the tie setting point (m)
$L_{a(p)2}$	: the arm length of $S_{a(p)2}$ from the tie setting point (m)
$P_{a(p)1}$	: the active (passive) side pressure at the top elevation of the soil layer (kPa)
$P_{a(p)1}$	: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)
h	: the thickness of the soil layer (m)
L	: the vertical distance from the tie setting point to the top elevation of the soil layer (m)
X	: the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -7.50 m	FOS×M_{a1} = 3118.869	> M_{p1} = 1782.214
at -12.00 m	FOS×M_{a2} = 5770.420	< M_{p2} = 10061.089

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \quad L_{a1} = 10.50 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 202.188X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{3.500}X\right]X}{2} = 0.000X^2 + 19.256X \ , \quad L_{a2} = 10.500 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 202.188X$$

$$M_a = 0.000X^3 + 19.256X^2 + 404.376X + 2599.058$$

[Moment on passive side]

$$S_{p1} = \frac{130.000X}{2} = 65.000X \ , \quad L_{p1} = 10.500 + \frac{1}{3}X$$

$$M_{p1'} = 21.667X^2 + 682.500X$$

$$S_{p2} = \frac{\left[130.000 + \frac{151.000 - 130.000}{3.500}X\right]X}{2} = 3.000X^2 + 65.000X \ , \quad L_{p2} = 10.500 + \frac{2}{3}X$$

$$M_{p2'} = 2.000X^3 + 74.833X^2 + 682.500X$$

$$M_p = 2.000X^3 + 96.500X^2 + 1365.000X + 1782.214$$

[Balancing depth]

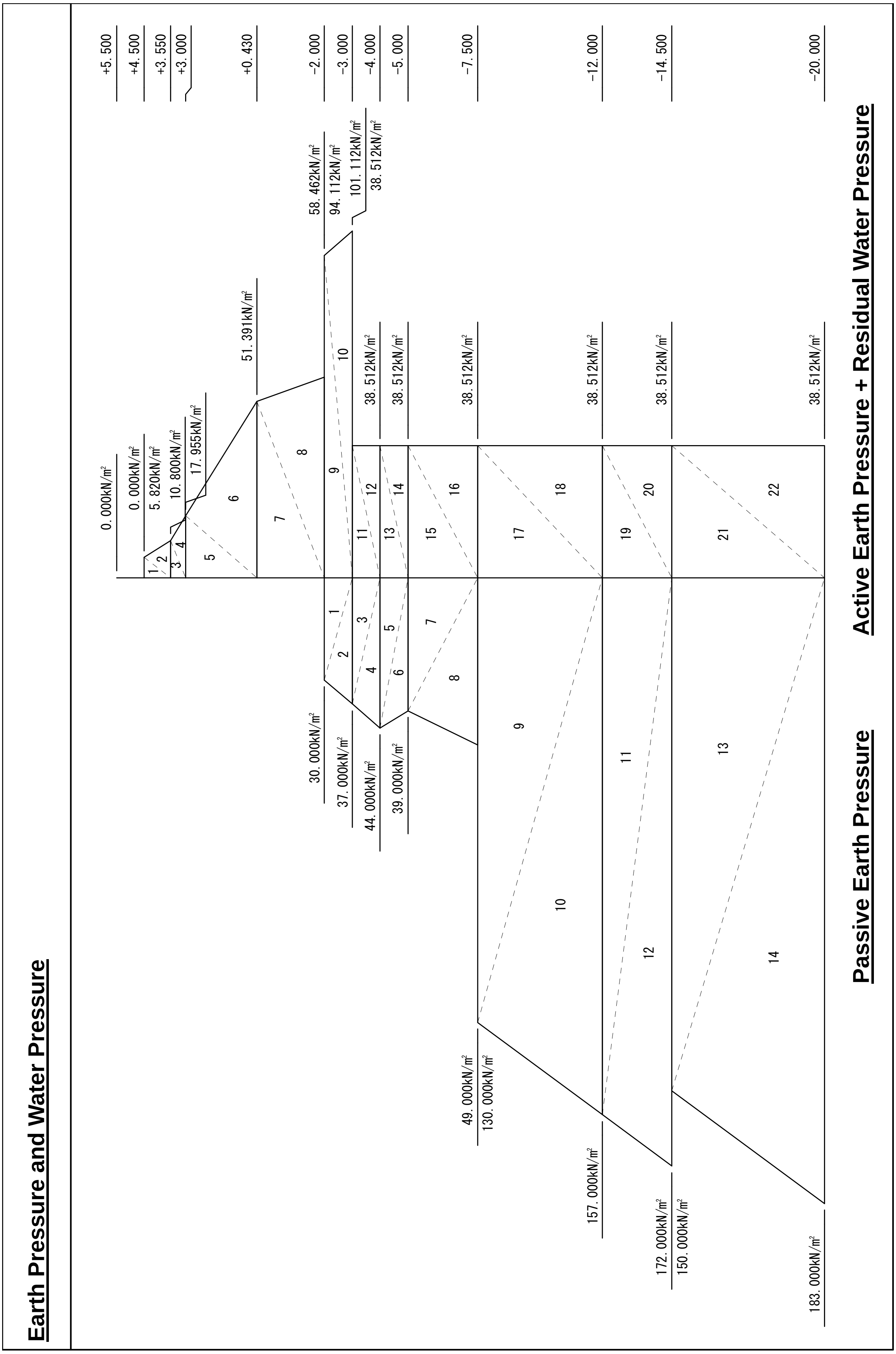
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 404.376X + 2599.058)$$

$$= 2.000X^3 + 96.500X^2 + 1365.000X + 1782.214$$

X= 1.359 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -7.500 – 1.359 = -8.859 m


Earth Pressure and Water Pressure
Passive Earth Pressure
Active Earth Pressure + Residual Water Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-7.500
-12.000
-14.500
-20.000
0.000kN/m²
0.000kN/m²
5.820kN/m²
10.800kN/m²
17.955kN/m²
51.391kN/m²
58.462kN/m²
94.112kN/m²
101.112kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
38.512kN/m²
30.000kN/m²
37.000kN/m²
44.000kN/m²
39.000kN/m²
49.000kN/m²
130.000kN/m²
157.000kN/m²
172.000kN/m²
150.000kN/m²
183.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 4.50	0.000 0.000	——— ———	0.000 0.000
4.50 3.55	5.820 10.800	——— ———	5.820 10.800
3.55 3.00	10.800 17.955	——— ———	10.800 17.955
3.00 0.43	17.955 51.391	——— ———	17.955 51.391
0.43 -2.00	51.391 58.462	——— ———	51.391 58.462
-2.00 -3.00	94.112 101.112	30.000 37.000	64.112 64.112
-3.00 -4.00	38.512 38.512	37.000 44.000	1.512 -5.488
-4.00 -5.00	38.512 38.512	44.000 39.000	-5.488 -0.488
-5.00 -7.50	38.512 38.512	39.000 49.000	-0.488 -10.488
-7.50 -12.00	38.512 38.512	130.000 157.000	-91.488 -118.488
-12.00 -14.50	38.512 38.512	157.000 172.000	-118.488 -133.488
-14.50 -20.00	38.512 38.512	150.000 183.000	-111.488 -144.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $P_{a1} = 38.512 \quad > \quad P_{p1} = 37.000$
at -4.00 m $P_{a2} = 38.512 \quad < \quad P_{p2} = 44.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{1.00 \times (37.000 - 38.512)}{(38.512 - 38.512) - (44.000 - 37.000)} = -3.216m$$

Accordingly, the virtual seabed level is obtained as -3.216m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	5.820x	0.950	2.764	-1.183	-3.270
2	1/2x	10.800x	0.950	5.130	-0.867	-4.448
3	1/2x	10.800x	0.550	2.970	-0.367	-1.090
4	1/2x	17.955x	0.550	4.938	-0.183	-0.904
5	1/2x	17.955x	2.570	23.072	0.857	19.773
6	1/2x	51.391x	2.570	66.037	1.713	113.121
7	1/2x	51.391x	2.430	62.440	3.380	211.047
8	1/2x	58.462x	2.430	71.031	4.190	297.620
9	1/2x	64.112x	1.000	32.056	5.333	170.955
10	1/2x	64.112x	1.000	32.056	5.667	181.661
11	1/2x	1.512x	0.216	0.163	6.072	0.990
Total				302.657	———	985.455

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

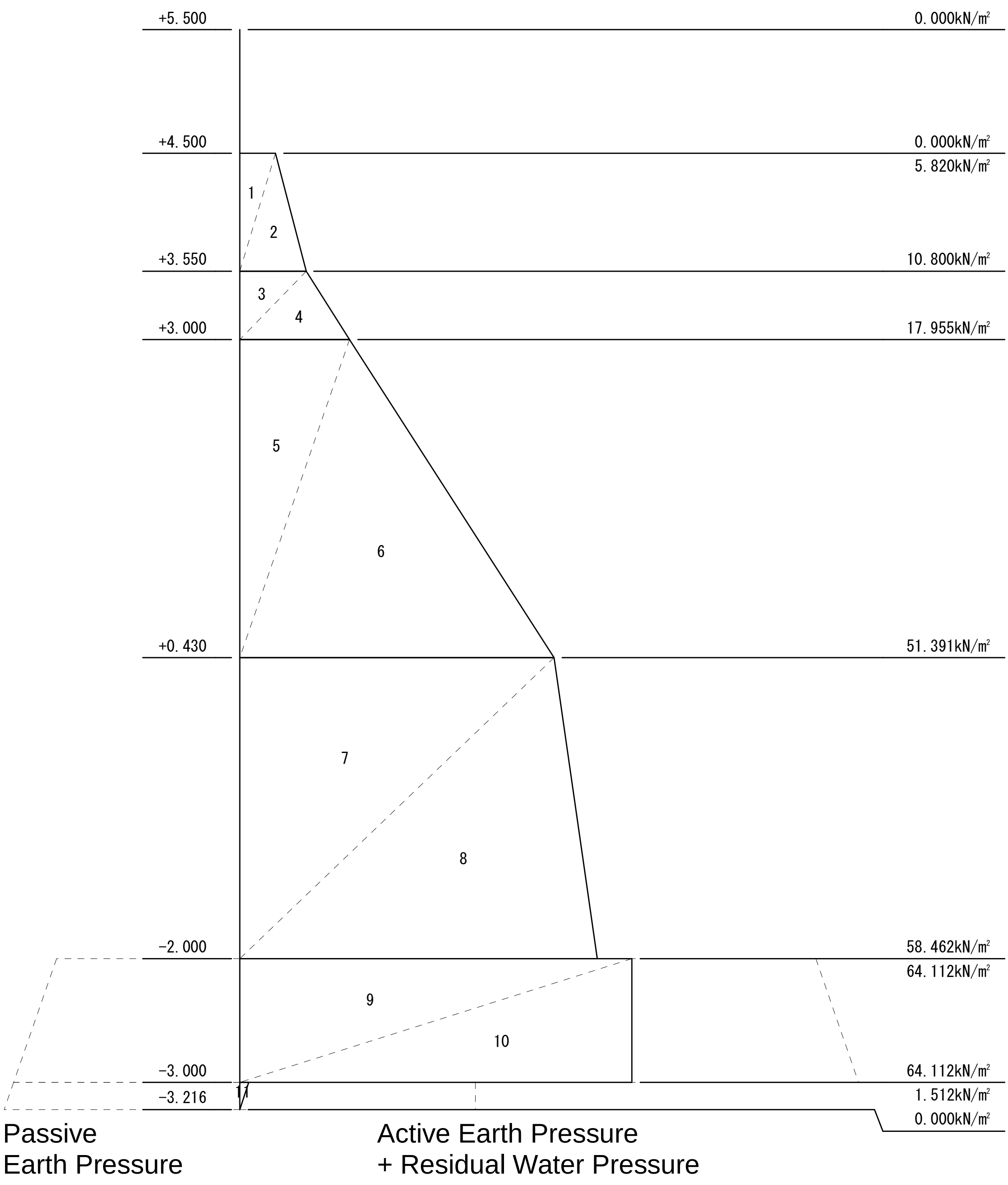
4) Reaction forces

Distance between supports : L_T = 3.000 – (-3.216) = 6.216 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{985.455}{6.216} = 158.535 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 302.657 - 158.535 = 144.122 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 4.50	0.000 0.000	0.000		0.000
4.50 3.55	2.764 5.130	7.894		-7.894
3.55 3.00	2.970 4.938	15.802	144.122	128.320
3.00 0.43	23.072 66.037	104.911	144.122	39.211
0.43 -2.00	62.440 71.031	238.382	144.122	-94.260
-2.00 -3.00	32.056 32.056	302.494	144.122	-158.372
-3.00 -3.22	0.163 0.000	302.657	144.122	-158.535

Shear force $Q = A_p - \sum S$

The above table suggests that the shear force zero point exists in between 0.430m and -1.000m.

$$Q = 39.211 - \frac{[51.391 + (51.391 + 2.910X)]X}{2} = 0$$

X = 0.747 m

Shear force zero point : DL = +0.43 – 0.747 = -0.371m

6) Calculation of moment

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	5.820x	0.950	-2.764	4.500	-12.438
2	1/2x	10.800x	0.950	-5.130	4.184	-21.464
3	1/2x	10.800x	0.550	-2.970	3.684	-10.941
4	1/2x	17.955x	0.550	-4.938	3.500	-17.283
5	1/2x	17.955x	2.570	-23.072	2.460	-56.757
6	1/2x	51.391x	2.570	-66.037	1.604	-105.923
7	1/2x	51.391x	0.747	-19.195	0.498	-9.559
8	1/2x	53.565x	0.747	-20.007	0.249	-4.982
Total				―――	―――	-239.347

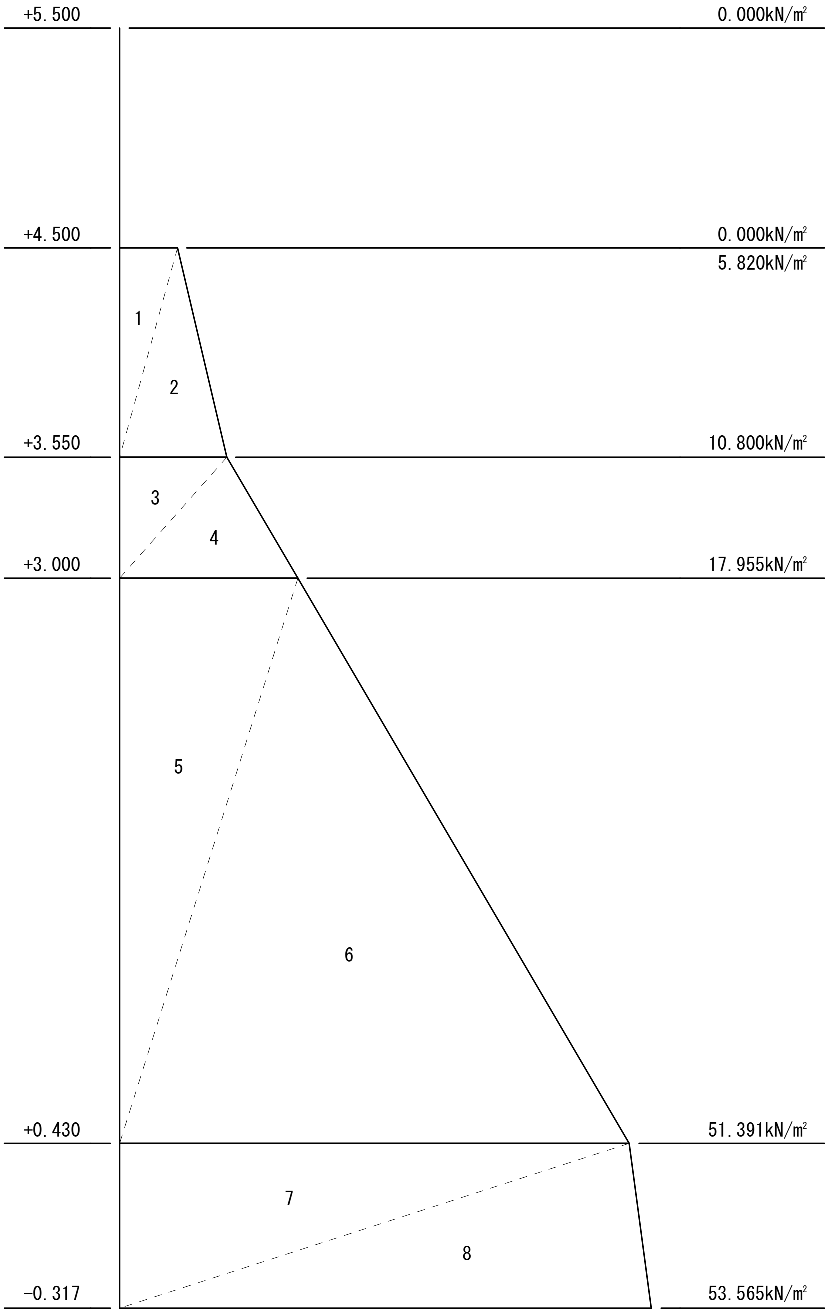
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.371) = 3.371 m

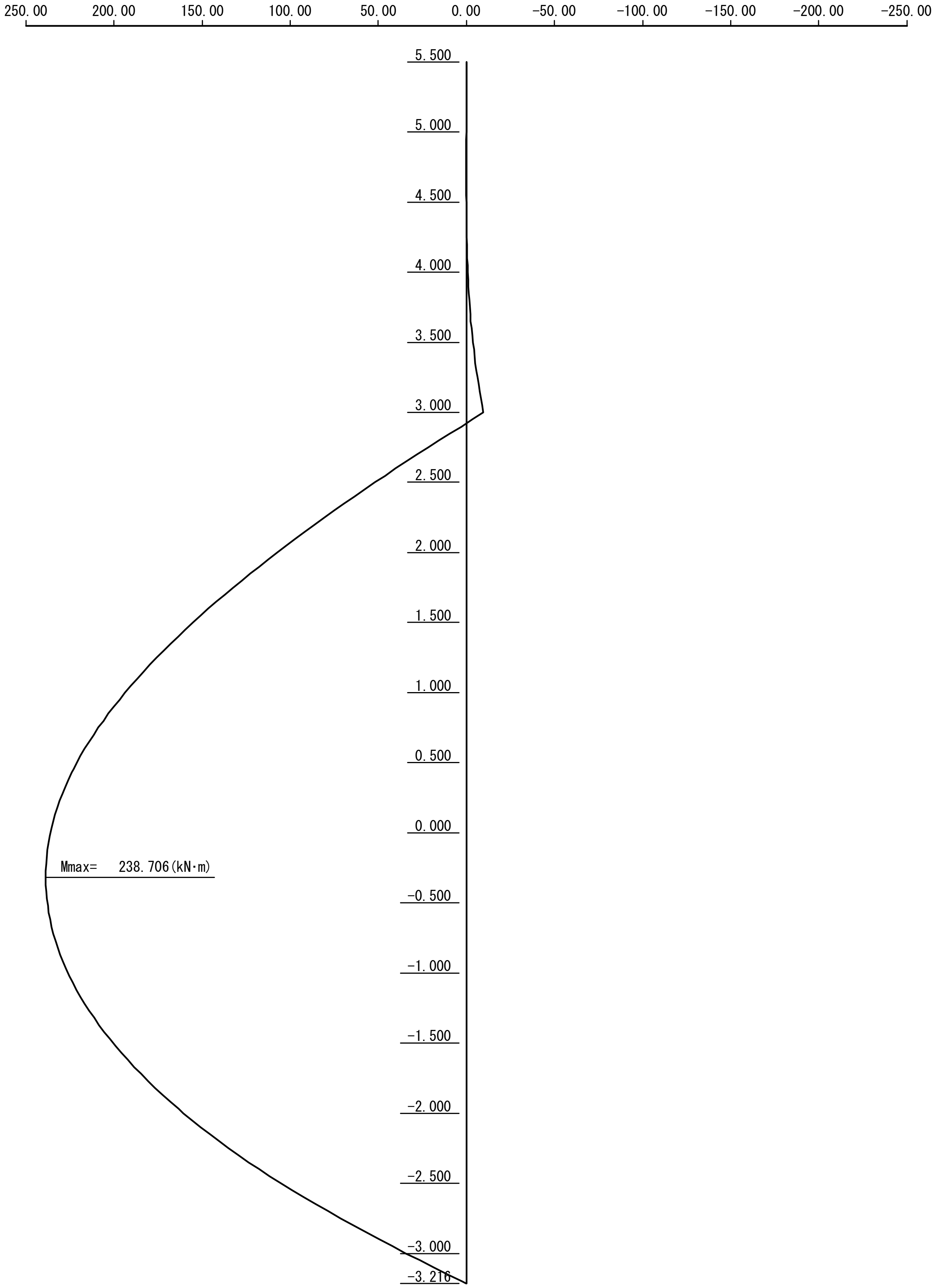
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 144.122 \times 3.371 - 239.347 = 238.706$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -12.451 m --- adopted
- Free earth support method : -8.859 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-12.451) = 15.951m → 16.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 16.000 = -12.500 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 399.711 kN-m/m --- adopted
- Free earth support method : 238.706 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{399.711\times10^6}{4191\times10^3} = 95.4MPa \quad \leq \quad \sigma_a = 140MPa \text{ .... } OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 187.448 kN-m/m --- adopted
- Free earth support method : 144.122 kN-m/m

T = 187.448 × 1.960 × sec(0.000) × sec (0.000) = 367.398 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 367.398 × 3.8 = 1396.112 kN/wire

Tie wire shall have minimum tensile strength of 1397 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M_{max}	is the maximum bending moment on the waling beam (kN-m)
T	is the tension force on tie wire (kN/wire)
L	is the spacing of the tie wire (m)

$$M_{maz} = \frac{367.398 \times 1.960}{10.0} = 72.010 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \le \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{72.010 \times 10^6}{2 \times 374.0 \times 10^3} = 96.3 MPa \quad \le \quad \sigma_a = 140 MPa \ \text{ .... } OK$$

Output of Design Software for Anchor Wall (During Construction) – Revetment Block b

1. Design Conditions

1-1 Dimensions

Ground elevation	+4.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-2.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	20 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water $10.10\ kN/m^3$

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

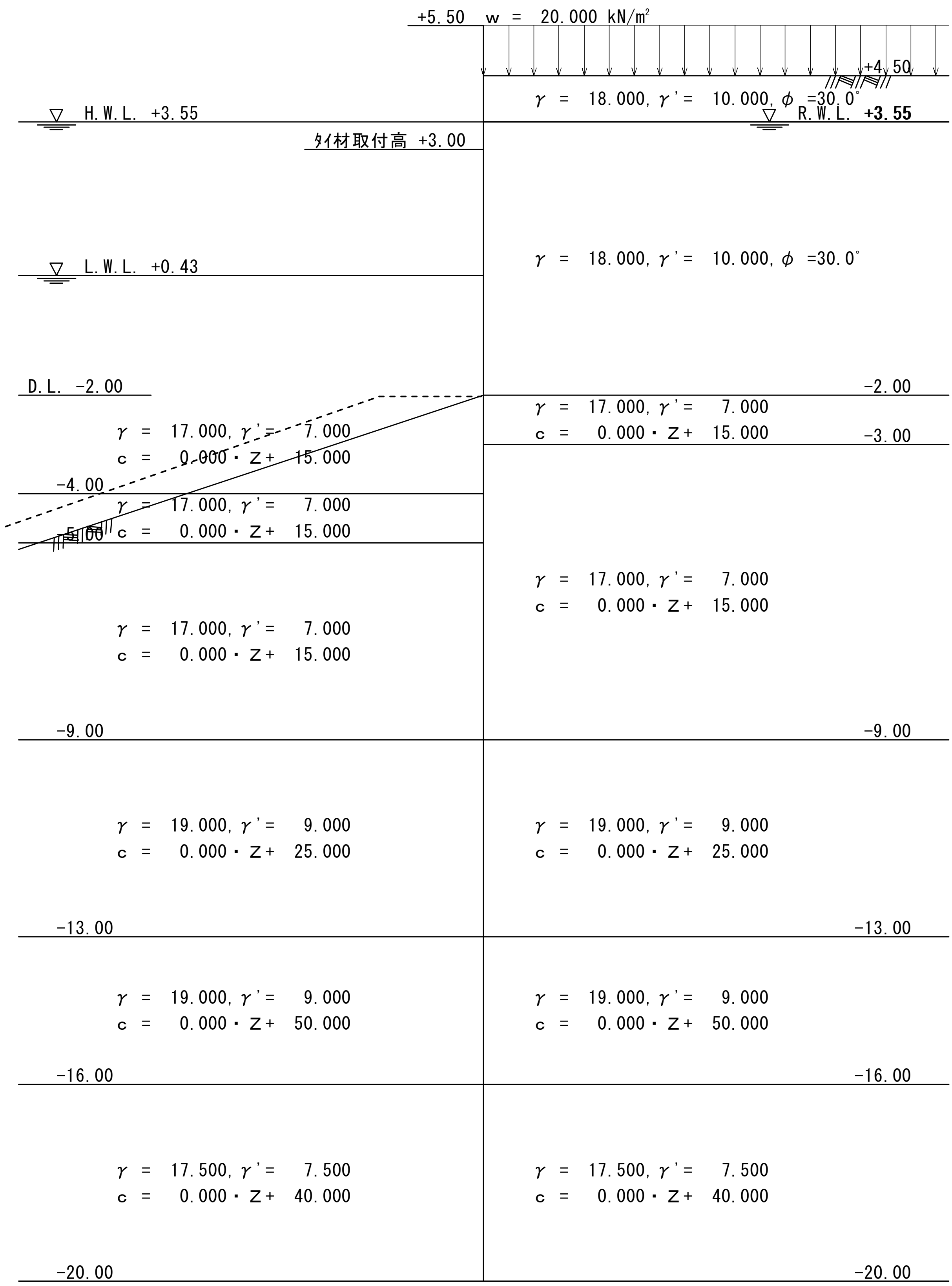
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
4.50 3.55	0.95	5.820 10.800
3.55 3.00	0.55	10.800 12.400
3.00 -2.00	5.00	12.400 26.950
-2.00 -3.00	1.00	62.600 69.600
-3.00 -9.00	6.00	7.000 7.000
-9.00 -12.00	3.00	7.000 7.000
-12.00 -13.00	1.00	7.000 7.000
-13.00 -16.00	3.00	7.000 7.000
-16.00 -20.00	4.00	7.000 7.000

(2) Residual water pressure

$p_w = \gamma_w \times h_w$

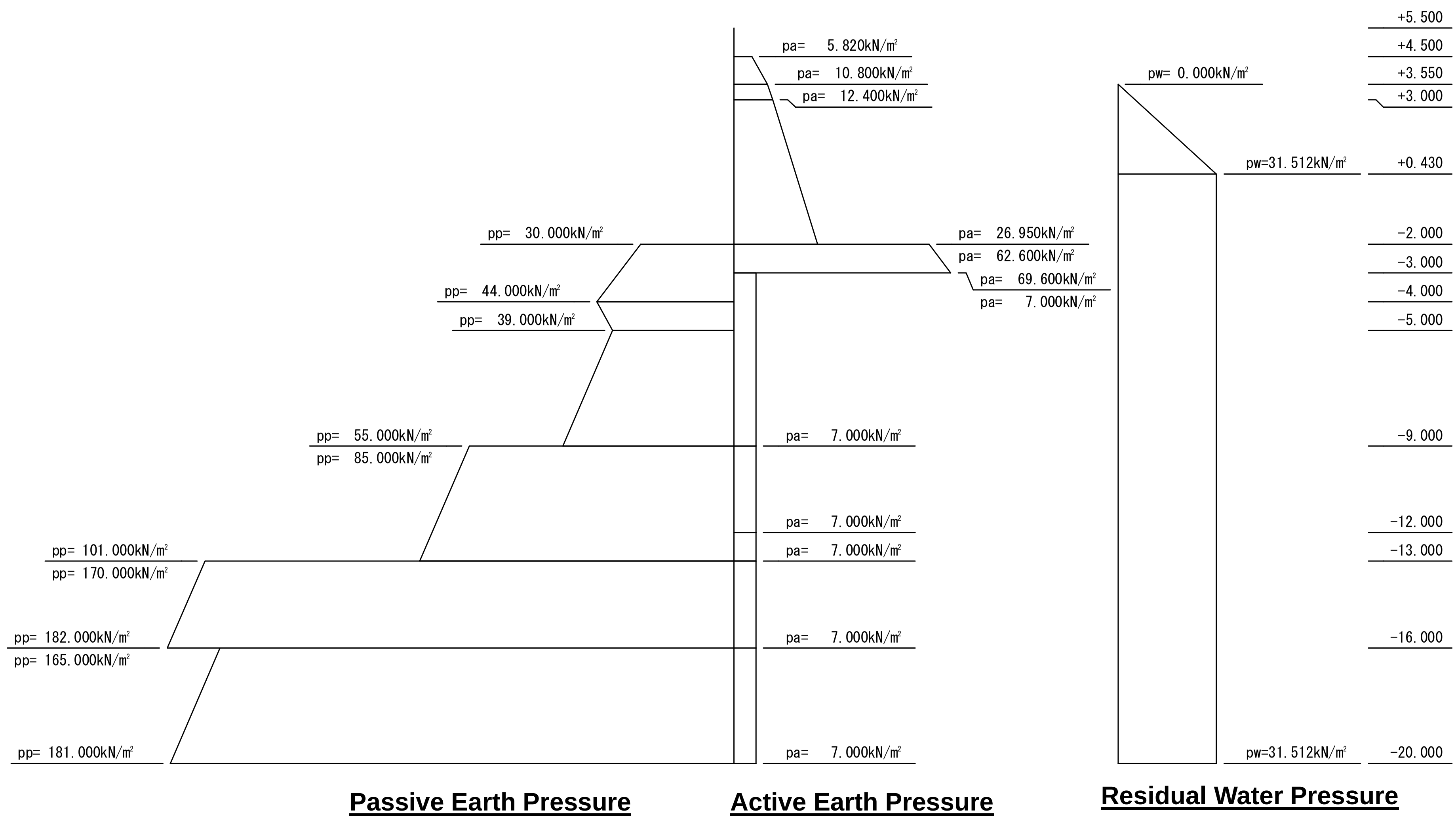
where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-2.00 -4.00	2.00	30.000 44.000
-4.00 -5.00	1.00	44.000 39.000
-5.00 -9.00	4.00	39.000 55.000
-9.00 -13.00	4.00	85.000 101.000
-13.00 -16.00	3.00	170.000 182.000
-16.00 -20.00	4.00	165.000 181.000

Earth Pressure and Water Pressure



(4) Summary of Forces on the Wall

Elev	Active side (kPa)	Passive side(kPa)
(m)	$P_a + P_w$	P_p
5.50 4.50	0.000+ 0.000= 0.000 0.000+ 0.000= 0.000	―― ――
4.50 3.55	5.820+ 0.000= 5.820 10.800+ 0.000= 10.800	―― ――
3.55 3.00	10.800+ 0.000= 10.800 12.400+ 5.555= 17.955	―― ――
3.00 0.43	12.400+ 5.555= 17.955 19.879+ 31.512= 51.391	―― ――
0.43 -2.00	19.879+ 31.512= 51.391 26.950+ 31.512= 58.462	―― ――
-2.00 -3.00	62.600+ 31.512= 94.112 69.600+ 31.512= 101.112	30.000 37.000
-3.00 -4.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	37.000 44.000
-4.00 -5.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	44.000 39.000
-5.00 -9.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	39.000 55.000
-9.00 -12.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	85.000 97.000
-12.00 -13.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	97.000 101.000
-13.00 -16.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	170.000 182.000
-16.00 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	165.000 181.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -2.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-2.000	-239.109	112.012	126.370
-3.000	-446.199	138.417	164.077
-4.000	-686.260	161.798	138.708
-5.000	-899.495	178.897	118.621
-6.000	-1095.816	191.976	103.054
-7.000	-1271.134	201.990	86.552
-8.000	-1414.513	209.412	68.642
-9.000	-1512.401	214.558	49.008
-10.000	-1488.716	216.488	-1.410
-11.000	-1264.008	214.536	-51.946
-12.000	-814.954	209.213	-103.111
-13.000	-114.823	200.899	-155.285
-14.000	1050.320	187.858	-275.732

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -13.00 and -14.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-13.131	0.000	199.570	-171.188

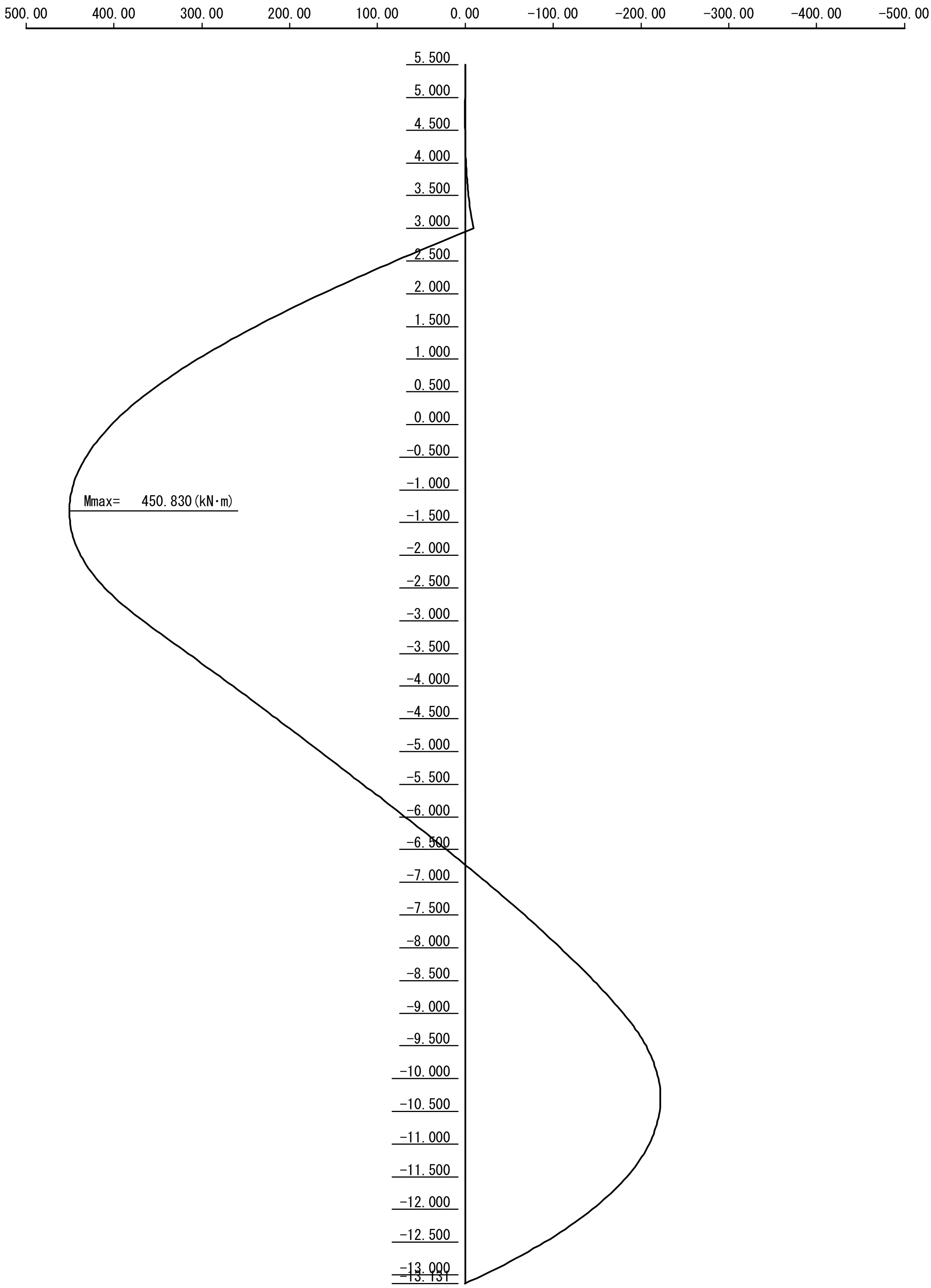
Embedded length of the pipe wall : L = 1.2 × (-2.000 + 13.131) = 13.357 m
Toe elevation : D = -2.000 – 13.357 = -15.357 m
Reaction force at tie setting point : 199.570 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	0.000	0.000
4.500	0.000	0.000
4.000	-0.837	-3.565
3.550	-3.375	-7.894
3.050	-8.944	-14.921
3.000	-9.712	-15.802
2.500	79.657	173.164
2.000	162.910	159.308
1.500	238.423	142.199
1.000	304.567	121.838
0.500	359.718	98.224
0.430	366.469	94.658
-0.070	407.314	68.599
-0.570	434.947	41.812
-1.070	449.005	14.298
-1.570	449.124	-13.943
-2.000	437.801	-38.813
-2.500	410.380	-70.869
-3.000	366.932	-102.925
-3.500	315.426	-102.806
-4.000	264.417	-100.937
-4.500	214.530	-98.818
-5.000	165.391	-97.949
-5.500	116.561	-97.205
-6.000	68.352	-95.461
-6.500	21.266	-92.717
-7.000	-24.198	-88.973
-7.500	-67.540	-84.229
-8.000	-108.261	-78.485
-8.500	-145.859	-71.741
-9.000	-179.835	-63.997
-9.500	-205.939	-40.253
-10.000	-219.922	-15.509
-10.500	-221.282	10.235
-11.000	-209.520	36.979
-11.500	-184.136	64.723
-12.000	-144.631	93.467
-12.500	-90.503	123.211
-13.000	-21.253	153.955
-13.131	0.000	171.188

Maximum bending moment : 450.830 kN-m/m
Elevation of maximum bending moment : -1.325 m
Depth of 1st steady point (moment=0) : -6.731 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

$$M_p = FOS \times M_a$$

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
—	5.50	1/2x	0.000x	1.000	0.000	-2.167	0.000	
—	4.50	1/2x	0.000x	1.000	0.000	-1.833	0.000	0.000
1	4.50	1/2x	5.820x	0.950	2.764	-1.183	-3.270	
2	3.55	1/2x	10.800x	0.950	5.130	-0.867	-4.448	-9.262
3	3.55	1/2x	10.800x	0.550	2.970	-0.367	-1.090	
4	3.00	1/2x	17.955x	0.550	4.938	-0.183	-0.904	-11.655
5	3.00	1/2x	17.955x	2.570	23.072	0.857	19.773	
6	0.43	1/2x	51.391x	2.570	66.037	1.713	113.121	147.818
7	0.43	1/2x	51.391x	2.430	62.440	3.380	211.047	
8	-2.00	1/2x	58.462x	2.430	71.031	4.190	297.620	758.218
9	-2.00	1/2x	94.112x	1.000	47.056	5.333	250.950	
10	-3.00	1/2x	101.112x	1.000	50.556	5.667	286.501	1403.159
11	-3.00	1/2x	38.512x	1.000	19.256	6.333	121.948	
12	-4.00	1/2x	38.512x	1.000	19.256	6.667	128.380	1703.553
13	-4.00	1/2x	38.512x	1.000	19.256	7.333	141.204	
14	-5.00	1/2x	38.512x	1.000	19.256	7.667	147.636	2050.161
15	-5.00	1/2x	38.512x	4.000	77.024	9.333	718.865	
16	-9.00	1/2x	38.512x	4.000	77.024	10.667	821.615	3898.737
17	-9.00	1/2x	38.512x	3.000	57.768	13.000	750.984	
18	-12.00	1/2x	38.512x	3.000	57.768	14.000	808.752	5770.420
19	-12.00	1/2x	38.512x	1.000	19.256	15.333	295.252	
20	-13.00	1/2x	38.512x	1.000	19.256	15.667	301.684	6486.743
21	-13.00	1/2x	38.512x	3.000	57.768	17.000	982.056	
22	-16.00	1/2x	38.512x	3.000	57.768	18.000	1039.824	8912.999
23	-16.00	1/2x	38.512x	4.000	77.024	20.333	1566.129	
24	-20.00	1/2x	38.512x	4.000	77.024	21.667	1668.879	12795.009

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

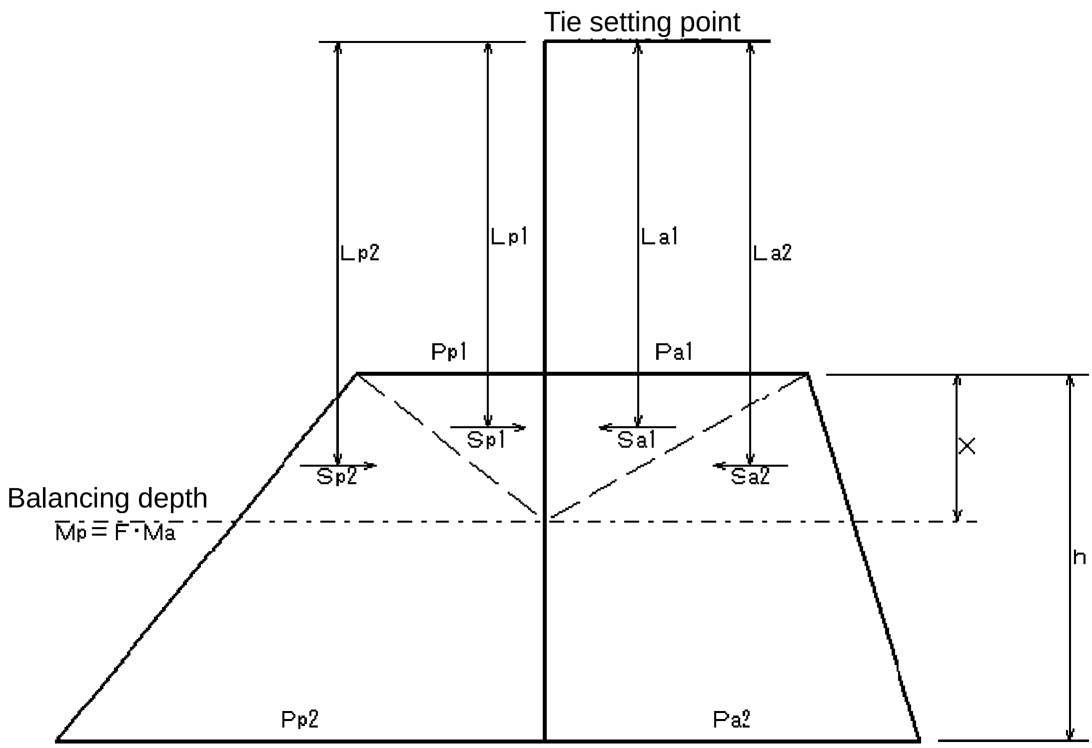
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-2.00	1/2x	30.000x	1.000	15.000	5.333	79.995	
2	-3.00	1/2x	37.000x	1.000	18.500	5.667	104.840	184.835
3	-3.00	1/2x	37.000x	1.000	18.500	6.333	117.160	
4	-4.00	1/2x	44.000x	1.000	22.000	6.667	146.674	448.669
5	-4.00	1/2x	44.000x	1.000	22.000	7.333	161.326	
6	-5.00	1/2x	39.000x	1.000	19.500	7.667	149.506	759.501
7	-5.00	1/2x	39.000x	4.000	78.000	9.333	727.974	
8	-9.00	1/2x	55.000x	4.000	110.000	10.667	1173.370	2660.845
9	-9.00	1/2x	85.000x	3.000	127.500	13.000	1657.500	
10	-12.00	1/2x	97.000x	3.000	145.500	14.000	2037.000	6355.345
11	-12.00	1/2x	97.000x	1.000	48.500	15.333	743.650	
12	-13.00	1/2x	101.000x	1.000	50.500	15.667	791.184	7890.179
13	-13.00	1/2x	170.000x	3.000	255.000	17.000	4335.000	
14	-16.00	1/2x	182.000x	3.000	273.000	18.000	4914.000	17139.179
15	-16.00	1/2x	165.000x	4.000	330.000	20.333	6709.890	
16	-20.00	1/2x	181.000x	4.000	362.000	21.667	7843.454	31692.523

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \dfrac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$	: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)
$M_{a(p)1}$	: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)
$M_{a(p)1'}$	: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$M_{a(p)2'}$	: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$S_{a(p)1}$	: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$S_{a(p)2}$	: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$L_{a(p)1}$	: the arm length of $S_{a(p)1}$ from the tie setting point (m)
$L_{a(p)2}$	: the arm length of $S_{a(p)2}$ from the tie setting point (m)
$P_{a(p)1}$	: the active (passive) side pressure at the top elevation of the soil layer (kPa)
$P_{a(p)1}$	: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)
h	: the thickness of the soil layer (m)
L	: the vertical distance from the tie setting point to the top elevation of the soil layer (m)
X	: the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -9.00 m	FOS×M_{a1} = 3898.737	> M_{p1} = 2660.845
at -12.00 m	FOS×M_{a2} = 5770.420	< M_{p2} = 6355.345

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \;,\quad L_{a1} = 12.000 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 231.072X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{2.000}X\right]X}{2} = 0.000X^2 + 19.256X \;,\quad L_{a2} = 12.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 231.072X$$

$$M_a = 0.000X^3 + 19.256X^2 + 462.144X + 3248.948$$

[Moment on passive side]

$$S_{p1} = \frac{85.000X}{2} = 42.500X \;,\quad L_{p1} = 12.000 + \frac{1}{3}X$$

$$M_{p1'} = 14.167X^2 + 510.000X$$

$$S_{p2} = \frac{\left[85.000 + \frac{97.000 - 85.000}{2.000}X\right]X}{2} = 2.000X^2 + 42.500X \;,\quad L_{p2} = 14.000 + \frac{2}{3}X$$

$$M_{p2'} = 1.333X^3 + 52.333X^2 + 510.000X$$

$$M_p = 1.333X^3 + 66.500X^2 + 1020.000X + 2660.845$$

[Balancing depth]

$$FOS \cdot M_a = M_p$$

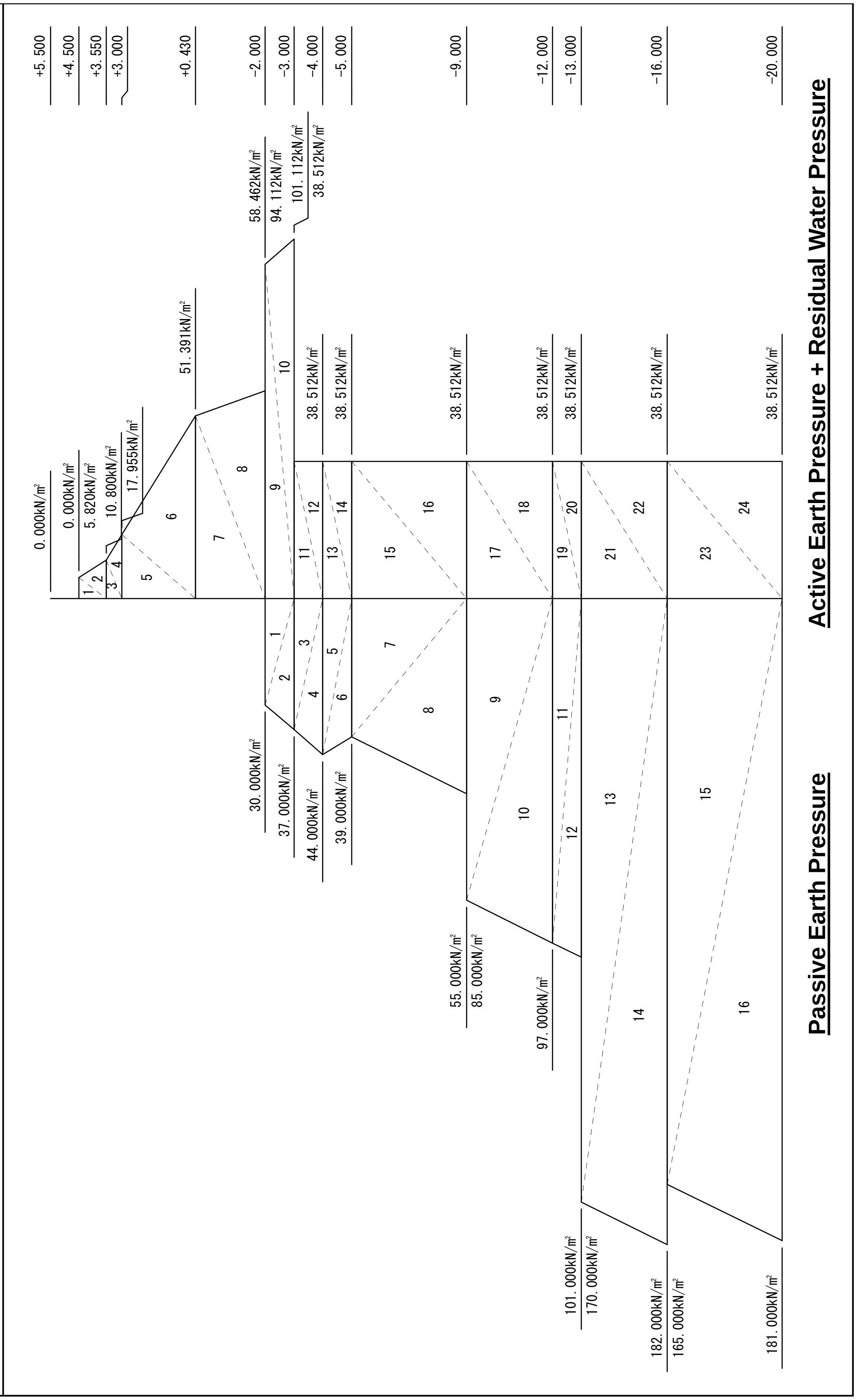
$$1.2 \times (0.000X^3 + 19.256X^2 + 462.144X + 3248.948)$$

$$= 1.333X^3 + 66.500X^2 + 1020.000X + 2660.845$$

X= 2.185 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -9.000 – 2.185 = -11.185 m

Earth Pressure and Water Pressure



(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 4.50	0.000 0.000	——— ———	0.000 0.000
4.50 3.55	5.820 10.800	——— ———	5.820 10.800
3.55 3.00	10.800 17.955	——— ———	10.800 17.955
3.00 0.43	17.955 51.391	——— ———	17.955 51.391
0.43 -2.00	51.391 58.462	——— ———	51.391 58.462
-2.00 -3.00	94.112 101.112	30.000 37.000	64.112 64.112
-3.00 -4.00	38.512 38.512	37.000 44.000	1.512 -5.488
-4.00 -5.00	38.512 38.512	44.000 39.000	-5.488 -0.488
-5.00 -9.00	38.512 38.512	39.000 55.000	-0.488 -16.488
-9.00 -12.00	38.512 38.512	85.000 97.000	-46.488 -58.488
-12.00 -13.00	38.512 38.512	97.000 101.000	-58.488 -62.488
-13.00 -16.00	38.512 38.512	170.000 182.000	-131.488 -143.488
-16.00 -20.00	38.512 38.512	165.000 181.000	-126.488 -142.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $P_{a1} = 38.512 > P_{p1} = 37.000$

at -4.00 m $P_{a2} = 38.512 < P_{p2} = 44.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{1.00 \times (37.000 - 38.512)}{(38.512 - 38.512) - (44.000 - 37.000)} = -3.216m$$

Accordingly, the virtual seabed level is obtained as -3.216m.

3) Moment about tie setting point

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 5.820x 0.950	2.764	-1.183	-3.270
2	1/2x 10.800x 0.950	5.130	-0.867	-4.448
3	1/2x 10.800x 0.550	2.970	-0.367	-1.090
4	1/2x 17.955x 0.550	4.938	-0.183	-0.904
5	1/2x 17.955x 2.570	23.072	0.857	19.773
6	1/2x 51.391x 2.570	66.037	1.713	113.121
7	1/2x 51.391x 2.430	62.440	3.380	211.047
8	1/2x 58.462x 2.430	71.031	4.190	297.620
9	1/2x 64.112x 1.000	32.056	5.333	170.955
10	1/2x 64.112x 1.000	32.056	5.667	181.661
11	1/2x 1.512x 0.216	0.163	6.072	0.990
Total		302.657	———	985.455

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

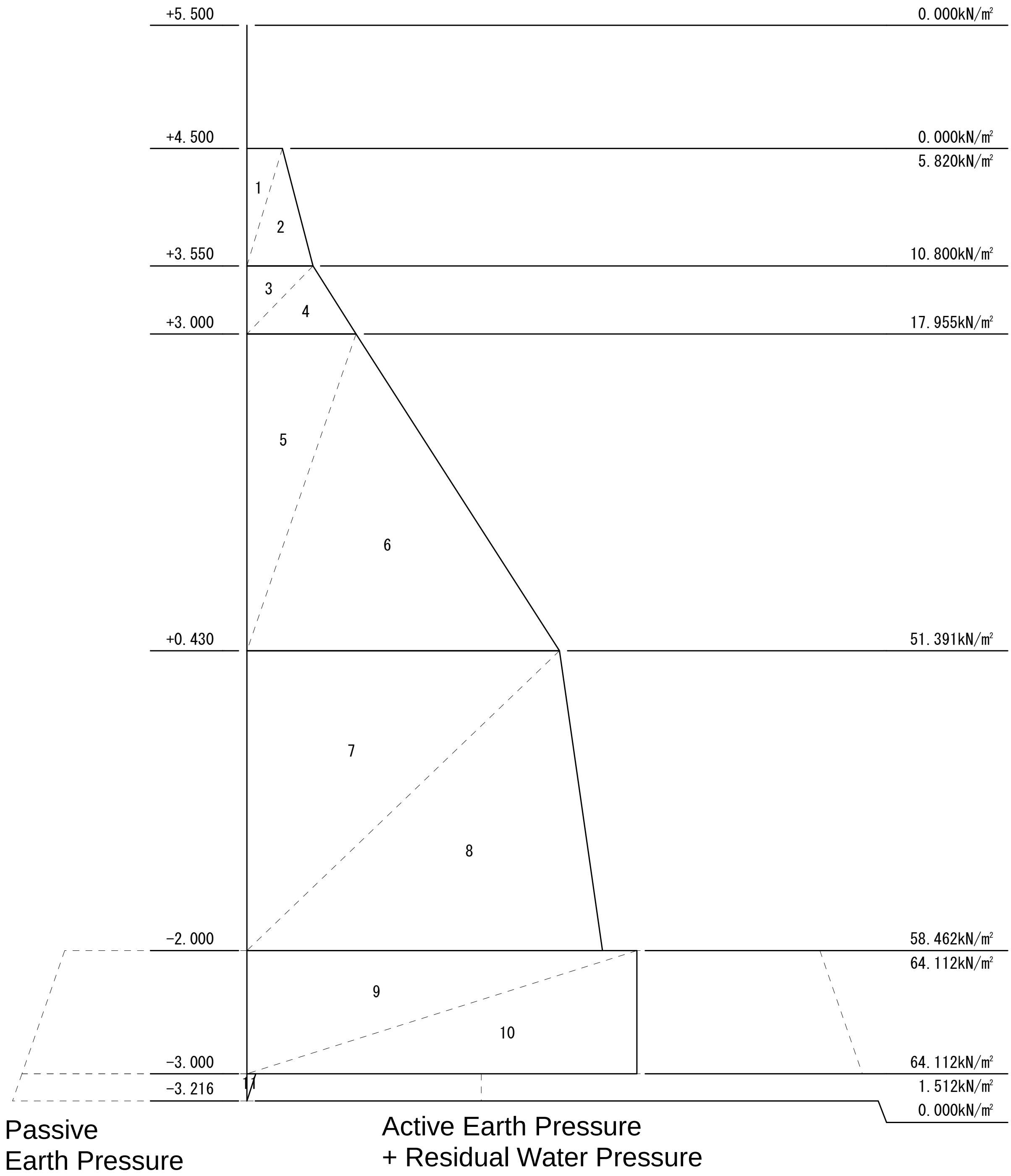
4) Reaction forces

Distance between supports : L_T = 3.000 – (-3.216) = 6.216 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{985.455}{6.216} = 158.535 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 302.657 - 158.535 = 144.122 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 4.50	0.000 0.000	0.000		0.000
4.50 3.55	2.764 5.130	7.894		-7.894
3.55 3.00	2.970 4.938	15.802	144.122	128.320
3.00 0.43	23.072 66.037	104.911	144.122	39.211
0.43 -2.00	62.440 71.031	238.382	144.122	-94.260
-2.00 -3.00	32.056 32.056	302.494	144.122	-158.372
-3.00 -3.22	0.163 0.000	302.657	144.122	-158.535

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -2.000m.

$$Q = 39.211 - \frac{[51.391 + (51.391 + 2.910X)]X}{2} = 0$$

X = 0.747 m

Shear force zero point : DL = +0.43 – 0.747 = -0.317m

6) Calculation of moment

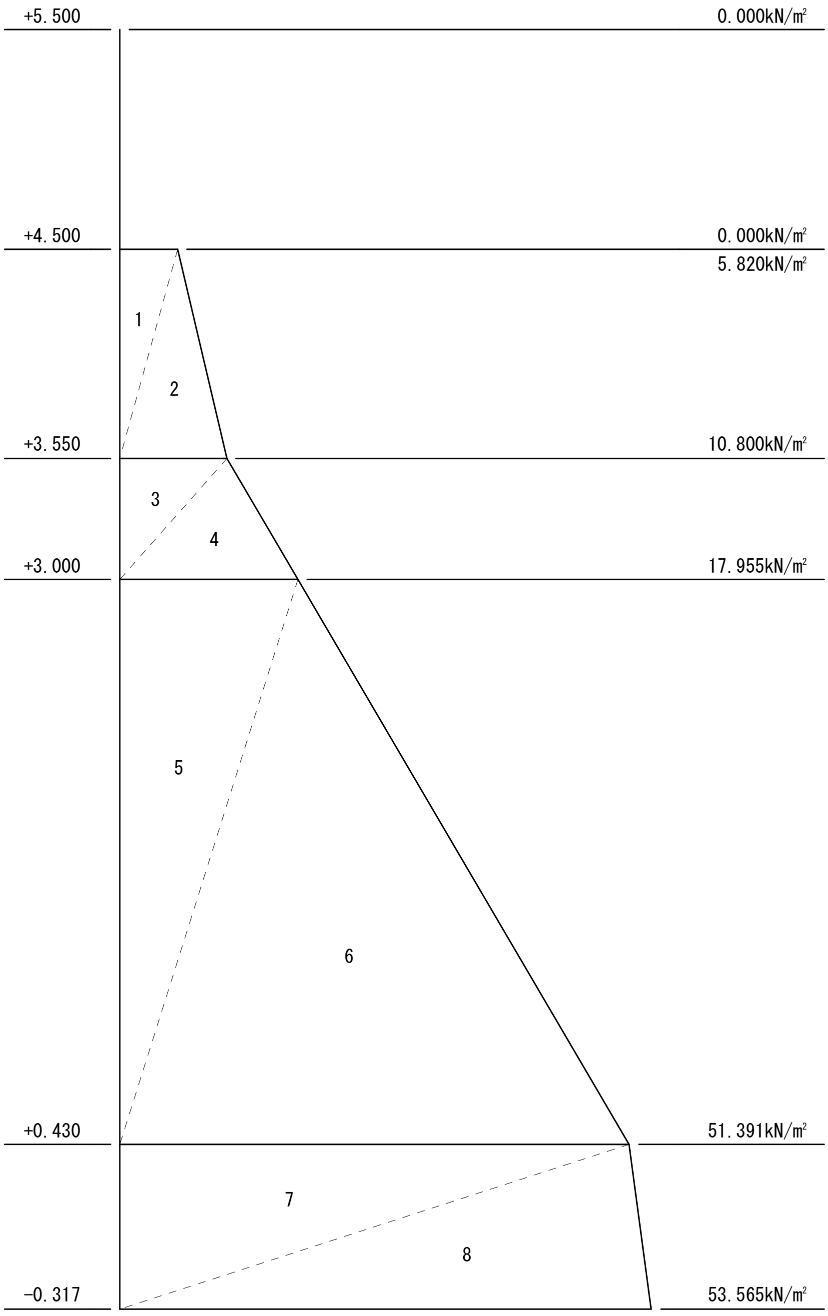
No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	5.820x	0.950	-2.764	4.500	-12.438
2	1/2x	10.800x	0.950	-5.130	4.184	-21.464
3	1/2x	10.800x	0.550	-2.970	3.684	-10.941
4	1/2x	17.955x	0.550	-4.938	3.500	-17.283
5	1/2x	17.955x	2.570	-23.072	2.460	-56.757
6	1/2x	51.391x	2.570	-66.037	1.604	-105.923
7	1/2x	51.391x	0.747	-19.195	0.498	-9.559
8	1/2x	53.565x	0.747	-20.007	0.249	-4.982
Total				———	———	-239.347

where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

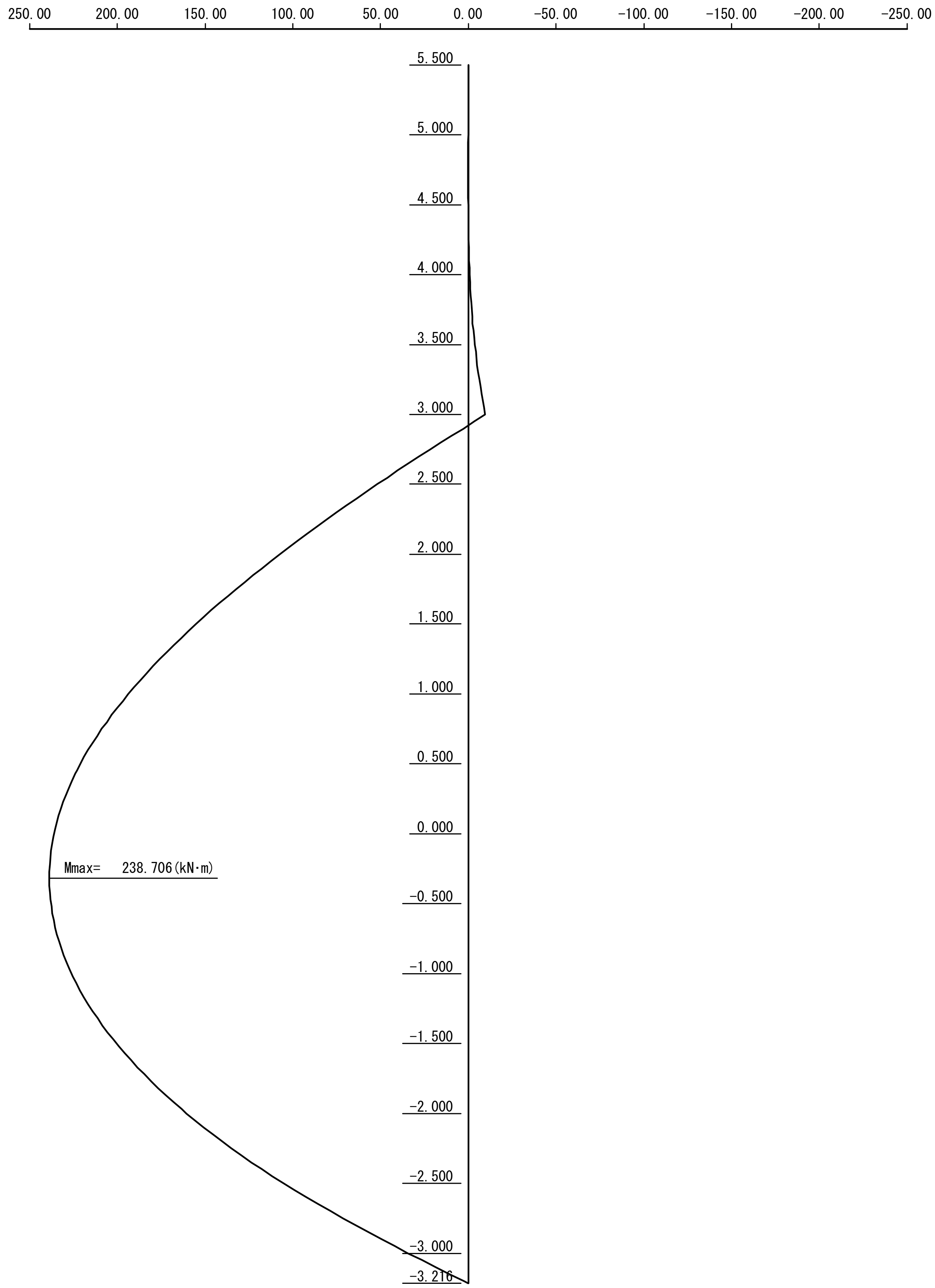
- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.317) = 3.317 m
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M$ = 144.122 × 3.317 – 239.347 = 238.706 kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -15.357 m --- adopted
- Free earth support method : -11.185 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-15.357) = 18.857m → 19.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 19.000 = -15.500 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 450.830 kN-m/m --- adopted
- Free earth support method : 238.706 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{450.830\times10^6}{4191\times10^3} = 107.6MPa \quad \leq \quad \sigma_a = 140MPa \;\; .... \; OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

- T is the tension force on tie wire (kN/wire)
- A_p is the reaction force of the pipe wall at tie setting point (kN/m)
- L is the spacing of the tie wire (m)
- θ_1 is the vertical angle of the tie wire to horizontal (deg.)
- θ_2 is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 199.570 kN-m/m --- adopted
- Free earth support method : 144.122 kN-m/m

T = 199.570 × 1.960 × sec(0.000) × sec (0.000) = 391.157 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 391.157 × 3.8 = 1486.400 kN/wire

Tie wire shall have minimum tensile strength of 1487 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{391.157 \times 1.960}{10.0} = 76.667 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \leq \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{76.667 \times 10^6}{2 \times 374.0 \times 10^3} = 102.5 MPa \quad \leq \quad \sigma_a = 140 MPa \ \ .... \ OK$$

<u>Output of Design Software for Anchor Wall (During Construction)</u>
– Revetment Block c

1. Design Conditions

1-1 Dimensions

Ground elevation	+4.50 m
Top of cope concrete	+5.50 m
Top of pile wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-2.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	20 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water 10.10 kN/m^3

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

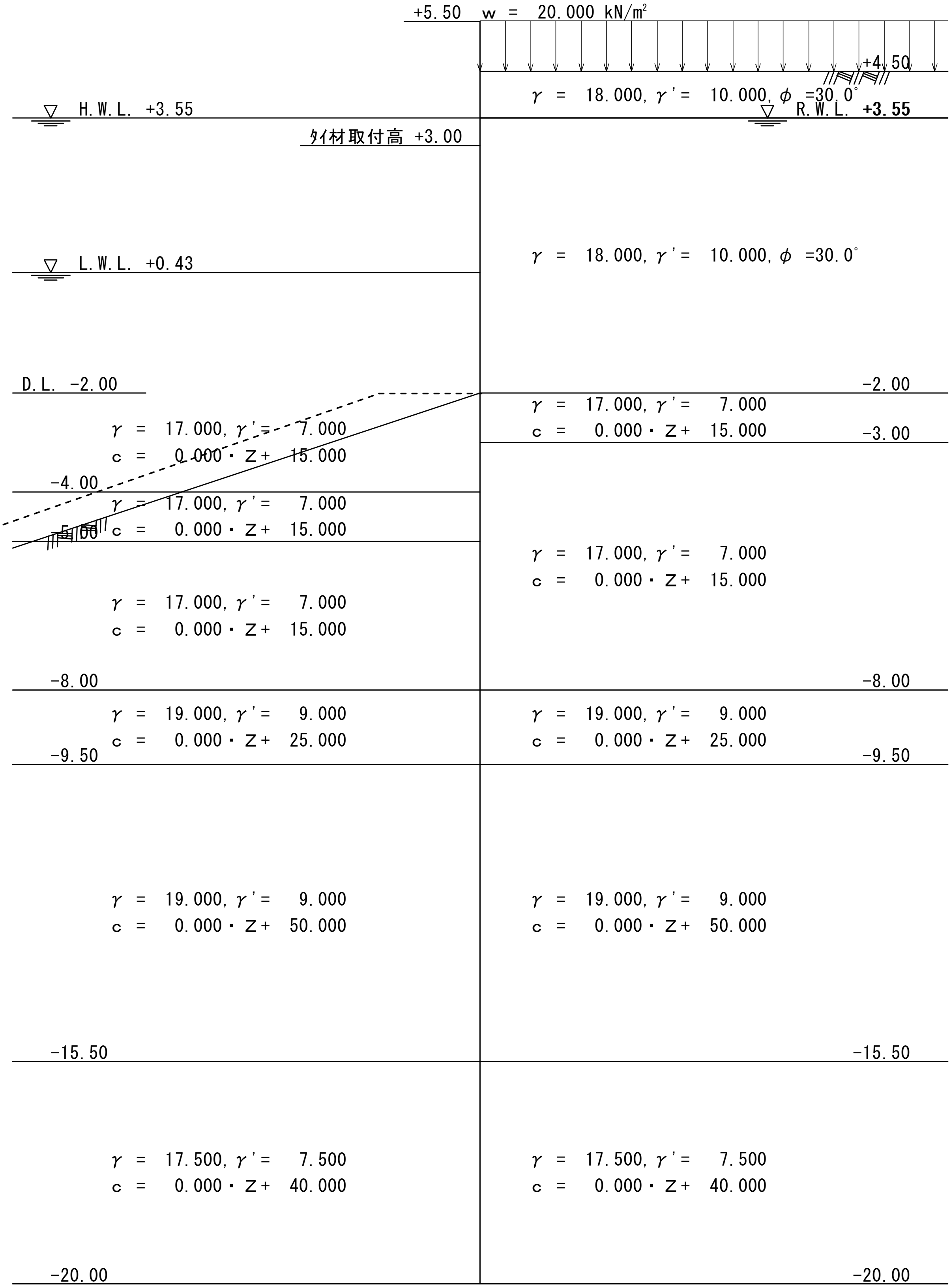
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
4.50 3.55	0.95	5.820 10.800
3.55 3.00	0.55	10.800 12.400
3.00 -2.00	5.00	12.400 26.950
-2.00 -3.00	1.00	62.600 69.600
-3.00 -8.00	5.00	7.000 7.000
-8.00 -9.50	1.50	7.000 7.000
-9.50 -12.00	2.50	7.000 7.000
-12.00 -15.50	3.50	7.000 7.000
-15.50 -20.00	4.50	7.000 7.000

(2) Residual water pressure

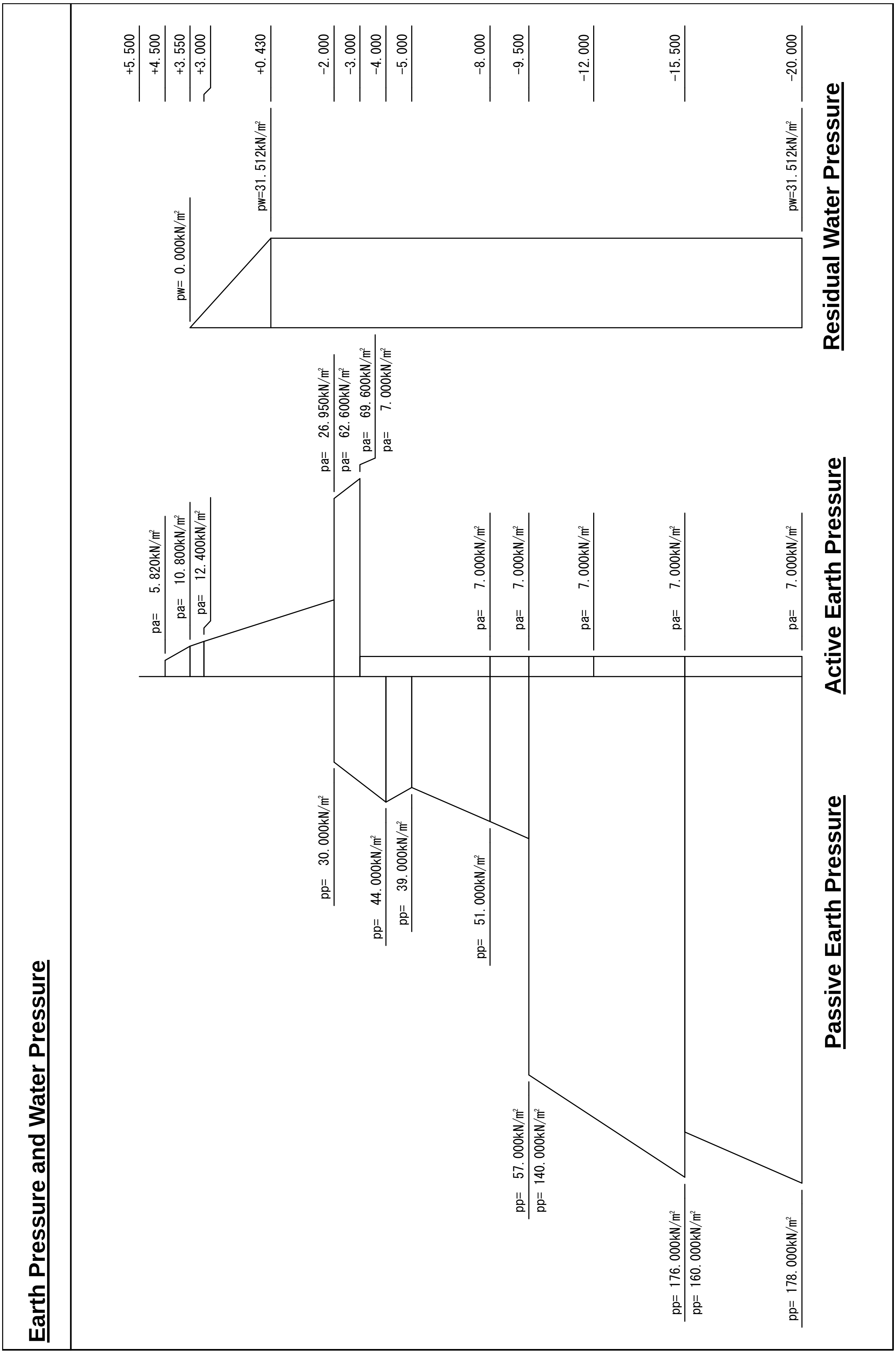
$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-2.00 -4.00	2.00	30.000 44.000
-4.00 -5.00	1.00	44.000 39.000
-5.00 -8.00	3.00	39.000 51.000
-8.00 -9.50	1.50	51.000 57.000
-9.50 -15.50	6.00	140.000 176.000
-15.50 -20.00	4.50	160.000 178.000


Earth Pressure and Water Pressure
Passive Earth Pressure
Active Earth Pressure
Residual Water Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-8.000
-9.500
-12.000
-15.500
-20.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
pa= 5.820kN/m²
pa= 10.800kN/m²
pa= 12.400kN/m²
pa= 26.950kN/m²
pa= 62.600kN/m²
pa= 69.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pp= 30.000kN/m²
pp= 44.000kN/m²
pp= 39.000kN/m²
pp= 51.000kN/m²
pp= 57.000kN/m²
pp= 140.000kN/m²
pp= 176.000kN/m²
pp= 160.000kN/m²
pp= 178.000kN/m²

(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$			Passive side(kPa) P_p
5.50	0.000+	0.000=	0.000	———
4.50	0.000+	0.000=	0.000	———
4.50	5.820+	0.000=	5.820	———
3.55	10.800+	0.000=	10.800	———
3.55	10.800+	0.000=	10.800	———
3.00	12.400+	5.555=	17.955	———
3.00	12.400+	5.555=	17.955	———
0.43	19.879+	31.512=	51.391	———
0.43	19.879+	31.512=	51.391	———
-2.00	26.950+	31.512=	58.462	———
-2.00	62.600+	31.512=	94.112	30.000
-3.00	69.600+	31.512=	101.112	37.000
-3.00	7.000+	31.512=	38.512	37.000
-4.00	7.000+	31.512=	38.512	44.000
-4.00	7.000+	31.512=	38.512	44.000
-5.00	7.000+	31.512=	38.512	39.000
-5.00	7.000+	31.512=	38.512	39.000
-8.00	7.000+	31.512=	38.512	51.000
-8.00	7.000+	31.512=	38.512	51.000
-9.50	7.000+	31.512=	38.512	57.000
-9.50	7.000+	31.512=	38.512	140.000
-12.00	7.000+	31.512=	38.512	155.000
-12.00	7.000+	31.512=	38.512	155.000
-15.50	7.000+	31.512=	38.512	176.000
-15.50	7.000+	31.512=	38.512	160.000
-20.00	7.000+	31.512=	38.512	178.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -2.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-2.000	-239.109	112.012	126.370
-3.000	-446.199	138.417	164.077
-4.000	-686.260	161.798	138.708
-5.000	-899.495	178.897	118.621
-6.000	-1095.816	191.976	103.054
-7.000	-1271.134	201.990	86.552
-8.000	-1414.513	209.412	68.642
-9.000	-1512.401	214.557	49.009
-10.000	-1505.391	216.840	-13.512
-11.000	-1110.287	212.072	-116.232
-12.000	-249.074	200.574	-218.222
-13.000	1120.106	183.232	-320.368

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -12.00 and -13.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-12.214	0.000	197.324	-240.070

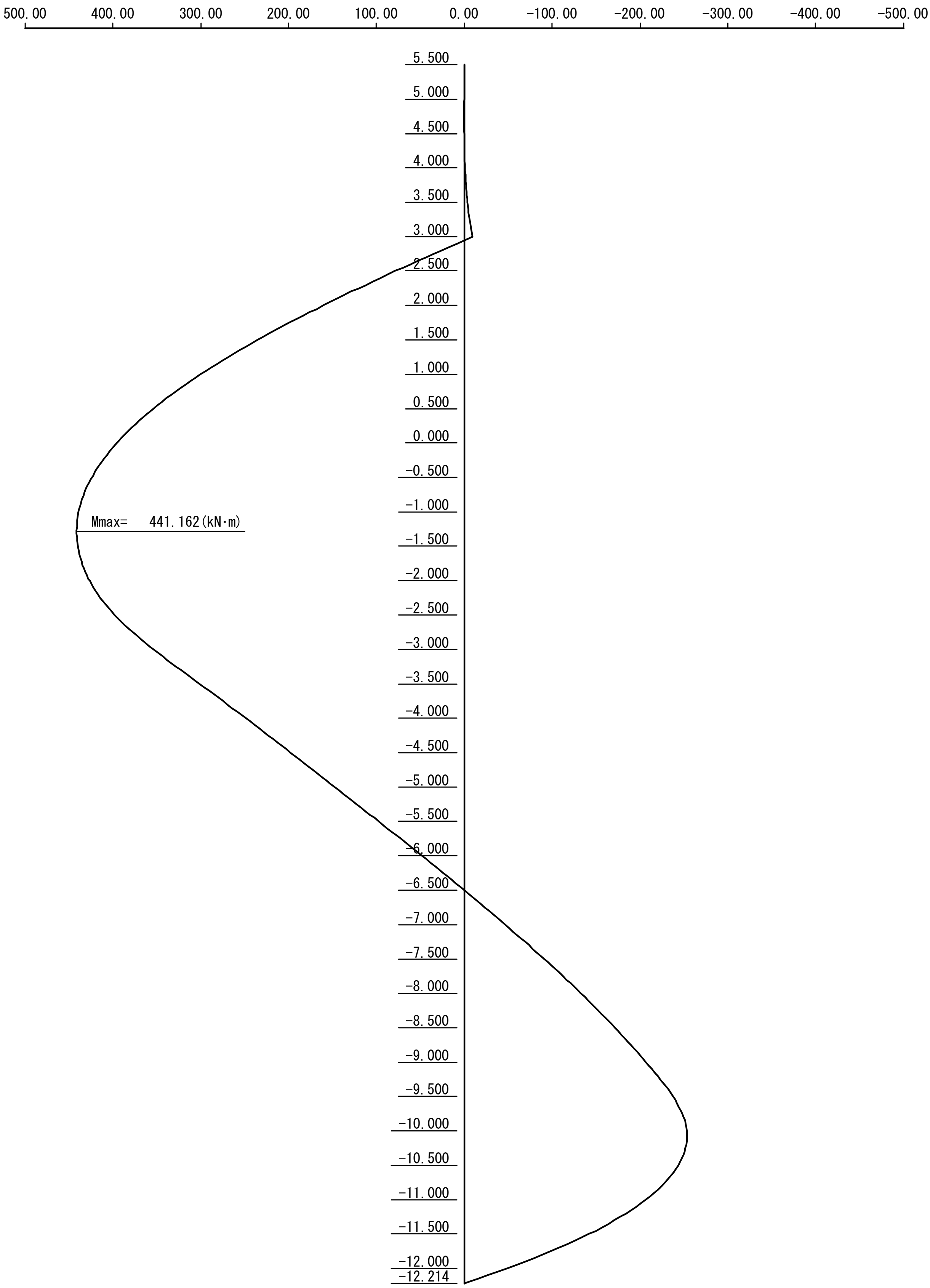
Embedded length of the pipe wall : L = 1.2 × (-2.000 + 12.214) = 12.257 m
Toe elevation : D = -2.000 – 12.257 = -14.257 m
Reaction force at tie setting point : 197.324 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	0.000	0.000
4.500	0.000	0.000
4.000	-0.837	-3.565
3.550	-3.375	-7.894
3.050	-8.944	-14.921
3.000	-9.712	-15.802
2.500	78.534	170.918
2.000	160.664	157.062
1.500	235.054	139.953
1.000	300.075	119.592
0.500	354.103	95.978
0.430	360.697	92.412
-0.070	400.419	66.353
-0.570	426.929	39.566
-1.070	439.864	12.052
-1.570	438.860	-16.189
-2.000	426.571	-41.059
-2.500	398.027	-73.115
-3.000	353.456	-105.171
-3.500	300.827	-105.052
-4.000	248.695	-103.183
-4.500	197.685	-101.064
-5.000	147.423	-100.195
-5.500	97.469	-99.451
-6.000	48.138	-97.707
-6.500	-0.071	-94.963
-7.000	-46.658	-91.219
-7.500	-91.123	-86.475
-8.000	-132.967	-80.731
-8.500	-171.688	-73.987
-9.000	-206.787	-66.243
-9.500	-237.764	-57.499
-10.000	-253.703	-6.005
-10.500	-243.519	46.989
-11.000	-206.464	101.483
-11.500	-141.787	157.477
-12.000	-48.737	214.971
-12.214	0.000	240.070

Maximum bending moment : 441.162 kN-m/m
Elevation of maximum bending moment : -1.285 m
Depth of 1st steady point (moment=0) : -6.499 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

M_p = FOS×M_a

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
—	5.50	1/2x	0.000x	1.000	0.000	-2.167	0.000	
—	4.50	1/2x	0.000x	1.000	0.000	-1.833	0.000	0.000
1	4.50	1/2x	5.820x	0.950	2.764	-1.183	-3.270	
2	3.55	1/2x	10.800x	0.950	5.130	-0.867	-4.448	-9.262
3	3.55	1/2x	10.800x	0.550	2.970	-0.367	-1.090	
4	3.00	1/2x	17.955x	0.550	4.938	-0.183	-0.904	-11.655
5	3.00	1/2x	17.955x	2.570	23.072	0.857	19.773	
6	0.43	1/2x	51.391x	2.570	66.037	1.713	113.121	147.818
7	0.43	1/2x	51.391x	2.430	62.440	3.380	211.047	
8	-2.00	1/2x	58.462x	2.430	71.031	4.190	297.620	758.218
9	-2.00	1/2x	94.112x	1.000	47.056	5.333	250.950	
10	-3.00	1/2x	101.112x	1.000	50.556	5.667	286.501	1403.159
11	-3.00	1/2x	38.512x	1.000	19.256	6.333	121.948	
12	-4.00	1/2x	38.512x	1.000	19.256	6.667	128.380	1703.553
13	-4.00	1/2x	38.512x	1.000	19.256	7.333	141.204	
14	-5.00	1/2x	38.512x	1.000	19.256	7.667	147.636	2050.161
15	-5.00	1/2x	38.512x	3.000	57.768	9.000	519.912	
16	-8.00	1/2x	38.512x	3.000	57.768	10.000	577.680	3367.271
17	-8.00	1/2x	38.512x	1.500	28.884	11.500	332.166	
18	-9.50	1/2x	38.512x	1.500	28.884	12.000	346.608	4181.800
19	-9.50	1/2x	38.512x	2.500	48.140	13.333	641.851	
20	-12.00	1/2x	38.512x	2.500	48.140	14.167	681.999	5770.420
21	-12.00	1/2x	38.512x	3.500	67.396	16.167	1089.591	
22	-15.50	1/2x	38.512x	3.500	67.396	17.333	1168.175	8479.739
23	-15.50	1/2x	38.512x	4.500	86.652	20.000	1733.040	
24	-20.00	1/2x	38.512x	4.500	86.652	21.500	1863.018	12795.009

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

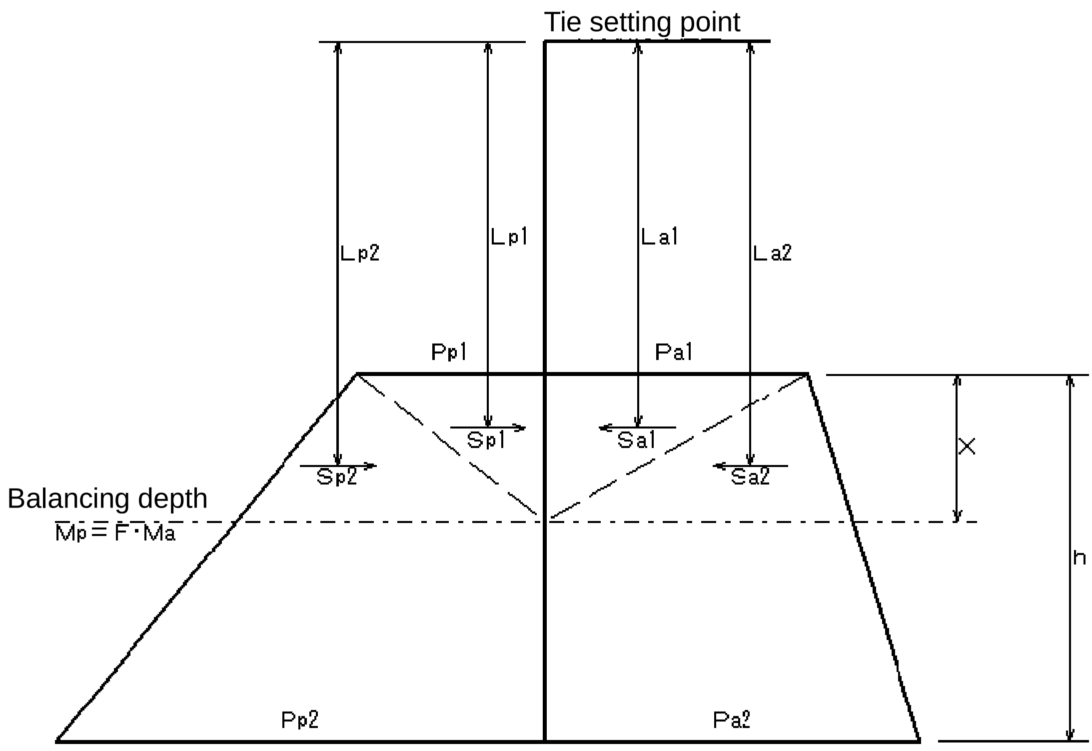
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-2.00	1/2x 30.000x 1.000	15.000	5.333	79.995	
2	-3.00	1/2x 37.000x 1.000	18.500	5.667	104.840	184.835
3	-3.00	1/2x 37.000x 1.000	18.500	6.333	117.160	
4	-4.00	1/2x 44.000x 1.000	22.000	6.667	146.674	448.669
5	-4.00	1/2x 44.000x 1.000	22.000	7.333	161.326	
6	-5.00	1/2x 39.000x 1.000	19.500	7.667	149.506	759.501
7	-5.00	1/2x 39.000x 3.000	58.500	9.000	526.500	
8	-8.00	1/2x 51.000x 3.000	76.500	10.000	765.000	2051.001
9	-8.00	1/2x 51.000x 1.500	38.250	11.500	439.875	
10	-9.50	1/2x 57.000x 1.500	42.750	12.000	513.000	3003.876
11	-9.50	1/2x 140.000x 2.500	175.000	13.333	2333.275	
12	-12.00	1/2x 155.000x 2.500	193.750	14.167	2744.856	8082.007
13	-12.00	1/2x 155.000x 3.500	271.250	16.167	4385.299	
14	-15.50	1/2x 176.000x 3.500	308.000	17.333	5338.564	17805.870
15	-15.50	1/2x 160.000x 4.500	360.000	20.000	7200.000	
16	-20.00	1/2x 178.000x 4.500	400.500	21.500	8610.750	33616.620

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \;,\;\; L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -9.50 m FOS×M_{a1} = 4181.800 > M_{p1} = 3003.876
at -12.00 m FOS×M_{a2} = 5770.420 < M_{p2} = 8082.007

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \ \ L_{a1} = 12.500 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 240.700X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{1.500}X\right]X}{2} = 0.000X^2 + 19.256X \ , \ \ L_{a2} = 12.500 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 240.700X$$

$$M_a = 0.000X^3 + 19.256X^2 + 481.400X + 3484.834$$

[Moment on passive side]

$$S_{p1} = \frac{140.000X}{2} = 70.000X \ , \ \ L_{p1} = 12.500 + \frac{1}{3}X$$

$$M_{p1'} = 23.333X^2 + 875.000X$$

$$S_{p2} = \frac{\left[140.000 + \frac{149.000 - 140.000}{1.500}X\right]X}{2} = 3.000X^2 + 70.000X \ , \ \ L_{p2} = 12.500 + \frac{2}{3}X$$

$$M_{p2'} = 2.000X^3 + 84.167X^2 + 875.000X$$

$$M_p = 2.000X^3 + 107.500X^2 + 1750.000X + 3003.876$$

[Balancing depth]

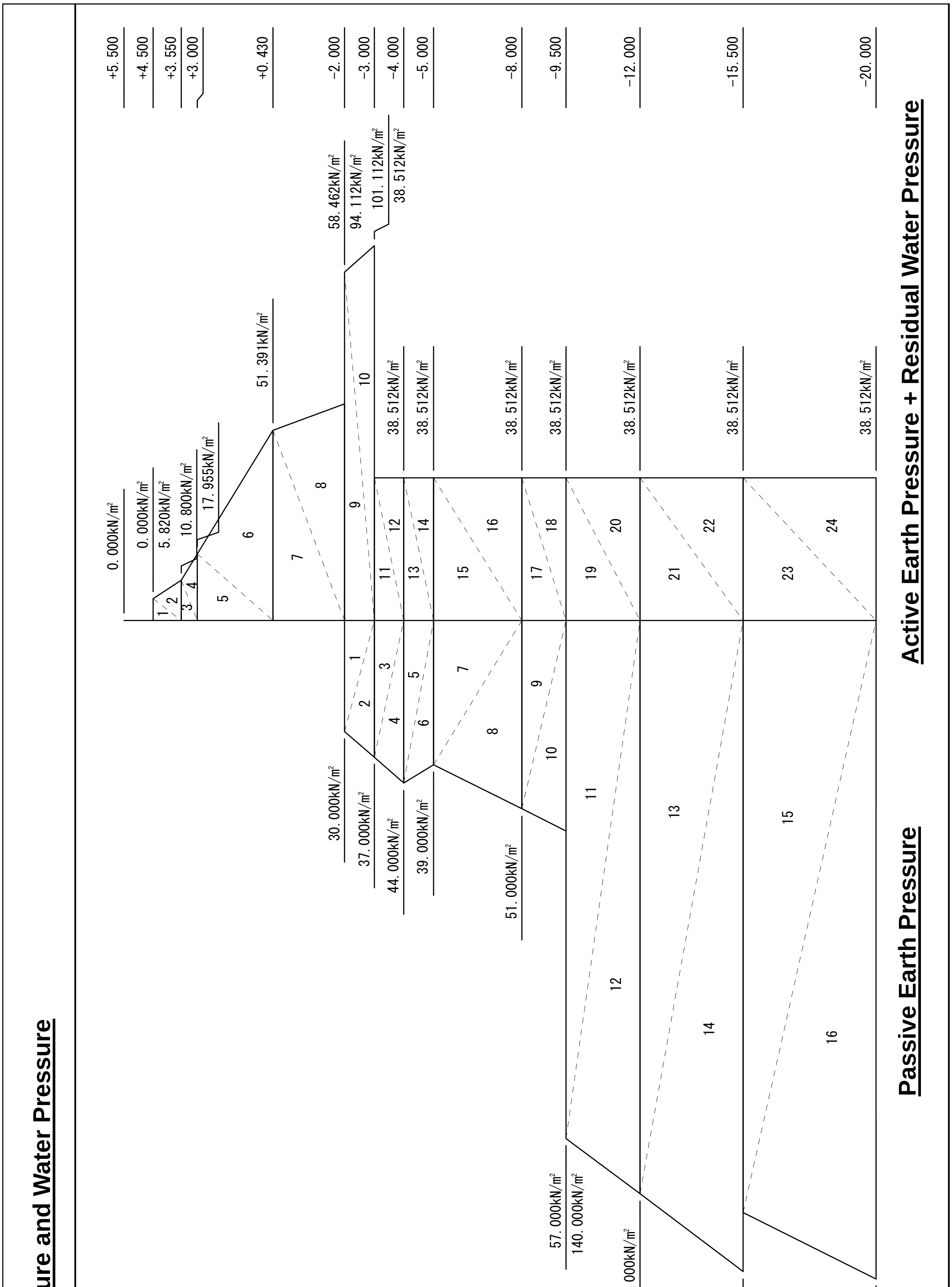
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 481.400X + 3484.834)$$

$$= 2.000X^3 + 107.500X^2 + 1750.000X + 3003.876$$

X= 0.940 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -9.500 – 0.940 = -10.440 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-8.000
-9.500
-12.000
-15.500
-20.000
0.000kN/m²
0.000kN/m²
5.820kN/m²
10.800kN/m²
17.955kN/m²
51.391kN/m²
58.462kN/m²
94.112kN/m²
101.112kN/m²
38.512kN/m²
30.000kN/m²
37.000kN/m²
44.000kN/m²
39.000kN/m²
51.000kN/m²
57.000kN/m²
140.000kN/m²
155.000kN/m²
176.000kN/m²
160.000kN/m²
178.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 4.50	0.000 0.000	——— ———	0.000 0.000
4.50 3.55	5.820 10.800	——— ———	5.820 10.800
3.55 3.00	10.800 17.955	——— ———	10.800 17.955
3.00 0.43	17.955 51.391	——— ———	17.955 51.391
0.43 -2.00	51.391 58.462	——— ———	51.391 58.462
-2.00 -3.00	94.112 101.112	30.000 37.000	64.112 64.112
-3.00 -4.00	38.512 38.512	37.000 44.000	1.512 -5.488
-4.00 -5.00	38.512 38.512	44.000 39.000	-5.488 -0.488
-5.00 -8.00	38.512 38.512	39.000 51.000	-0.488 -12.488
-8.00 -9.50	38.512 38.512	51.000 57.000	-12.488 -18.488
-9.50 -12.00	38.512 38.512	140.000 155.000	-101.488 -116.488
-12.00 -15.50	38.512 38.512	155.000 176.000	-116.488 -137.488
-15.50 -20.00	38.512 38.512	160.000 178.000	-121.488 -139.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $P_{a1} = 38.512 > P_{p1} = 37.000$
at -4.00 m $P_{a2} = 38.512 < P_{p2} = 44.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{1.00 \times (37.000 - 38.512)}{(38.512 - 38.512) - (44.000 - 37.000)} = -3.216m$$

Accordingly, the virtual seabed level is obtained as -3.216m.

3) Moment about tie setting point

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 5.820x 0.950	2.764	-1.183	-3.270
2	1/2x 10.800x 0.950	5.130	-0.867	-4.448
3	1/2x 10.800x 0.550	2.970	-0.367	-1.090
4	1/2x 17.955x 0.550	4.938	-0.183	-0.904
5	1/2x 17.955x 2.570	23.072	0.857	19.773
6	1/2x 51.391x 2.570	66.037	1.713	113.121
7	1/2x 51.391x 2.430	62.440	3.380	211.047
8	1/2x 58.462x 2.430	71.031	4.190	297.620
9	1/2x 64.112x 1.000	32.056	5.333	170.955
10	1/2x 64.112x 1.000	32.056	5.667	181.661
11	1/2x 1.512x 0.216	0.163	6.072	0.990
Total		302.657	——	985.455

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

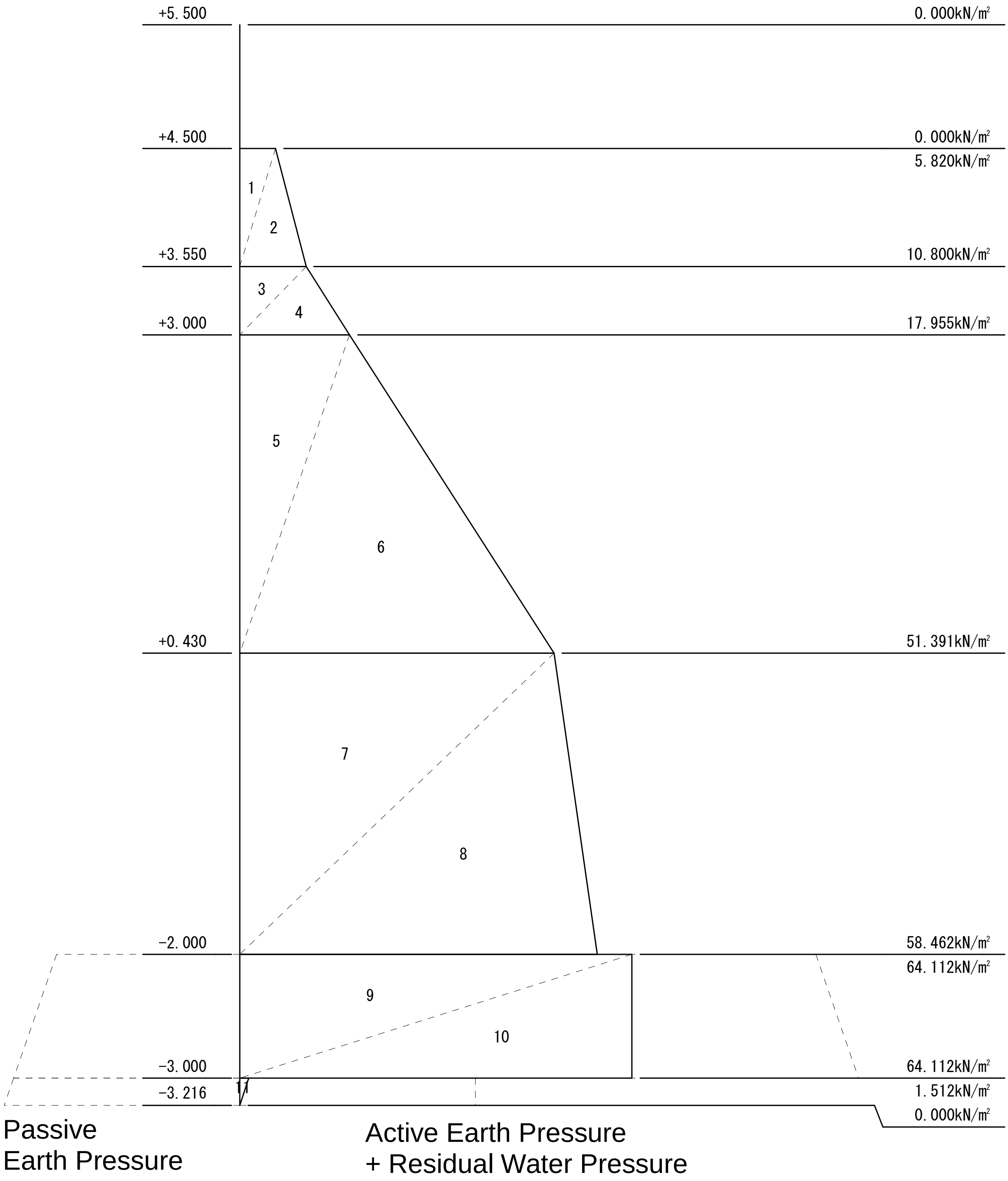
4) Reaction forces

Distance between supports : L_T = 3.000 – (-3.216) = 6.216 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{985.455}{6.216} = 158.535kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 302.657 - 158.535 = 144.122kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 4.50	0.000 0.000	0.000		0.000
4.50 3.55	2.764 5.130	7.894		-7.894
3.55 3.00	2.970 4.938	15.802	144.122	128.320
3.00 0.43	23.072 66.037	104.911	144.122	39.211
0.43 -2.00	62.440 71.031	238.382	144.122	-94.260
-2.00 -3.00	32.056 32.056	302.494	144.122	-158.372
-3.00 -3.22	0.163 0.000	302.657	144.122	-158.535

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -2.000m.

$$Q = 39.211 - \frac{[51.391 + (51.391 + 2.910X)]X}{2} = 0$$

X = 0.747 m

Shear force zero point : DL = +0.43 – 0.747 = -0.317m

6) Calculation of moment

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 5.820x 0.950	-2.764	4.500	-12.438
2	1/2x 10.800x 0.950	-5.130	4.184	-21.464
3	1/2x 10.800x 0.550	-2.970	3.684	-10.941
4	1/2x 17.955x 0.550	-4.938	3.500	-17.283
5	1/2x 17.955x 2.570	-23.072	2.460	-56.757
6	1/2x 51.391x 2.570	-66.037	1.604	-105.923
7	1/2x 51.391x 0.747	-19.195	0.498	-9.559
8	1/2x 53.565x 0.747	-20.007	0.249	-4.982
Total		———	———	-239.347

where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

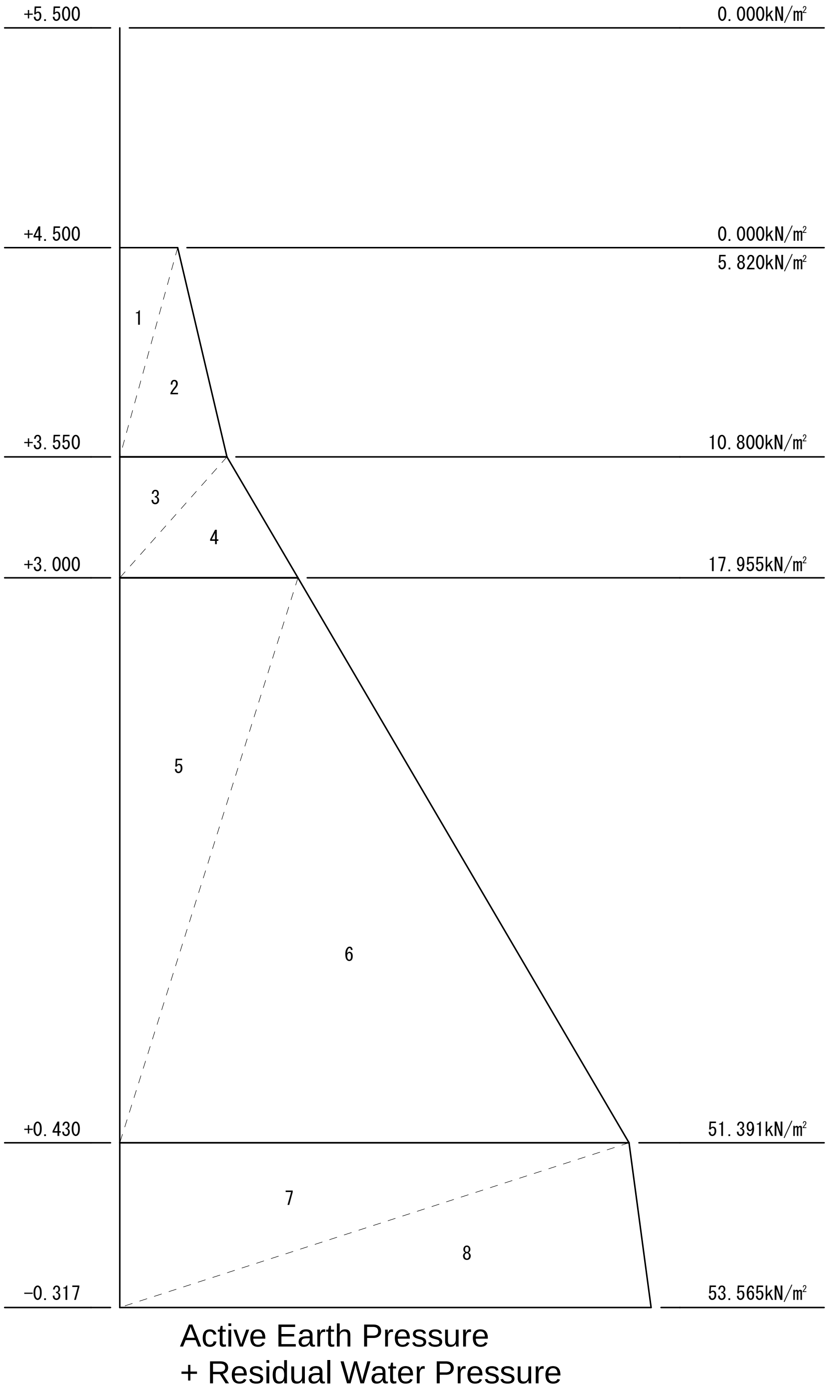
- Distance of the shear zero point to the tie setting point

h = 3.000 – (-0.317) = 3.317 m

- Maximum bending moment

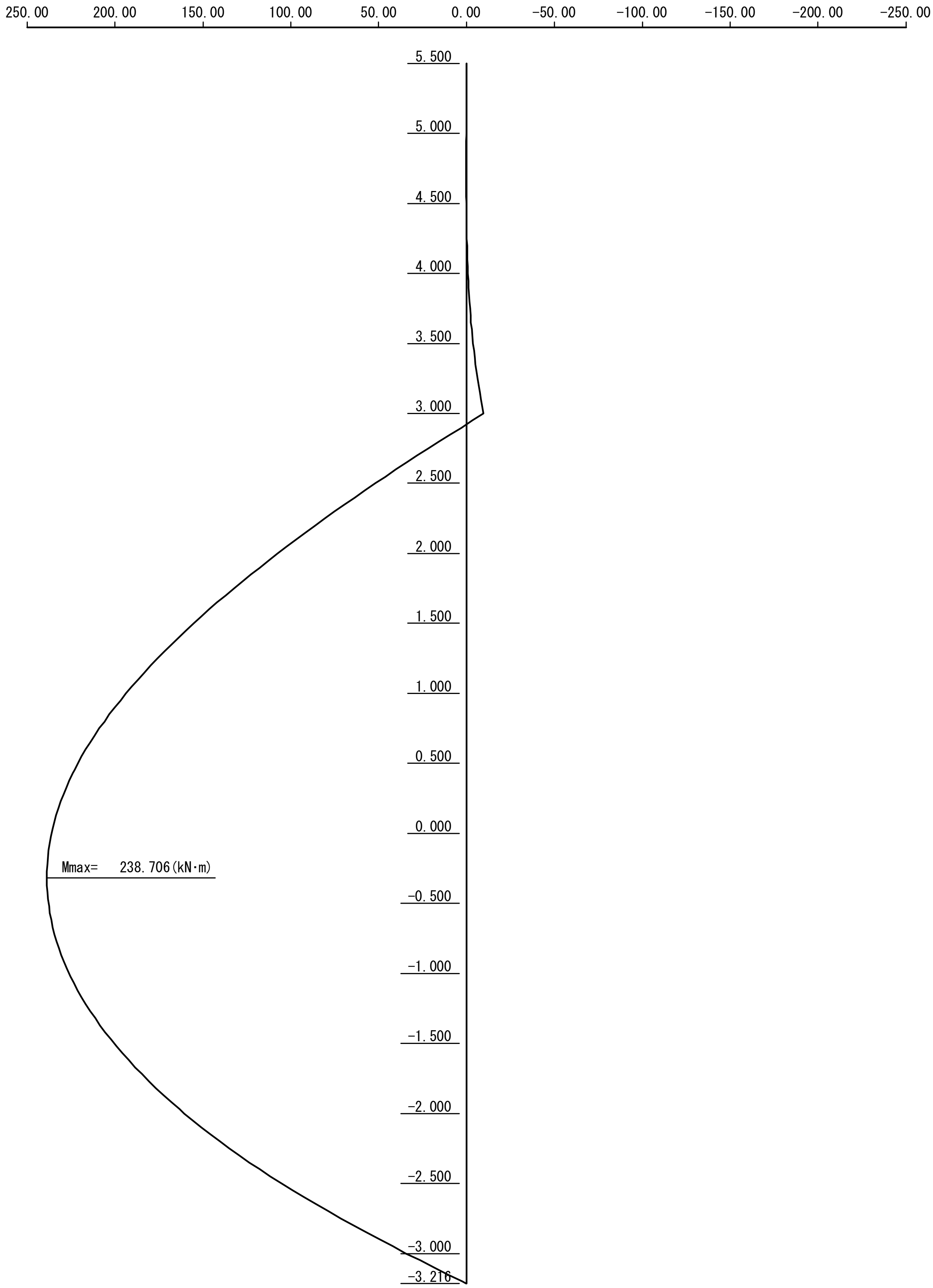
$M_{max} = A_p \times h + \sum M = 144.122 \times 3.317 - 239.347 = 238.706$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion

- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall

Toe level of the pipe wall is obtained as follows:

- Deflection curve method : -14.257 m --- adopted
- Free earth support method : -10.440 m

Therefore, the minimum length of the pipe wall is;

L = 3.500 – (-14.257) = 17.757m → 18.000 m

Accordingly the design toe level of the pipe wall is;

EL = +3.500 – 18.000 = -14.500 m

(3) Stress on pipe wall section

Maximum bending moment on the pipe wall is as follows:

- Deflection curve method : 441.162 kN-m/m --- adopted
- Free earth support method : 238.706 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{441.162 \times 10^6}{4191 \times 10^3} = 105.3 MPa \quad \leq \quad \sigma_a = 140 MPa \text{ .... } OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

T	is the tension force on tie wire (kN/wire)
A_p	is the reaction force of the pipe wall at tie setting point (kN/m)
L	is the spacing of the tie wire (m)
θ_1	is the vertical angle of the tie wire to horizontal (deg.)
θ_2	is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 197.324 kN-m/m --- adopted
- Free earth support method : 144.122 kN-m/m

T = 197.324 × 1.960 × sec(0.000) × sec (0.000) = 386.755 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 386.755 × 3.8 = 1469.669 kN/wire

Tie wire shall have minimum tensile strength of 1470 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{386.755 \times 1.960}{10.0} = 75.804 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \le \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{75.804 \times 10^6}{2 \times 374.0 \times 10^3} = 101.3 MPa \quad \le \quad \sigma_a = 140 MPa \text{ .... } OK$$

Output of Design Software for Anchor Wall (During Construction) – Revetment Block d

1. Design Conditions

1-1 Dimensions

Ground elevation	+4.50 m
Top of cope concrete	+5.50 m
Top of pipe wall	+3.50 m
Tie elevation	+3.00 m
Seabed elevation	-2.00 m
Angle of seabed slope	18.4°

1-2 Tide Levels

H.W.L	+3.55 m
L.W.L	+0.43 m

1-3 Residual Water Level

R.W.L	+3.55 m

1-4 Base Elevation of Cohesion Increment +0.00 m (not used)

1-5 Surcharge Load	(Active side)	20 kPa
	(Passive side)	0 kPa

1-6 Unit Weight of Water $10.10\ kN/m^3$

1-7 Allowable Stresses

Pipe Wall	(SKY400)	140 MPa
Structural Steel	(SS400)	140 MPa

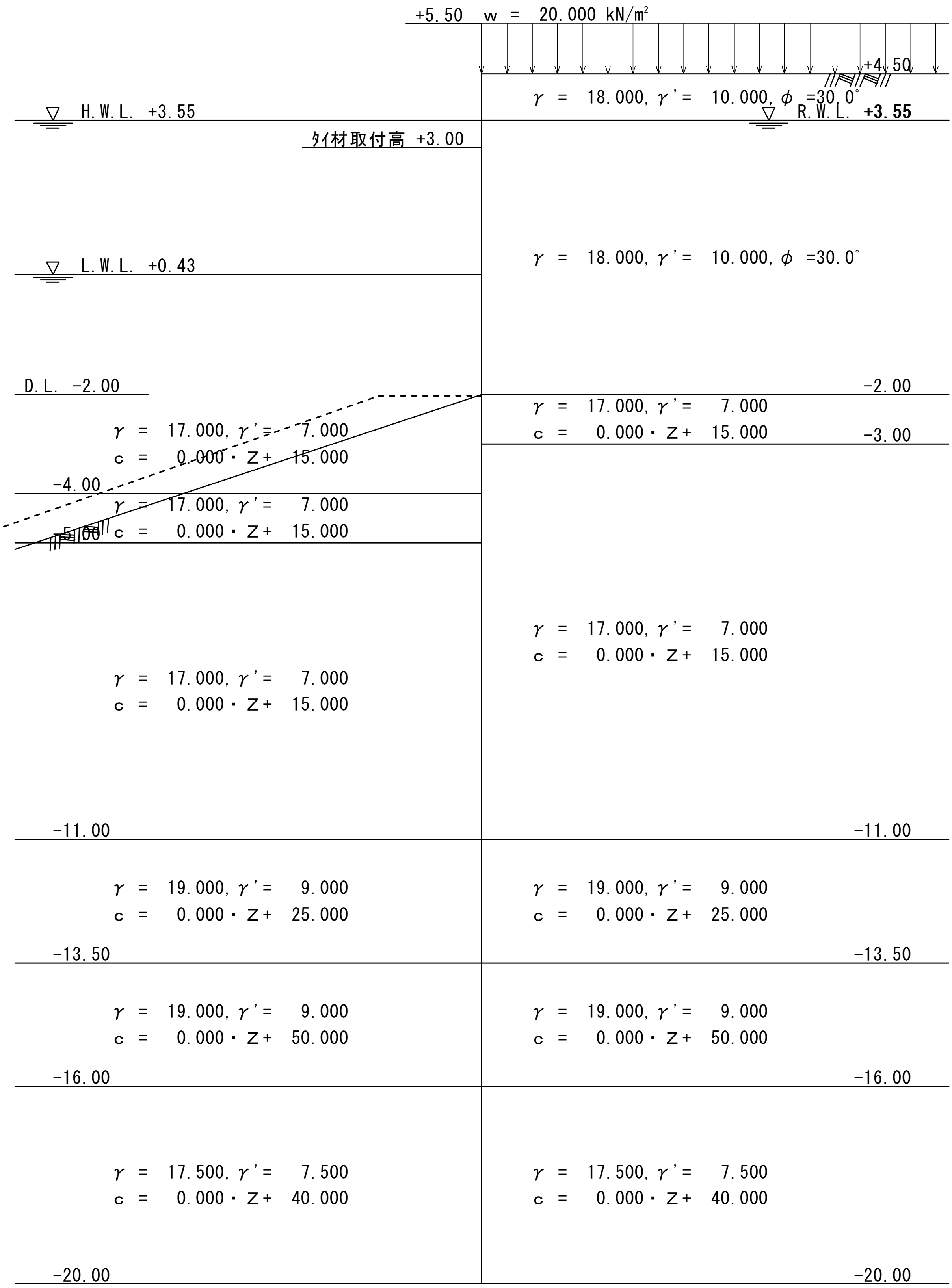
1-8 Corrosion Allowance

Pipe Wall	0.03 mm/yr × 50 yrs =	1.50 mm (outside only)

1-9 Embedded Length of Pipe Wall

Calculation Method 1	:	Deflection Curve Method
Calculation Method 2	:	Free Earth Support Method
Factor of Safety	:	FOS = 1.2

Profile of Revetment and Soil Conditions



2. Pipe Wall

2-1 Earth Pressure and Water Pressure

(1) Active earth pressure

Elev. (m)	h (m)	P_a (kPa)
4.50 3.55	0.95	5.820 10.800
3.55 3.00	0.55	10.800 12.400
3.00 -2.00	5.00	12.400 26.950
-2.00 -3.00	1.00	62.600 69.600
-3.00 -11.00	8.00	7.000 7.000
-11.00 -12.00	1.00	7.000 7.000
-12.00 -13.50	1.50	7.000 7.000
-13.50 -16.00	2.50	7.000 7.000
-16.00 -20.00	4.00	7.000 7.000

(2) Residual water pressure

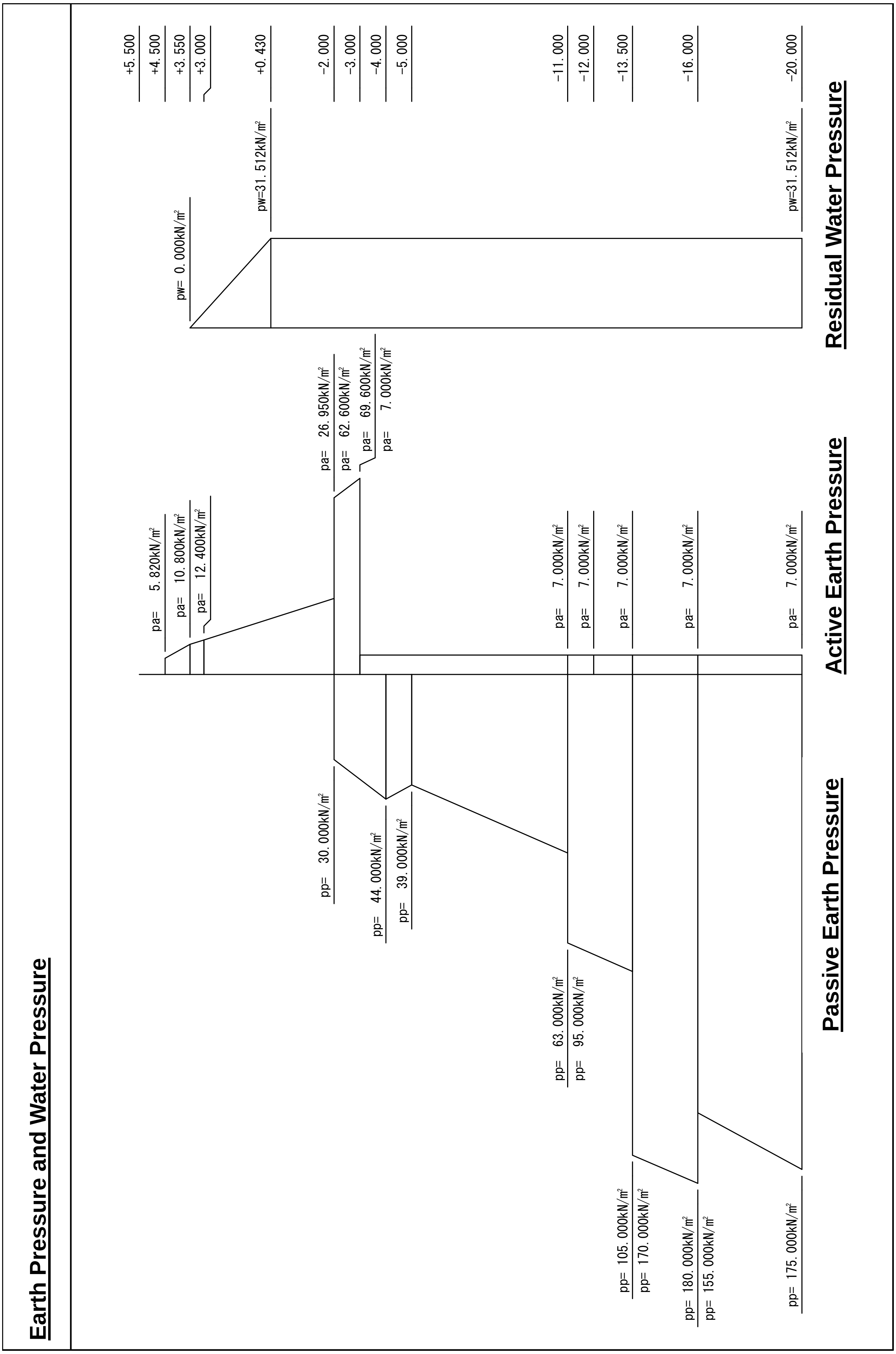
$p_w = \gamma_w \times h_w$

where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

pw = 10.100 × 3.120 = 31.512 kPa

(3) Passive earth pressure

Elev. (m)	h (m)	P_p (kPa)
-2.00 -4.00	2.00	30.000 44.000
-4.00 -5.00	1.00	44.000 39.000
-5.00 -11.00	6.00	39.000 63.000
-11.00 -13.50	2.50	95.000 105.000
-13.50 -16.00	2.50	170.000 180.000
-16.00 -20.00	4.00	155.000 175.000


Earth Pressure and Water Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-11.000
-12.000
-13.500
-16.000
-20.000
pw= 0.000kN/m²
pw=31.512kN/m²
pw=31.512kN/m²
Residual Water Pressure
pa= 5.820kN/m²
pa= 10.800kN/m²
pa= 12.400kN/m²
pa= 26.950kN/m²
pa= 62.600kN/m²
pa= 69.600kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
pa= 7.000kN/m²
Active Earth Pressure
pp= 30.000kN/m²
pp= 44.000kN/m²
pp= 39.000kN/m²
pp= 63.000kN/m²
pp= 95.000kN/m²
pp= 105.000kN/m²
pp= 170.000kN/m²
pp= 180.000kN/m²
pp= 155.000kN/m²
pp= 175.000kN/m²
Passive Earth Pressure

(4) Summary of Forces on the Wall

Elev (m)	Active side (kPa) $P_a + P_w$	Passive side(kPa) P_p
5.50 4.50	0.000+ 0.000= 0.000 0.000+ 0.000= 0.000	——— ———
4.50 3.55	5.820+ 0.000= 5.820 10.800+ 0.000= 10.800	——— ———
3.55 3.00	10.800+ 0.000= 10.800 12.400+ 5.555= 17.955	——— ———
3.00 0.43	12.400+ 5.555= 17.955 19.879+ 31.512= 51.391	——— ———
0.43 -2.00	19.879+ 31.512= 51.391 26.950+ 31.512= 58.462	——— ———
-2.00 -3.00	62.600+ 31.512= 94.112 69.600+ 31.512= 101.112	30.000 37.000
-3.00 -4.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	37.000 44.000
-4.00 -5.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	44.000 39.000
-5.00 -11.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	39.000 63.000
-11.00 -12.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	95.000 99.000
-12.00 -13.50	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	99.000 105.000
-13.50 -16.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	170.000 180.000
-16.00 -20.00	7.000+ 31.512= 38.512 7.000+ 31.512= 38.512	155.000 175.000

2-2 Deflection Curve Method

(1) Calculation of embedded length

The elastic equations are solved under the external force conditions obtained in 2-1 with the conditions that the displacement and deflection angle is zero at the tie setting point and the toe of the pipe wall.

For this purpose, the pipe wall is assumed to be simply supported at the tie setting point and the toe level of the pipe wall and the deflection angle at the toe of the pipe wall is calculated by changing the embedded length of the pipe wall. Once the embedded length of the pipe wall with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the pipe wall shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

Seabed elevation: -2.000 m

Toe of pipe wall (m)	Deflection angle θ×EI	Reaction at Tie (kN/m)	Reaction at Toe (kN/m)
-2.000	-239.109	112.012	126.370
-3.000	-446.199	138.417	164.077
-4.000	-686.260	161.798	138.708
-5.000	-899.495	178.897	118.621
-6.000	-1095.816	191.976	103.054
-7.000	-1271.134	201.990	86.552
-8.000	-1414.513	209.412	68.642
-9.000	-1512.401	214.558	49.008
-10.000	-1548.812	217.640	27.438
-11.000	-1505.437	218.822	3.768
-12.000	-1286.949	217.145	-53.043
-13.000	-794.823	211.898	-110.284
-14.000	43.246	202.997	-200.371

The above results suggest that the toe level of the pipe wall by which the deflection angle at the toe of the pipe wall becomes zero would be in between -13.00 and -14.00. Such depth was obtained by the iterative calculation as follows.

Toe of pipe wall (m)	Deflection angle at toe (deg)	Reaction at tie (kN/m)	Reaction at toe (kN/m)
-13.961	0.000	203.449	-195.669

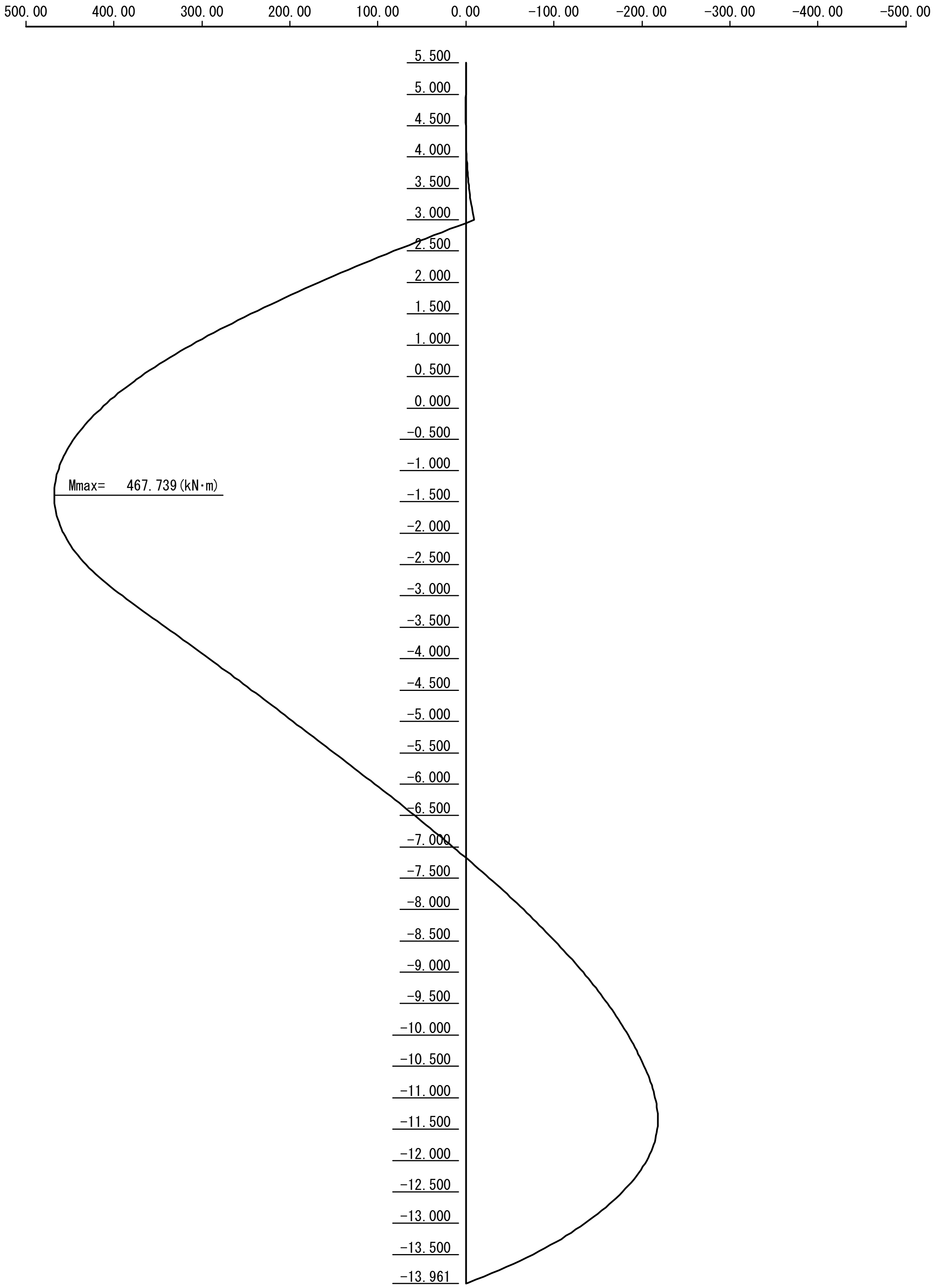
Embedded length of the pipe wall : $L = 1.2 \times (-2.000 + 13.961) = 14.353$ m
Toe elevation : $D = -2.000 - 14.353 = -16.353$ m
Reaction force at tie setting point : 203.449 kN/m

(2) Calculation of maximum bending moment

Elev (m)	Bending moment (kN-m/m)	Shear force (kN/m)
5.500	0.000	0.000
5.000	0.000	0.000
4.500	0.000	0.000
4.000	-0.837	-3.565
3.550	-3.375	-7.894
3.050	-8.944	-14.921
3.000	-9.712	-15.802
2.500	81.596	177.043
2.000	166.789	163.187
1.500	244.241	146.078
1.000	312.325	125.717
0.500	369.416	102.103
0.430	376.439	98.537
-0.070	419.223	72.478
-0.570	448.795	45.691
-1.070	464.793	18.177
-1.570	466.851	-10.064
-2.000	457.196	-34.934
-2.500	431.715	-66.990
-3.000	390.206	-99.046
-3.500	340.639	-98.927
-4.000	291.570	-97.058
-4.500	243.623	-94.939
-5.000	196.423	-94.070
-5.500	149.532	-93.326
-6.000	103.263	-91.582
-6.500	58.117	-88.838
-7.000	14.592	-85.094
-7.500	-26.811	-80.350
-8.000	-65.592	-74.606
-8.500	-101.250	-67.862
-9.000	-133.287	-60.118
-9.500	-161.202	-51.374
-10.000	-184.495	-41.630
-10.500	-202.665	-30.886
-11.000	-215.214	-19.142
-11.500	-217.641	9.602
-12.000	-205.446	39.346
-12.500	-178.128	70.090
-13.000	-135.189	101.834
-13.500	-76.128	134.578
-13.961	0.000	195.669

Maximum bending moment : 467.739 kN-m/m
Elevation of maximum bending moment : -1.393 m
Depth of 1st steady point (moment=0) : -7.173 m

Bending moment diagram – Deflection Curve Method



2-3 Free Earth Support Method

(1) Calculation of embedded length

The embedded length of the pipe wall is obtained so that the following formula is satisfied.

M_p = FOS×M_a

where;

M_p is the moment due to the passive earth pressure about tie setting point (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the tie setting point (kN-m/m)

FOS =1.2

1) Calculation of FOS×M_a

NO	Elev (m)	Calculation			S (kN/m)	L (m)	M (kN-m/m)	FOS×M_a (kN-m/m)
─	5.50	1/2x	0.000x	1.000	0.000	-2.167	0.000	
─	4.50	1/2x	0.000x	1.000	0.000	-1.833	0.000	0.000
1	4.50	1/2x	5.820x	0.950	2.764	-1.183	-3.270	
2	3.55	1/2x	10.800x	0.950	5.130	-0.867	-4.448	-9.262
3	3.55	1/2x	10.800x	0.550	2.970	-0.367	-1.090	
4	3.00	1/2x	17.955x	0.550	4.938	-0.183	-0.904	-11.655
5	3.00	1/2x	17.955x	2.570	23.072	0.857	19.773	
6	0.43	1/2x	51.391x	2.570	66.037	1.713	113.121	147.818
7	0.43	1/2x	51.391x	2.430	62.440	3.380	211.047	
8	-2.00	1/2x	58.462x	2.430	71.031	4.190	297.620	758.218
9	-2.00	1/2x	94.112x	1.000	47.056	5.333	250.950	
10	-3.00	1/2x	101.112x	1.000	50.556	5.667	286.501	1403.159
11	-3.00	1/2x	38.512x	1.000	19.256	6.333	121.948	
12	-4.00	1/2x	38.512x	1.000	19.256	6.667	128.380	1703.553
13	-4.00	1/2x	38.512x	1.000	19.256	7.333	141.204	
14	-5.00	1/2x	38.512x	1.000	19.256	7.667	147.636	2050.161
15	-5.00	1/2x	38.512x	6.000	115.536	10.000	1155.360	
16	-11.00	1/2x	38.512x	6.000	115.536	12.000	1386.432	5100.311
17	-11.00	1/2x	38.512x	1.000	19.256	14.333	275.996	
18	-12.00	1/2x	38.512x	1.000	19.256	14.667	282.428	5770.420
19	-12.00	1/2x	38.512x	1.500	28.884	15.500	447.702	
20	-13.50	1/2x	38.512x	1.500	28.884	16.000	462.144	6862.235
21	-13.50	1/2x	38.512x	2.500	48.140	17.333	834.411	
22	-16.00	1/2x	38.512x	2.500	48.140	18.167	874.559	8912.999
23	-16.00	1/2x	38.512x	4.000	77.024	20.333	1566.129	
24	-20.00	1/2x	38.512x	4.000	77.024	21.667	1668.879	12795.009

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the tie point (m)

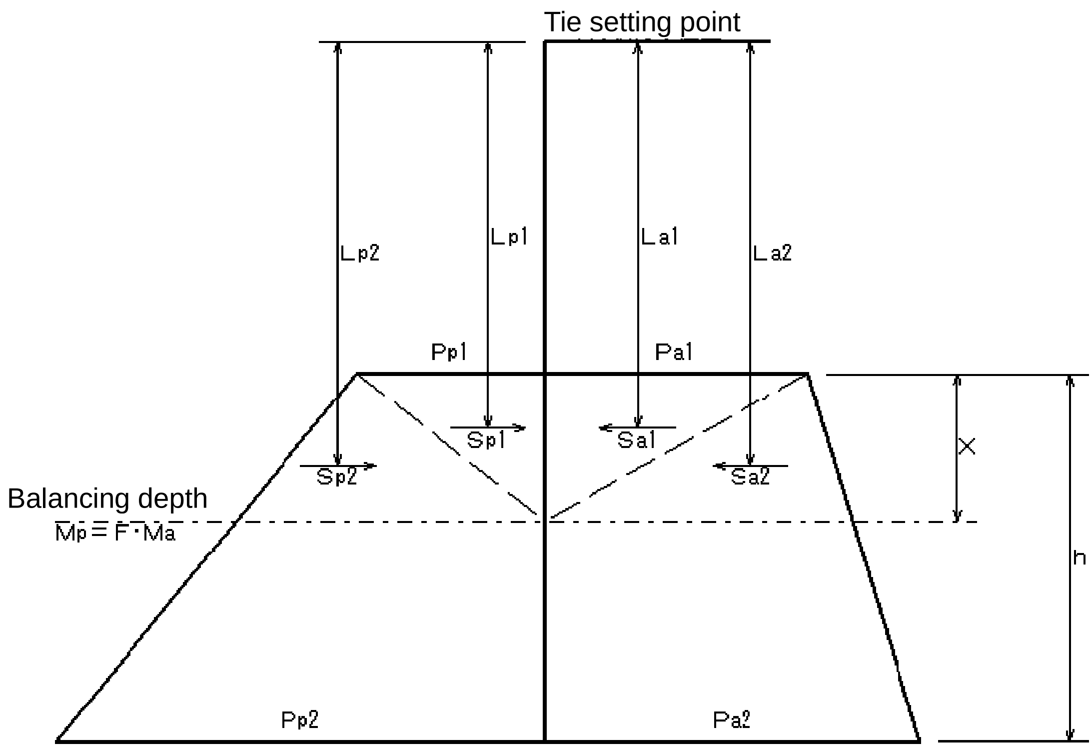
M_a is the moment about the tie setting point (kN-m/m)

2) Calculation of M_p

NO	Elev (m)	Calculation	S (kN/m)	L (m)	M (kN-m/m)	FOS×M_p (kN-m/m)
1	-2.00	1/2x 30.000x 1.000	15.000	5.333	79.995	
2	-3.00	1/2x 37.000x 1.000	18.500	5.667	104.840	184.835
3	-3.00	1/2x 37.000x 1.000	18.500	6.333	117.160	
4	-4.00	1/2x 44.000x 1.000	22.000	6.667	146.674	448.669
5	-4.00	1/2x 44.000x 1.000	22.000	7.333	161.326	
6	-5.00	1/2x 39.000x 1.000	19.500	7.667	149.506	759.501
7	-5.00	1/2x 39.000x 6.000	117.000	10.000	1170.000	
8	-11.00	1/2x 63.000x 6.000	189.000	12.000	2268.000	4197.501
9	-11.00	1/2x 95.000x 1.000	47.500	14.333	680.818	
10	-12.00	1/2x 99.000x 1.000	49.500	14.667	726.016	5604.335
11	-12.00	1/2x 99.000x 1.500	74.250	15.500	1150.875	
12	-13.50	1/2x 105.000x 1.500	78.750	16.000	1260.000	8015.210
13	-13.50	1/2x 170.000x 2.500	212.500	17.333	3683.262	
14	-16.00	1/2x 180.000x 2.500	225.000	18.167	4087.575	15786.047
15	-16.00	1/2x 155.000x 4.000	310.000	20.333	6303.230	
16	-20.00	1/2x 175.000x 4.000	350.000	21.667	7583.450	29672.727

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M_p is the moment about the tie setting point (kN-m/m)

3) Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X \right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)

$M_{a(p)1}$: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)

$M_{a(p)1'}$: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$M_{a(p)2'}$: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)

$S_{a(p)1}$: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$S_{a(p)2}$: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)

$L_{a(p)1}$: the arm length of $S_{a(p)1}$ from the tie setting point (m)

$L_{a(p)2}$: the arm length of $S_{a(p)2}$ from the tie setting point (m)

$P_{a(p)1}$: the active (passive) side pressure at the top elevation of the soil layer (kPa)

$P_{a(p)1}$: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)

h : the thickness of the soil layer (m)

L : the vertical distance from the tie setting point to the top elevation of the soil layer (m)

X : the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -12.00 m FOS×M_{a1} = 5770.420 > M_{p1} = 5604.335
at -13.50 m FOS×M_{a2} = 6862.235 > M_{p2} = 8015.210

[Moment on active side]

$$S_{a1} = \frac{38.512X}{2} = 19.256X \ , \quad L_{a1} = 15.000 + \frac{1}{3}X$$

$$M_{a1'} = 6.419X^2 + 288.840X$$

$$S_{a2} = \frac{\left[38.512 + \frac{38.512 - 38.512}{2.500}X\right]X}{2} = 0.000X^2 + 19.256X \ , \quad L_{a2} = 15.000 + \frac{2}{3}X$$

$$M_{a2'} = 0.000X^3 + 12.837X^2 + 288.840X$$

$$M_a = 0.000X^3 + 19.256X^2 + 577.680X + 4808.684$$

[Moment on passive side]

$$S_{p1} = \frac{99.000X}{2} = 49.500X \ , \quad L_{p1} = 15.000 + \frac{1}{3}X$$

$$M_{p1'} = 16.500X^2 + 742.500X$$

$$S_{p2} = \frac{\left[99.000 + \frac{105.000 - 99.000}{1.500}X\right]X}{2} = 2.000X^2 + 49.500X \ , \quad L_{p2} = 15.000 + \frac{2}{3}X$$

$$M_{p2'} = 1.333X^3 + 63.000X^2 + 742.500X$$

$$M_p = 1.333X^3 + 79.500X^2 + 1485.000X + 5604.335$$

[Balancing depth]

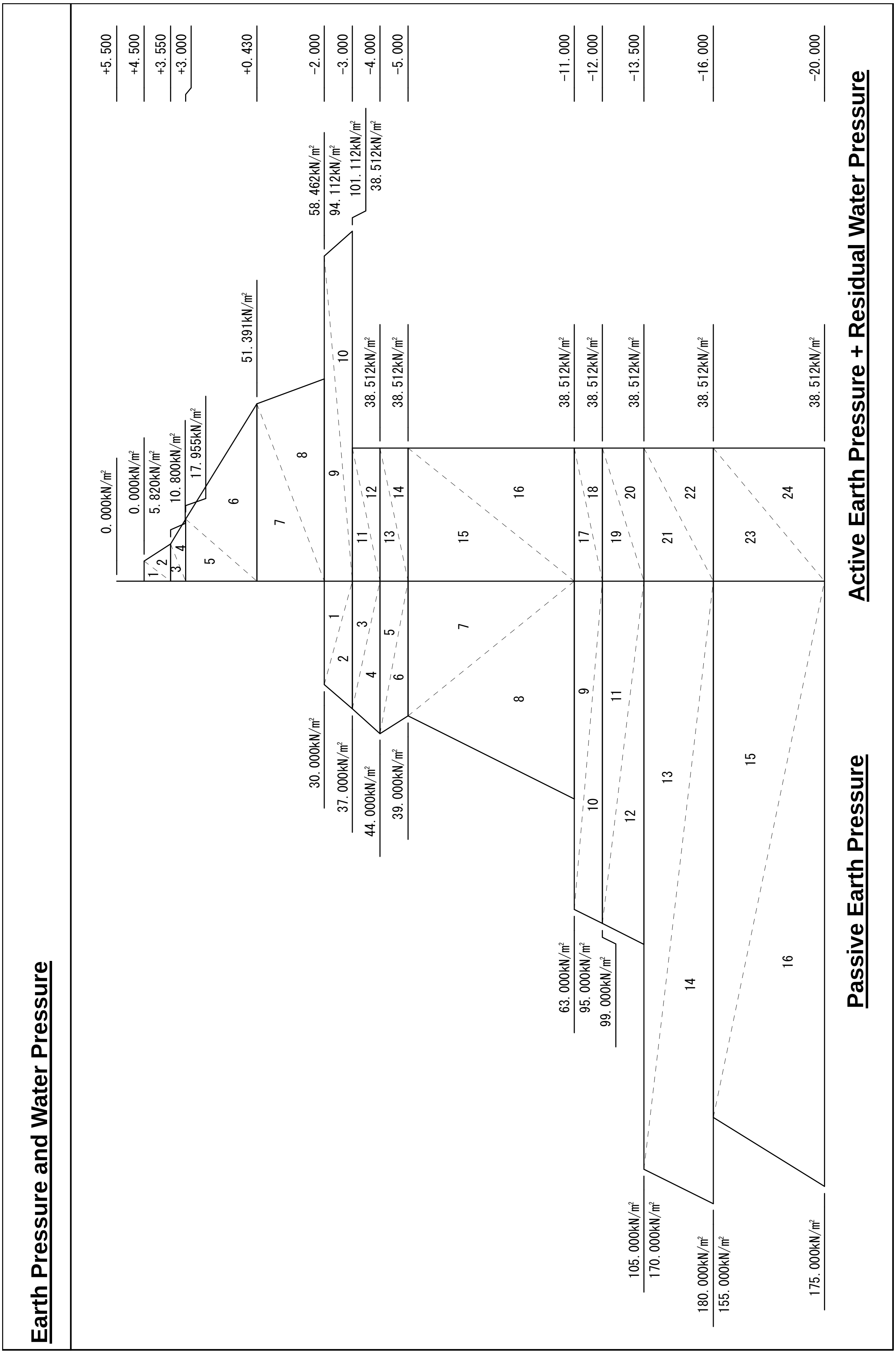
$$FOS \cdot M_a = M_p$$

$$1.2 \times (0.000X^3 + 19.256X^2 + 577.680X + 4808.684)$$

$$= 1.333X^3 + 79.500X^2 + 1485.000X + 5604.335$$

X= 0.207 m

Accordingly, toe elevation of the pipe wall shall be as follows:
D = -12.000 – 0.207 = -12.207 m


Earth Pressure and Water Pressure
Active Earth Pressure + Residual Water Pressure
Passive Earth Pressure
+5.500
+4.500
+3.550
+3.000
+0.430
-2.000
-3.000
-4.000
-5.000
-11.000
-12.000
-13.500
-16.000
-20.000
0.000kN/m²
0.000kN/m²
5.820kN/m²
10.800kN/m²
17.955kN/m²
51.391kN/m²
58.462kN/m²
94.112kN/m²
101.112kN/m²
38.512kN/m²
30.000kN/m²
37.000kN/m²
44.000kN/m²
39.000kN/m²
63.000kN/m²
95.000kN/m²
99.000kN/m²
105.000kN/m²
170.000kN/m²
180.000kN/m²
155.000kN/m²
175.000kN/m²

(2) Calculation of maximum bending moment

1) External forces

Elev (m)	$P_a + P_w$ (kN/m^2)	P_p (kN/m^2)	$P_a - P_p$ (kN/m^2)
5.50 4.50	0.000 0.000	——— ———	0.000 0.000
4.50 3.55	5.820 10.800	——— ———	5.820 10.800
3.55 3.00	10.800 17.955	——— ———	10.800 17.955
3.00 0.43	17.955 51.391	——— ———	17.955 51.391
0.43 -2.00	51.391 58.462	——— ———	51.391 58.462
-2.00 -3.00	94.112 101.112	30.000 37.000	64.112 64.112
-3.00 -4.00	38.512 38.512	37.000 44.000	1.512 -5.488
-4.00 -5.00	38.512 38.512	44.000 39.000	-5.488 -0.488
-5.00 -11.00	38.512 38.512	39.000 63.000	-0.488 -24.488
-11.00 -12.00	38.512 38.512	95.000 99.000	-56.488 -60.488
-12.00 -13.50	38.512 38.512	99.000 105.000	-60.488 -66.488
-13.50 -16.00	38.512 38.512	170.000 180.000	-131.488 -141.488
-16.00 -20.00	38.512 38.512	155.000 175.000	-116.488 -136.488

2) Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

at -3.00 m $\quad P_{a1} = 38.512 \quad > \quad P_{p1} = 37.000$
at -4.00 m $\quad P_{a2} = 38.512 \quad < \quad P_{p2} = 44.000$

$$X = EL_{top} - \frac{h(P_{p1} - P_{a1})}{(P_{a2} - P_{a1}) - (P_{p2} - P_{p1})}$$

Where;

EL_{top} : the top elevation of the soil layer in question (m)

P_{a1} : the active earth pressure plus residual water pressure at the top of the soil layer (kPa)

P_{a2} : the active earth pressure plus residual water pressure at the bottom of the soil layer (kPa)

P_{p1} : the passive earth pressure at the top of the soil layer (kPa)

P_{p2} : the passive earth pressure at the bottom of the soil layer (kPa)

h : the thickness of the soil layer (m)

$$X = -3.00 - \frac{1.00 \times (37.000 - 38.512)}{(38.512 - 38.512) - (44.000 - 37.000)} = -3.216m$$

Accordingly, the virtual seabed level is obtained as -3.216m.

3) Moment about tie setting point

No	Calculation			S (kN/m)	L (m)	M (kN-m/m)
1	1/2x	5.820x	0.950	2.764	-1.183	-3.270
2	1/2x	10.800x	0.950	5.130	-0.867	-4.448
3	1/2x	10.800x	0.550	2.970	-0.367	-1.090
4	1/2x	17.955x	0.550	4.938	-0.183	-0.904
5	1/2x	17.955x	2.570	23.072	0.857	19.773
6	1/2x	51.391x	2.570	66.037	1.713	113.121
7	1/2x	51.391x	2.430	62.440	3.380	211.047
8	1/2x	58.462x	2.430	71.031	4.190	297.620
9	1/2x	64.112x	1.000	32.056	5.333	170.955
10	1/2x	64.112x	1.000	32.056	5.667	181.661
11	1/2x	1.512x	0.216	0.163	6.072	0.990
Total				302.657	———	985.455

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the tie point (m)
M is the moment about the tie setting point (kN-m/m)

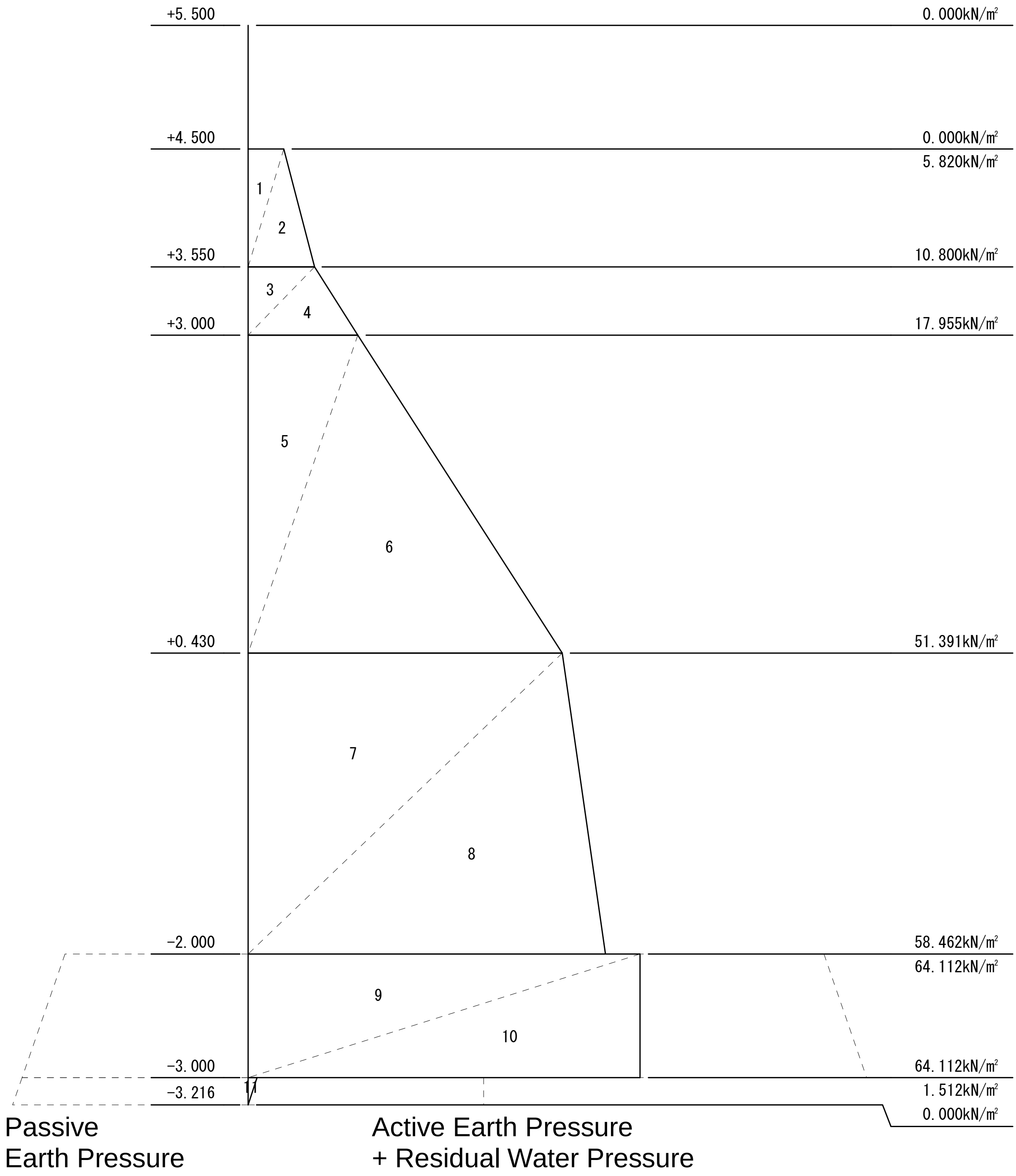
4) Reaction forces

Distance between supports : L_T = 3.000 – (-3.216) = 6.216 m

Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{985.455}{6.216} = 158.535 kN/m$

Reaction at tie setting point : $A_p = \Sigma S - R_0 = 302.657 - 158.535 = 144.122 kN/m$

External Force Diagram



5) Calculation of shear force zero point

Elev (m)	Horizontal Force S(kN/m)	∑S (kN/m)	Reaction at Tie A_p (kN/m)	Shear Force Q(kN/m)
5.50 4.50	0.000 0.000	0.000		0.000
4.50 3.55	2.764 5.130	7.894		-7.894
3.55 3.00	2.970 4.938	15.802	144.122	128.320
3.00 0.43	23.072 66.037	104.911	144.122	39.211
0.43 -2.00	62.440 71.031	238.382	144.122	-94.260
-2.00 -3.00	32.056 32.056	302.494	144.122	-158.372
-3.00 -3.22	0.163 0.000	302.657	144.122	-158.535

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between 0.430m and -2.000m.

$$Q = 39.211 - \frac{[51.391 + (51.391 + 2.910X)]X}{2} = 0$$

X = 0.747 m

Shear force zero point : DL = 0.430 – 0.747 = -0.317m

6) Calculation of moment

No	Calculation	S (kN/m)	L (m)	M (kN-m/m)
1	1/2x 5.820x 0.950	-2.764	4.500	-12.438
2	1/2x 10.800x 0.950	-5.130	4.184	-21.464
3	1/2x 10.800x 0.550	-2.970	3.684	-10.941
4	1/2x 17.955x 0.550	-4.938	3.500	-17.283
5	1/2x 17.955x 2.570	-23.072	2.460	-56.757
6	1/2x 51.391x 2.570	-66.037	1.604	-105.923
7	1/2x 51.391x 0.747	-19.195	0.498	-9.559
8	1/2x 53.565x 0.747	-20.007	0.249	-4.982
Total		———	———	-239.347

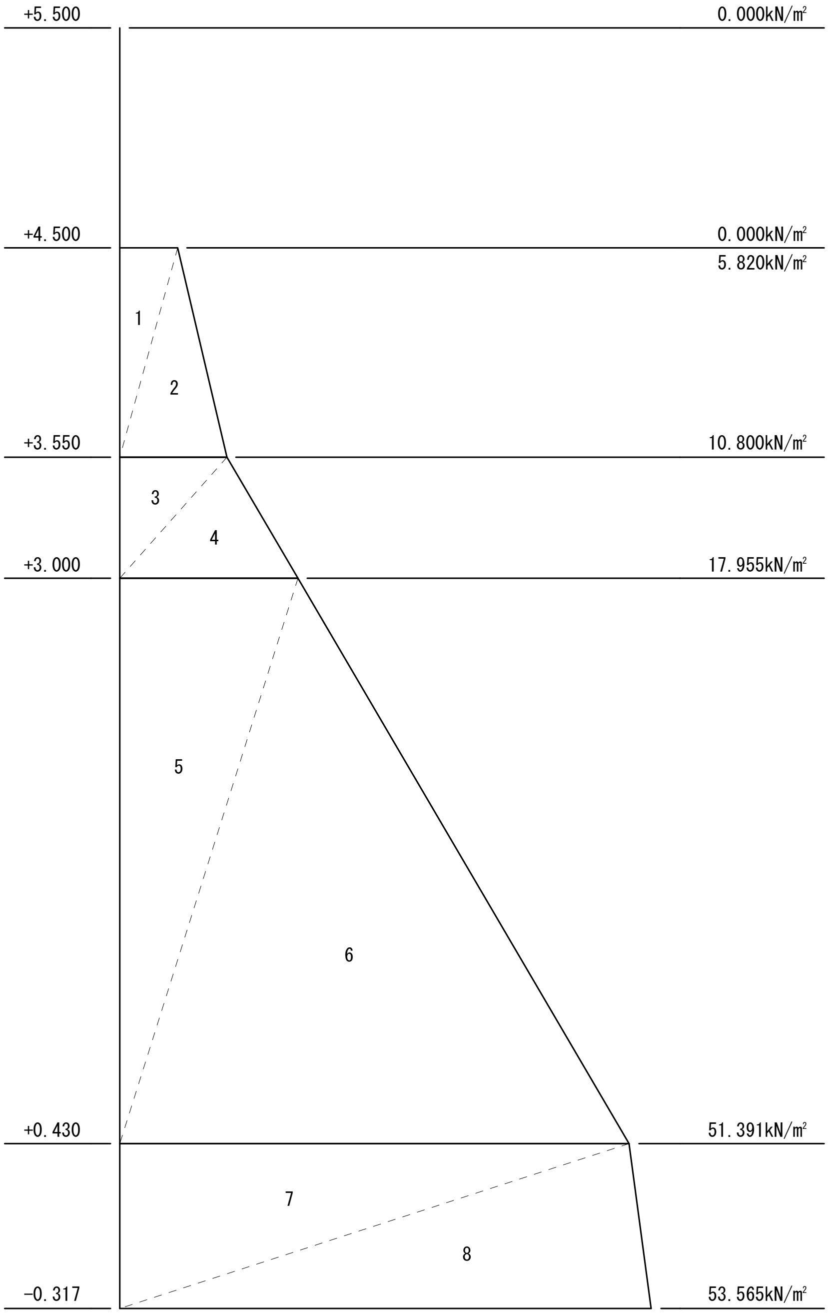
where; S is the horizontal force (kN/m)
L is the distance to the shear force zero point (m)
M is the moment about the shear force zero point (kN-m/m)

7) Maximum bending moment

- Distance of the shear zero point to the tie setting point
  h = 3.000 – (-0.317) = 3.317 m

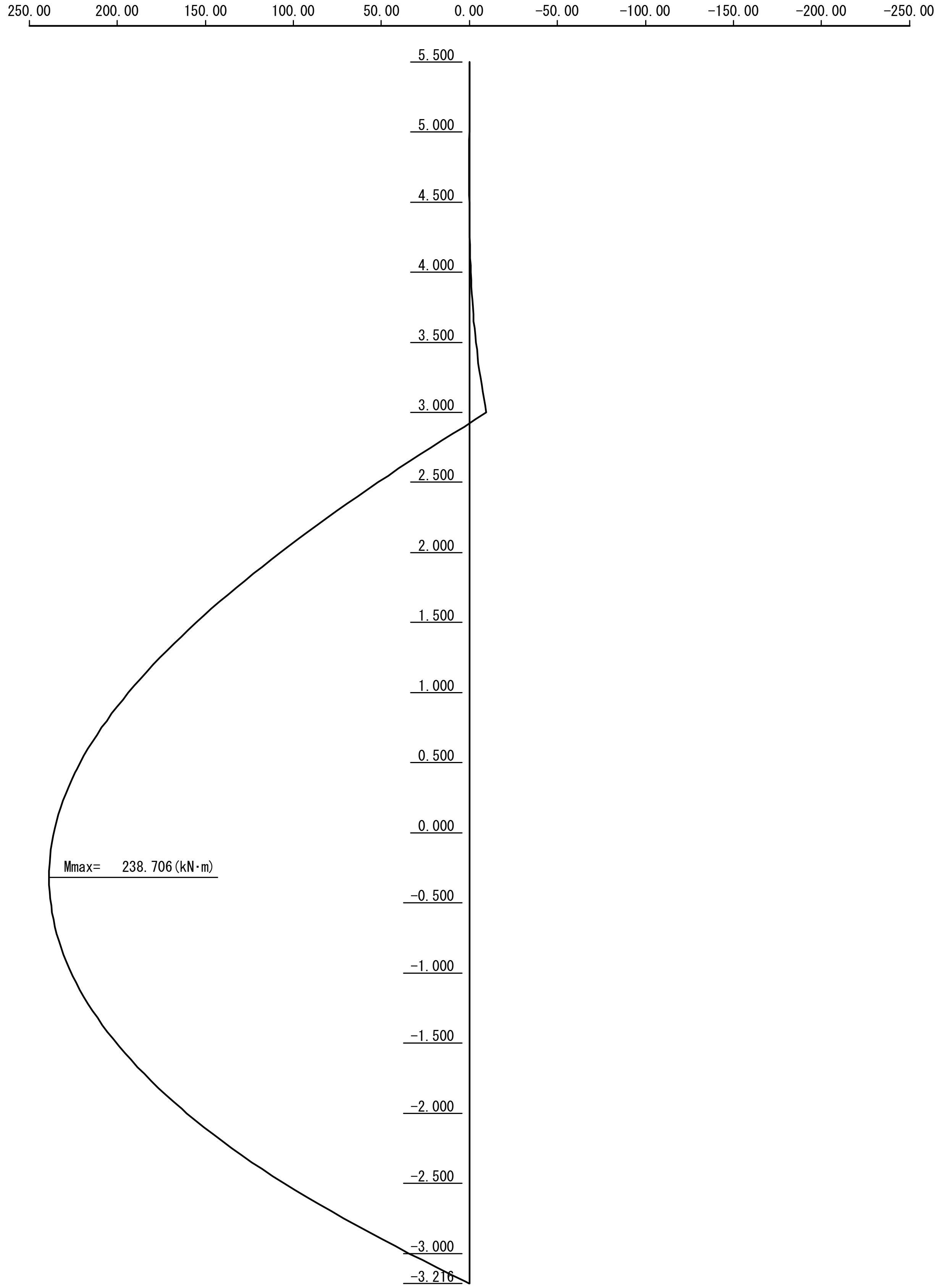
- Maximum bending moment
  $M_{max} = A_p \times h + \sum M = 144.122 \times 3.317 - 239.347 = 238.706$ kN-m/m

External Force Diagram



Active Earth Pressure
+ Residual Water Pressure

Bending moment diagram – Free Earth Support Method



2-4 Determination of Pipe Wall Section

(1) Section properties

Dimensions and joint type : ϕ800 × t10 (P-T joint)

Modulus of elasticity : E = 200.0 kN/mm^2
Moment of inertia : I = 193647 cm^4
Section modulus : Z = 4841 cm^3

Section property (per 1m) before corrosion
- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 197599 cm^4/m
- Section modulus : Z = 4940 cm^3/m

Section property (per 1m) after corrosion
- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm

- Spacing of pipes : Pipe 800mm + Joint 180mm = 980mm
- Moment of inertia : I = 166997 cm^4/m
- Section modulus : Z = 4191 cm^3/m

(2) Embedded depth of pipe wall
Toe level of the pipe wall is obtained as follows:
- Deflection curve method : -16.353 m --- adopted
- Free earth support method : -12.207 m

Therefore, the minimum length of the pipe wall is;
L = 3.500 – (-16.353) = 19.853m → 20.000 m

Accordingly the design toe level of the pipe wall is;
EL = +3.500 – 20.000 = -16.500 m

(3) Stress on pipe wall section
Maximum bending moment on the pipe wall is as follows:
- Deflection curve method : 467.739 kN-m/m --- adopted
- Free earth support method : 238.706 kN-m/m

$$\sigma = \frac{M_{\max}}{Z} = \frac{467.739 \times 10^6}{4666 \times 10^3} = 111.6 MPa \quad \leq \quad \sigma_a = 140 MPa \quad .... OK$$

3. Tie Wire

3-1 Tension Force on Tie Wire

$$T = A_p \cdot L \cdot \sec\theta_1 \cdot \sec\theta_2$$

Where;

- T is the tension force on tie wire (kN/wire)
- A_p is the reaction force of the pipe wall at tie setting point (kN/m)
- L is the spacing of the tie wire (m)
- θ_1 is the vertical angle of the tie wire to horizontal (deg.)
- θ_2 is the plan angle of the tie wire to the direction perpendicular to the revetment line (deg.)

The reaction force is obtained as follows:

- Deflection curve method : 203.449 kN-m/m --- adopted
- Free earth support method : 144.122 kN-m/m

T = 203.449 × 1.96 × sec(0.000) × sec (0.000) = 398.760 (kN/wire)

3-2 Requirement of Tie Wire

Factor of safety = 3.8

Required tensile strength of tie wire = 398.760 × 3.8 = 1515.288 kN/wire

Tie wire shall have minimum tensile strength of 1516 kN.

4. Study of Waling Beam

4-1 Maximum Bending Moment

$$M_{maz} = \frac{T \cdot L}{10.0}$$

Where;

M

M_{max} is the maximum bending moment on the waling beam (kN-m)
T is the tension force on tie wire (kN/wire)
L is the spacing of the tie wire (m)

$$M_{maz} = \frac{398.760 \times 1.960}{10.0} = 78.157 kN - m$$

4-2 Waling Beam of Seaside Wall

Provide 2×[250×90×11.0×14.5

Steel grade : SS400
Section modulus : Z = 374.0 cm^3

$$\sigma = \frac{M_{\max}}{2 \cdot Z} \le \sigma_a$$

$$\sigma = \frac{M_{\max}}{2 \cdot Z} = \frac{78.157 \times 10^6}{2 \times 374.0 \times 10^3} = 104.5 MPa \quad \le \quad \sigma_a = 140 MPa \quad .... \ OK$$

Appendix 7-9

Calculation of Bearing Capacity of Steel Sheet Pipe Pile

Bearing Capacity Check

1) Block-1

Block-1

Self Weight of Cope Concrete/SPSP and Skin Friction on SPSP

Dead weight of coping concrete

Formula
V = 1.5 x 1.5 x 1/2(1.5 + 1.0) x 2.0 = 4.75 m3/m W = 4.75 x 24 kN/m3 = 114 kN/m

↓ Wc= 114.0 kN/m

Pile weight

Dimensions		Unit	Elevation		Length	Weight
Dia	Thk	weight	Top	Bottom		per pile
mm	mm	kg/m	m CD	m CD	m	kN
800	14	271.4	3.5	-15.5	19.0	50.6

↓ Wp= 51.6 kN/m

Negative friction on land side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Sand Fill	5.50	2.00	3.50	10.00		20.00	0.98 x h =	3.43	68.60
Sand Fill	2.00	-2.00	4.00	10.00		20.00	π/2x0.8x h=	5.03	100.53
Layer-2	-2.00	-8.00	6.00		15.00	15.00	π/2x0.8x h=	7.54	113.10
Layer-3b	-8.00	-10.00	2.00		25.00	25.00	π/2x0.8x h=	2.51	62.83
Layer-4	-10.00	-15.00	5.00		50.00	50.00	π/2x0.8x h=	6.28	314.16
Layer-5	-15.00	-15.50	0.50		40.00	40.00	π/2x0.8x h=	0.63	25.13
								Total	684.35

↓ Fin= 698.32 kN/m

Skin friction on sea side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Layer-2	-5.00	-8.00	3.00		15.00	15.00	π/2x0.8x h=	3.77	56.55
Layer-3b	-8.00	-10.00	2.00		25.00	25.00	π/2x0.8x h=	2.51	62.83
Layer-4	-10.00	-15.00	5.00		50.00	50.00	π/2x0.8x h=	6.28	314.16
Layer-5	-15.00	-15.50	0.50		40.00	40.00	π/2x0.8x h=	0.63	25.13
								Total	458.67

↑ Fout= 468.03 kN/m

Total downward load

= Wc + Wp + Fin = 863.9 kN/m > Fout … Therefore, SPSP with cope concrete will settle by the consolidation of the soil between SPSP and DMM after construction.

Bearing Capacity Check

Bearing capacity

$Ru = 300 \times N \times Ap$

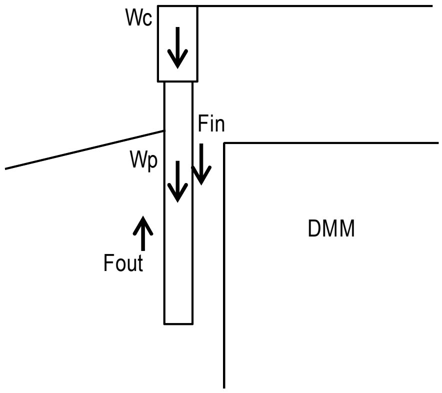
$= 300 \times 40 \times \pi/4 \times 0.8^2 =$ 6,031.9 kN/pile

$Ra = Ru / 2.5 =$ $\boxed{2{,}412.7}$ kN/pile

Design load

$P = Wc + Wp + Fin - Fout =$ 395.9 kN/m

$P_{\_4piles} = P \times 0.98m \times 4piles =$ $\boxed{1{,}551.9}$ kN/pile < Ra ... OK



Stress Check for Bearing Pile

Pile diameter	D =	800	mm
Wall thickness	t =	19	mm
Corrosion loss	Δt =	1.5	mm
Outer dia after corrosion	OD =	797	mm
Inner dia after corrosion	ID =	762	mm (no change)
Area	A =	42,855	mm2
Moment of inertia	I =	3,256,605,327	mm4
Section modulus	Z =	8,172,159	mm3
Maximum bending moment	M =	786.6	kNm
Axial compression force	P =	1,551.9	kN
Stress due to bending moment	M/Z =	96.3	MPa
Stress due to axial force	P/A =	36.2	MPa
Maximum fibre stress	M/Z + P/A =	$\boxed{132.5}$	MPa < 140 MPa ... OK

2) Block-a

Block-a

Self Weight of Cope Concrete/SPSP and Skin Friction on SPSP

Dead weight of coping concrete

Formula
V = 1.5 x 1.5 x 1/2(1.5 + 1.0) x 2.0 = 4.75 m3/m W = 4.75 x 24 kN/m3 = 114 kN/m

↓ Wc= 114.0 kN/m

Pile weight

Dimensions		Unit	Elevation		Length	Weight
Dia	Thk	weight	Top	Bottom		per pile
mm	mm	kg/m	m CD	m CD	m	kN
800	10	194.8	3.5	-13.5	17.0	32.5

↓ Wp= 33.2 kN/m

Negative friction on land side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Sand Fill	5.50	2.00	3.50	10.00		20.00	0.98 x h =	3.43	68.60
Sand Fill	2.00	-2.00	4.00	10.00		20.00	π/2x0.8x h=	5.03	100.53
Layer-2	-2.00	-7.50	5.50		15.00	15.00	π/2x0.8x h=	6.91	103.67
Layer-4	-7.50	-13.50	6.00		50.00	50.00	π/2x0.8x h=	7.54	376.99
								Total	649.79

↓ Fin= 663.06 kN/m

Skin friction on sea side surface of SPSP

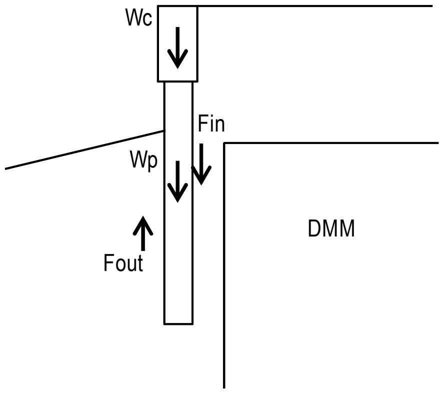
	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Layer-2	-2.00	-7.50	5.50		15.00	15.00	π/2x0.8x h=	6.91	103.67
Layer-4	-7.50	-13.50	6.00		50.00	50.00	π/2x0.8x h=	7.54	376.99
								Total	480.66

↑ Fout= 490.47 kN/m

Total downward load

= Wc + Wp + Fin = 810.2 kN/m   > Fout … Therefore, SPSP with cope concrete will settle by the consolidation of the soil between SPSP and DMM after construction.

Bearing Capacity Check



Bearing capacity

Ru = 300 x N x Ap

= 300 x 40 x π/4 x 0.8^2 = 6,031.9 kN/pile

Ra = Ru / 2.5 = $\boxed{2{,}412.7}$ kN/pile

Design load

P = Wc + Wp + Fin - Fout = 319.7 kN/m

$P_{\_4piles}$ = P x 0.98m x 4piles = $\boxed{1{,}253.4}$ kN/pile < Ra ... OK

Stress Check for Bearing Pile

Pile diameter	D =	800	mm
Wall thickness	t =	12	mm
Corrosion loss	Δt =	1.5	mm
Outer dia after corrosion	OD =	797	mm
Inner dia after corrosion	ID =	776	mm (no change)
Area	A =	25,944	mm2
Moment of inertia	I =	2,006,424,227	mm4
Section modulus	Z =	5,034,942	mm3
Maximum bending moment	M =	457.1	kNm
Axial compression force	P =	1,253.4	kN
Stress due to bending moment	M/Z =	90.8	MPa
Stress due to axial force	P/A =	48.3	MPa
Maximum fibre stress	M/Z + P/A =	$\boxed{139.1}$	MPa < 140 MPa ... OK

3) Block-b

Block-b

Self Weight of Cope Concrete/SPSP and Skin Friction on SPSP

Dead weight of coping concrete

Formula
V = 1.5 x 1.5 x 1/2(1.5 + 1.0) x 2.0 = 4.75 m3/m W = 4.75 x 24 kN/m3 = 114 kN/m

↓ Wc= 114.0 kN/m

Pile weight

Dimensions		Unit	Elevation		Length	Weight
Dia	Thk	weight	Top	Bottom		per pile
mm	mm	kg/m	m CD	m CD	m	kN
800	10	194.8	3.5	-16.5	20.0	38.2

↓ Wp= 39.0 kN/m

Negative friction on land side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Sand Fill	5.50	2.00	3.50	10.00		20.00	0.98 x h =	3.43	68.60
Sand Fill	2.00	-2.00	4.00	10.00		20.00	π/2x0.8x h=	5.03	100.53
Layer-2	-2.00	-9.00	7.00		15.00	15.00	π/2x0.8x h=	8.80	131.95
Layer-3b	-9.00	-13.00	4.00		25.00	25.00	π/2x0.8x h=	5.03	125.66
Layer-4	-13.00	-16.00	3.00		50.00	50.00	π/2x0.8x h=	3.77	188.50
Layer-5	-16.00	-16.50	0.50		40.00	40.00	π/2x0.8x h=	0.63	25.13
								Total	640.37

↓ Fin= 653.44 kN/m

Skin friction on sea side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Layer-2	-2.00	-9.00	7.00		15.00	15.00	π/2x0.8x h=	8.80	131.95
Layer-3b	-9.00	-13.00	4.00		25.00	25.00	π/2x0.8x h=	5.03	125.66
Layer-4	-13.00	-16.00	3.00		50.00	50.00	π/2x0.8x h=	3.77	188.50
Layer-5	-16.00	-16.50	0.50		40.00	40.00	π/2x0.8x h=	0.63	25.13
								Total	471.24

↑ Fout= 480.86 kN/m

Total downward load

= Wc + Wp + Fin = 806.4 kN/m > Fout … Therefore, SPSP with cope concrete will settle by the consolidation of the soil between SPSP and DMM after construction.

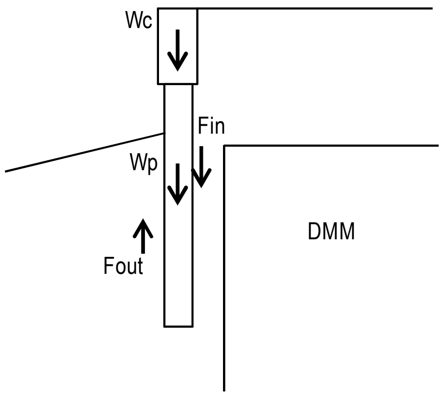
Bearing Capacity Check

Bearing capacity

Ru = 300 x N x Ap

= 300 x 40 x π/4 x 0.8^2 = 6,031.9 kN/pile

Ra = Ru / 2.5 = $\boxed{2,412.7}$ kN/pile



Design load

P = Wc + Wp + Fin - Fout = 325.6 kN/m

$P_{\_4piles}$ = P x 0.98m x 4piles = $\boxed{1,276.3}$ kN/pile < Ra ... OK

Stress Check for Bearing Pile

Pile diameter	D =	800	mm
Wall thickness	t =	14	mm
Corrosion loss	Δt =	1.5	mm
Outer dia after corrosion	OD =	797	mm
Inner dia after corrosion	ID =	772	mm (no change)
Area	A =	30,807	mm2
Moment of inertia	I =	2,370,603,868	mm4
Section modulus	Z =	5,948,818	mm3
Maximum bending moment	M =	524.7	kNm
Axial compression force	P =	1,276.3	kN
Stress due to bending moment	M/Z =	88.2	MPa
Stress due to axial force	P/A =	41.4	MPa
Maximum fibre stress	M/Z + P/A =	$\boxed{129.6}$	MPa < 140 MPa ... OK

4) Block-c

Block-c

Self Weight of Cope Concrete/SPSP and Skin Friction on SPSP

Dead weight of coping concrete

Formula
V = 1.5 x 1.5 x 1/2(1.5 + 1.0) x 2.0 = 4.75 m3/m W = 4.75 x 24 kN/m3 = 114 kN/m

↓ Wc= 114.0 kN/m

Pile weight

Dimensions		Unit	Elevation		Length	Weight
Dia	Thk	weight	Top	Bottom		per pile
mm	mm	kg/m	m CD	m CD	m	kN
800	10	194.8	3.5	-15.0	18.5	35.4

↓ Wp= 36.1 kN/m

Negative friction on land side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Sand Fill	5.50	2.00	3.50	10.00		20.00	0.98 x h =	3.43	68.60
Sand Fill	2.00	-2.00	4.00	10.00		20.00	π/2x0.8x h=	5.03	100.53
Layer-2	-2.00	-8.00	6.00		15.00	15.00	π/2x0.8x h=	7.54	113.10
Layer-3a	-8.00	-9.50	1.50	4.00		8.00	π/2x0.8x h=	1.88	15.08
Layer-4	-9.50	-15.00	5.50		50.00	50.00	π/2x0.8x h=	6.91	345.58
								Total	642.88

↓ Fin= 656.00 kN/m

Skin friction on sea side surface of SPSP

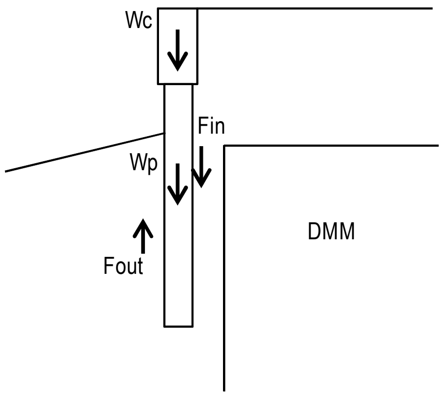
	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Layer-2	-2.00	-8.00	6.00		15.00	15.00	π/2x0.8x h=	7.54	113.10
Layer-3a	-8.00	-9.50	1.50	4.00		8.00	π/2x0.8x h=	1.88	15.08
Layer-4	-9.50	-15.00	5.50		50.00	50.00	π/2x0.8x h=	6.91	345.58
								Total	473.75

↑ Fout= 483.42 kN/m

Total downward load

= Wc + Wp + Fin = 806.1 kN/m > Fout … Therefore, SPSP with cope concrete will settle by the consolidation of the soil between SPSP and DMM after construction.

Bearing Capacity Check



Bearing capacity

$Ru = 300 \times N \times Ap$		
$= 300 \times 40 \times \pi/4 \times 0.8^2$ =	6,031.9	kN/pile
Ra = Ru / 2.5 =	$\boxed{2,412.7}$	kN/pile

Design load

P = Wc + Wp + Fin - Fout =	322.7	kN/m	
$P_{\_4piles}$ = P x 0.98m x 4piles =	$\boxed{1,264.8}$	kN/pile	< Ra ... OK

Stress Check for Bearing Pile

Pile diameter	D =	800	mm	
Wall thickness	t =	14	mm	
Corrosion loss	Δt =	1.5	mm	
Outer dia after corrosion	OD =	797	mm	
Inner dia after corrosion	ID =	772	mm (no change)	
Area	A =	30,807	mm2	
Moment of inertia	I =	2,370,603,868	mm4	
Section modulus	Z =	5,948,818	mm3	
Maximum bending moment	M =	509.3	kNm	
Axial compression force	P =	1,264.8	kN	
Stress due to bending moment	M/Z =	85.6	MPa	
Stress due to axial force	P/A =	41.1	MPa	
Maximum fibre stress	M/Z + P/A =	$\boxed{126.67}$	MPa	< 140 MPa ... OK

5) Block-d

Block-d

Self Weight of Cope Concrete/SPSP and Skin Friction on SPSP

Dead weight of coping concrete

Formula
V = 1.5 x 1.5 x 1/2(1.5 + 1.0) x 2.0 = 4.75 m3/m W = 4.75 x 24 kN/m3 = 114 kN/m

↓ Wc= 114.0 kN/m

Pile weight

Dimensions		Unit	Elevation		Length	Weight
Dia	Thk	weight	Top	Bottom		per pile
mm	mm	kg/m	m CD	m CD	m	kN
800	10	194.8	3.5	-17.5	21.0	40.1

↓ Wp= 41.0 kN/m

Negative friction on land side surface of SPSP

	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Sand Fill	5.50	2.00	3.50	10.00		20.00	0.98 x h =	3.43	68.60
Sand Fill	2.00	-2.00	4.00	10.00		20.00	π/2x0.8x h=	5.03	100.53
Layer-2	-2.00	-11.00	9.00		15.00	15.00	π/2x0.8x h=	11.31	169.65
Layer-3b	-11.00	-13.50	2.50		25.00	25.00	π/2x0.8x h=	3.14	78.54
Layer-4	-13.50	-16.00	2.50		50.00	50.00	π/2x0.8x h=	3.14	157.08
Layer-5	-16.00	-17.50	1.50		40.00	40.00	π/2x0.8x h=	1.88	75.40
								Total	649.79

↓ Fin= 663.06 kN/m

Skin friction on sea side surface of SPSP

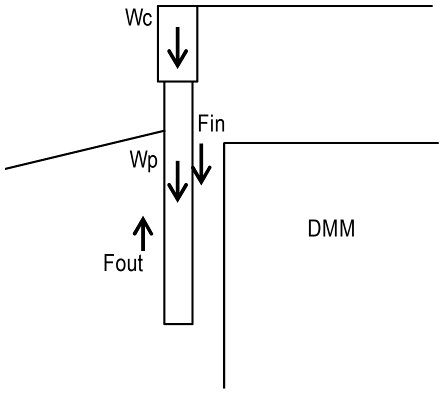
	Elev		Layer	Soil Condition		f	As		F
	Top	Bottom	Thk	SPT-N	Cu	2N or Cu			per pile
	mCD	mCD	m		kPa	kPa	m2		kN
Layer-2	-2.00	-11.00	9.00		15.00	15.00	π/2x0.8x h=	11.31	169.65
Layer-3b	-11.00	-13.50	2.50		25.00	25.00	π/2x0.8x h=	3.14	78.54
Layer-4	-13.50	-16.00	2.50		50.00	50.00	π/2x0.8x h=	3.14	157.08
Layer-5	-16.00	-17.50	1.50		40.00	40.00	π/2x0.8x h=	1.88	75.40
								Total	480.66

↑ Fout= 490.47 kN/m

Total downward load

= Wc + Wp + Fin = 818.0 kN/m > Fout … Therefore, SPSP with cope concrete will settle by the consolidation of the soil between SPSP and DMM after construction.

Bearing Capacity Check



Bearing capacity

$R_u = 300 \times N \times A_p$

$= 300 \times 40 \times \pi/4 \times 0.8^2 =$ 6,031.9 kN/pile

$R_a = R_u / 2.5 =$ $\boxed{2{,}412.7}$ kN/pile

Design load

$P = W_c + W_p + F_{in} - F_{out} =$ 327.5 kN/m

$P_{\_4piles} = P \times 0.98m \times 4piles =$ $\boxed{1{,}283.9}$ kN/pile $< R_a$ ... OK

Stress Check for Bearing Pile

Pile diameter	D =	800	mm
Wall thickness	t =	14	mm
Corrosion loss	Δt =	1.5	mm
Outer dia after corrosion	OD =	797	mm
Inner dia after corrosion	ID =	772	mm (no change)
Area	A =	30,807	mm2
Moment of inertia	I =	2,370,603,868	mm4
Section modulus	Z =	5,948,818	mm3
Maximum bending moment	M =	577.49	kNm
Axial compression force	P =	1,283.9	kN
Stress due to bending moment	M/Z =	97.1	MPa
Stress due to axial force	P/A =	41.7	MPa
Maximum fibre stress	M/Z + P/A =	$\boxed{138.75}$	MPa < 140 MPa ... OK

Appendix 10-1

Design Calculation for Fender

DESIGN CALCULATION FOR FENDER

1.0 DESIGN CONDITION

1.1 Specification of Vessel

Berthing vessel to be 4,000ps Tug Boat.,

	Tug Boat
Displacement Tonnage : W (Ton)	544
Length Overall : Loa (m)	36.2
Length Perpendicular : Lpp (m)	31.5
Breadth : B (m)	9.8
Depth : D (m)	4.4
Draft : d (m)	3.2
Bow Radius : R (m)	10.13
Berthing Velocity : V (m/s)	0.30
Max. Berthing Angle : (Degree)	10^0

2.0 BERTHING ENERGY CALCULATION

2.1 Berthing Energy

Effective berthing energy is calculated as follows :

$$E = \frac{W \times V^2}{2g} \text{ x Ce x Cm x Cs X Cc}$$

Where ;

E : Effective Berthing Energy (ton-m)
W : Displacement Tonnage (ton)
V : Berthing velocity (m/s)
Ce : Eccentricity Coefficient
Cm : Hydrodynamic Mass Coefficient
Cs : Softness Coefficient (Generally value taken as 1.0)
Cc : Berthing Configuration Coefficient (Generally taken as 1.0)
g : Acceleration of Gravity (m/s^2)

2.2 Hydrodynamic Mass Coefficient (Cm)

The hydrodynamic mass coefficient allows the movement of water around the vessel to be taken into account when calculating the total energy of the vessel by increasing the mass of the system. The hydrodynamic mass coefficient Cm may be calculated from the following equation.

$$Cm = 1 + \frac{2d}{B}$$

Where ;

d : Draft of Vessel (m)

B : Breadth of Vessel (m)

2.3 Eccentricity Coefficient (Ce)

A vessel normally berthed at a certain angle. Therefore, the vessel turns simultaneously at the time of first contact with the fender. Some of the kinetic energy held by the vessel is consumed as its turning energy and the remained energy is thus transferred to the berth.

The eccentricity factor (Ce) presents the proportion of the remainder energy to the kinetic energy of the vessel at berthing.

$$Ce = \frac{K^2 + R^2 Cos^2 \gamma}{K^2 + R}$$

Where ;

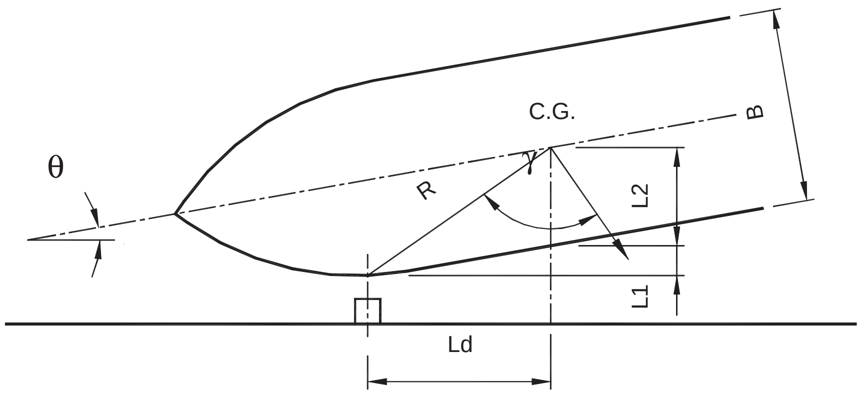
K : The radius gyration of the ship. (0.19Cb+0.11)Lpp

Lpp : Length of hull between perpendicular

Cb : Block Coefficient

R : Distance(m) of the point of contact from the center of mass.

γ : The angle between the line joining the point of contact to the center of mass and the velocity vector. ($\gamma = 90^0$)



For continuous wharf, point of contact is assumed to be 0.25Lpp

$$L1 = Ld \times \tan\theta$$

$$L2 = \frac{0.5B}{\cos\theta}$$

$$R = \sqrt{Ld \;\; + (L1 + L2)^2}$$

$$K = (0.19Cb + 0.11)\ Lpp$$

Where ;

$$Cb = \frac{W}{L \times B \times d \times \rho}$$

($\rho = 1.025$ Ton/m^3)

	Tug Boat
W (Ton)	**544**
R (m)	10.13
K (m)	6.66
Ce	0.500
Cm	1.959
Cc	1.00
Cs	1.00
V (m/s)	0.30
Berthing Energy (kN-m)	**24.0**

3.0 SELECTION OF FENDER

Model : SX-300H x 3500L (H2+5)

Deflection	: 52.5%
Rubber Grade	: H2 + 5
Reaction Force, R.F.	: 213 KN/meter
Energy Absorption, E.A.	: 26.9 KN-m/meter
Tolerance	: 10%

Appendix 10-2

Design Calculation for Relieving Platform (Standard Section)

Design Calculation for Quay Wall with Relieving Platform

1. Design Conditions

1-1 Design Code Applied

This design calculation was based on mainly following design standard.

No	Title	Publisher
1	Technical Standards and Commentaries for Port and Harbour Facilities in Japan - 1999	The Overseas Coastal Area Development Institute of Japan
2	Specifications for Highway Bridges - March 2002	Japan Road Association
3	Standard Specification for Concrete Structures - 2007	Japan Society of Civil Engineers

1-2 Dimension

Ground elevation	+5.50 m
Top of the sheet pile	+1.50 m
Seabed elevation	-5.00 m

1-3 Environmental Data

High Water Level (H.W.L)	+3.55 m
Low Water Level (L.W.L)	+0.43 m
Residual Water Level (R.W.L)	+2.51 m

1-4 Material

(1) Concrete

Characteristic 28 days compressive strength of concrete is as follows.

Characteristic 28 days compressive strength		Modulus of elasticity
Cube strength	Cylinder strength	
40 MPa	30 MPa	28 kN/mm^2

(2) Reinforcement Bar

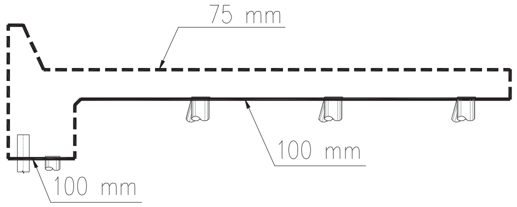
a) Grade of Reinforcement

Grade	Modulus of elasticity
SD295	200 kN/mm^2

b) Cover to Reinforcement

C = 75mm

However, the concrete cover for the bottom surface will be 100mm to allow for a gap between the rebar and the embedded pile head.



c) Design Crack Width

0.004 × C (mm)   C:the cover to reinforcement

(3) Steel Sheet Pile

Type	Grade	Allowable Stress
IVw	SY295	180 MPa

- Design life time : 50 years
- Effect of cathodic protection : 90%
- Corrosion rate : 0.030 mm/year (0.3mm/year × 10%)
- Corrosion allowance : t = 0.030 × 50 years = 1.50 mm (all surfaces)

(4) Prestressed Concrete Spun Pile

Type	Effective Prestress	Compressive Strength	Modulus of Elasticity
Type-B	8.0 MPa	85 MPa	37 kN/mm^2

1-5 Design Load

Live Load 10.0kPa

Mooring Load (150kN bollard @12m) 12.5kN/m

1-6 Load Case

	Design Load					Increase of Allowable Stress	Factor of Safety				
	DL	LL	EP	ML	SL		Circular Slip	Sliding	Over--turning	Bearing Capacity	
										Comp.	Pulling.
Ordinary	X	10kPa	X			1.0	1.3	1.2	1.2	2.5	3.0
Mooring	X	10kPa	X	X		1.0	-*	1.2	1.2	2.5	3.0

DL : Dead Load

LL : Live Load

EP : Earth Pressure & Water Pressure

ML : Mooring Load

*note : Mooring load will be taken by the deck and piles and will not contribute the driving moment.

1-7 Structural Software

The analysis was made by SAP2000, developed by Computers & Structures Inc.

1-8 Method of Earth Pressure Calculation

The earth pressure will be calculated based on the "Technical Standards and Commentaries for Port and Harbour Facilities in Japan (TSCPHFJ)".

[For sandy soil – ordinary conditions]

14.2 Earth Pressure under Ordinary Conditions

14.2.1 Earth Pressure of Sandy Soil under Ordinary Conditions (Notification Article 11, Clause 1, Number 1)

The earth pressure of sandy soil acting on the backface wall of structure and the angle of sliding surface shall be calculated by the following equations:

(1) Active Earth Pressure and the Angle of Failure Surface.

$$p_{ai} = K_{ai}\left[\Sigma\gamma_i h_i + \frac{w\cos\psi}{\cos(\psi-\beta)}\right]\cos\psi \qquad (14.2.1)$$

$$\cot(\zeta_i-\beta) = -\tan(\phi_i+\delta+\psi-\beta)+\sec(\phi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\psi+\delta)\sin(\phi_i+\delta)}{\cos(\psi-\beta)\sin(\phi_i-\beta)}} \qquad (14.2.2)$$

where

$$K_{ai} = \frac{\cos^2(\phi_i-\psi)}{\cos^2\psi\cos(\delta+\psi)\left[1+\sqrt{\frac{\sin(\phi_i+\delta)\sin(\phi_i-\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}\right]^2}$$

with

p_{ai}, p_{pi} : active and passive earth pressure, respectively, acting on the bottom level of the i-th soil layer (kN/m^2)
ϕ_i : angle of internal friction of the i-th soil layer (°)
γ_i : unit weight of the i-th soil layer (kN/m^3)
h_i : thickness of the i-th soil layer (m)
K_{ai}, K_{pi} : coefficients of active and passive earth pressures, respectively, in the i-th soil layer
ψ : angle of batter of backface wall from vertical line (°)
β : angle of backfill ground surface from horizontal line (°)
δ : angle of friction between backfilling material and backface wall (°)
ζ_i : angle of failure surface of the i-th soil layer (°)
w : uniformly distributed surcharge (kN/m^2)

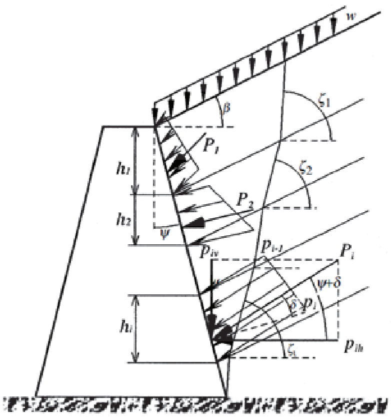


Fig. 14.2.1 Schematic Diagram of Earth Pressure Acting on Retaining Wall

[For cohesive soil – ordinary conditions]

14.2.2 Earth Pressure of Cohesive Soil under Ordinary Conditions (Notification Article 11, Clause 1, Number 2)

The earth pressure of cohesive soil acting on the backface wall of structure and the angle of failure surface shall be calculated by following equations:

(1) Active Earth Pressure

$$p_a = \Sigma\gamma_i h_i + w - 2c \qquad (14.2.5)$$

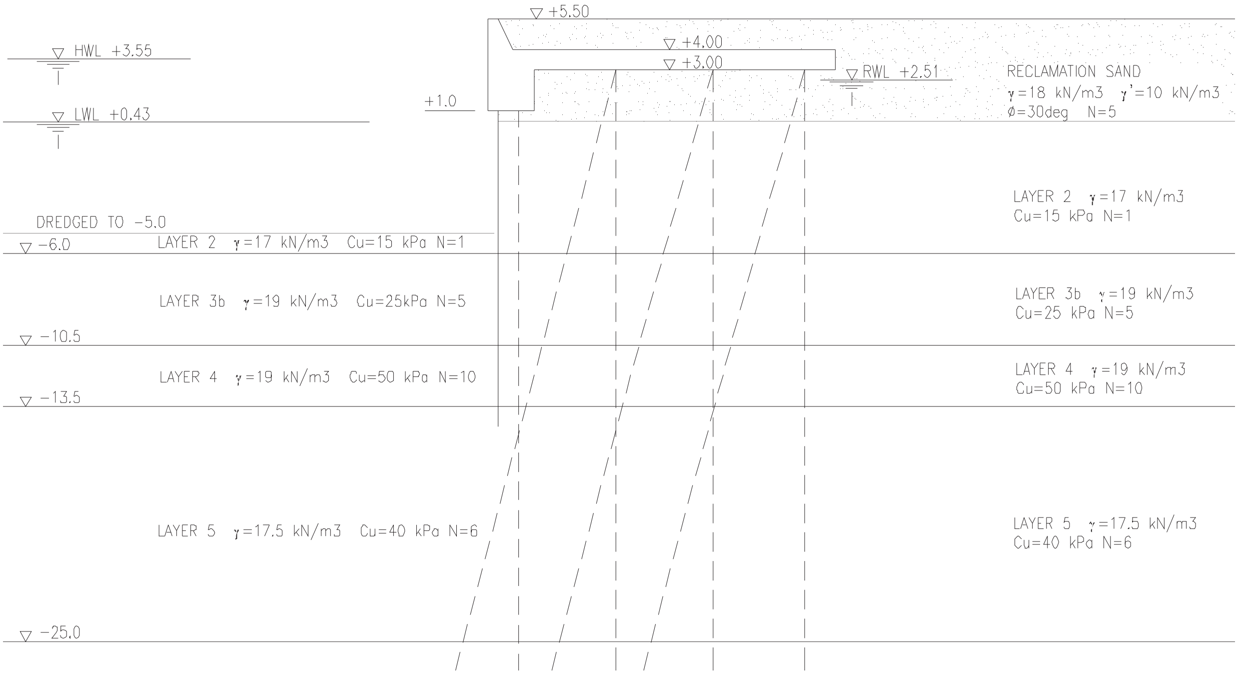
(2) Passive Earth Pressure

$$p_p = \Sigma\gamma_i h_i + w + 2c \qquad (14.2.6)$$

where

p_a : active earth pressure acting on the bottom level of the i-th soil layer (kN/m^2)
p_p : passive earth pressure acts on the bottom level of the i-th soil layer (kN/m^2)
γ_i : unit weight of the i-th soil layer (kN/m^3)
h_i : thickness of the i-th soil layer (m)
w : uniformly distributed surcharge (kN/m^2)
c : cohesion of soil in the i-th layer (kN/m^2)

1-9 Profile of Revetment and Soil Conditions



2. Design of the Relieving Platform

2-1 Width of the Relieving Platform

(1) Summary

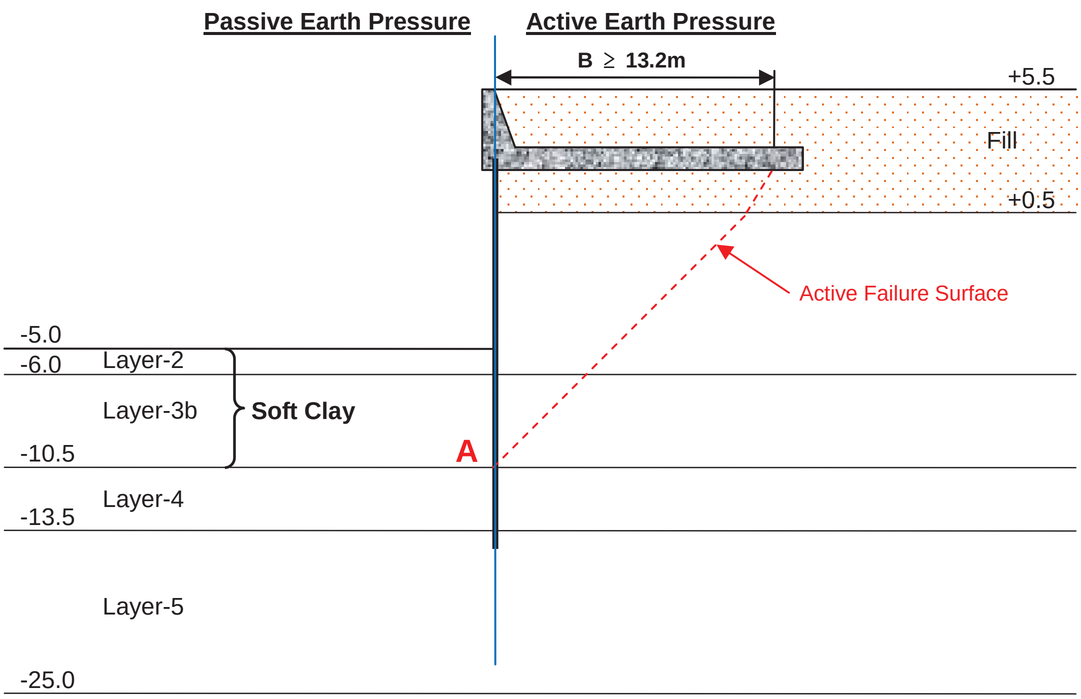
The width of the relieving platform (B) is determined so that all of the following conditions are satisfied.

- The check of the intersection with the active failure surface
- The stability check against circular slip
- The stability check as the gravity-type wall structure

These checks were done in section (2)~(4) and the width (B) is determined as 16.5m.

(2) The check of the intersection with the active failure surface

According to the soil investigation data, soil of Layer-2 and Layer-3b are soft (Cu<30kPa). The width of the relieving platform was determined so that the platform will intersects the active failure surface drawn from the point A in the figure below.

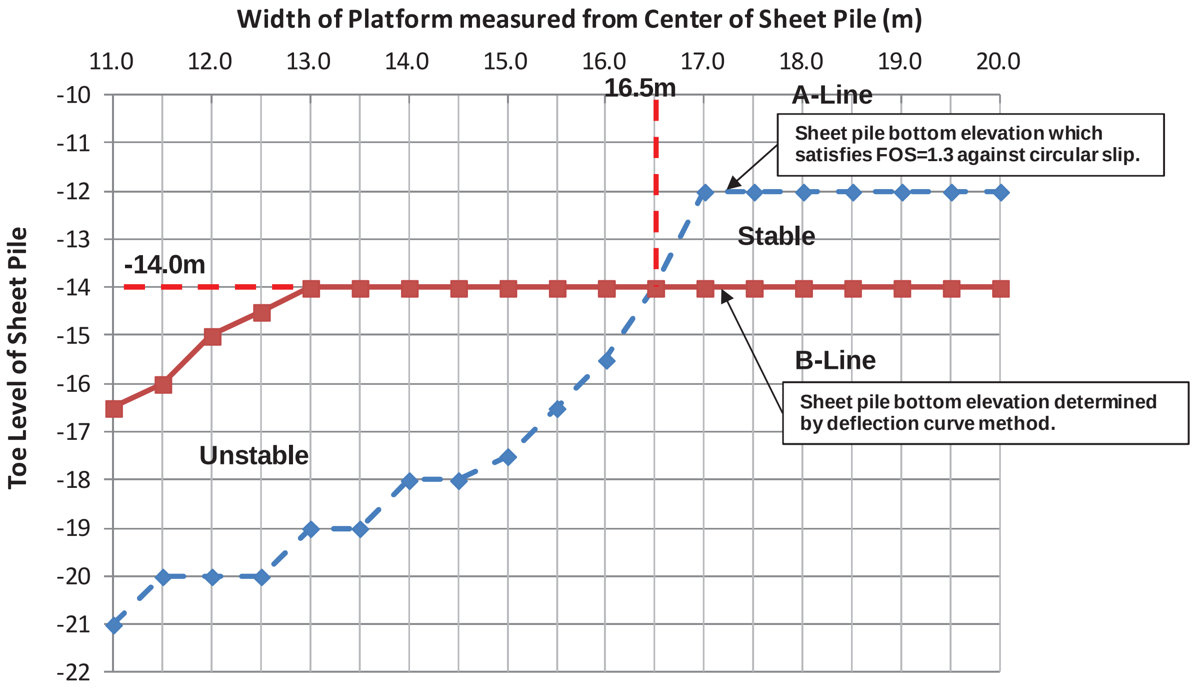


A minimum width of the relieving platform is calculated as 13.2m from the center line of the sheet pile.

(3) The stability check against circular slip

The required factor of safety is FOS ≥ 1.3.

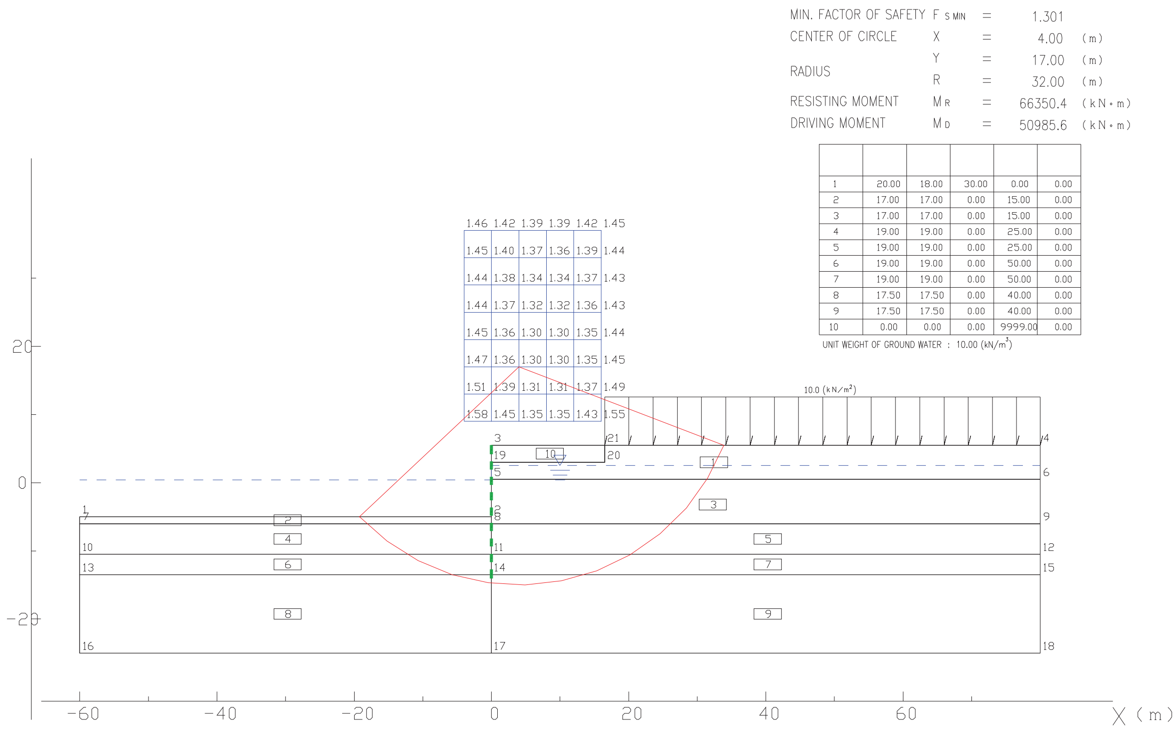
Figure below presents the preliminary check results of the circular slip analysis and sheet pile penetration calculation.
The relieving platform has effects to reduce the earth pressure on the sheet pile as well as the driving moment in the circular slip analysis.
Generally, wider the platform, shallower the sheet pile toe level.



The A-Line is the required toe level of the sheet pile which satisfies factor of safety against circular slip failure FOS=1.3, and the B-Line is that determined by pile penetration calculation (deflection curve method).
Design code recommends that the embedded length of the sheet pile will not be extended from the required embedded length determined by the equilibrium of the earth pressure on the sheet pile for a purpose to prevent the circular slip. Therefore, the minimum width of the platform is <u>16.5m</u> which is the intersection of A-Line and B-Line.

The results of the stability check against circular slip (The width of platform is 16.5m and the toe level of sheet pile is -14.0m) are shown as follows.

FOS = 1.301 ≥ 1.3



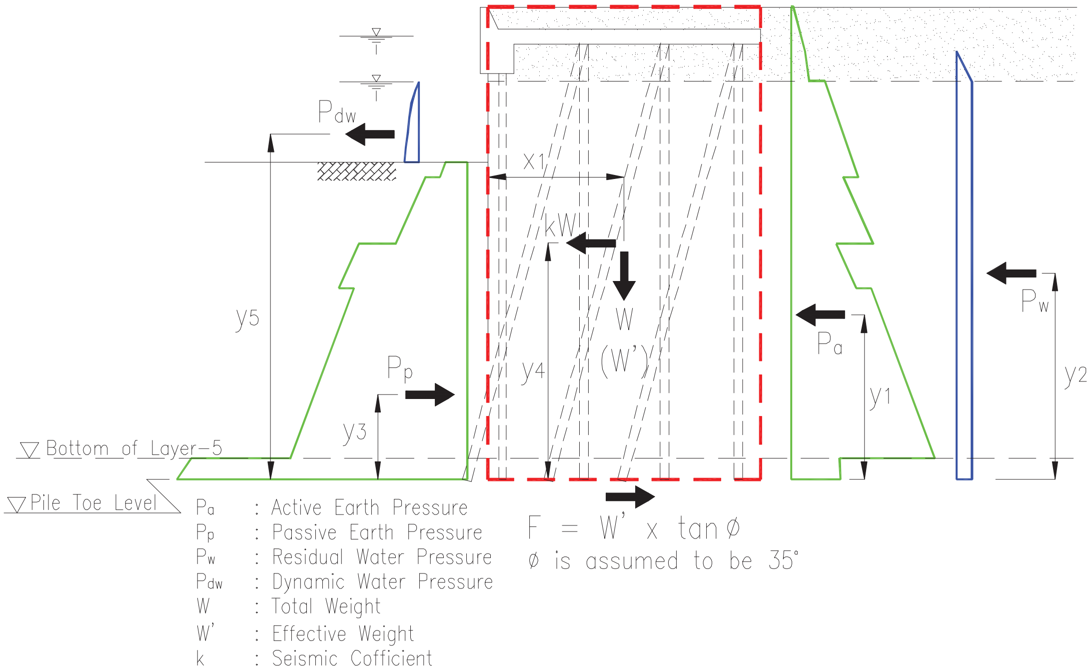
(4) The stability check as the gravity-type wall structure

The stability against sliding and overturning as the gravity-type wall structure was checked the platform width of B=16.5m (measured from center of sheet pile) as obtained in section (3).

Required factors of safety are as follows;

	Sliding	Overturning
Ordinary/Mooring Case	FOS ≥ 1.2	FOS ≥ 1.2

a) Case 1 : Stability at pile toe level



Factor of safety against sliding

Ordinary/Mooring Case : $FOS = \dfrac{F + P_p}{P_a + P_w}$

Seismic Case : $FOS = \dfrac{F + P_p}{P_a + P_w + kW + P_{dw}}$

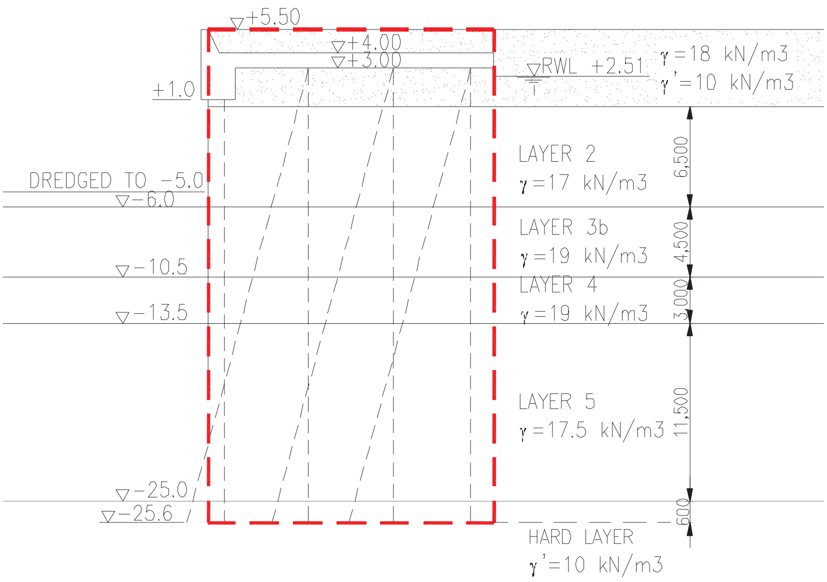
Factor of safety against overturning

Ordinary/Mooring Case : $FOS = \dfrac{P_p \times y_3 + W' \times x_1}{P_a \times y_1 + P_w \times y_2}$

Seismic Case : $FOS = \dfrac{P_p \times y_3 + W' \times x_1}{P_a \times y_1 + P_w \times y_2 + kW \times y_4 + P_{dw} \times y_5}$

(Case 1 : Ordinary Case)

1) Self Weight



Width = 16.5 m

	Thick (m)	Area (m2/m)	γt	γ	W (kN/m)	W' (kN/m)
Reclamation Sand		24.23	18.0	18.0	436	436
Deck Concrete above RWL		19.38	24.0	24.0	465	465
Deck Concrete below RWL		3.4	24.0	14.0	82	48
Reclamation Sand above RWL		7.23	20.0	10.0	145	72
Reclamation Sand below RWL		30.52	20.0	10.0	610	305
Layer 2	6.5	107.25	17.0	7.0	1,823	751
Layer 3b	4.5	74.25	19.0	9.0	1,411	668
Layer 4	3	49.5	19.0	9.0	941	446
Layer 5	11.5	189.75	17.5	7.5	3,321	1,423
Hard Layer	0.6	9.9	20.0	10.0	198	99
					9,431	4,713

Total Weight W = 9,431 kN/m
Effective Weight W' = 4,713 kN/m

2) Active Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pa (kPa)	Pa (kN/m)	y (m)	Pay (kNm/m)
5.5	2.99	18		10	2.9	4.4	30.10	131
2.51	2.99	18		63.82	18.6	27.8	29.11	808
2.51	2.01	10		63.82	18.6	18.7	27.44	512
0.5	2.01	10		83.92	24.4	24.6	26.77	657
0.5	6.5	7	15	83.92	53.9	175.2	23.93	4,194
-6	6.5	7	15	129.42	99.4	323.1	21.77	7,033
-6	4.5	9	25	129.42	79.4	178.7	18.10	3,234
-10.5	4.5	9	25	169.92	119.9	269.8	16.60	4,479
-10.5	3	9	50	169.92	69.9	104.9	14.10	1,479
-13.5	3	9	50	196.92	96.9	145.4	13.10	1,904
-13.5	11.5	7.5	40	196.92	116.9	672.3	8.27	5,558
-25	11.5	7.5	40	283.17	203.2	1,168.2	4.43	5,179
-25	0.6	10		283.17	67.8	20.3	0.40	8
-25.6	0.6	10		289.17	69.2	20.8	0.20	4
						3,154.1		35,182

3) Residual Water Pressure

Elev. (mCD)	h (m)	γ			pw (kPa)	Pw (kN/m)	y (m)	Pay (kNm/m)
2.51	2.08	10.1						
0.43	2.08	10.1			21.008	21.8	26.72	584
0.43	26.03	10.1			21.008	273.4	17.35	4,745
-25.6	26.03	10.1			21.008	273.4	8.68	2,372
						568.7		7,701

4) Passive Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pp (kPa)	Pp (kN/m)	y (m)	Ppy (kNm/m)
-5	1	7	15	0	30.0	15.0	20.27	304
-6	1	7	15	7	37.0	18.5	19.93	369
-6	4.5	9	25	7	57.0	128.3	18.10	2,321
-10.5	4.5	9	25	47.5	97.5	219.4	16.60	3,642
-10.5	3	9	50	47.5	147.5	221.3	14.10	3,120
-13.5	3	9	50	74.5	174.5	261.8	13.10	3,429
-13.5	11.5	7.5	40	74.5	154.5	888.4	8.27	7,344
-25	11.5	7.5	40	160.75	240.8	1,384.3	4.43	6,137
-25	0.6	10		160.75	1,017.8	305.3	0.40	122
-25.6	0.6	10		166.75	1,055.8	316.7	0.20	63
						3,758.9		26,851

5) Bottom Resistance

$$F = W' \times \tan\phi = 4713\ \text{kN/m} \times \tan(35\text{deg.}) = 3300\ \text{kN/m}$$

6) Stability Check

		Force		Moment	
		Horizontal Driving	Horizontal Resisting	Driving	Resisting
Active Earth Pressure	Pa	3,154		35,182	
Residual Water Pressure	Pw	569		7,701	
Passive Earth Pressure	Pp		3,759		26,851
Bottom Resistance	F		3,300		
Moment Resistance due to Dead Load	DL				38,882
Total		3,723	7,059	42,883	65,733

Sliding			Overturning		
	Driving	3,723		Driving	42,883
	Resisting	7,059		Resisting	65,733
	FOS	1.896		FOS	1.533

(Case 1 : Mooring Case)

1) Self Weight

Same as ordinary case
Total Weight W = 9,431 kN/m
Effective Weight W' = 4,713 kN/m

2) Active Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pa (kPa)	Pa (kN/m)	y (m)	Pay (kNm/m)
5.5	2.99	18		10	2.9	4.4	30.10	131
2.51	2.99	18		63.82	18.6	27.8	29.11	808
2.51	2.01	10		63.82	18.6	18.7	27.44	512
0.5	2.01	10		83.92	24.4	24.6	26.77	657
0.5	6.5	7	15	83.92	53.9	175.2	23.93	4,194
-6	6.5	7	15	129.42	99.4	323.1	21.77	7,033
-6	4.5	9	25	129.42	79.4	178.7	18.10	3,234
-10.5	4.5	9	25	169.92	119.9	269.8	16.60	4,479
-10.5	3	9	50	169.92	69.9	104.9	14.10	1,479
-13.5	3	9	50	196.92	96.9	145.4	13.10	1,904
-13.5	11.5	7.5	40	196.92	116.9	672.3	8.27	5,558
-25	11.5	7.5	40	283.17	203.2	1,168.2	4.43	5,179
-25	0.6	10		283.17	67.8	20.3	0.40	8
-25.6	0.6	10		289.17	69.2	20.8	0.20	4
						3,154.1		35,182

3) Residual Water Pressure

Elev. (mCD)	h (m)	γ			pw (kPa)	Pw (kN/m)	y (m)	Pay (kNm/m)
2.51	2.08	10.1						
0.43	2.08	10.1			21.008	21.8	26.72	584
0.43	26.03	10.1			21.008	273.4	17.35	4,745
-25.6	26.03	10.1			21.008	273.4	8.68	2,372
						568.7		7,701

4) Passive Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pp (kPa)	Pp (kN/m)	y (m)	Ppy (kNm/m)
-5	1	7	15	0	30.0	15.0	20.27	304
-6	1	7	15	7	37.0	18.5	19.93	369
-6	4.5	9	25	7	57.0	128.3	18.10	2,321
-10.5	4.5	9	25	47.5	97.5	219.4	16.60	3,642
-10.5	3	9	50	47.5	147.5	221.3	14.10	3,120
-13.5	3	9	50	74.5	174.5	261.8	13.10	3,429
-13.5	11.5	7.5	40	74.5	154.5	888.4	8.27	7,344
-25	11.5	7.5	40	160.75	240.8	1,384.3	4.43	6,137
-25	0.6	10		160.75	1,017.8	305.3	0.40	122
-25.6	0.6	10		166.75	1,055.8	316.7	0.20	63
						3,758.9		26,851

5) Bottom Resistance

$F = W' \times \tan\phi = 4713\ kN/m \times \tan(35deg.) = 3300\ kN/m$

6) Mooring Load

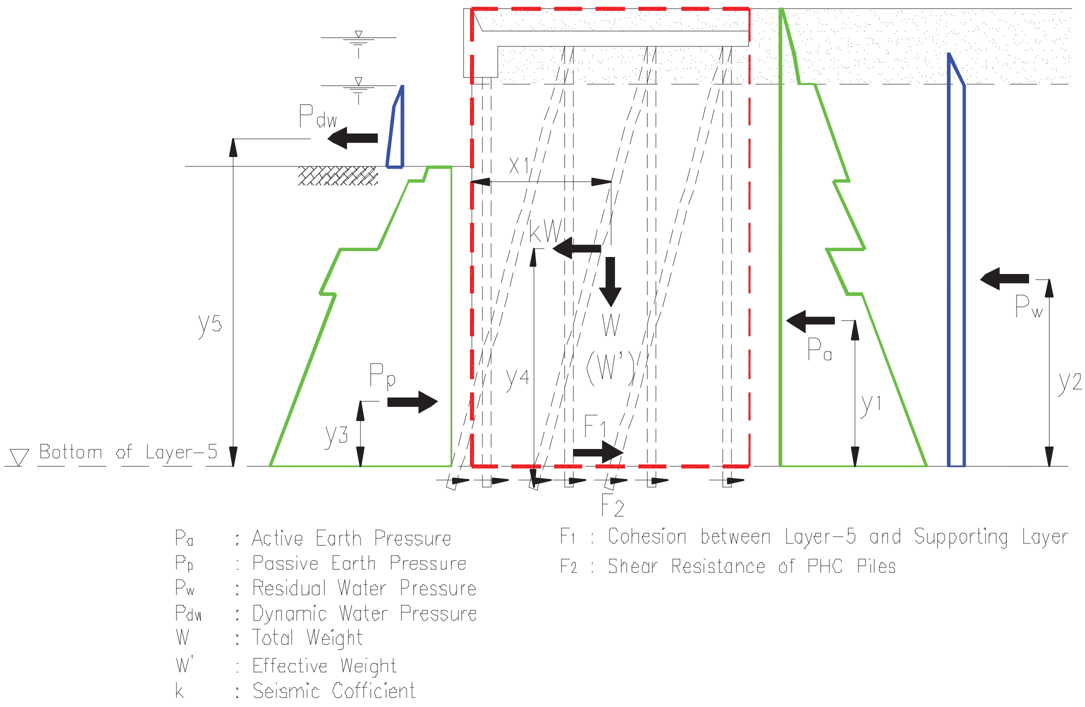
$P_{ML} = 12.5\ kN/m$

7) Stability Check

		Force		Moment	
		Horizontal Driving	Horizontal Resisting	Driving	Resisting
Active Earth Pressure	Pa	3,154		35,182	
Residual Water Pressure	Pw	569		7,701	
Passive Earth Pressure	Pp		3,759		26,851
Bottom Resistance	F		3,300		
Moment Resistance due to Dead Load	DL				38,882
Mooring Load	P_{ML}	13		389	
Total		3,735	7,059	43,272	65,733

Sliding			Overturning		
	Driving	3,735		Driving	43,272
	Resisting	7,059		Resisting	65,733
	FOS	1.890		FOS	1.519

b) Case 2 : Stability at bottom of Layer-5



Factor of safety against sliding

Ordinary/Mooring Case : $FOS = \dfrac{F_1 + F_2 + P_p}{P_a + P_w}$

Seismic Case : $FOS = \dfrac{F_1 + F_2 + P_p}{P_a + P_w + kW + P_{dw}}$

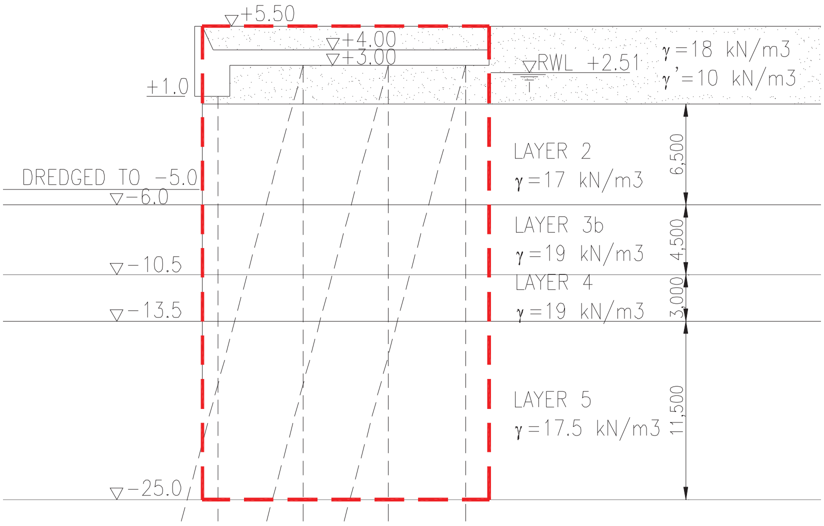
Factor of safety against overturning

Ordinary/Mooring Case : $FOS = \dfrac{P_p \times y_3 + W' \times x_1}{P_a \times y_1 + P_w \times y_2}$

Seismic Case : $FOS = \dfrac{P_p \times y_3 + W' \times x_1}{P_a \times y_1 + P_w \times y_2 + kW \times y_4 + P_{dw} \times y_5}$

(Case 2 : Ordinary Case)

1) Self Weight



Width = 16.5 m

	Thick (m)	Area (m2/m)	γt	γ	W (kN/m)	W' (kN/m)
Reclamation Sand		24.23	18.0	18.0	436	436
Deck Concrete above RWL		19.38	24.0	24.0	465	465
Deck Concrete below RWL		3.4	24.0	14.0	82	48
Reclamation Sand above RWL		7.23	20.0	10.0	145	72
Reclamation Sand below RWL		30.52	20.0	10.0	610	305
Layer 2	6.5	107.25	17.0	7.0	1,823	751
Layer 3b	4.5	74.25	19.0	9.0	1,411	668
Layer 4	3	49.5	19.0	9.0	941	446
Layer 5	11.5	189.75	17.5	7.5	3,321	1,423
					9,233	4,614

Total Weight W = 9,233 kN/m
Effective Weight W' = 4,614 kN/m

2) Active Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pa (kPa)	Pa (kN/m)	y (m)	Pay (kNm/m)
5.5	2.99	18		10	2.9	4.4	29.50	128
2.51	2.99	18		63.82	18.6	27.8	28.51	792
2.51	2.01	10		63.82	18.6	18.7	26.84	501
0.5	2.01	10		83.92	24.4	24.6	26.17	643
0.5	6.5	7	15	83.92	53.9	175.2	23.33	4,089
-6	6.5	7	15	129.42	99.4	323.1	21.17	6,839
-6	4.5	9	25	129.42	79.4	178.7	17.50	3,127
-10.5	4.5	9	25	169.92	119.9	269.8	16.00	4,317
-10.5	3	9	50	169.92	69.9	104.9	13.50	1,416
-13.5	3	9	50	196.92	96.9	145.4	12.50	1,817
-13.5	11.5	7.5	40	196.92	116.9	672.3	7.67	5,154
-25	11.5	7.5	40	283.17	203.2	1,168.2	3.83	4,478
						3,113.0		33,302

3) Residual Water Pressure

Elev. (mCD)	h (m)	γ			pw (kPa)	Pw (kN/m)	y (m)	Pay (kNm/m)
2.51	2.08	10.1						
0.43	2.08	10.1			21.008	21.8	26.12	571
0.43	25.43	10.1			21.008	267.1	16.95	4,529
-25	25.43	10.1			21.008	267.1	8.48	2,264
						556.1		7,364

4) Passive Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pp (kPa)	Pp (kN/m)	y (m)	Ppy (kNm/m)
-5	1	7	15	0	30.0	15.0	19.67	295
-6	1	7	15	7	37.0	18.5	19.33	358
-6	4.5	9	25	7	57.0	128.3	17.50	2,244
-10.5	4.5	9	25	47.5	97.5	219.4	16.00	3,510
-10.5	3	9	50	47.5	147.5	221.3	13.50	2,987
-13.5	3	9	50	74.5	174.5	261.8	12.50	3,272
-13.5	11.5	7.5	40	74.5	154.5	888.4	7.67	6,811
-25	11.5	7.5	40	160.75	240.8	1,384.3	3.83	5,307
						3,136.8		24,783

5) Bottom Resistance

F_1 : Cohesion between Layer-5 and supporting Layer

$F_1 = C \times L = 40\ \text{kPa} \times 16.5\text{m} = 660\ \text{kN/m}$

where, C is the cohesion of Layer-5
L is the width of platform

F_2 : Shear resistance of PHC piles

F_2 is considered as the total shear resistance of PHC pile

	shear capacity	nos	Resisntance	
			kN/4m	kN/m
ϕ600	392	6	2,352	588
ϕ500	288	1	288	72
			2,640	660

* note : The shear capacities of PHC pile were taken from Catalogue.

6) Stability Check

		Force		Moment	
		Horizontal Driving	Horizontal Resisting	Driving	Resisting
Active Earth Pressure	Pa	3,113		33,302	
Residual Water Pressure	Pw	556		7,364	
Passive Earth Pressure	Pp		3,137		24,783
Cohesion between Layer-5 and supporting Layer	F1		660		
Shear Resistance of PHC pile	F2		660		
Moment Resistance due to Dead Load	DL				38,065
Total		3,669	4,457	40,665	62,849

Sliding			Overturning		
	Driving	3,669		Driving	40,665
	Resisting	4,457		Resisting	62,849
	FOS	1.215		FOS	1.546

(Case 2 : Mooring Case)

1) Self Weight

Same as ordinary case.
Total Weight W = 9,233 kN/m
Effective Weight W' = 4,614 kN/m

2) Active Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pa (kPa)	Pa (kN/m)	y (m)	Pay (kNm/m)
5.5	2.99	18		10	2.9	4.4	29.50	128
2.51	2.99	18		63.82	18.6	27.8	28.51	792
2.51	2.01	10		63.82	18.6	18.7	26.84	501
0.5	2.01	10		83.92	24.4	24.6	26.17	643
0.5	6.5	7	15	83.92	53.9	175.2	23.33	4,089
-6	6.5	7	15	129.42	99.4	323.1	21.17	6,839
-6	4.5	9	25	129.42	79.4	178.7	17.50	3,127
-10.5	4.5	9	25	169.92	119.9	269.8	16.00	4,317
-10.5	3	9	50	169.92	69.9	104.9	13.50	1,416
-13.5	3	9	50	196.92	96.9	145.4	12.50	1,817
-13.5	11.5	7.5	40	196.92	116.9	672.3	7.67	5,154
-25	11.5	7.5	40	283.17	203.2	1,168.2	3.83	4,478
						3,113.0		33,302

3) Residual Water Pressure

Elev. (mCD)	h (m)	γ			pw (kPa)	Pw (kN/m)	y (m)	Pay (kNm/m)
2.51	2.08	10.1						
0.43	2.08	10.1			21.008	21.8	26.12	571
0.43	25.43	10.1			21.008	267.1	16.95	4,529
-25	25.43	10.1			21.008	267.1	8.48	2,264
						556.1		7,364

4) Passive Earth Pressure

Elev. (mCD)	h (m)	γ	Cu	Σγh	pp (kPa)	Pp (kN/m)	y (m)	Ppy (kNm/m)
-5	1	7	15	0	30.0	15.0	19.67	295
-6	1	7	15	7	37.0	18.5	19.33	358
-6	4.5	9	25	7	57.0	128.3	17.50	2,244
-10.5	4.5	9	25	47.5	97.5	219.4	16.00	3,510
-10.5	3	9	50	47.5	147.5	221.3	13.50	2,987
-13.5	3	9	50	74.5	174.5	261.8	12.50	3,272
-13.5	11.5	7.5	40	74.5	154.5	888.4	7.67	6,811
-25	11.5	7.5	40	160.75	240.8	1,384.3	3.83	5,307
						3,136.8		24,783

5) Bottom Resistance

F_1 : Cohesion between Layer-5 and supporting Layer

$F_1 = C \times L = 40\ \text{kPa} \times 16.5\text{m} = 660\ \text{kN/m}$

where, C is the cohesion of Layer-5
L is the width of platform

F_2 : Shear resistance of PHC piles

F_2 is considered as the total shear resistance of PHC pile

	shear capacity	nos	Resisntance	
			kN/4m	kN/m
φ600	392	6	2,352	588
φ500	288	1	288	72
			2,640	660

* note : The shear capacities of PHC pile were taken from Catalogue.

6) Mooring Load

P_{ML} = 12.5 kN/m

7) Stability Check

		Force		Moment	
		Horizontal Driving	Horizontal Resisting	Driving	Resisting
Active Earth Pressure	Pa	3,113		33,302	
Residual Water Pressure	Pw	556		7,364	
Passive Earth Pressure	Pp		3,137		24,783
Cohesion between Layer-5 and supporting Layer	F1		660		
Shear Resistance of PHC pile	F2		660		
Moment Resistance due to Dead Load	DL				38,065
Mooring Load	P_{ML}	13		381	
Total		3,682	4,457	41,047	62,849

Sliding			Overturning		
	Driving	3,682		Driving	41,047
	Resisting	4,457		Resisting	62,849
	FOS	1.211		FOS	1.531

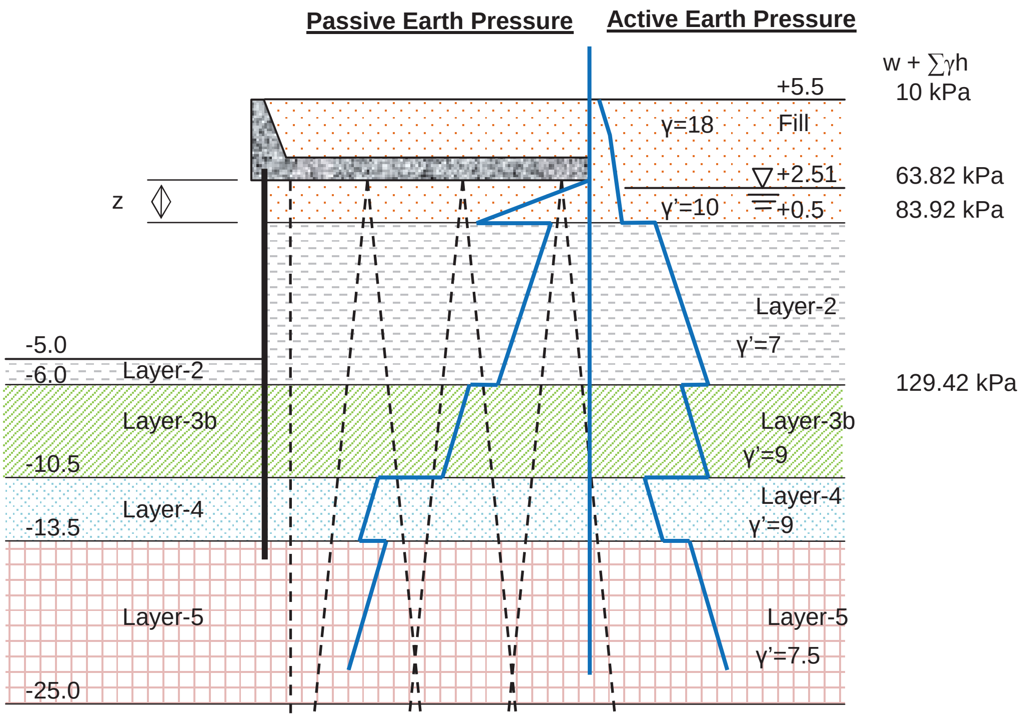
The calculation results are summarized as below.

		Sliding		Overturning	
		FOS	Required	FOS	Required
Ordinary Case	Case-1	1.896	>1.2	1.533	>1.2
	Case-2	1.215		1.546	
Mooring Case	Case-1	1.890	>1.2	1.519	>1.2
	Case-2	1.211		1.531	

The above results show that the all results are more than the required FOS.
Therefore, the width of platform of 16.5m is adequate.

2-2 The Elevation of Relieving Platform

The bottom elevation of the relieving platform was determined so that the active earth pressure of the Layer-2 at the rear end of the platform does not exceed the passive earth pressure.



At the top level of Layer-2 (+0.5m)

Active earth pressure : $p_A = w + \sum\gamma h - 2C = 83.92$ kPa – 2×15kPa
$= 53.92$ kPa

Passive earth pressure : $p_P = \gamma z + 2C = \gamma z + 30$ kPa

Accordingly, $p_P - p_A = \gamma z + 30\text{kPa} - 53.92\ \text{kPa} > 0$ … $\gamma z > 23.92\text{kPa}$
$\gamma = 10\ \text{kN/m}^3$, $z > 2.392\text{m}$
… the bottom elevation of the platform shall be higher than +2.892m.

From the above, the bottom elevation of the relieving platform of +3.0m was adopted.

2-3 Structural Analysis and Check for the Sheet Pile

(1) General

The embedded length and bending moment of the sheet pile was checked by “Deflection Curve Method” and “Free Earth Support Method”. The factor of safety for the embedded length of sheet pile was adopted as 1.2 for both methods.

(2) Earth Pressure and Water Pressure

Earth pressure and water pressure acting on the sheet pile was calculated as follows.

Active earth pressure

Elev. (m)	h (m)	ϕ (deg.)	c (kN/m2)	γ (kN/m3)	γh (kN/m2)	w +Σγh (kN/m2)	Ka cosδ	Pa (kN/m2)
3.00 2.51	0.49	30		18.00	8.820	0.000 8.820	0.2911 0.2911	0.000 2.568
2.51 1.00	1.51	30		10.00	15.100	8.820 23.920	0.2911 0.2911	2.568 6.963
1.00 0.50	0.50	30		10.00	5.000	23.920 28.920	0.2911 0.2911	6.963 8.419
0.50 0.35	0.15		15.00 15.00	7.00	1.050	28.920 29.970		0.000 0.000
0.35 -5.00	5.35		15.00 15.00	7.00	37.450	29.970 67.420		0.000 37.420
-5.00 -6.00	1.00		15.00 15.00	7.00	7.000	67.420 74.420		37.420 44.420
-6.00 -10.50	4.50		25.00 25.00	9.00	40.500	74.420 114.920		24.420 64.920
-10.50 -13.50	3.00		50.00 50.00	9.00	27.000	114.920 141.920		14.920 41.920
-13.50 -14.37	0.87		40.00 40.00	7.50	6.525	141.920 148.445		61.920 68.445
-14.37 -25.00	10.63		40.00 40.00	7.50	79.725	203.445 283.170		123.445 203.170

*note : The active earth pressure below -14.37m was calculated taking into account the soil weight and live load above +3.0m.

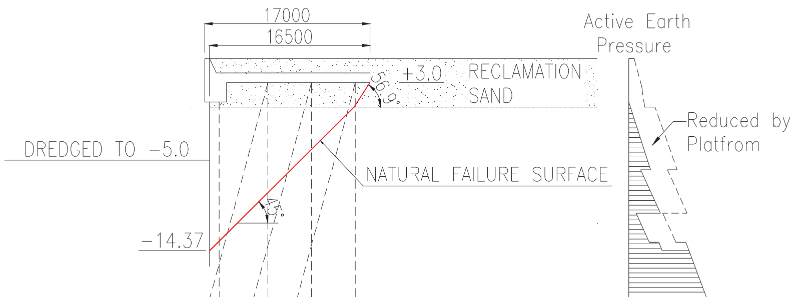
The additional overburden pressure due to this is calculated as follows.

The additional overburden puressure

- 10.00kPa (Live Load)
- +5.50m to +3.00m : 2.5m × 18kN/m^3 = 45.00kN/m^2
- w+Σγh = 10.00kN/m^2 + 45.00kN/m^2 = 55.00kN/m^2

At -16.37m

w+Σγh = 148.445kN/m^2 + 55.00kN/m^2 = 203.445kN/m^2



Residual water pressure

$p_w = \gamma_w \times h_w$

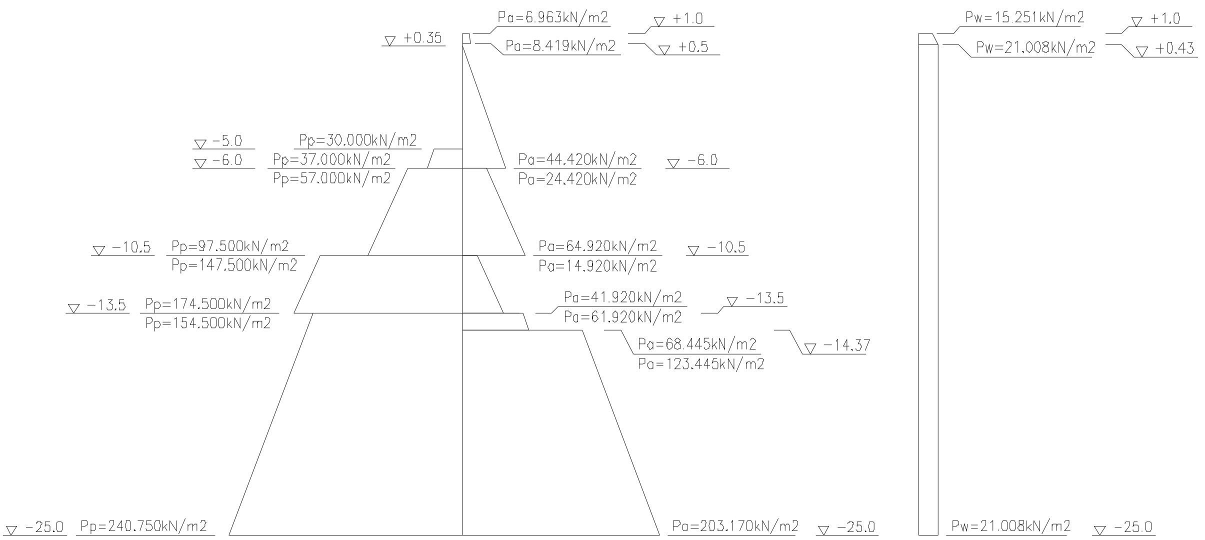
where; p_w is the residual water pressure (kPa)
h_w is the height of the tidal lag (m)
γ_w is the unit weight of water (kN/m^3)

$h_w = +2.51 - (+0.43) = 2.08$ m
$p_w = 10.100 \times 2.080 = 21.008$ kPa

Passive earth pressure

Elev. (m)	h (m)	ϕ (deg.)	c (kN/m2)	γ (kN/m3)	γh (kN/m2)	Σγh (kN/m2)	Kp cosδ	Pp (kN/m2)
-5.00 -6.00	1.00	30	15.00 15.00	7.00	7.000	0.000 7.000		30.000 37.000
-6.00 -10.50	4.50	30	25.00 25.00	9.00	40.500	7.000 47.500		57.000 97.500
-10.50 -13.50	3.00		50.00 50.00	9.00	27.000	47.500 74.500		147.500 174.500
-13.50 -14.37	0.87		40.00 40.00	7.50	6.525	74.500 81.025		154.500 161.025
-14.37 -25.00	10.63		40.00 40.00	7.50	79.725	81.025 160.750		161.025 240.750

Earth pressure and water pressure



Passive Earth Pressure　　Active Earth Pressure　　Residual Water Pressure

(3) Deflection Curve Method

Calculation of embedded length

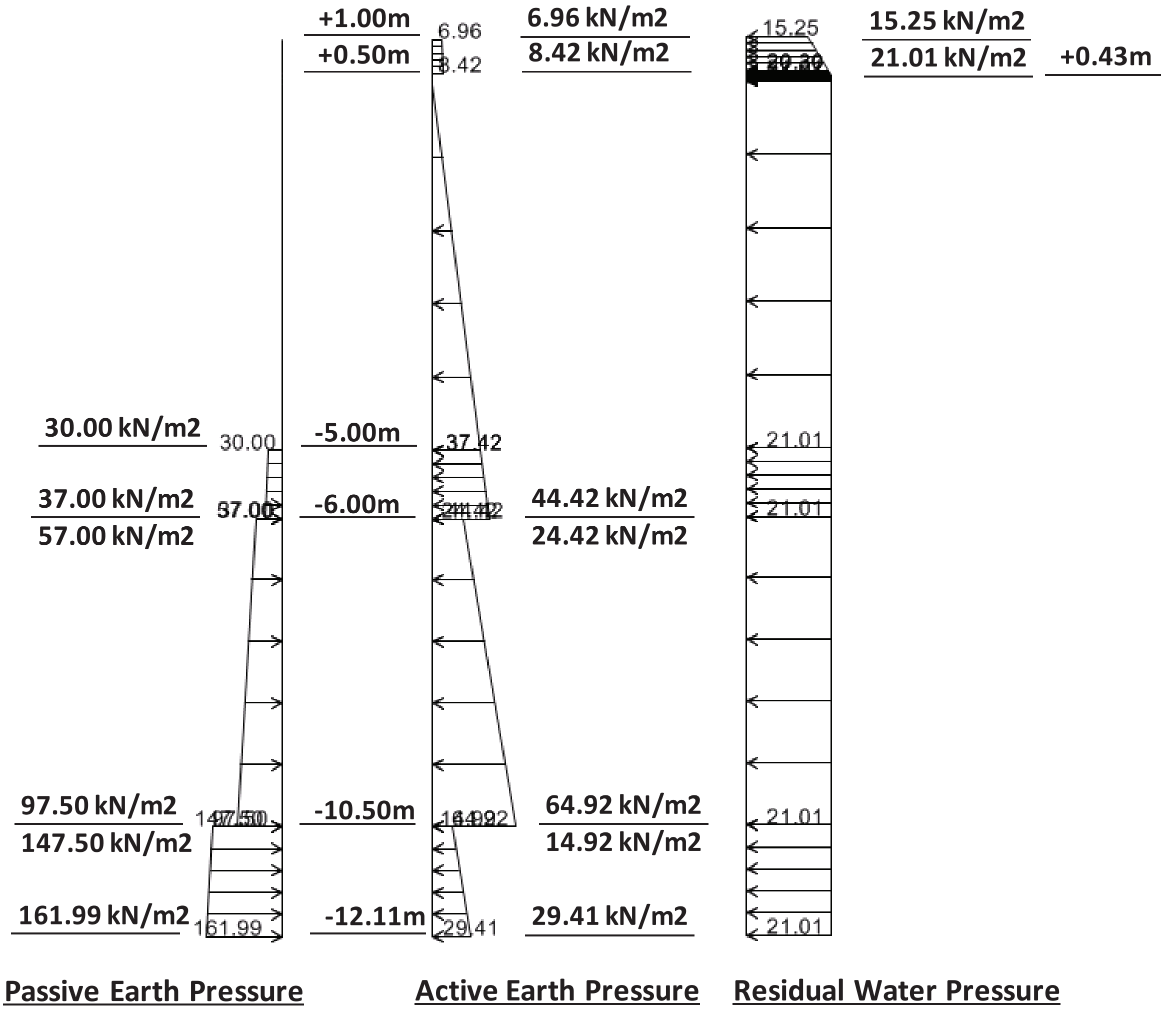
The elastic equations are solved under the external force conditions obtained above with the conditions that the displacement and deflection angle is zero at the top and the toe of the sheet pile.

For this purpose, the sheet pile is assumed to be fix supported at the top of the sheet pile and pin supported at the toe of the sheet pile and the deflection angle at the toe of the sheet pile is calculated by changing the embedded length of the sheet pile. Once the embedded length of the sheet pile with which the deflection angle becomes zero is obtained, it is considered as the minimum embedded length. The embedded length of the sheet pile shall be 1.2 times (i.e. factor of safety) the minimum embedded length.

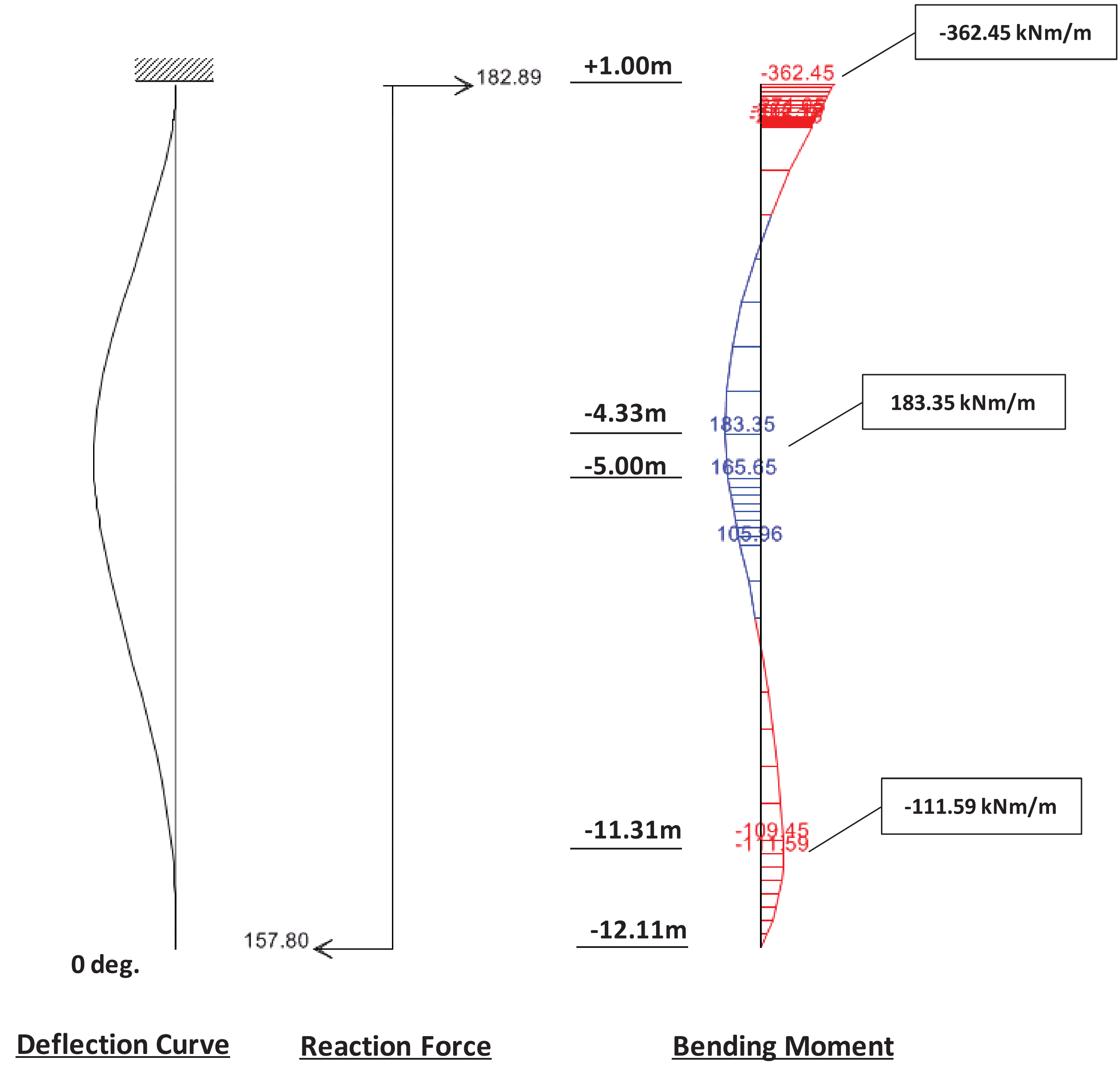
The analysis was done using the SAP2000. The result of analysis is shown below.

Toe of sheet pile : -12.11 m

Input - Design Load



Output – Deflection Curve Method



Toe of sheet pile (m)	Deflection angle at toe (deg)	Reaction at top level (kN/m)	Reaction at toe level (kN/m)
-12.11	0.00	182.89	-157.80

Seabed elevation : -5.000 m
Embedded length of the sheet pile : L = 1.2 × (-5.00 + 12.11) = 8.532 m
Toe elevation : D = -5.00 – 8.532 = -13.532 m
Reaction force at bottom of platform : 182.89 kN/m

Maximum bending moment : -362.45 kN-m/m
Elevation of maximum bending moment : +1.000 m

(4) Free Earth Support Method

Calculation of embedded length

The embedded length of the sheet pile is obtained so that the following formula is satisfied.

$M_p = FOS \times M_a$

where;

M_p is the moment due to the passive earth pressure about the top of the sheet pile (kN-m/m)

Ma is the moment due to the active earth pressure and the residual water pressure about the top of the sheet pile (kN-m/m)

FOS =1.2

Calculation of $FOS \times M_a$

Elev. (m)	Pa (kN/m2)	Pw (kN/m2)	Pa+Pw (kN/m2)	S (kN/m)	L (m)	M (kNm/m)	FOS×Ma (kNm/m)
1.00	6.963	15.251	22.214	5.554	0.167	0.926	3.983
0.50	8.419	20.301	28.720	7.180	0.333	2.393	
0.50	0.000	20.301	20.301	0.711	0.523	0.372	4.911
0.43	0.000	21.008	21.008	0.735	0.547	0.402	
0.43	0.000	21.008	21.008	0.840	0.597	0.501	6.141
0.35	0.000	21.008	21.008	0.840	0.623	0.524	
0.35	0.000	21.008	21.008	56.196	2.433	136.745	961.087
-5.00	37.420	21.008	58.428	156.295	4.217	659.043	
-5.00	37.420	21.008	58.428	29.214	6.333	185.022	1,444.826
-6.00	44.420	21.008	65.428	32.714	6.667	218.093	
-6.00	24.420	21.008	45.428	102.213	8.500	868.811	4,807.454
-10.50	64.920	21.008	85.928	193.338	10.000	1,933.380	
-10.50	14.920	21.008	35.928	53.892	12.500	673.650	7,144.985
-13.50	41.920	21.008	62.928	94.392	13.500	1,274.292	
-13.50	61.920	21.008	82.928	36.074	14.790	533.530	8,489.373
-14.37	68.445	21.008	89.453	38.912	15.080	586.794	
-14.37	123.445	21.008	144.453	767.768	18.913	14,521.046	58,023.334
-25.00	203.170	21.008	224.178	1,191.506	22.457	26,757.255	

where; S is the horizontal force (kN/m)

L is the distance of the horizontal force acting point to the top of the sheet pile (m)

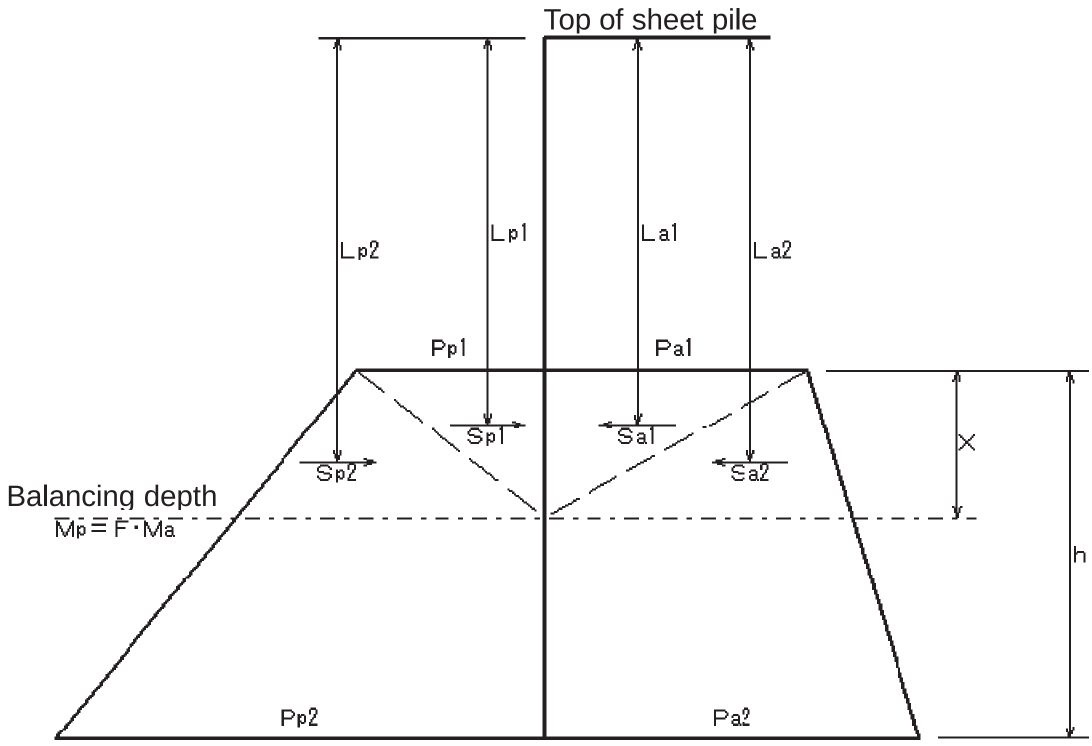
M_a is the moment about the top of the sheet pile (kN-m/m)

Calculation of M_p

Elev. (m)	Pp (kN/m2)	S (kN/m)	L (m)	M (kNm/m)	Mp (kNm/m)
-5.00 -6.00	30.000 37.000	15.000 18.500	6.333 6.667	95.000 123.333	218.333
-6.00 -10.50	57.000 97.500	128.250 219.375	8.500 10.000	1,090.125 2,193.750	3,502.208
-10.50 -13.50	147.500 174.500	221.250 261.750	12.500 13.500	2,765.625 3,533.625	9,801.458
-13.50 -14.37	154.500 161.025	67.207 70.046	14.790 15.080	993.999 1,056.292	11,851.749
-14.37 -25.00	161.025 240.750	855.848 1279.586	18.913 22.457	16,186.936 28,735.242	56,773.927

where; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting point to the top of the sheet pile (m)
M_p is the moment about the top of the sheet pile (kN-m/m)

Calculation of embedded length



Horizontal force and moment (upper triangle)

$$S_{a(p)1} = \frac{P_{a(p)1}X}{2}, \quad L_{a(p)1} = L + \frac{1}{3}X$$

$$M_{a(p)1'} = S_{a(p)1} \times L_{a(p)1}$$

Horizontal force and moment (lower triangle)

$$S_{a(p)2} = \frac{\left[P_{a(p)1} + \frac{P_{a(p)2} - P_{a(p)1}}{h} X\right] X}{2} \ , \quad L_{a(p)2} = L + \frac{2}{3} X$$

$$M_{a(p)2'} = S_{a(p)2} \times L_{a(p)2}$$

Moment due to forces above the balancing depth

$$M_{a(p)} = M_{a(p)1'} + M_{a(p)2'} + M_{a1}$$

Where;

$M_{a(p)}$	: the moment due to active (passive) side pressure above the balancing depth (kN-m/m)
$M_{a(p)1}$	: the moment due to active (passive) side pressure above the top elevation of the soil layer (kN-m/m)
$M_{a(p)1'}$	: the moment due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$M_{a(p)2'}$	: the moment due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN-m/m)
$S_{a(p)1}$	: the horizontal force due to upper triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$S_{a(p)2}$	: the horizontal force due to lower triangle of the active (passive) side pressure above the balancing depth in the soil layer (kN/m)
$L_{a(p)1}$	: the arm length of $S_{a(p)1}$ from the top of the sheet pile (m)
$L_{a(p)2}$	: the arm length of $S_{a(p)2}$ from the top of the sheet pile (m)
$P_{a(p)1}$	: the active (passive) side pressure at the top elevation of the soil layer (kPa)
$P_{a(p)2}$	: the active (passive) side pressure at the bottom elevation of the soil layer (kPa)
h	: the thickness of the soil layer (m)
L	: the vertical distance from the top of the sheet pile to the top elevation of the soil layer (m)
X	: the vertical distance from the top elevation of the soil layer to the balancing depth (m)

The balancing depth is obtained by comparing the moments due to the active side pressure and the passive side pressure.

at -10.50 m	FOS×M_{a1} = 4807.454	> M_{p1} = 3502.208
at -13.50 m	FOS×M_{a2} =7144.985	< M_{p2} = 9801.458

[Moment on active side]

$$S_{a1} = \frac{35.928X}{2} = 17.964X \quad , \quad L_{a1} = 11.50 + \frac{1}{3}X$$

$$M_{a1'} = 5.988X^2 + 206.586X$$

$$S_{a2} = \frac{\left[35.928 + \frac{62.928 - 35.928}{3.000}X\right]X}{2} = 4.500X^2 + 17.964X \ , L_{a2} = 11.500 + \frac{2}{3}X$$

$$M_{a2'} = 3.000X^3 + 63.726X^2 + 206.586X$$

$$M_a = 3.000X^3 + 69.714X^2 + 413.172X + 4006.212$$

[Moment on passive side]

$$S_{p1} = \frac{147.500X}{2} = 73.750X \ , \quad L_{p1} = 11.500 + \frac{1}{3}X$$

$$M_{p1'} = 24.583X^2 + 848.125X$$

$$S_{p2} = \frac{\left[147.500 + \frac{174.500 - 147.500}{3.000}X\right]X}{2} = 4.500X^2 + 73.750X \ , \quad L_{p2} = 11.500 + \frac{2}{3}X$$

$$M_{p2'} = 3.000X^3 + 100.917X^2 + 848.125X$$

$$M_p = 3.000X^3 + 125.500X^2 + 1696.250X + 3502.208$$

[Balancing depth]
FOS Ma = Mp
$1.2 \times (3.000X^3 + 69.714X^2 + 413.172X + 4006.212)$
$= 3.000X^3 + 125.500X^2 + 1696.250X + 3502.208$

X= 1.049 m

Accordingly, toe elevation of the sheet pile shall be as follows:
D = -10.500 – 1.049 = -11.549 m

Calculation of maximum bending moment

Virtual seabed level

The elevation at which the active side pressure equals to the passive side pressure is defined as the virtual seabed level.

Just above -6.00m $P_{a1} = 65.428 > P_{p1} = 37.000$
Just below -6.00m $P_{a2} = 45.428 < P_{p2} = 57.000$

Accordingly, the virtual seabed level is obtained as -6.00m.

Moment about the top of the sheet pile

Elev. (m)	h (m)	Pa (kN/m2)	Pw (kN/m2)	Pp (kN/m2)	P (kN/m2)	S (kN/m)	L (m)	M (kNm/m)
1.00 0.50	0.50	6.963 8.419	15.251 20.301		22.214 28.720	5.554 7.180	0.167 0.333	0.926 2.393
0.50 0.43	0.07	0.000 0.000	20.301 21.008		20.301 21.008	0.711 0.735	0.523 0.547	0.372 0.402
0.43 0.35	0.08	0.000 0.000	21.008 21.008		21.008 21.008	0.840 0.840	0.597 0.623	0.501 0.524
0.35 -5.00	5.35	0.000 37.420	21.008 21.008		21.008 58.428	56.196 156.295	2.433 4.217	136.745 659.043
-5.00 -6.00	1.00	37.420 44.420	21.008 21.008	30.000 37.000	28.428 28.428	14.214 14.214	6.333 6.667	90.022 94.760
-6.00 -10.50	4.50	24.420 64.920	21.008 21.008	57.000 97.500	-11.572 -11.572			
Total						256.779	-	985.688

where ; S is the horizontal force (kN/m)
L is the distance of the horizontal force acting on the top of the sheet pile (m)
M is the moment about the top of the sheet pile (kN-m/m)

Reaction forces

Design seabed level : -5.000m
Virtual seabed level : -6.000m
Distance between supports : $L_T = 1.000 - (-6.000) = 7.000$ m
Reaction at virtual seabed : $R_0 = \frac{\Sigma M}{L_T} = \frac{985.688}{7.000} = 140.813 kN/m$
Reaction at the top of sheet pile : $A_p = \Sigma S - R_0 = 256.779 - 140.813 = 115.966 kN/m$

Calculation of shear force zero point

Elev. (m)	h (m)	P (kN/m2)	S (kN/m)	ΣS (kN/m)	Ap (kN/m)	Q (kN/m)
1.00 0.50	0.50	22.214 28.720	5.554 7.180	12.733	115.966	103.233
0.50 0.43	0.07	20.301 21.008	0.711 0.735	14.179	115.966	101.787
0.43 0.35	0.08	21.008 21.008	0.840 0.840	15.860	115.966	100.106
0.35 -5.00	5.35	21.008 58.428	56.196 156.295	228.351	115.966	-112.385
-5.00 -6.00	1.00	28.428 28.428	14.214 14.214	256.779	115.966	-140.813

Shear force Q = A_p - ∑S

The above table suggests that the shear force zero point exists in between +0.35m and -5.00m.

$$Q = 100.106 - \frac{[21.008 + (21.008 + 6.994X)]X}{2} = 0$$

= 100.106-21.008X-3,497X^2 = 0

X = 3.132m

Shear force zero point : DL = +0.350 – 3.132 = -2.782 m

Calculation of moment

Elev. (m)	h (m)	P (kN/m2)	S (kN/m)	L (m)	M (kNm/m)
1.00 0.50	0.50	22.214 28.720	-5.554 -7.180	3.615 3.449	-20.078 -24.761
0.50 0.43	0.07	20.301 21.008	-0.711 -0.735	3.259 3.235	-2.315 -2.379
0.43 0.35	0.08	21.008 21.008	-0.840 -0.840	3.185 3.159	-2.677 -2.654
0.35 -2.78	3.13	21.008 42.902	-32.899 -67.185	2.088 1.044	-68.692 -70.141
Total					-193.697

where; S is the horizontal force (kN/m)

L is the distance to the shear force zero point (m)

M is the moment about the shear force zero point (kN-m/m)

Maximum bending moment

- Distance of the shear zero point to the top of the sheet pile

  h = 1.000 – (-2.782) = 3.782 m

- Maximum bending moment

  M_{max} = A_p×h + ∑M = 115.966 × 3.782 – 193.697 = 244.886 kN-m/m

(5) Determination of Sheet Pile Section

Section properties

Steel Sheet Pile Type SP-IVw

- Modulus of elasticity : $E = 200.0$ kN/mm^2
- Moment of inertia : $I = 56700$ cm^4/m
- Section modulus : $Z = 2700$ cm^3/m

After corrosion

- Design life : 50 years
- Corrosion rate : 0.030 mm/year (effect of cathodic protection = 90%)
- Corrosion allowance : $t = 2 \times 0.030 \times 50$ years = 3.000 mm
- Moment of inertia : $I = 46454$ cm^4/m
- Section modulus : $Z = 2228$ cm^3/m

Embedded depth of sheet pile

Toe level of the sheet pile is obtained as follows:

- Deflection curve method : -13.532 m --- adopted
- Free earth support method : -11.549 m

Therefore, the minimum length of the sheet pile is;

L = 1.500 – (-13.532) = 15.032 m → 15.500 m

Accordingly the design toe level of the sheet pile is;

EL = +1.500 – 15.500 = -14.000 m ------------------ design toe level

Stress on the sheet pile

Maximum bending moment on the sheet pile is as follows:

- Deflection curve method : 362.450 kN-m/m --- adopted
- Free earth support method : 244.886 kN-m/m

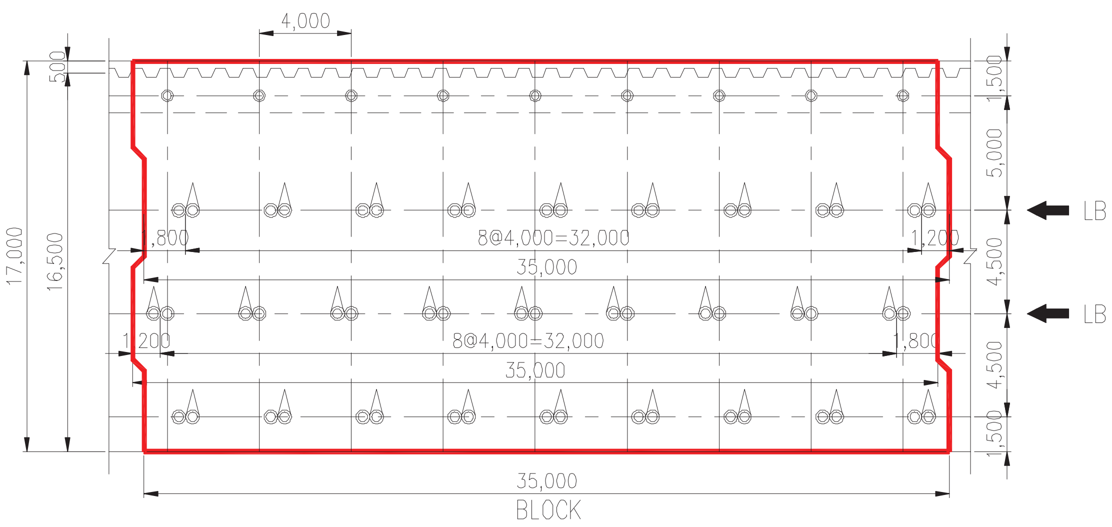
$$\sigma = \frac{362.450 \times 10^6}{2228 \times 10^3} = 162.7 \leq \sigma_a = 180.0 ... OK$$

2-4 Structural Analysis for PHC Pile and Deck

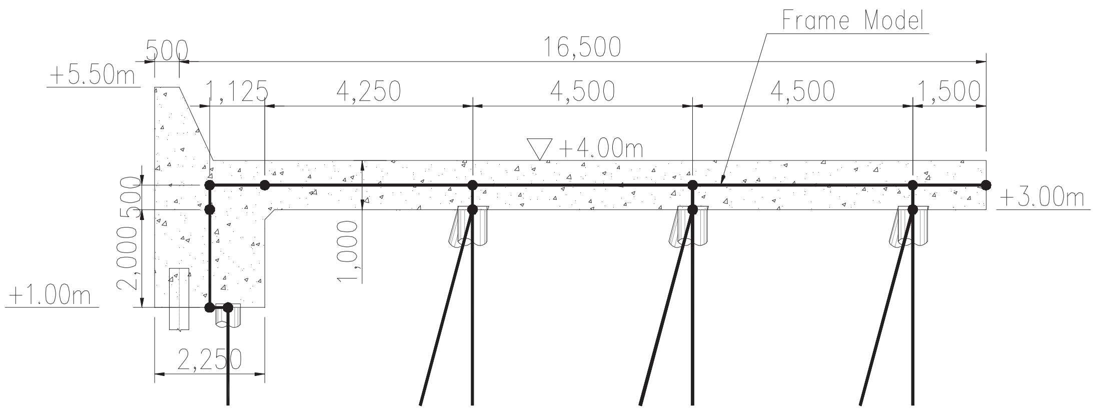
(1) Structural Analysis Model

2-dimensional frame model was built up with all structural components and piles were considered as rigidly connected to the deck structure. The pile-soil interaction was modeled by providing horizontal spring.

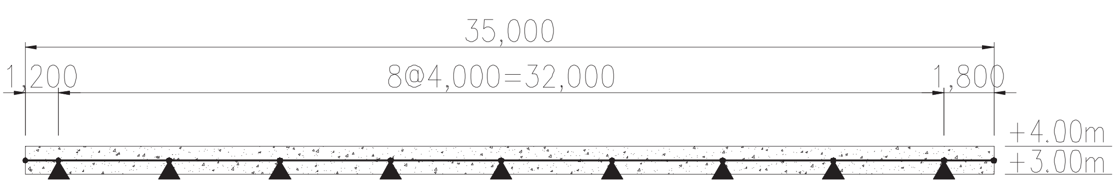
The analysis model is shown below.



<u>Transversal Direction</u>



<u>Longitudinal Direction</u>



The value of horizontal spring was calculated as following formula.

$K_h = 1500 \times N \times D$ (kN/m^2)

where,

K_h : Horizontal spring (kN/m^2)

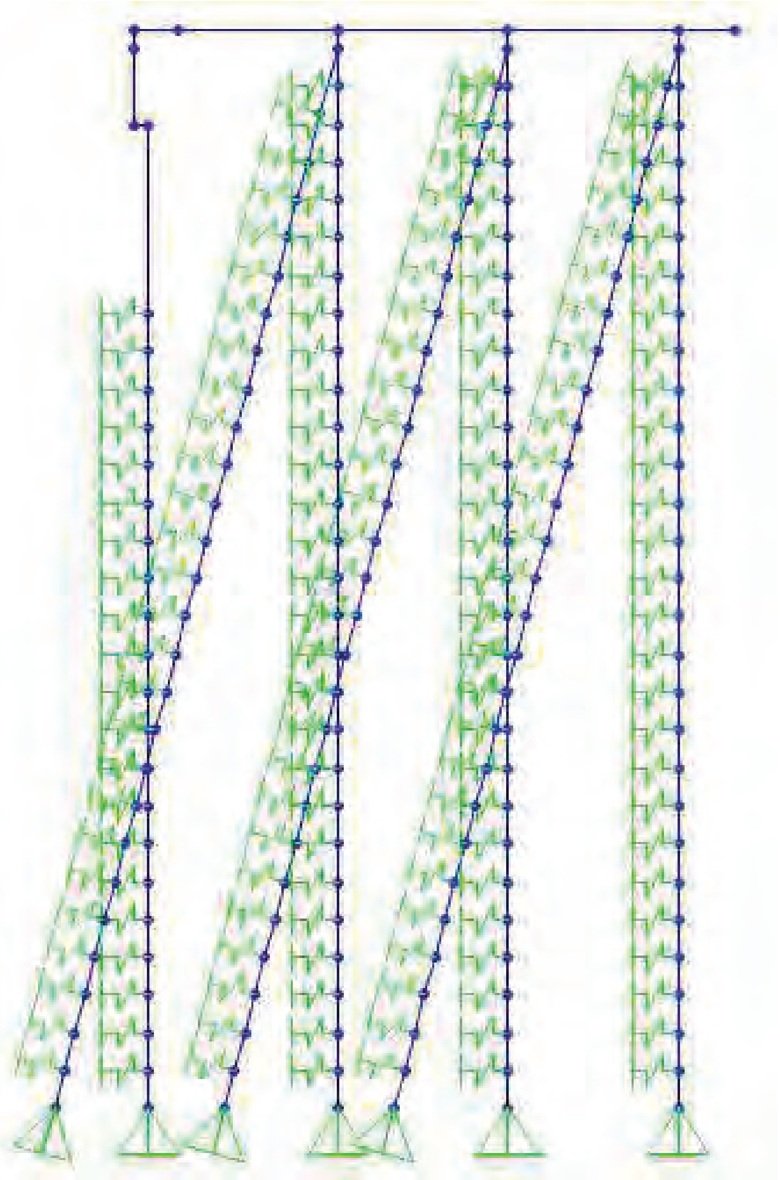
N : the value of SPT-N

D : the diameter of PHC pile (m)

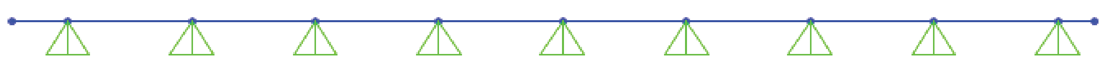
Layer	Elev. (m)	N	Kh (kN/m2) PHC ϕ500	Kh (kN/m2) PHC ϕ600
Reclamation Sand	5.0 0.5	5	3750	4500
Layer 2	0.5 -6.0	1	750	900
Layer 3b	-6.0 -10.5	5	3750	4500
Layer 4	-10.5 -13.5	10	7500	9000
Layer 5	-13.5 -25.0	6	4500	5400

The analysis model made by SAP2000 is shown below.

Transversal Direction



Longitudinal Direction



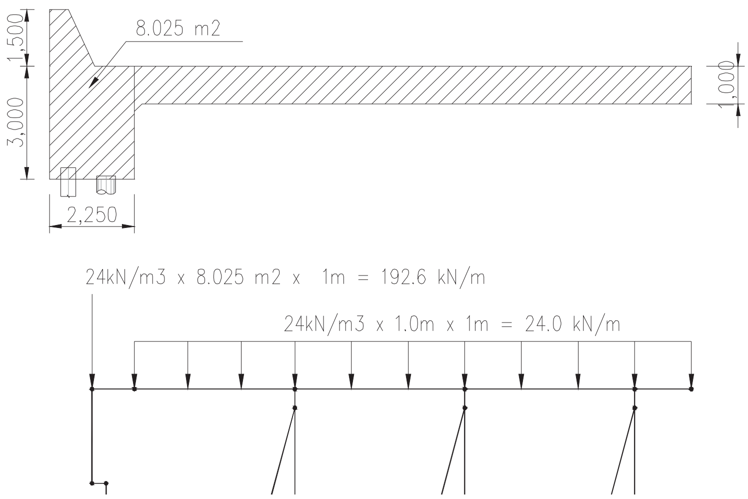
(2) Design Load

Load Combination is shown below.

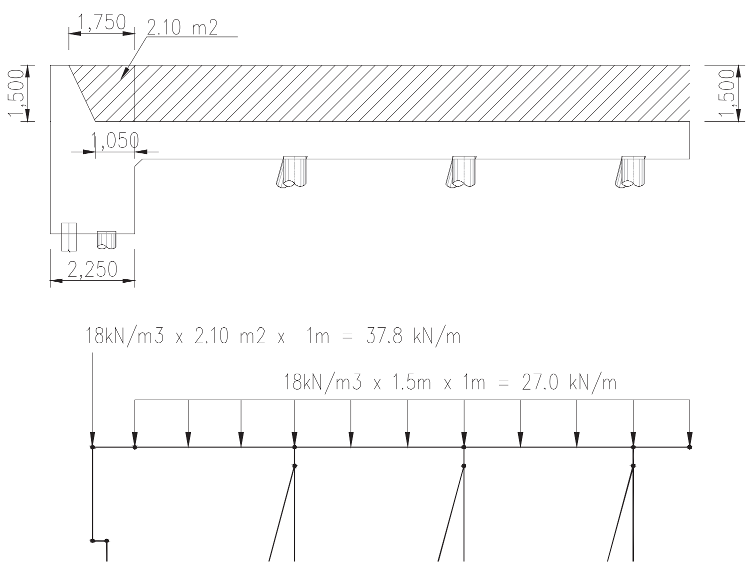
Case		DL of Con. (a)	DL of Soil (b)	Live Load (c)	Water Pressure. (d)	Soil Pressure (e)	Mooring Load (f)	Reaction Force (g)	Seismic Load (h)	Increase of Allowable Stress
Ordinary Case	Case1	x	x	x	x	x		x		1.0
	Case2	x	x		x	x		x		
Mooring Case	Case3	x	x	x	x	x	x	x		1.0
	Case4	x	x		x	x	x	x		

Each design load applied for the analysis model is shown as follows.

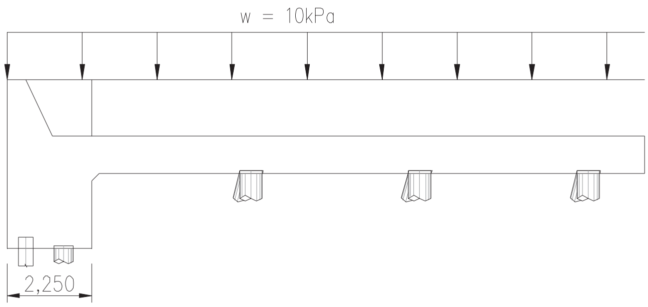
a) Dead Load of Concrete

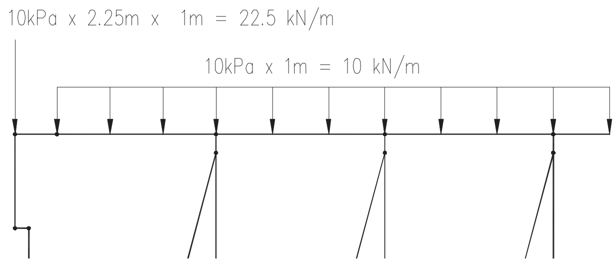


b) Dead Load of Soil

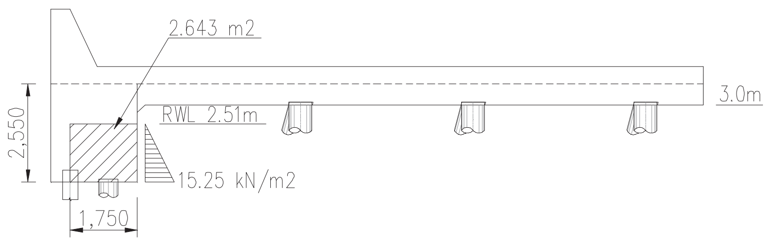


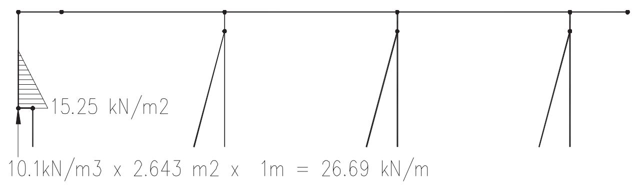
c) Live Load



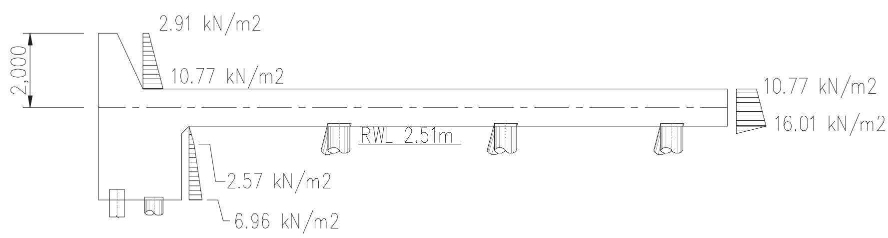


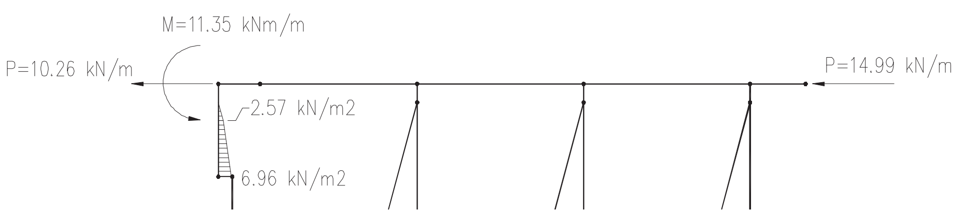
d) Water Pressure



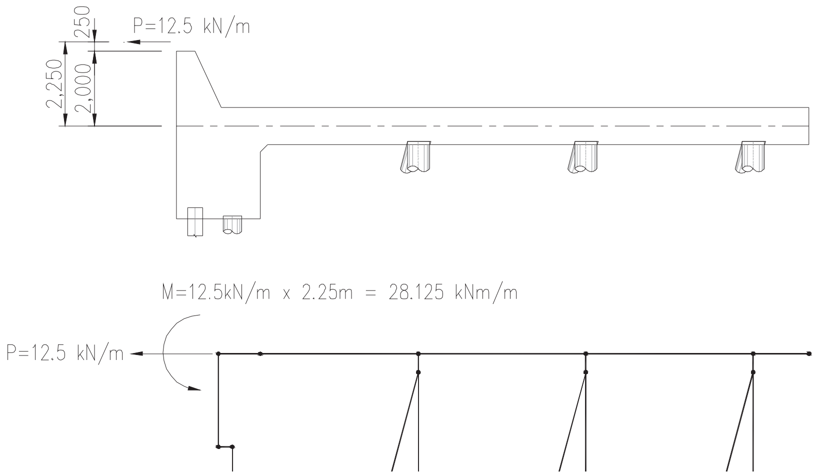


e) Soil Pressure

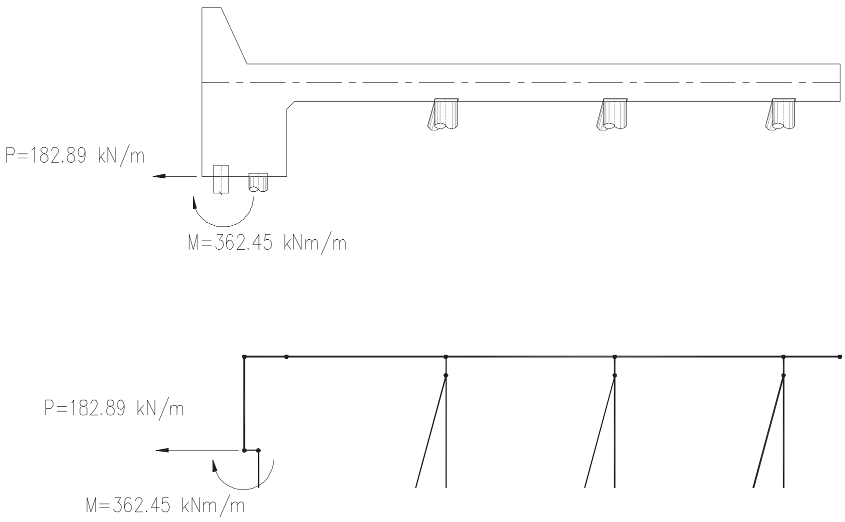




f) Mooring Load



g) Reaction Force Transmitted from Sheet Pile



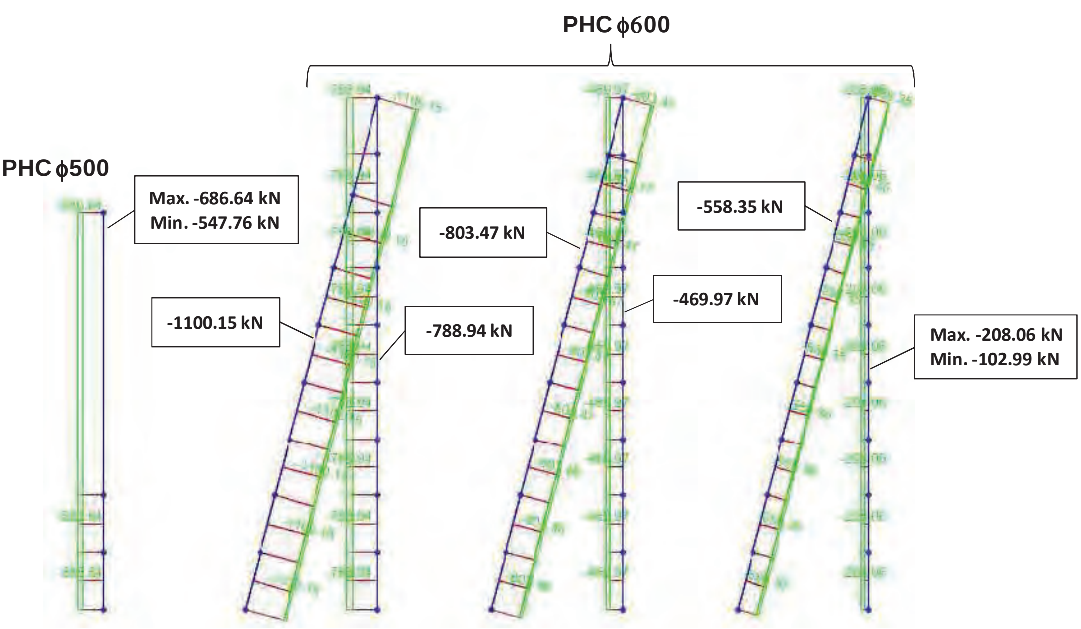
(3) Check of Prestressed Concrete Spun Pile

The prestressed concrete spun piles were checked by working stress design. The results of maximum axial forces and bending moments are shown below.

(Ordinary Case)

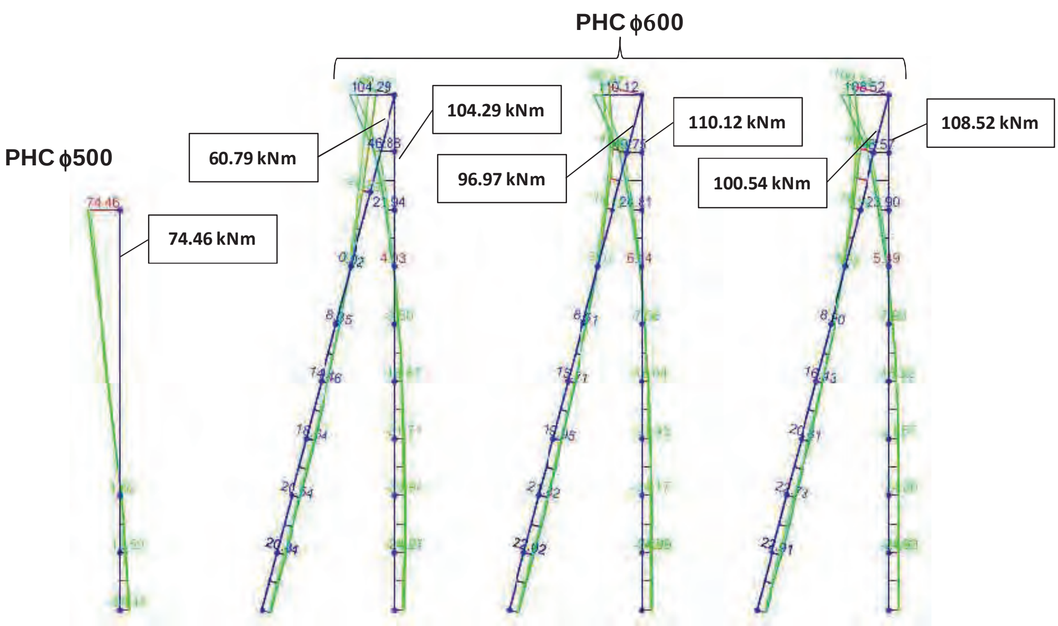
	Axial Force (kN)	Bending Moment (kNm)
PHC ϕ500	687	74
PHC ϕ600	1100	110

Axial Force (Envelope of Case1 & 2)



note : Negative value is compressive force.

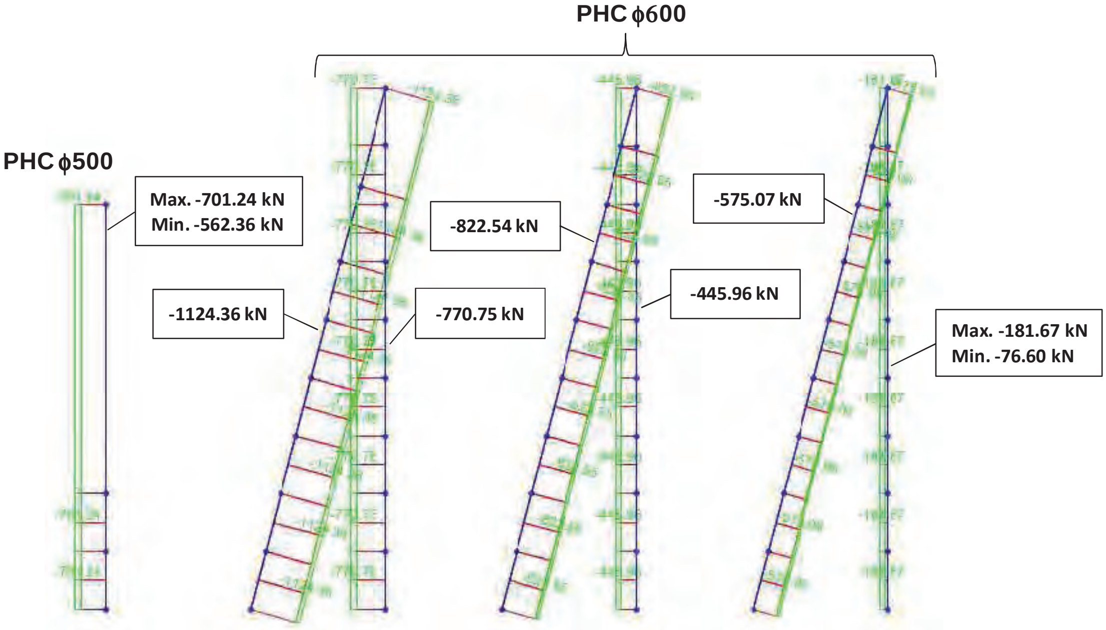
Bending Moment (Envelope of Case1 & 2)



(Mooring Case)

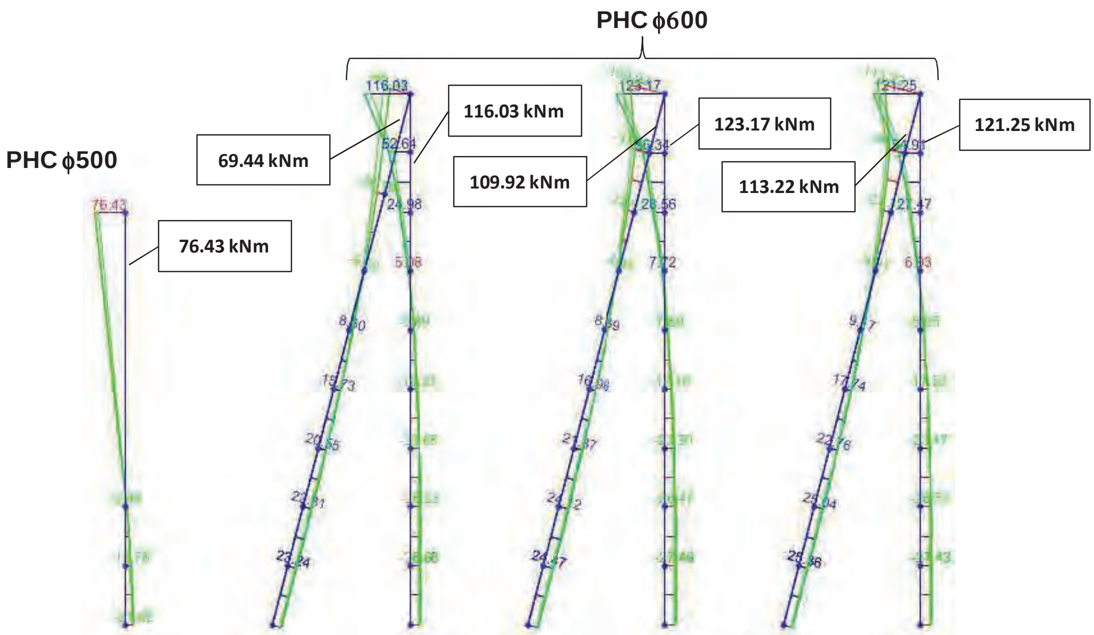
	Axial Force (kN)	Bending Moment (kNm)
PHC ϕ500	701	76
PHC ϕ600	1124	123

Axial Force (Envelope of Case3 & 4)



note : Negative value is compressive force.

Bending Moment (Envelope of Case3 & 4)



The interaction curve diagrams were obtained based on the following formula.

Compression : $\sigma_c = \frac{P}{A} + \frac{M}{Z} + \sigma_e \leq \sigma_{ca}$

Tension : $\sigma_t = \frac{P}{A} - \frac{M}{Z} + \sigma_e \geq \sigma_{ta}$

where,

σ_c: maximum compressive stress (N/mm^2)

σ_t: maximum tensile stress (N/mm^2)

P : axial force (N) … positive in compression and negative in tension

M : bending moment (N-mm)

A : cross sectional area (mm^2)

Z : elastic modulus (mm^3)

σ_e: effective prestress (N/mm^2)

σ_{ca}: allowable compressive stress (N/mm^2)

$$\sigma_{ca} = \begin{cases} 27.0MPa: \text{ for Long Term} \\ 40.5MPa: \text{ for Short Term} \end{cases}$$

σ_{ta}: allowable tensile stress (N/mm^2)

$$\sigma_{ta} = \begin{cases} 0MPa: \text{ for Long Term} \\ 5MPa: \text{ for Short Term} \end{cases}$$

All combinations of bending moment and axial force under working load condition were plotted on the interaction curve diagrams in the following pages.

The plotted results show that the pile sections are adequate.

(Ordinary Case – PHC ϕ500)

Pile Check by Working Stress Design (PHC ϕ500)

Allowable Strength

	Long Term MPa	Short Trem MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	500 mm	OD	500 mm
Thickness	80 mm	ID	340 mm
Outer Length of PC Steel	420 mm	PC OD	420.83 mm
Diameter of PC	10.0 mm	PC ID	419.17 mm
Sectional Area of PC Steel	78.5 mm2/nos	A all	105,558 mm2
Number of PC Steel	14 nos	A conc	104,458 mm2
Total Sectional area of PC Steel	1,100 mm2	A p	1,100 mm2
Effective Prestress	8 MPa	n	5.405
		I conc	2.388.E+09 mm4
Concrete Data		I p	2.425.E+07 mm4
Design Strength	85 MPa	I comp	2.519.E+09 mm4
Modulus of Elasticity	37,000 MPa	A comp	110,402 mm2
		Z comp	10,075,199 mm3

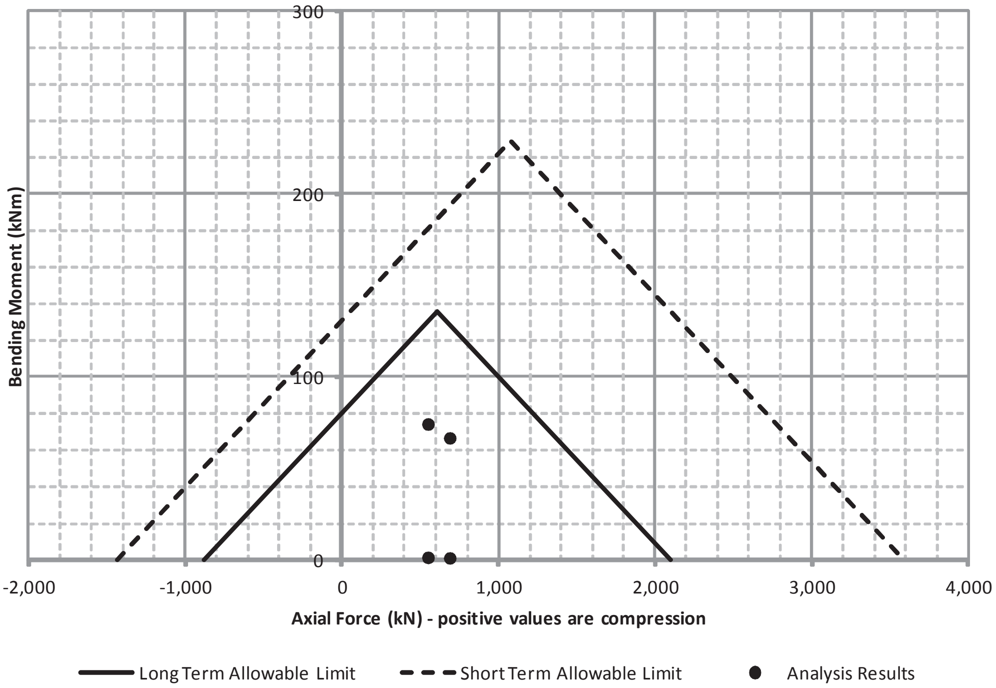
PC Steel Data

Tensile strength of PC	1,420 MPa
Yield strength of PC	1,275 MPa
Modulus of Elasticity	200,000 MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-883	0	-1,435	0
607	136	1,076	229
2,098	0	3,588	0

Interaction Curve for Prestressed Concrete Spun Pile



(Ordinary Case – PHC ϕ600)

Pile Check by Working Stress Design (PHC ϕ600)

Allowable Strength

	Long Term MPa	Short Term MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	600	mm	OD	600	mm
Thickness	90	mm	ID	420	mm
Outer Length of PC Steel	510	mm	PC OD	510.98	mm
Diameter of PC	10.0	mm	PC ID	509.02	mm
Sectional Area of PC Steel	78.5	mm2/nos	A all	144,199	mm2
Number of PC Steel	20	nos	A conc	142,628	mm2
Total Sectional area of PC Steel	1,571	mm2	A p	1,571	mm2
Effective Prestress	8	MPa	n	5.405	
			I conc	4.783.E+09	mm4
Concrete Data			I p	5.107.E+07	mm4
Design Strength	85	MPa	I comp	5.059.E+09	mm4
Modulus of Elasticity	37,000	MPa	A comp	151,119	mm2
			Z comp	16,864,207	mm3

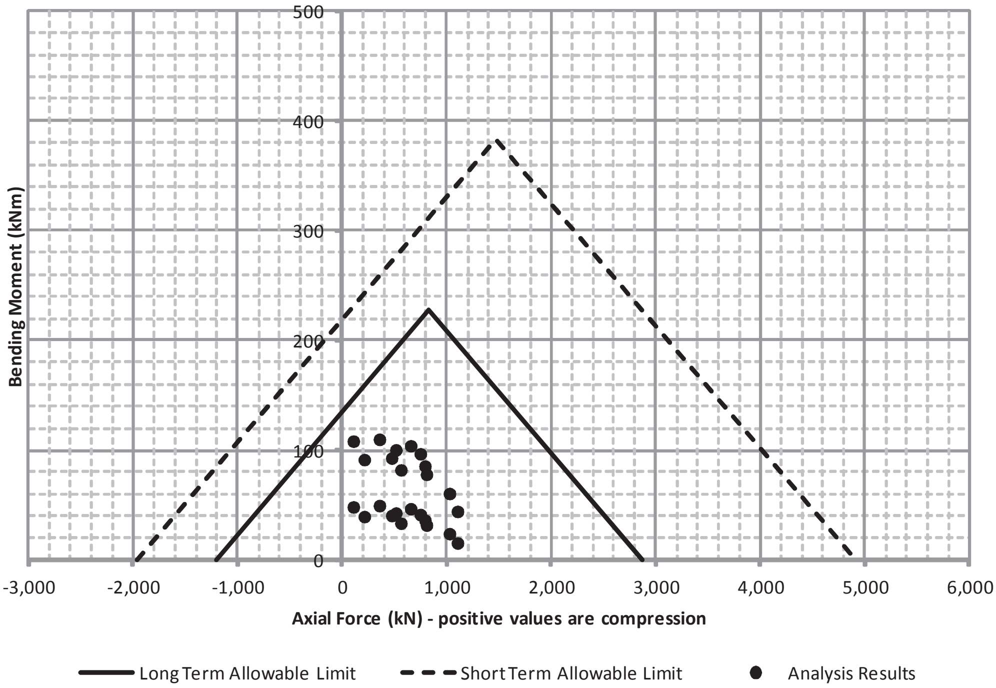
PC Steel Data

Tensile strength of PC	1,420	MPa
Yield strength of PC	1,275	MPa
Modulus of Elasticity	200,000	MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-1,209	0	-1,965	0
831	228	1,473	384
2,871	0	4,911	0

Interaction Curve for Prestressed Concrete Spun Pile



(Mooring Case – PHC ϕ500)

Pile Check by Working Stress Design (PHC ϕ500)

Allowable Strength

	Long Term MPa	Short Trem MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	500 mm	OD	500 mm
Thickness	80 mm	ID	340 mm
Outer Length of PC Steel	420 mm	PC OD	420.83 mm
Diameter of PC	10.0 mm	PC ID	419.17 mm
Sectional Area of PC Steel	78.5 mm2/nos	A all	105,558 mm2
Number of PC Steel	14 nos	A conc	104,458 mm2
Total Sectional area of PC Steel	1,100 mm2	A p	1,100 mm2
Effective Prestress	8 MPa	n	5.405
		I conc	2.388.E+09 mm4
Concrete Data		I p	2.425.E+07 mm4
Design Strength	85 MPa	I comp	2.519.E+09 mm4
Modulus of Elasticity	37,000 MPa	A comp	110,402 mm2
		Z comp	10,075,199 mm3

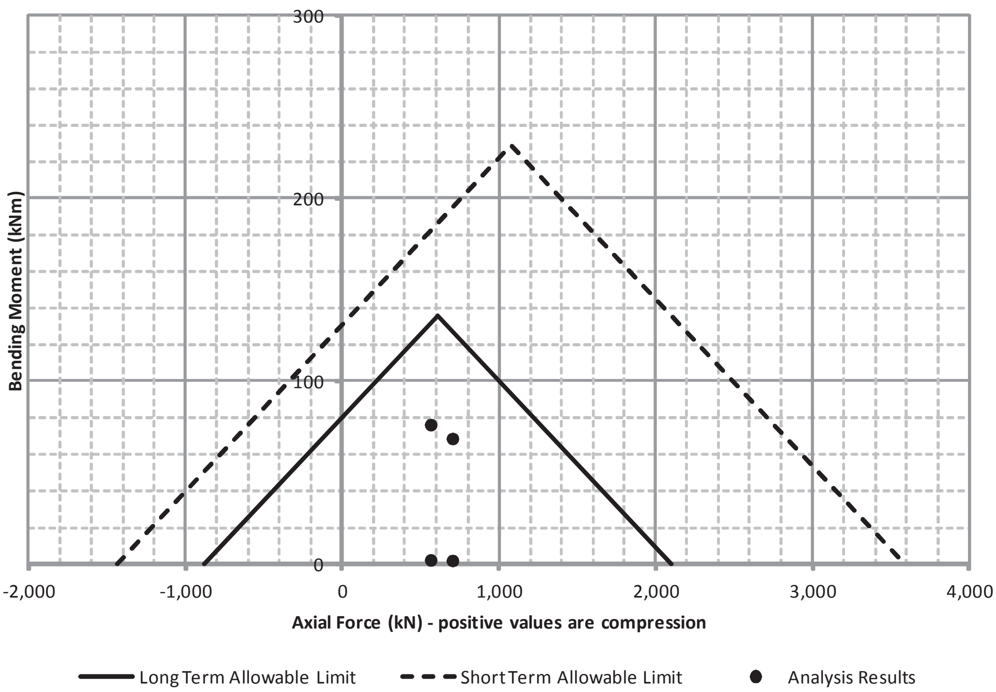
PC Steel Data

Tensile strength of PC	1,420 MPa
Yield strength of PC	1,275 MPa
Modulus of Elasticity	200,000 MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-883	0	-1,435	0
607	136	1,076	229
2,098	0	3,588	0

Interaction Curve for Prestressed Concrete Spun Pile



(Mooring Case – PHC ϕ600)

Pile Check by Working Stress Design (PHC ϕ600)

Allowable Strength

	Long Term MPa	Short Term MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	600 mm	OD	600 mm
Thickness	90 mm	ID	420 mm
Outer Length of PC Steel	510 mm	PC OD	510.98 mm
Diameter of PC	10.0 mm	PC ID	509.02 mm
Sectional Area of PC Steel	78.5 mm2/nos	A all	144,199 mm2
Number of PC Steel	20 nos	A conc	142,628 mm2
Total Sectional area of PC Steel	1,571 mm2	A p	1,571 mm2
Effective Prestress	8 MPa	n	5.405
		I conc	4.783.E+09 mm4
Concrete Data		I p	5.107.E+07 mm4
Design Strength	85 MPa	I comp	5.059.E+09 mm4
Modulus of Elasticity	37,000 MPa	A comp	151,119 mm2
		Z comp	16,864,207 mm3

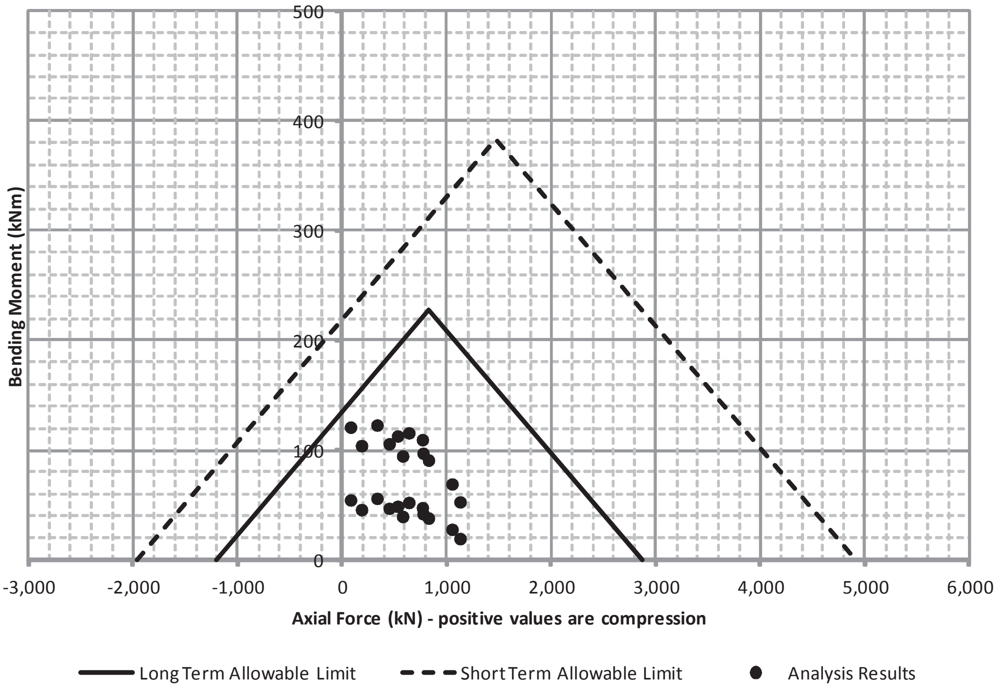
PC Steel Data

Tensile strength of PC	1,420 MPa
Yield strength of PC	1,275 MPa
Modulus of Elasticity	200,000 MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-1,209	0	-1,965	0
831	228	1,473	384
2,871	0	4,911	0

Interaction Curve for Prestressed Concrete Spun Pile



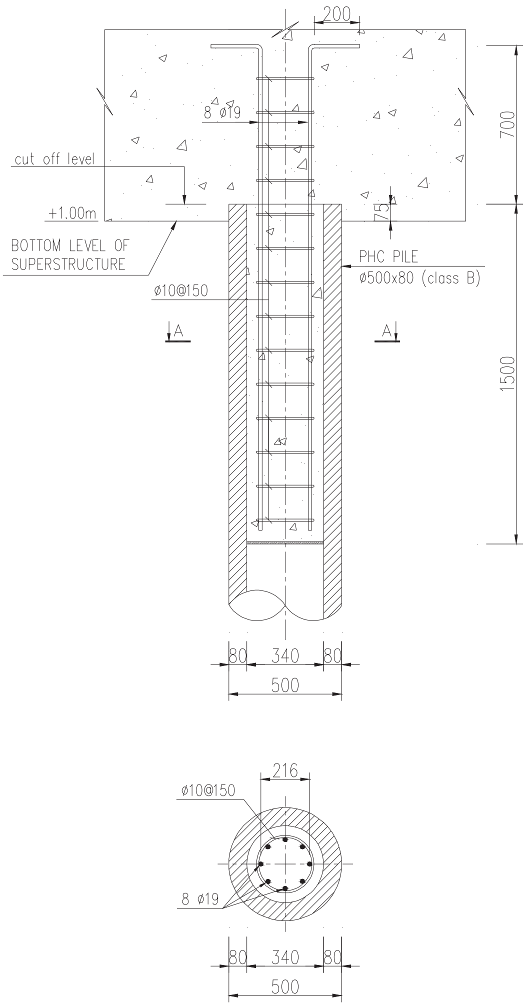
(4) Check of Pile Head Reinforcement

The pile head connection was checked for the reinforced concrete section having a diameter 150mm (embedment to deck of 75mm on one side) larger than PHC pile diameter.

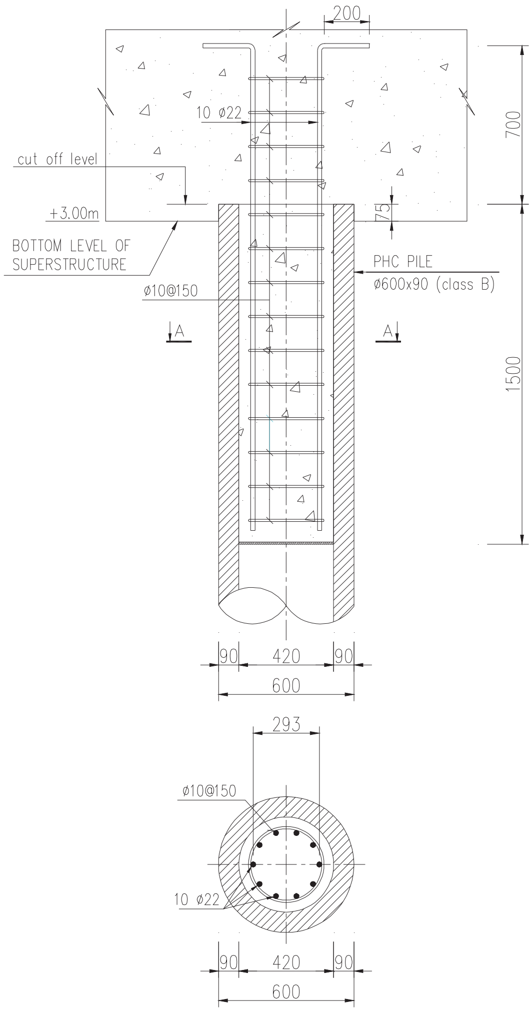
Forces acting on the pile head are shown below.

		Axial Force		Bending Moment
		Max.(kN)	Min.(kN)	(kNm)
Ordinary Case	PHC ϕ500	687	548	74
	PHC ϕ600	1100	103	110
Mooring Case	PHC ϕ500	701	562	76
	PHC ϕ600	1124	77	123

The steel reinforcement to be provided at the pile head is shown below.



PHC ø500



PHC ø600

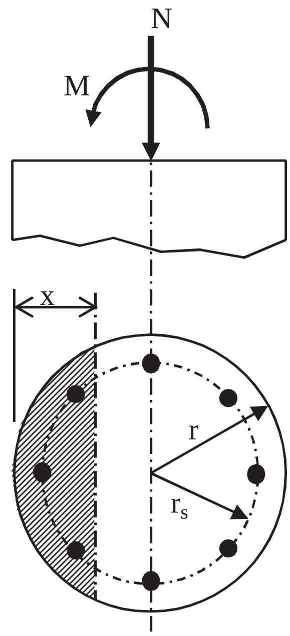
The calculation results are shown in the following pages which are all satisfactory.

(Ordinary Case – PHC ϕ500)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	74	kNm
Axial Force (positive = compression)	N =	548	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	466.43339	mm
Extreme fiber strain (concrete)		0.000167	

Maximum stress

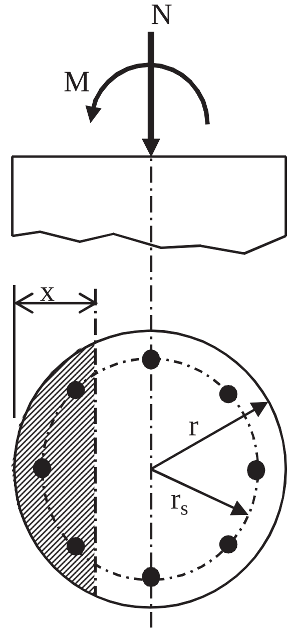
Concrete	fc =	4.68	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Ordinary Case – PHC ϕ500)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	74	kNm
Axial Force (positive = compression)	N =	687	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	545.98191	mm
Extreme fiber strain (concrete)		0.0001722	

Maximum stress

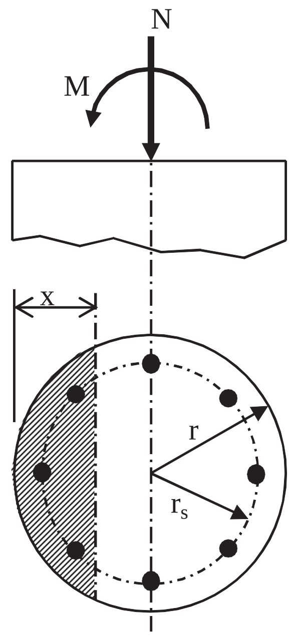
Concrete	fc =	4.82	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Ordinary Case – PHC ϕ600)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	147	mm

1.3 Design forces

Bending Moment	M =	110	kNm
Axial Force (positive = compression)	N =	103	kN

1.4 Rebar arrangement

Dia		D22	mm
Nos		10	nos
As	As =	3801.3	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	193.56244	mm	
Extreme fiber strain (concrete)		0.0003174		
Maximum stress				
Concrete	fc =	8.89	MPa	**OK**
Rebar	fs =	105.35	MPa	**OK**

(Ordinary Case – PHC ϕ600)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input

1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	147	mm

1.3 Design forces

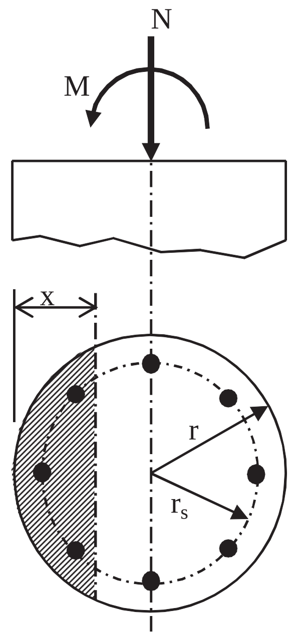
Bending Moment	M =	110	kNm
Axial Force (positive = compression)	N =	1110	kN

1.4 Rebar arrangement

Dia		D22	mm
Nos		10	nos
As	As =	3801.3	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa



2. Results of Calculation

Neutral axis	x	716.19766	mm	
Extreme fiber strain (concrete)		0.000179		
Maximum stress				
Concrete	fc =	5.01	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC φ500)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input

1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

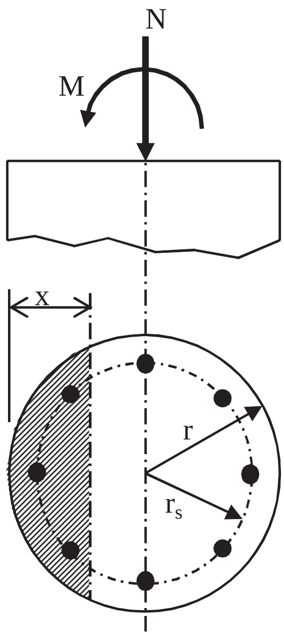
Bending Moment	M =	76	kNm
Axial Force (positive = compression)	N =	562	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa



2. Results of Calculation

Neutral axis	x	465.79654	mm	
Extreme fiber strain (concrete)		0.0001716		
Maximum stress				
Concrete	fc =	4.80	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC ϕ500)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input

1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

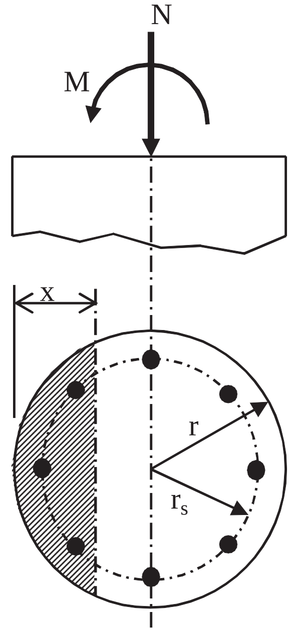
Bending Moment	M =	76	kNm
Axial Force (positive = compression)	N =	701	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa



2. Results of Calculation

Neutral axis	x	544.9915	mm
Extreme fiber strain (concrete)		0.0001761	

Maximum stress

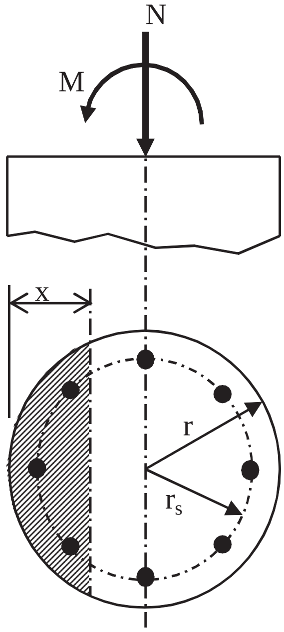
Concrete	fc =	4.93	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC ϕ600)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	147	mm

1.3 Design forces

Bending Moment	M =	123	kNm
Axial Force (positive = compression)	N =	77	kN

1.4 Rebar arrangement

Dia		D22	mm
Nos		10	nos
As	As =	3801.3	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	186.17772	mm	
Extreme fiber strain (concrete)		0.0003672		
Maximum stress				
Concrete	fc =	10.28	MPa	**OK**
Rebar	fs =	129.64	MPa	**OK**

(Mooring Case – PHC ϕ600)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input

1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	147	mm

1.3 Design forces

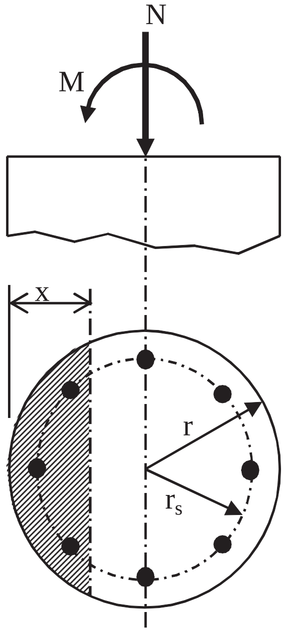
Bending Moment	M =	123	kNm
Axial Force (positive = compression)	N =	1124	kN

1.4 Rebar arrangement

Dia		D22	mm
Nos		10	nos
As	As =	3801.3	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa



2. Results of Calculation

Neutral axis	x	678.83478	mm
Extreme fiber strain (concrete)		0.0001922	

Maximum stress

Concrete	fc =	5.38	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(5) Check of Pile Bearing Capacity

1) Check of Ultimate Bearing Capacity

Ultimate bearing capacity is calculated as following formulas.

<u>The piles driven into sandy ground by hammer</u>

$$R_u = 300NA_p + 2\overline{N}A_s$$

where,

R_u : ultimate bearing capacity of pile (kN)

A_p : toe area of pile (m^2)

A_s : total circumferential area of pile (m^2)

N : nominal N-value for end bearing

$\overline{N}$: mean N-value for total penetration length of pile

The nominal N-value for end bearing is calculated by equation below.

$$N = \frac{N_1 + \overline{N_2}}{2}$$

where,

N_1 : N-value at the toe of pile

$\overline{N_2}$: mean N-value in the range from the toe of pile to the level 4B above

B : diameter or width of pile (m)

<u>The piles driven into clayey ground by hammer</u>

$$R_u = 8c_pA_p + \overline{c_a}A_s$$

where,

c_p : cohesion at pile toe (kN/m^2)

$\overline{c_a}$: mean adhesion for total embedded length of pile (kN/m^2)

The adhesion value may be calculated as follows:

$c_a = c$: $c \leq 100$ kN/m^2

$c_a = 100$kN/m^2 : $c > 100$ kN/m^2

<u>Factor of Safty</u>

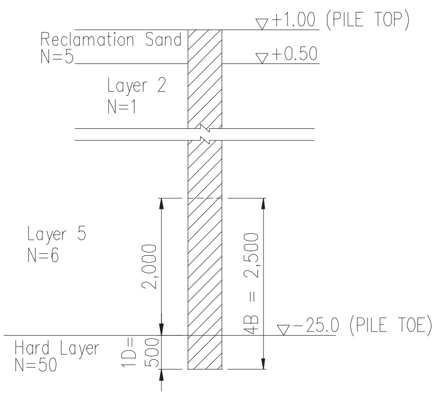
Allowable working axial loads on piles are obtained by dividing ultimate pile capacity by factors of safety which is shown below.

	Compressive	Pulling
Ordinary/Mooring Case	2.5	3.0

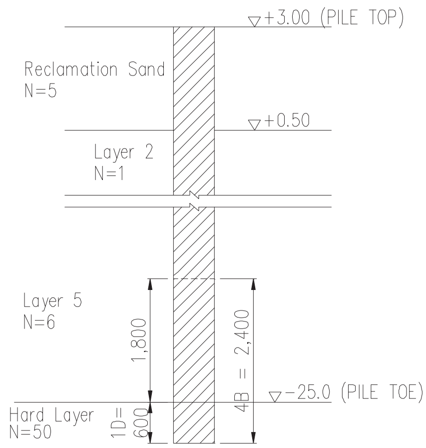
PHC piles are planned to be penetrated 1D into hard layer below Layer-5.
Pile dimensions and the elevations are shown below.

Pile dimensions

	D (m)	t (m)	A (m2)	Ap (m2)	As (m2)	Pile Top (m)	Pile Toe (m)	Weight (kN)
PHC φ500	0.50	0.08	0.106	0.196	1.571	1.00	-25.50	39.05
PHC φ600	0.60	0.09	0.144	0.283	1.885	3.00	-25.60	57.25



PHC ∅500



PHC ∅600

The soil parameter and the calculation results of skin friction are shown below.

Soil parameter and Skin friction

	Elev. (m)	Thick (m)	N	C (kN/m2)	2N or Ca (kN/m2)	Skin Friction (kN) PHC φ500	Skin Friction (kN) PHC φ600
Reclamation Sand	3.0 1.0	2.0	5.0		10.0	-	38
	1.0 0.5	0.5	5.0		10.0	8	9
Layer-2	0.5 -6.0	6.5	1.0	15.0	15.0	153	184
Layer-3b	-6.0 -10.5	4.5	5.0	25.0	25.0	177	212
Layer-4	-10.5 -13.5	3.0	10.0	50.0	50.0	236	283
Layer-5	-13.5 -25.0	11.5	6.0	40.0	40.0	723	867
Hard Layer	-25.0		50.0				
Total						1,297	1,593

End bearing was calculated as follows.

PHCφ500 : $\overline{N_2} = \dfrac{50 \times 0.5m + 6 \times 1.5m}{2.0m} = 17$

PHCφ600 : $\overline{N_2} = \dfrac{50 \times 0.6m + 6 \times 1.8m}{2.4m} = 17$

$$N_1 = 50, \quad \overline{N_2} = 17$$

$$N = \frac{N_1 + \overline{N_2}}{2} = \frac{50 + 17}{2} = 33.5$$

PHC φ500

Ru = 300NAp = 300 × 33.5 × 0.196 = 1970 kN

PHC φ600

Ru = 300NAp = 300 × 33.5 × 0.283 = 2844 kN

Accordingly, the calculation results of the bearing capacity are shown below which are all satisfactory.

Calculation of Allowable Bearing Capacity

		End Bearing	Skin Friction	Ultimate Bearing Capacity Ru		FOS		Allowable Bearing Capacity Ra	
		(kN)	(kN)	Comp. (kN)	Pulling (kN)	Comp.	Pulling	Comp. (kN)	Pulling (kN)
PHC φ500	Ordinary Case / Mooring Case	1,970	1,297	3,267	1,297	2.5	3.0	1,307	432
PHC φ600	Ordinary Case / Mooring Case	2,844	1,593	4,437	1,593	2.5	3.0	1,775	531

Check of the Bearing Capacity

		Allowable Bearing Capacity Ra		Max. Axial Force P		Pile Weight	Check		Judge
		Comp. (kN)	Pulling (kN)	Comp (kN)	Pulling (kN)	W (kN)	P+W Comp (kN)	P-W Pulling (kN)	< Ra ?
PHC φ500	Ordinary Case	1,307	432	687	-	39	726	-	OK
	Mooring Case			701	-		740	-	
PHC φ600	Ordinary Case	1,775	531	1100	-	57	1,157	-	OK
	Mooring Case			1124	-		1,181	-	

2) Check of the bearing capacity against negative skin friction

The negative skin friction is calculated as following formula.

$$R_{nf,\max} = \varphi L_2 \overline{f_s}$$

where, $R_{nf,\max}$ is maximum negative skin friction (for single pile) (kN)

φ is circumference of pile (m)

L_2 is length of piles in the consolidation layer (m)

$\overline{f_s}$ is mean skin friction intensity in the consolidation layer (kN/m^2)

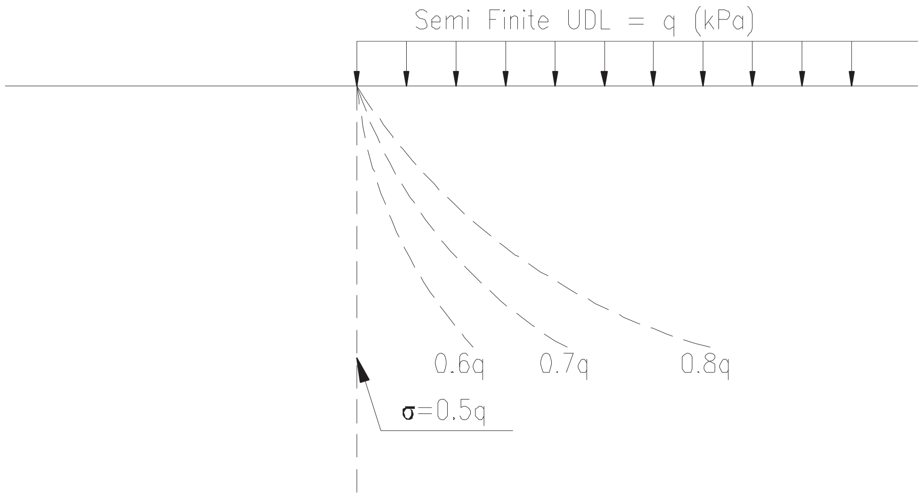
The bearing capacity against negative skin friction was checked by following formula.

$$R_a \leq \frac{1}{1.2} R_p - R_{nf,\max}$$

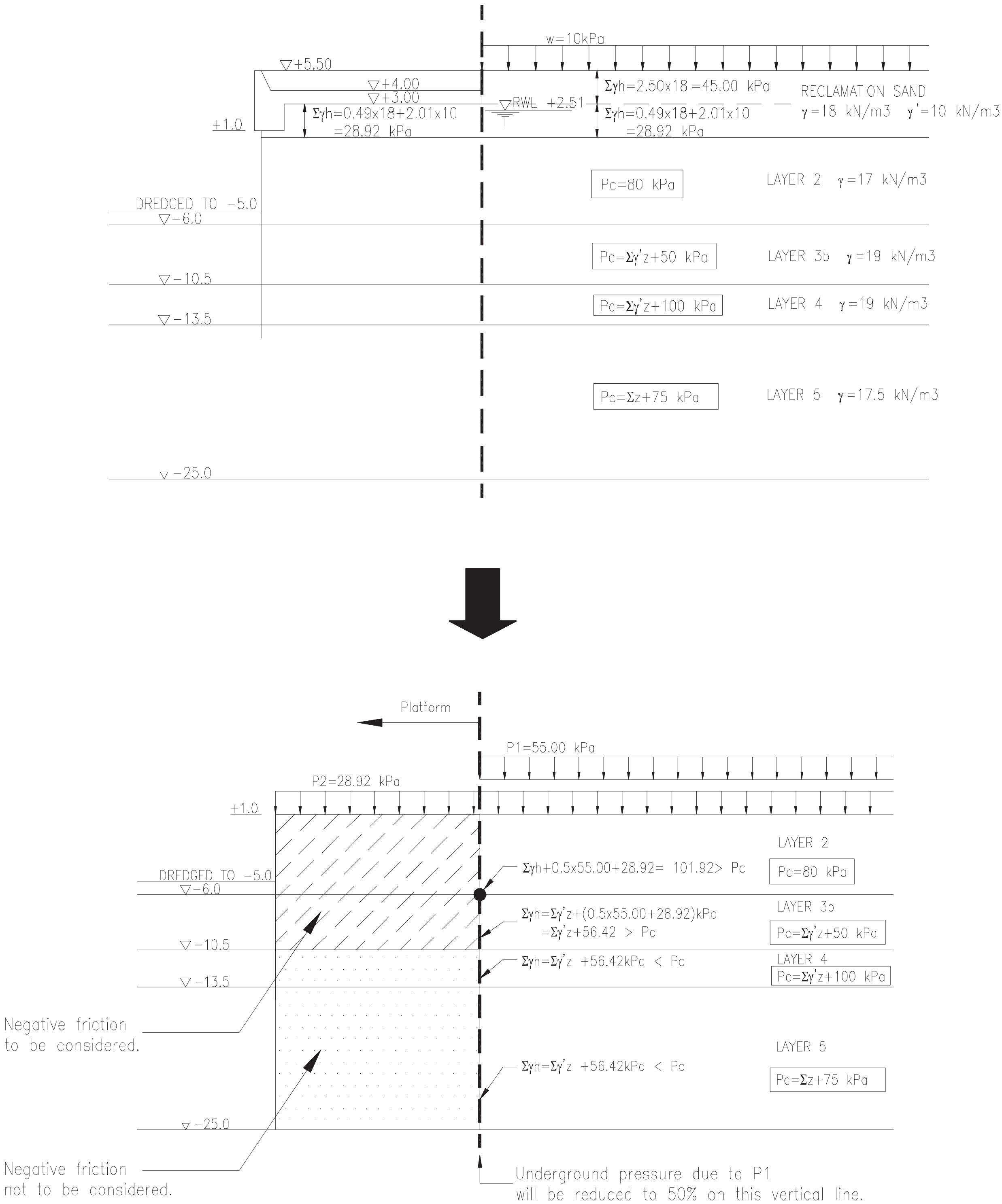
where, R_a is allowable axial bearing capacity (ordinary) (kN)

R_p is end bearing capacity of pile (ultimate value) (kN)

The sketch below presents the underground stress distribution due to semi finite UDL. As shown in the figure below, UDL will be reduced to 50% of UDL in the boundary line.



Considering the effect of the overburden pressure behind the relieving platform (i.e. underground stress distributed below the platform), Layer 2 and 3b are considered to be the consolidation layers which contribute to the negative friction to the piles.



The negative skin friction was calculated as follows.

	Elev. (m)	Thick (m)	N	C (kN/m2)	2N or Ca (kN/m2)	NegativeSkin Friction (kN) PHC f500	PHC f600
Reclamation Sand	3.0 1.0	2.0	5.0		10.0	-	38
	1.0 0.5	0.5	5.0		10.0	8	9
Layer-2	0.5 -6.0	6.5	1.0	15.0	15.0	153	184
Layer-3b	-6.0 -10.5	4.5	5.0	25.0	25.0	177	212
Total						338	443

Accordingly, the check of bearing capacity against the negative skin friction is shown below which are all satisfactory.

<u>PHC ϕ500</u>

R_p = 1970 kN

P_{max} = 726 kN(Ordinary), 740kN(Mooring)

$R_p/1.2 - R_{nf,max} = 1970/1.2 - 338 = 1304 \geq P_{max}$… OK

<u>PHC ϕ600</u>

R_p = 2844 kN

P_{max} = 1157 kN(Ordinary), 1181kN(Mooring)

$R_p/1.2 - R_{nf,max} = 2844/1.2 - 443 = 1927 \geq P_{max}$… OK

(6) Check of Reinforcement for Deck Slab

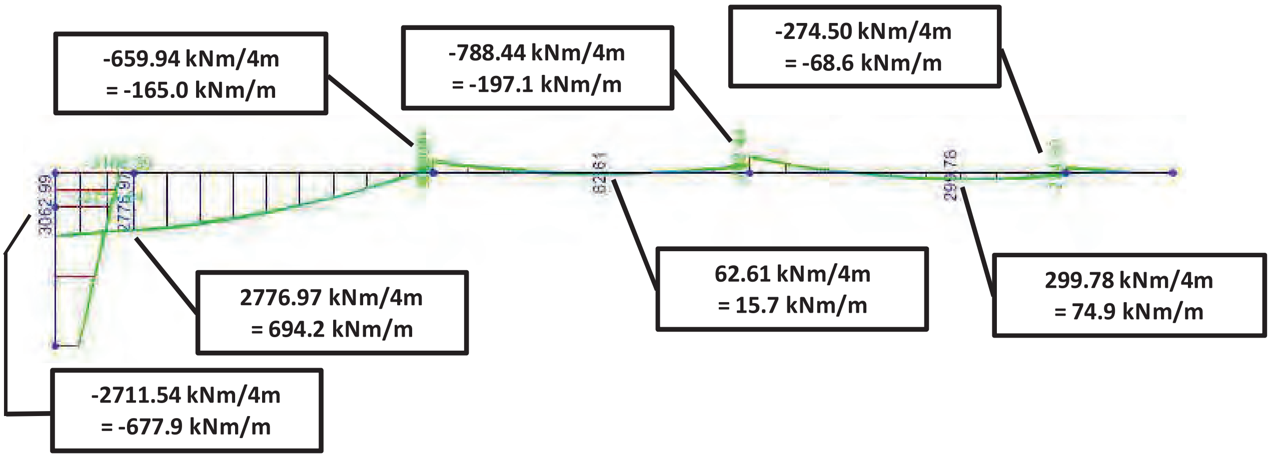
The deck slab was checked the bending moment and shear force.

The analysis results are shown below.

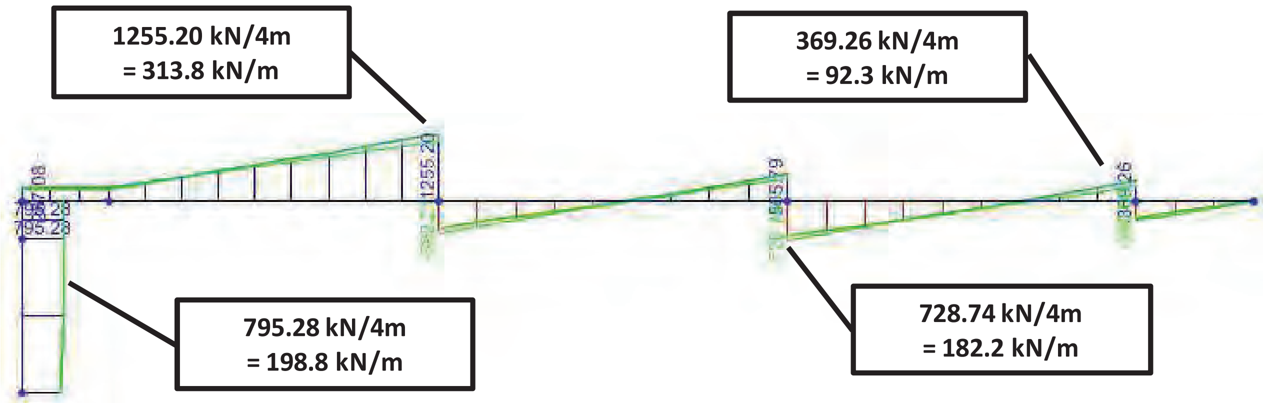
(Ordinary Case)

Transversal Direction

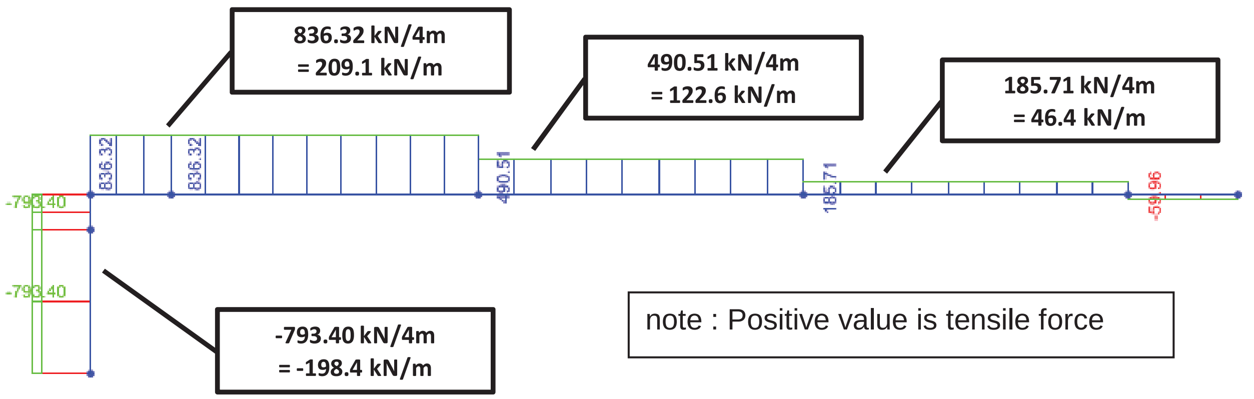
- Bending Moment (Envelope of Case 1 & 2)



- Shear Force (Envelope of Case 1 & 2)

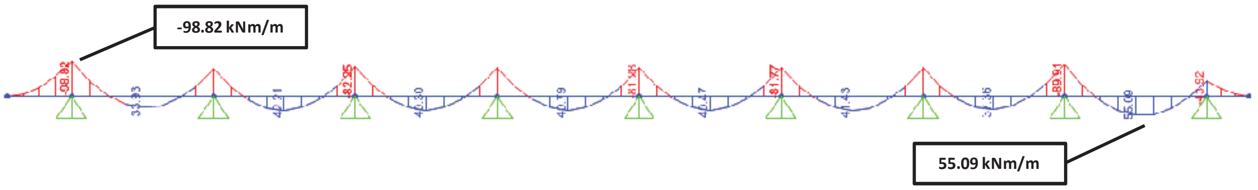


- Axial Force (Envelope of Case 1 & 2)



<u>Longitudinal Direction</u>

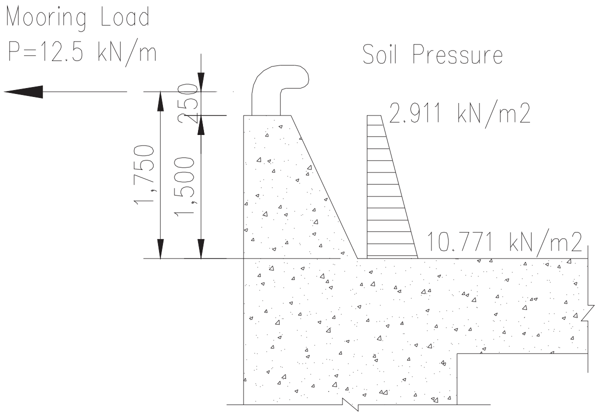
- Bending Moment



- Shear Force

Parapet

The force acting for the parapet is calculated as follows.



- Bending Moment

Bending Moment due to Mooring Load

Md 1 = 12.5 kN/m×1.75m = 21.880 kNm/m

Bending Moment due to Soil Pressure

Md 2 = (2.911 kN/m^2×1.5m / 2)×1.5m×2/3 + (10.771 kN/m2×1.5m/2)×1.5m×1/3

= 2.183 kNm/m + 4.039 kNm/m = 6.222 kNm/m

Total Bending Moment

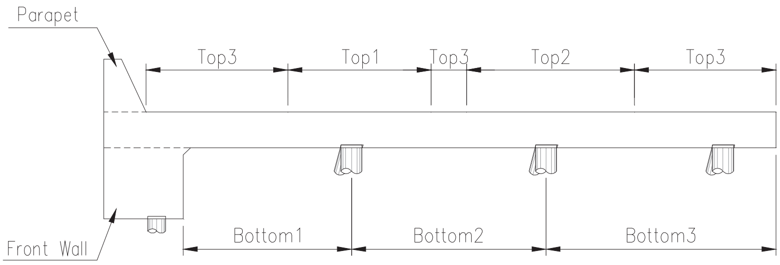
Md = Md1 + Md2 = 28.10 kNm/m

- Shear Force

Vd = 12.5 kN/m + (2.911 kM/m^2+10.771 kN/m^2)×1.5m / 2 = 22.76 kN/m

The concrete sections were checked as following pages and the results are all satisfactory.

(Ordinary Case)



Calculation Sheet for Reinforced Concrete Section

		Sym.	Unit	Transversal Direction					
				Bottom 1	**Bottom 2**	**Bottom 3**	**Top 1**	**Top 2**	**Top 3**
Type of Member									
				Slab	**Slab**	**Slab**	**Slab**	**Slab**	**Slab**
Section Property									
Breadth of Section		b	mm	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**
Height of Section		h	mm	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**
Effective height for tensile bar		d	mm	**850**	**891**	**891**	**916**	**916**	**916**
		d-out	mm	**884**	**891**	**891**	**916**	**916**	**916**
Design Load									
Bending Moment		M	kNm	**694.2**	**15.7**	**74.9**	**165.0**	**197.1**	**68.6**
Axial Force (positive=compression)		N	kN	**-209.1**	**-122.6**	**-46.4**	**-209.1**	**-122.6**	**-46.4**
Shear Force		S	kN	**313.8**	**313.8**	**182.2**	**313.8**	**182.2**	**92.3**
Allowable Stress - Reinforcement									
Material				**SD295**	**SD295**	**SD295**	**SD295**	**SD295**	**SD295**
Allowable stress		σsa	MPa	180	180	180	180	180	180
Allowable Stress - Concrete									
Standard Strength 28 days cylinder		σck	MPa	**30.0**	**30.0**	**30.0**	**30.0**	**30.0**	**30.0**
Allowable bending compressive stress		σca	MPa	11.00	11.00	11.00	11.00	11.00	11.00
Allowable shear stress without links		τa	MPa	1.00	1.00	1.00	1.00	1.00	1.00
Young's modulus ratio		n		7.14	7.14	7.14	7.14	7.14	7.14
Rebar Arrangement									
<Tensile bar>		1st	Bar-1	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**
			Bar-2	**5 -D32**			**5 -D19**	**5 -D19**	
		2nd	Bar-1	**5 -D19**					
			Bar-2	**5 -D19**					
		As	mm^2	8,274	1,418	1,418	2,835	2,835	1,418
	Cover	C	mm	100.00	100.00	100.00	75.00	75.00	75.00
	Spacing	Cs	mm	100.00	200.00	200.00	100.00	100.00	200.00
<Link>				**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**
		Area	mm^2	166	166	166	166	166	166
		Spacing	mm	800	800	800	800	800	800
Working Stress									
Increment Factor for Allowable Stress				**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
Concrete Compressive Stress	Working stress	σc	MPa	6.7	0.0	1.4	1.8	2.5	1.2
	Allowable stress	σca	MPa	11.0	11.0	11.0	11.0	11.0	11.0
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Steel Reinforcement	Working stress	σs	MPa	129.6	61.0	79.4	105.0	102.5	72.1
	Allowable stress	σsa	MPa	180.0	180.0	180.0	180.0	180.0	180.0
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Shear	Working stress	τ	MPa	0.41	0.35	0.21	0.36	0.21	0.10
	Allowable stress	τa	MPa	1.00	1.00	1.00	1.00	1.00	1.00
	Req'd area of rebar	Aw req	mm^2	0.00	0.00	0.00	0.00	0.00	0.00
	Rebar provided	Aw	mm^2	165.92	165.92	165.92	165.92	165.92	165.92
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Crack Width									
Flexural Crack	Calculated crack width	w	mm	0.364	0.264	0.317	0.265	0.260	0.240
	Allowable crack width	wa	mm	0.400	0.400	0.400	0.300	0.300	0.300
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

(Ordinary Case)

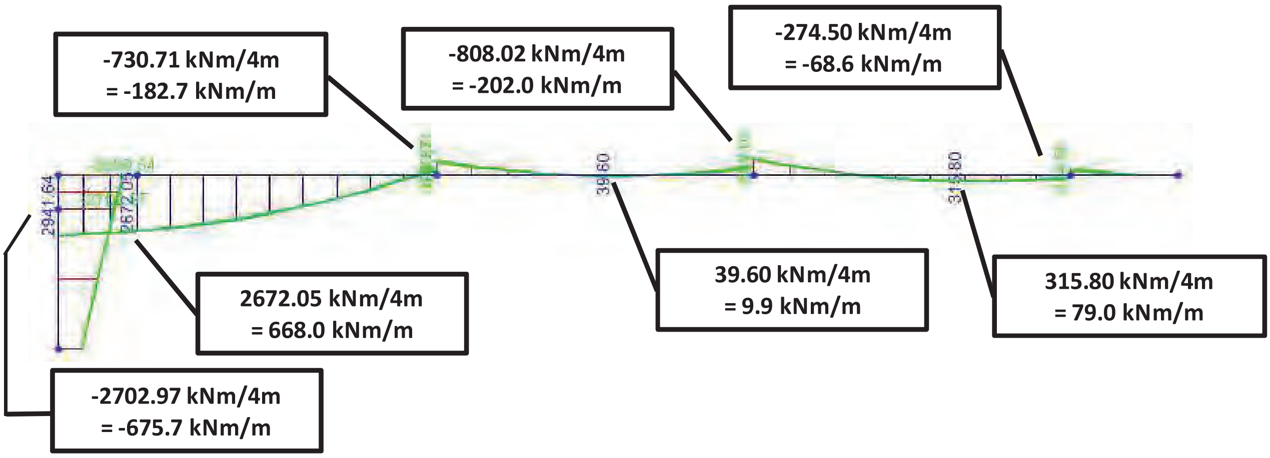
Calculation Sheet for Reinforced Concrete Section

		Sym.	Unit	Front Wall	Longitudinal Direction Bottom	Longitudinal Direction Top
Type of Member						
				Slab	**Slab**	**Slab**
Section Property						
Breadth of Section		b	mm	**1,000**	**1,000**	**1,000**
Height of Section		h	mm	**2,250**	**1,000**	**1,000**
Effective height for tensile bar		d	mm	**2,167**	**859**	**897**
		d-out	mm	**2,167**	**859**	**897**
Design Load						
Bending Moment		M	kNm	**677.9**	**55.1**	**98.8**
Axial Force (positive=compression)		N	kN	**198.4**		
Shear Force		S	kN	**198.8**	**133.5**	**133.5**
Allowable Stress - Reinforcement						
Material				**SD295**	**SD295**	**SD295**
Allowable stress		σsa	MPa	180	180	180
Allowable Stress - Concrete						
Standard Strength 28 days cylinder		σck	MPa	**30.0**	**30.0**	**30.0**
Allowable bending compressive stress		σca	MPa	11.00	11.00	11.00
Allowable shear stress without links		τa	MPa	1.00	1.00	1.00
Young's modulus ratio		n		7.14	7.14	7.14
Rebar Arrangement						
<Tensile bar>		1st	Bar-1	**5 -D16**	**5 -D19**	**5 -D19**
			Bar-2	**5 -D16**		
		2nd	Bar-1			
			Bar-2			
		As	mm^2	2,011	1,418	1,418
Cover		C	mm	75.00	132.00	94.00
Spacing		Cs	mm	100.00	200.00	200.00
<Link>				**1.25 -D13**	**1.25 -D13**	**1.25 -D13**
		Area	mm^2	166	166	166
		Spacing	mm	800	800	800
Working Stress						
Increment Factor for Allowable Stress				**1.00**	**1.00**	**1.00**
Concrete Compressive Stress	Working stress	σc	MPa	2.7	1.1	1.8
	Allowable stress	σca	MPa	11.0	11.0	11.0
	Check !!			**OK**	**OK**	**OK**
Steel Reinforcement	Working stress	σs	MPa	114.7	47.5	81.5
	Allowable stress	σsa	MPa	180.0	180.0	180.0
	Check !!			**OK**	**OK**	**OK**
Shear	Working stress	τ	MPa	0.10	0.16	0.16
	Allowable stress	τa	MPa	1.00	1.00	1.00
	Req'd area of rebar	Aw req	mm^2	0.00	0.00	0.00
	Rebar provided	Aw	mm^2	165.92	165.92	165.92
	Check !!			**OK**	**OK**	**OK**
Crack Width						
Flexural Crack	Calculated crack width	w	mm	0.286	0.279	0.308
	Allowable crack width	wa	mm	0.300	0.400	0.376
	Check !!			**OK**	**OK**	**OK**

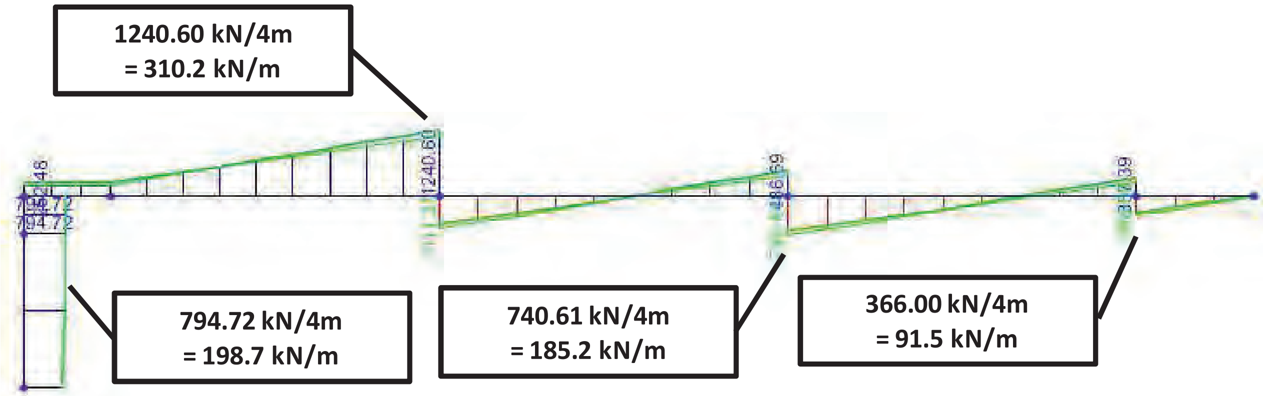
(Mooring Case)

Transversal Direction

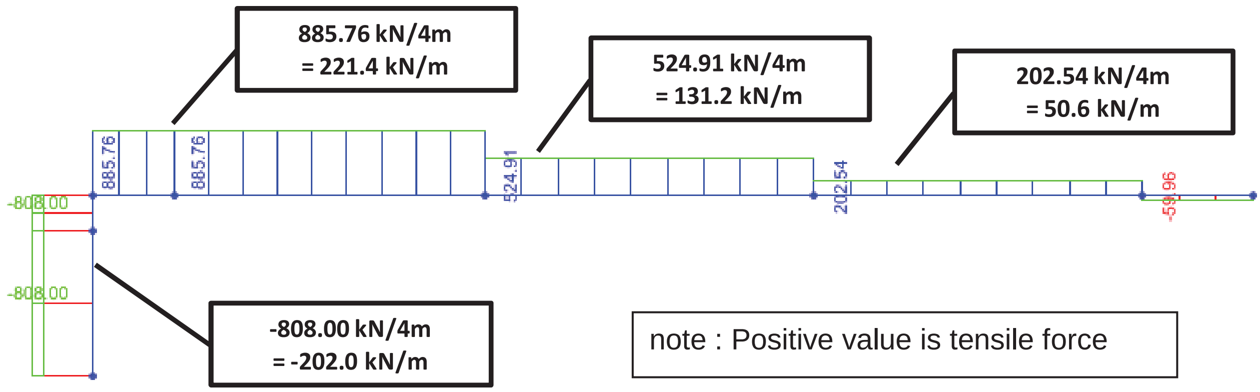
- Bending Moment (Envelope of Case 3 & 4)



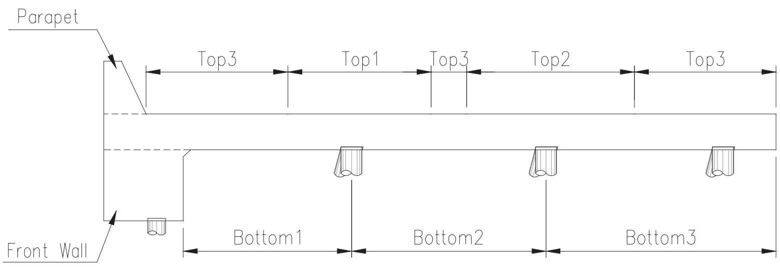
- Shear Force (Envelope of Case 3 & 4)



- Axial Force (Envelope of Case 3 & 4)



(Mooring Case)



Calculation Sheet for Reinforced Concrete Section

		Sym.	Unit	Transversal Direction					
				Bottom 1	**Bottom 2**	**Bottom 3**	**Top 1**	**Top 2**	**Top 3**
Type of Member									
				Slab	**Slab**	**Slab**	**Slab**	**Slab**	**Slab**
Section Property									
Breadth of Section		b	mm	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**
Height of Section		h	mm	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**
Effective height for tensile bar		d	mm	**850**	**891**	**891**	**916**	**916**	**916**
		d-out	mm	**884**	**891**	**891**	**916**	**916**	**916**
Design Load									
Bending Moment		M	kNm	**668.0**	**9.9**	**79.0**	**182.7**	**202.0**	**68.6**
Axial Force (positive=compression)		N	kN	**-221.4**	**-131.2**	**-50.6**	**-221.4**	**-131.2**	**-50.6**
Shear Force		S	kN	**310.2**	**310.2**	**185.2**	**310.2**	**185.2**	**91.5**
Allowable Stress - Reinforcement									
Material				**SD295**	**SD295**	**SD295**	**SD295**	**SD295**	**SD295**
Allowable stress		σsa	MPa	180	180	180	180	180	180
Allowable Stress - Concrete									
Standard Strength 28 days cylinder		σck	MPa	**30.0**	**30.0**	**30.0**	**30.0**	**30.0**	**30.0**
Allowable bending compressive stress		σca	MPa	11.00	11.00	11.00	11.00	11.00	11.00
Allowable shear stress without links		τa	MPa	1.00	1.00	1.00	1.00	1.00	1.00
Young's modulus ratio		n		7.14	7.14	7.14	7.14	7.14	7.14
Rebar Arrangement									
<Tensile bar>		1st	Bar-1	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**	**5 -D19**
			Bar-2	**5 -D32**			**5 -D19**	**5 -D19**	
		2nd	Bar-1	**5 -D19**					
			Bar-2	**5 -D19**					
		As	mm^2	8,274	1,418	1,418	2,835	2,835	1,418
Cover		C	mm	100.00	100.00	100.00	75.00	75.00	75.00
Spacing		Cs	mm	100.00	200.00	200.00	100.00	100.00	200.00
<Link>				**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**	**1.25 -D13**
		Area	mm^2	166	166	166	166	166	166
		Spacing	mm	800	800	800	800	800	800
Working Stress									
Increment Factor for Allowable Stress				**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
Concrete Compressive Stress	Working stress	σc	MPa	6.5	0.0	1.4	2.0	2.5	1.2
	Allowable stress	σca	MPa	11.0	11.0	11.0	11.0	11.0	11.0
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Steel Reinforcement	Working stress	σs	MPa	125.9	59.8	84.4	114.4	106.0	73.6
	Allowable stress	σsa	MPa	180.0	180.0	180.0	180.0	180.0	180.0
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Shear	Working stress	τ	MPa	0.40	0.35	0.22	0.35	0.21	0.10
	Allowable stress	τa	MPa	1.00	1.00	1.00	1.00	1.00	1.00
	Req'd area of rebar	Aw req	mm^2	0.00	0.00	0.00	0.00	0.00	0.00
	Rebar provided	Aw	mm^2	165.92	165.92	165.92	165.92	165.92	165.92
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**
Crack Width									
Flexural Crack	Calculated crack width	w	mm	0.356	0.260	0.331	0.283	0.267	0.243
	Allowable crack width	wa	mm	0.400	0.400	0.400	0.300	0.300	0.300
	Check !!			**OK**	**OK**	**OK**	**OK**	**OK**	**OK**

(Mooring Case)

Calculation Sheet for Reinforced Concrete Section

		Sym.	Unit	Parapet	Front Wall
Type of Member					
				Slab	**Slab**
Section Property					
Breadth of Section		b	mm	**1,000**	**1,000**
Height of Section		h	mm	**1,200**	**2,250**
Effective height for tensile bar		d	mm	**1,117**	**2,167**
		d-out	mm	**1,117**	**2,167**
Design Load					
Bending Moment		M	kNm	**28.1**	**675.7**
Axial Force (positive=compression)		N	kN		**202.0**
Shear Force		S	kN	**22.8**	**198.7**
Allowable Stress - Reinforcement					
Material				**SD295**	**SD295**
Allowable stress		σsa	MPa	180	180
Allowable Stress - Concrete					
Standard Strength 28 days cylinder		σck	MPa	**30.0**	**30.0**
Allowable bending compressive stress		σca	MPa	11.00	11.00
Allowable shear stress without links		τa	MPa	1.00	1.00
Young's modulus ratio		n		7.14	7.14
Rebar Arrangement					
<Tensile bar>		1st	Bar-1	**5 -D16**	**5 -D16**
			Bar-2		**5 -D16**
		2nd	Bar-1		
			Bar-2		
		As	mm^2	1,005	2,011
Cover		C	mm	75.00	75.00
Spacing		Cs	mm	200.00	100.00
<Link>				**1.25 -D13**	**1.25 -D13**
		Area	mm^2	166	166
		Spacing	mm	800	800
Working Stress					
Increment Factor for Allowable Stress				**1.00**	**1.00**
Concrete Compressive Stress	Working stress	σc	MPa	0.4	2.7
	Allowable stress	σca	MPa	11.0	11.0
	Check !!			**OK**	**OK**
Steel Reinforcement	Working stress	σs	MPa	26.0	113.3
	Allowable stress	σsa	MPa	180.0	180.0
	Check !!			**OK**	**OK**
Shear	Working stress	τ	MPa	0.02	0.10
	Allowable stress	τa	MPa	1.00	1.00
	Req'd area of rebar	Aw req	mm^2	0.00	0.00
	Rebar provided	Aw	mm^2	165.92	165.92
	Check !!			**OK**	**OK**
Crack Width					
Flexural Crack	Calculated crack width	w	mm	0.132	0.283
	Allowable crack width	wa	mm	0.300	0.300
	Check !!			**OK**	**OK**

Appendix 10-3

Design Calculation for Relieving Platform (Corner Section)

Supplementary Check for Quay Wall with Relieving Platform (Corner Block)

1. Pile Arrangement

For the corner block, the length of the sheet pile (55.33m) is longer than the block length (44.95m) as shown in Figure 1.

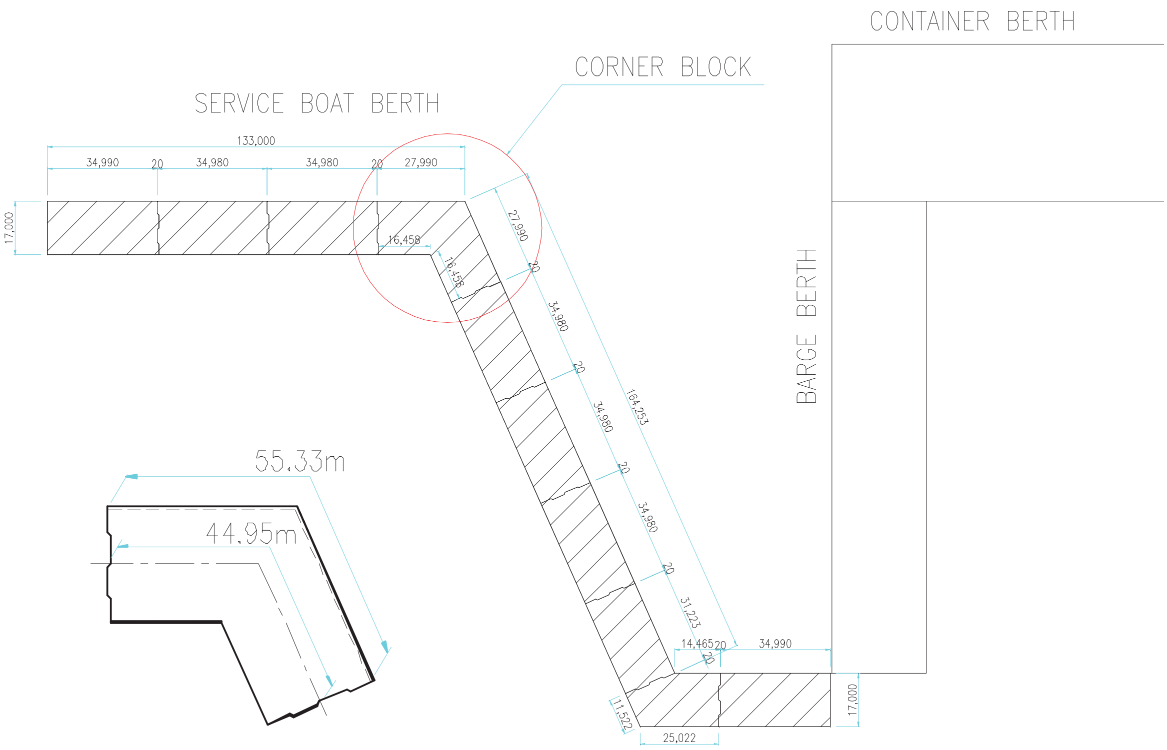


Figure 1 Corner Block

Consequently, the piles need to be provided much densely than the typical blocks as shown in Figure 2.

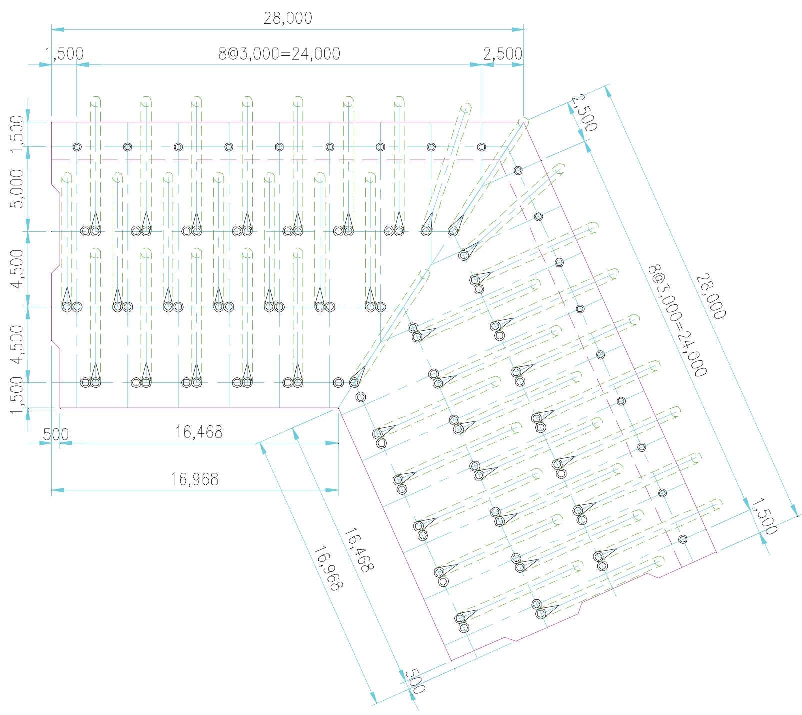


Figure 2 Pile Arrangement Plan

2. Structural Analysis for PHC Pile

(1) Structural Analysis Model
3-dimensional analysis model was built up by SAP2000 as shown in Figure 3.

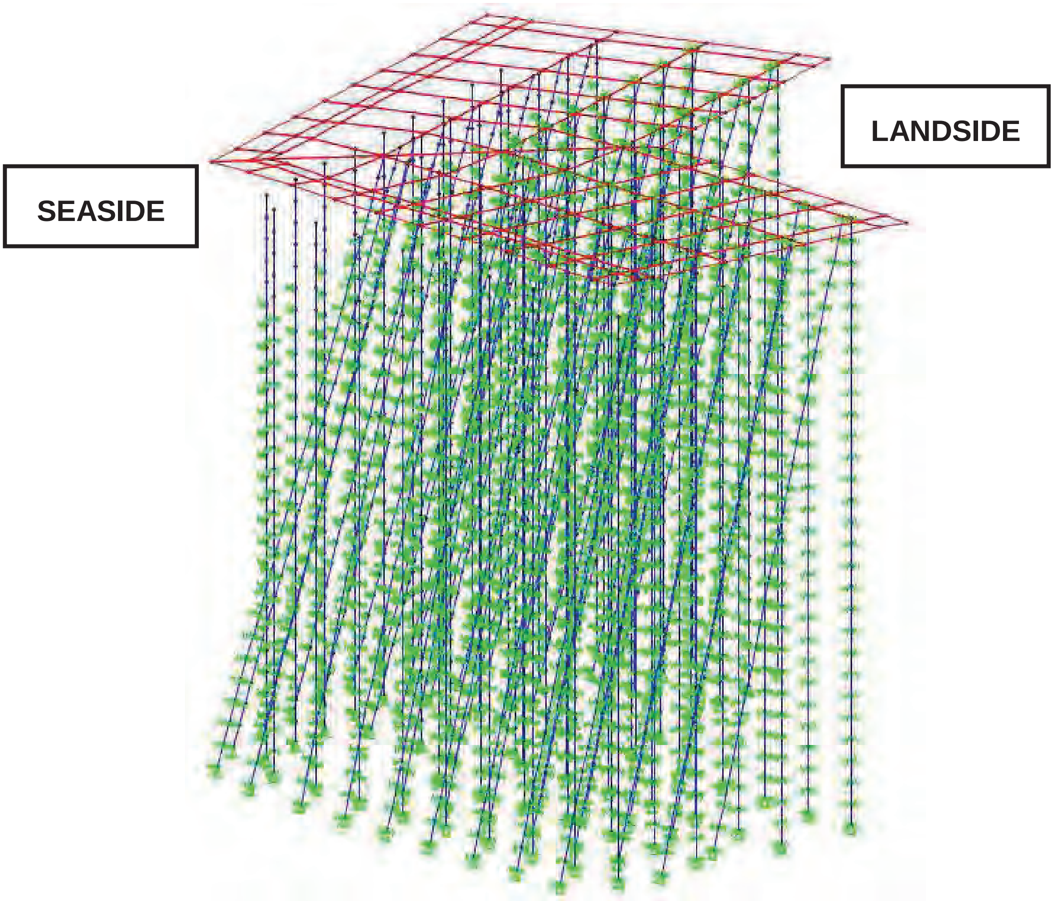


Figure 3 Analysis Model

(2) Check of Prestressed Concrete Spun Pile
The prestressed concrete spun piles were checked by working stress design.
The results of maximum axial forces and bending moments are shown below.

		Axial Force		Bending Moment
		Max. (kN)	Min. (kN)	(kNm)
Ordinary Case	PHC ϕ500	607	438	54
	PHC ϕ600	972	91	126
Mooring Case	PHC ϕ500	616	447	55
	PHC ϕ600	989	76	138

All combinations of bending moment and axial force under working load condition were plotted on the interaction curve diagrams in the following pages.
The plotted results show that the pile sections are adequate.

(Ordinary Case – PHC ϕ500)

Pile Check by Working Stress Design (PHC ϕ500)

Allowable Strength

	Long Term MPa	Short Trem MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	500	mm	OD	500	mm
Thickness	80	mm	ID	340	mm
Outer Length of PC Steel	420	mm	PC OD	420.83	mm
Diameter of PC	10.0	mm	PC ID	419.17	mm
Sectional Area of PC Steel	78.5	mm2/nos	A all	105,558	mm2
Number of PC Steel	14	nos	A conc	104,458	mm2
Total Sectional area of PC Steel	1,100	mm2	A p	1,100	mm2
Effective Prestress	8	MPa	n	5.405	
			I conc	2.388.E+09	mm4

Concrete Data

			I p	2.425.E+07	mm4
Design Strength	85	MPa	I comp	2.519.E+09	mm4
Modulus of Elasticity	37,000	MPa	A comp	110,402	mm2
			Z comp	10,075,199	mm3

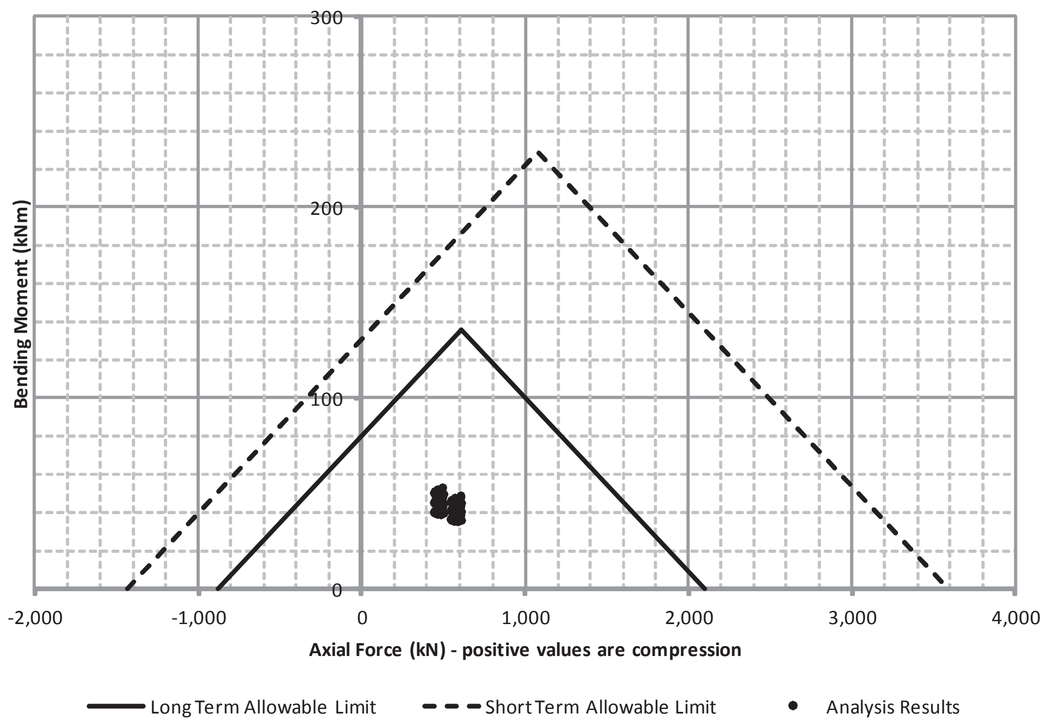
PC Steel Data

Tensile strength of PC	1,420	MPa
Yield strength of PC	1,275	MPa
Modulus of Elasticity	200,000	MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-883	0	-1,435	0
607	136	1,076	229
2,098	0	3,588	0

Interaction Curve for Prestressed Concrete Spun Pile



(Ordinary Case – PHC ϕ600)

Pile Check by Working Stress Design (PHC ϕ600)

Allowable Strength

	Long Term MPa	Short Term MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	600	mm	OD	600	mm
Thickness	90	mm	ID	420	mm
Outer Length of PC Steel	510	mm	PC OD	510.98	mm
Diameter of PC	10.0	mm	PC ID	509.02	mm
Sectional Area of PC Steel	78.5	mm2/nos	A all	144,199	mm2
Number of PC Steel	20	nos	A conc	142,628	mm2
Total Sectional area of PC Steel	1,571	mm2	A p	1,571	mm2
Effective Prestress	8	MPa	n	5.405	
			I conc	4.783.E+09	mm4

Concrete Data

			I p	5.107.E+07	mm4
Design Strength	85	MPa	I comp	5.059.E+09	mm4
Modulus of Elasticity	37,000	MPa	A comp	151,119	mm2
			Z comp	16,864,207	mm3

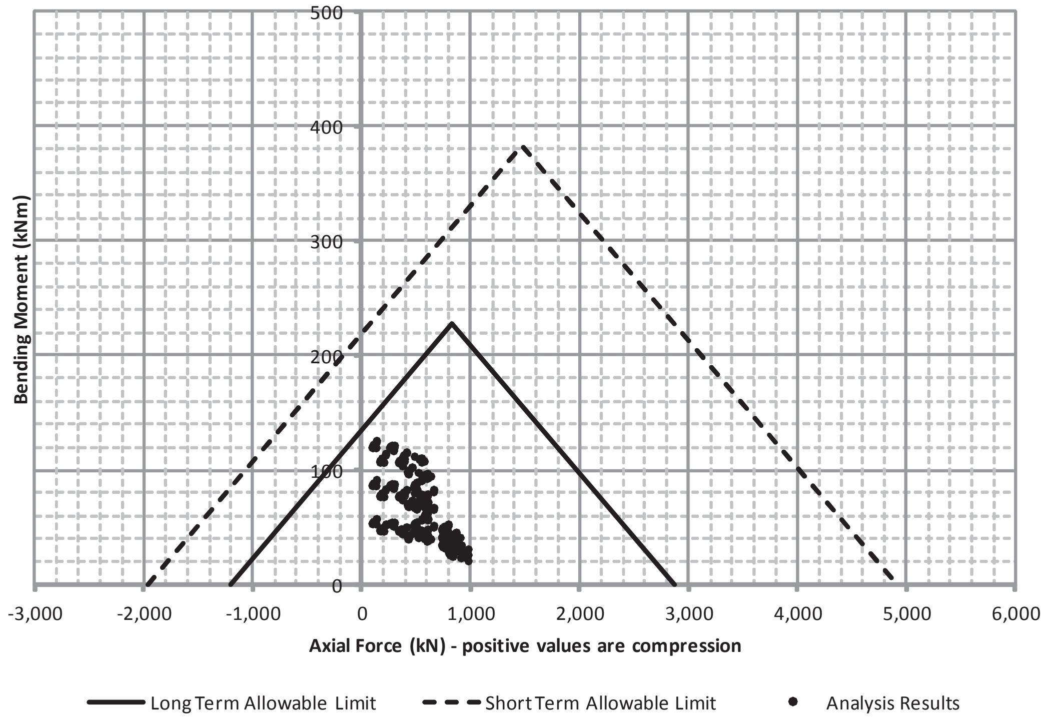
PC Steel Data

Tensile strength of PC	1,420	MPa
Yield strength of PC	1,275	MPa
Modulus of Elasticity	200,000	MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-1,209	0	-1,965	0
831	228	1,473	384
2,871	0	4,911	0

Interaction Curve for Prestressed Concrete Spun Pile



(Mooring Case – PHC ϕ500)

Pile Check by Working Stress Design (PHC ϕ500)

Allowable Strength

	Long Term MPa	Short Trem MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	500 mm	OD	500 mm
Thickness	80 mm	ID	340 mm
Outer Length of PC Steel	420 mm	PC OD	420.83 mm
Diameter of PC	10.0 mm	PC ID	419.17 mm
Sectional Area of PC Steel	78.5 mm2/nos	A all	105,558 mm2
Number of PC Steel	14 nos	A conc	104,458 mm2
Total Sectional area of PC Steel	1,100 mm2	A p	1,100 mm2
Effective Prestress	8 MPa	n	5.405
		I conc	2.388.E+09 mm4
Concrete Data		I p	2.425.E+07 mm4
Design Strength	85 MPa	I comp	2.519.E+09 mm4
Modulus of Elasticity	37,000 MPa	A comp	110,402 mm2
		Z comp	10,075,199 mm3

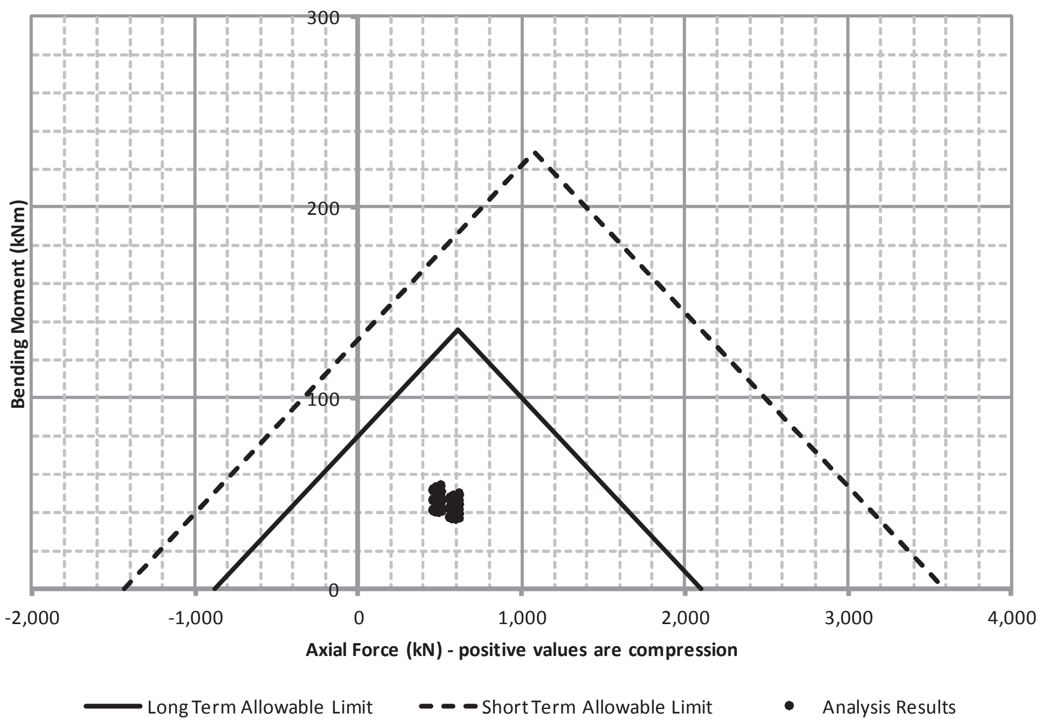
PC Steel Data

Tensile strength of PC	1,420 MPa
Yield strength of PC	1,275 MPa
Modulus of Elasticity	200,000 MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-883	0	-1,435	0
607	136	1,076	229
2,098	0	3,588	0

Interaction Curve for Prestressed Concrete Spun Pile



(Mooring Case – PHC ϕ600)

Pile Check by Working Stress Design (PHC ϕ600)

Allowable Strength

	Long Term MPa	Short Term MPa
Comp.	27	40.5
Tens.	0	5

Pile Data

Diameter	600	mm	OD	600	mm
Thickness	90	mm	ID	420	mm
Outer Length of PC Steel	510	mm	PC OD	510.98	mm
Diameter of PC	10.0	mm	PC ID	509.02	mm
Sectional Area of PC Steel	78.5	mm2/nos	A all	144,199	mm2
Number of PC Steel	20	nos	A conc	142,628	mm2
Total Sectional area of PC Steel	1,571	mm2	A p	1,571	mm2
Effective Prestress	8	MPa	n	5.405	
			I conc	4.783.E+09	mm4

Concrete Data

			I p	5.107.E+07	mm4
Design Strength	85	MPa	I comp	5.059.E+09	mm4
Modulus of Elasticity	37,000	MPa	A comp	151,119	mm2
			Z comp	16,864,207	mm3

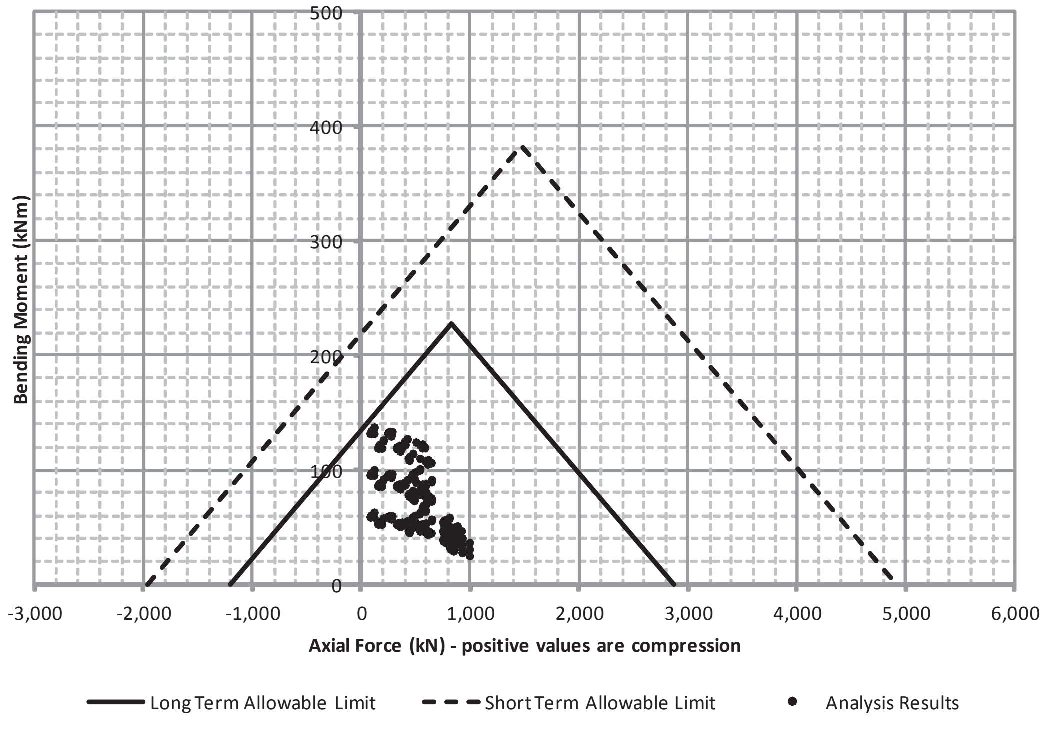
PC Steel Data

Tensile strength of PC	1,420	MPa
Yield strength of PC	1,275	MPa
Modulus of Elasticity	200,000	MPa

Interaction Curve

Long Term		Short Term	
P	M	P'	M'
-1,209	0	-1,965	0
831	228	1,473	384
2,871	0	4,911	0

Interaction Curve for Prestressed Concrete Spun Pile



(3) Check of Pile Head Reinforcement

The pile head connection was checked for the reinforced concrete section having a diameter 150mm (embedment to deck of 75mm on one side) larger than PHC pile diameter.

Forces acting on the pile head are shown below.

		Axial Force		Bending Moment
		Max. (kN)	Min. (kN)	(kNm)
Ordinary Case	PHC ϕ500	607	438	54
	PHC ϕ600	972	91	126
Mooring Case	PHC ϕ500	616	447	55
	PHC ϕ600	989	76	138

The steel reinforcements to be provided at the pile head are shown in Figure 4.

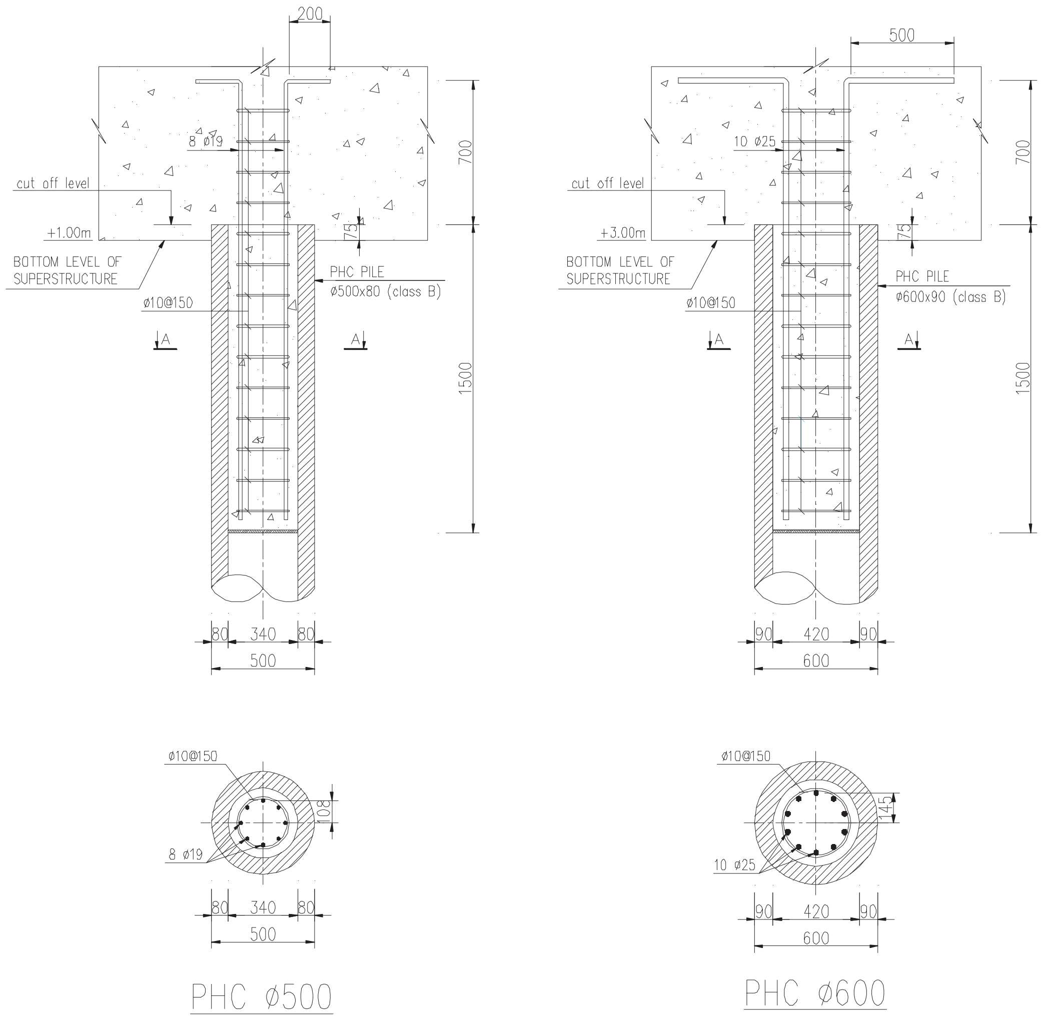


Figure 4 Pile Head Reinforcement

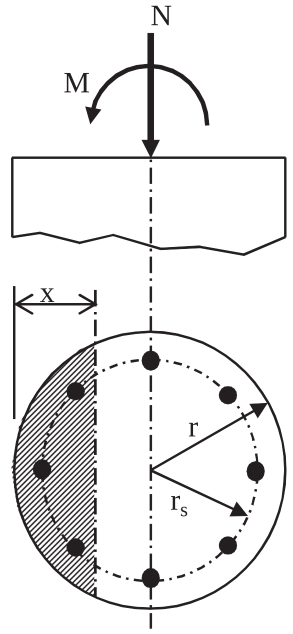
The calculation results are shown in the following pages which are all satisfactory.

(Ordinary Case – PHC ϕ500)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	54	kNm
Axial Force (positive = compression)	N =	438	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

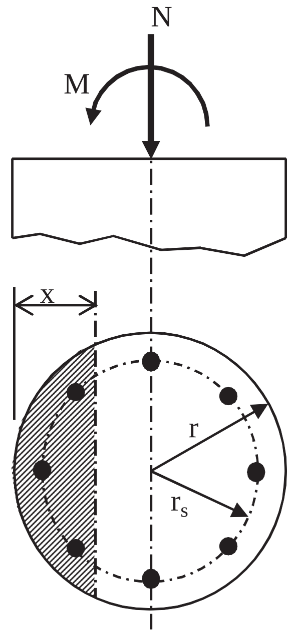
Neutral axis	x	501.48042	mm	
Extreme fiber strain (concrete)		0.0001218		
Maximum stress				
Concrete	fc =	3.41	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Ordinary Case – PHC ϕ500)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	54	kNm
Axial Force (positive = compression)	N =	607	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	610.07339	mm	
Extreme fiber strain (concrete)		0.0001341		
Maximum stress				
Concrete	fc =	3.76	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Ordinary Case – PHC ϕ600)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input

1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	145	mm

1.3 Design forces

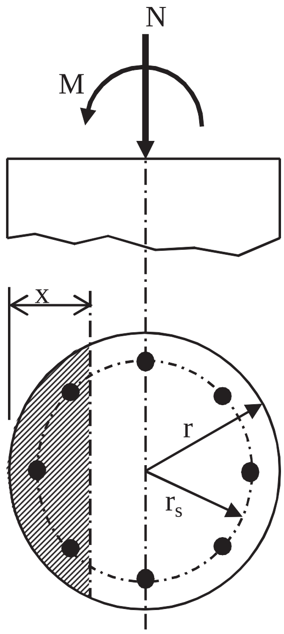
Bending Moment	M =	126	kNm
Axial Force (positive = compression)	N =	91	kN

1.4 Rebar arrangement

Dia		D25	mm
Nos		10	nos
As	As =	4909	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa



2. Results of Calculation

Neutral axis	x	202.39633	mm
Extreme fiber strain (concrete)		0.000339	

Maximum stress

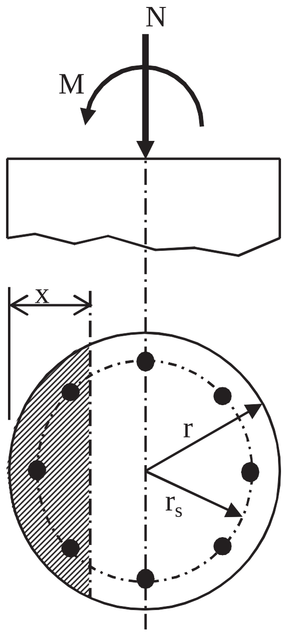
Concrete	fc =	9.49	MPa	**OK**
Rebar	fs =	104.01	MPa	**OK**

(Ordinary Case – PHC ϕ600)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	145	mm

1.3 Design forces

Bending Moment	M =	126	kNm
Axial Force (positive = compression)	N =	972	kN

1.4 Rebar arrangement

Dia		D25	mm
Nos		10	nos
As	As =	4909	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

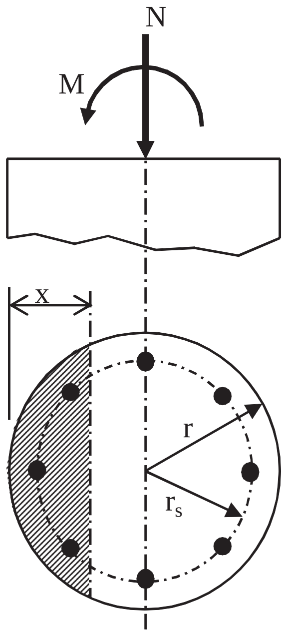
Neutral axis	x	608.37904	mm	
Extreme fiber strain (concrete)		0.0001863		
Maximum stress				
Concrete	fc =	5.22	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC ϕ500)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	55	kNm
Axial Force (positive = compression)	N =	447	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

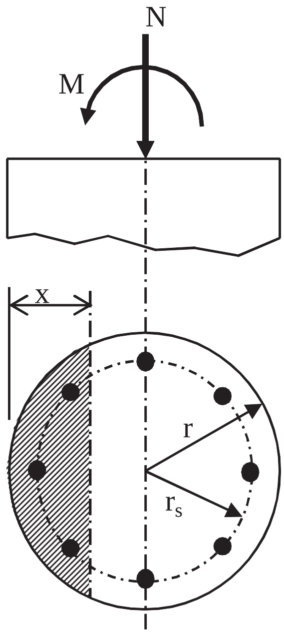
Neutral axis	x	500.82274	mm	
Extreme fiber strain (concrete)		0.0001245		
Maximum stress				
Concrete	fc =	3.48	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC ϕ500)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	325	mm
Radius of reinforcement position	rs =	108	mm

1.3 Design forces

Bending Moment	M =	55	kNm
Axial Force (positive = compression)	N =	616	kN

1.4 Rebar arrangement

Dia		D19	mm
Nos		8	nos
As	As =	2268	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

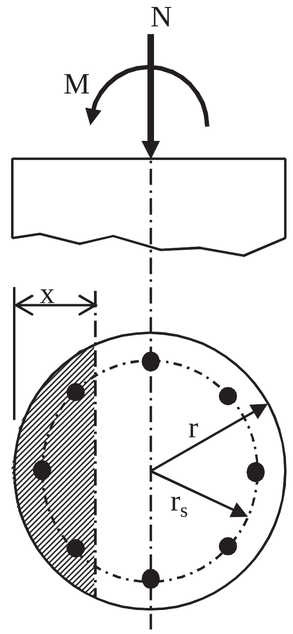
Neutral axis	x	609.59248	mm	
Extreme fiber strain (concrete)		0.0001362		
Maximum stress				
Concrete	fc =	3.81	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(Mooring Case – PHC ϕ600)

Pmin + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	145	mm

1.3 Design forces

Bending Moment	M =	138	kNm
Axial Force (positive = compression)	N =	76	kN

1.4 Rebar arrangement

Dia		D25	mm
Nos		10	nos
As	As =	4909	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

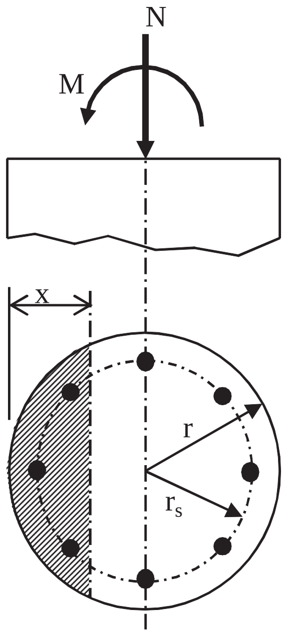
Neutral axis	x	198.4323	mm	
Extreme fiber strain (concrete)		0.0003777		
Maximum stress				
Concrete	fc =	10.58	MPa	**OK**
Rebar	fs =	119.73	MPa	**OK**

(Mooring Case – PHC ϕ600)

Pmax + Mmax

Results of Stress Check for RC Pile Reinforcement

1. Design Input



1.1 Allowable stress

Compressive stress in concrete	σca =	11.00	MPa
Tensile stres in rebar	σsa =	180.00	MPa

1.2 Section dimensions

Radius of concrete section	r =	375	mm
Radius of reinforcement position	rs =	145	mm

1.3 Design forces

Bending Moment	M =	138	kNm
Axial Force (positive = compression)	N =	989	kN

1.4 Rebar arrangement

Dia		D25	mm
Nos		10	nos
As	As =	4909	mm2

1.5 Young's mudulus

Concrete	Ec	28,000	MPa
Rebar	Es	200,000	MPa

2. Results of Calculation

Neutral axis	x	578.84562	mm
Extreme fiber strain (concrete)		0.0002016	

Maximum stress

Concrete	fc =	5.64	MPa	**OK**
Rebar	fs =	0.00	MPa	**OK**

(4) Check of Bearing Capacity

1) Check of ultimate bearing capacity

Based on the “Design Calculation for Quay Wall with Relieving Platform”, the allowable bearing capacities are calculated as shown below.

		Allowable Bearing Capacity	
		Comp. (kN)	Pulling. (kN)
PHC ϕ500	Ordinary/Mooring Case	1,307	432
PHC ϕ600	Ordinary/Mooring Case	1,775	531

The maximum compressive forces are less than the allowable bearing capacities as shown below, thus the results are all satisfactory.

		Allowable Bearing Capacity Ra		Max. Axial Force P		Pile Weight W	Check		Judge
		Comp. (kN)	Pulling (kN)	Comp (kN)	Pulling (kN)	(kN)	P+W Comp (kN)	P-W Pulling (kN)	< Ra ?
PHC ϕ500	Ordinary Case	1,307	432	607	-	39	646	-	OK
	Mooring Case	1,307	432	616	-		655	-	
PHC ϕ600	Ordinary Case	1,775	531	972	-	57	1,029	-	OK
	Mooring Case	1,775	531	989	-		1,046	-	

2) Check of the bearing capacity against negative skin friction

Based on the “Design Calculation for Quay Wall with Relieving Platform”, the bearing capacities against negative skin friction were checked as follows which are all satisfactory.

		End Bearing R_p (kN)	Negative Friction $R_{nf,max}$ (kN)	Allowable Bearing $R_a = R_p/1.2 - R_{nf,max}$ (kN)	Max. Axial Force P+W (kN)	Judge $P < R_a$?
PHC ϕ500	Ordinary Case	1,970	338	1,304	646	OK
	Mooring Case				655	
PHC ϕ600	Ordinary Case	2,844	443	1,927	1029	OK
	Mooring Case				1046	