#### 3.2 Structural design of revetment

#### Structural Calculation of PC Sheet Pile Revetment

Location: WF.37 L

1.DESIGN CONDITION

(1) Calculation Method

Loading Combination

: Normal and Earthquake

Position of Support

: Balancing Point of Active and Passive Soil Pressure

Length of Sheet Pile

: Balancing Point of Moment 1.0 (Normal)

Balancing Point of Moment 1.0 (Earthquake)

Earth Pressure of Cohesive Soil

 $\Sigma \gamma_h - 2C$ 

(Normal)

Port Formula

(Earthquake)

Seismic Coefficient

K,

= 0.12

K,

= 0.24 (Constant, Submarged)

Reduction of Seismic Forth under River-bed

: Non

(2) Soil Pressure

Background Surface

: Horizontally

Live Load

	<u>·                                      </u>				
	Passive	Active Down	Active Up		
Normal	$0.00 \text{ t/m}^2$	0.00 t/m <sup>2</sup>	1.00 t/m <sup>2</sup>		
Earthquake	0.00 t/m <sup>2</sup>	$0.00 \text{ t/m}^2$	0.50 t/m <sup>2</sup>		

Friction Angle to Wall

: Non

Reduction of Water Pressure

: Non (Trapezoid)

(3) Dimensions of Steel Sheet Pile

Туре

: PC Sheet Pile (t=220 mm)

Efficiency of Splice

: 0.800

Thickness of Corrosion

: Non

Allowable Stress

: Normal 160.0 kg/cm<sup>2</sup> , Earthquake 240 kg/cm<sup>2</sup>

Allowable Deformation at Pile Head

: Normal 5.0 cm, Earthquake 7.5 cm

(4) Soil Condition

(·),	JUM COMMI	1011							
	Elevation	Thick-	Avg.	1	Unit Weigh	t ·	Internal Friction	Cohesion	Horizontal K
Na	m	ness m	N- Value	Wet y t/m³	Sat. y t/m³	Sub. y t/m³	Angle  (°)	Со	Value K
1	0.650 -2.800	3.450	15.0	1.80	2.00	1.00	30.0	0.000	2.07
2	-2.800 -3.700	0.900	2.0	1.40	1.40	0.50	0,0	2.500	0.92
3	-3.700 -17,800	14.100	2.0	1.40	1.40	0.50	0.0	2.500	0.92
4	-17.800 -999.999	982.199	24.0	1.80	2.00	1.00	34.0	0,000	2.51

[Normal Case]

<u> 연설하다 하는 학교의 교육 회사회 설립 환경</u> 변경 및	0,650 m 22,7,7,7, 0,2,60 m√
∇ = 27000 f = 35000 der 1.750 m N = 27000 f = 35000 der 1.750 m N = 27000 f = 35000 der 1.750 m N = 27000 f = 35000 der 1.750 m	N= 15.000 f= 30.000 deg = 1800 t/m3 rw= 1.000 t/m3 Co= 0.000 Cz= 0.000 t/m3 Co= 0.000 cz= 0.000 t/m3 co= 2.000 cz= 0.000 t/m3
	OS 2500 Cz 0.000  N 2000 f 0000 deg  '" 1400 t/m3 rm 0500 t/m3  OS 2500 Cz 0.000
	-17800 m
	N= 24000 r= 1800 / m3

#### [Seismic Case]

######################################	0,650 m	—————————————————————————————————————
∇-0.230 m  N= 27000 f= 35.000 dec	N= 15.000 r= 1800 t/m3 0= 0.000 -72800 m N= 2800 -3,700 m r= 1400 t/m3	f= 30.000 der rw= 1.000 t/m3 Oz= 0.000 -f= 0.000-der
	03° 2500 N° 2000 r≈ 1400 t/m3 Co= 2500	f= 0.000 der rw= 0.000 t/m3 Cz= 0.000
	-17800 m	C2= 0,000
	N≃ 24.000 r= 1.800 t/m3 co≃ 0.000	f= 34.000 deg rw= 1.000 t/m3 Oz= 0.000

#### 2. Load Calculation

#### (1) Formula of Soil Pressure

#### 1) Sandy Soil: Active Soil Pressure

$$P_a = K_a \cdot \left( \sum \gamma h + \frac{q_a}{\cos \beta_a} \right) - 2 \cdot C \cdot \sqrt{K_a}$$

Where:

P<sub>a</sub>: Strength of Active Soil Pressure  $(t/m^2)$ : Coefficient of Active Soil Pressure : Unit Weight of Soil  $(t/m^3)$ : Thickness of Layer h (m) : Active Load  $(t/m^2)$  $q_a$ (°)  $\beta$ : Angle between G-Surface and Level Surface C  $(t/m^2)$ : Cohesive  $\cos(\phi - \theta) \cdot \cos \sigma$ 

$$K_{a} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{a} - \theta)}{\cos(\sigma + \theta) \cdot \cos \beta_{a}}}\right)}$$

Where:

φ : Internal Friction Angle (°)

 $\theta$ : Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_h)$ ,  $K_h$ : Horizontal Seismic Degree

## δ: Friction Angle between Soil and Wall (°)

#### 2) Sandy Soil: Passive Soil Pressure

$$P_{p} = K_{p} \cdot \left( \sum \gamma h + \frac{q_{p}}{\cos \beta_{p}} \right) + 2 \cdot C \cdot \sqrt{K_{p}}$$

Where:

$P_{\mathfrak{p}}$	: Strength of Passive Soil Pressure	(t/m²)
K <sub>n</sub>	: Coefficient of Passive Soil Pressure	
γ	: Unit Weight of Soil	$(t/m^3)$
h	: Thickness of Layer	(m)
$q_{\rho}$	: Active Load	$(t/m^2)$
Ĉ	: Cohesive	(t/m²)
$\beta_{p}$	: Angle between G-Surface and Level Surface	(°)

$$K_{p} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{p} - \theta)}{\cos(\sigma + \theta) \cdot \cos\beta_{p}}}\right)}$$

#### Where:

 $\phi$ : Internal Friction Angle (°)

: Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_h)$ ,

K<sub>h</sub>: Horizontal Seismic Degree

 $\delta$ : Friction Angle between Soil and Wall ( $^{\circ}$ )

## (2) Load Calculation (Normal Condition)

#### 1) Soil Pressure & Water Pressure

[Normal · Active]

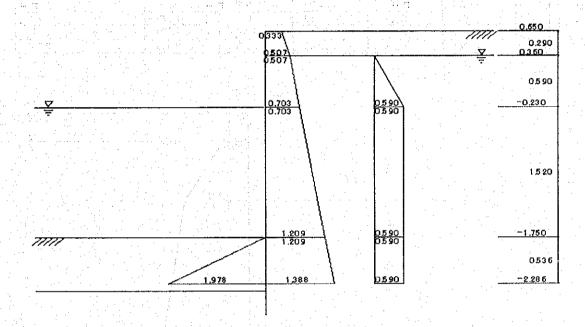
 $q = 0.00 \text{ t/m}^2$ 

Na	Elevation El.m	Thickness m	γ t/m³	ģ	c t/m³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P.
]	0.650			22.0	0.0	0.0	0.00	0.333	0.000	0.333	0.000
	0.360	0.290	1.80	30.0	0.0	0.0	0.00	0.333	0.522	0.507	0.000
2	0.360	0.000	4.00	20.0	0.0	0.0	0.00	0.333	0.522	0.507	0.000
	-0.230	0.590	1,00	30.0	0.0	0.0	0,00	0.333	1.112	0.703	0.590
3	-0.230			20.0	0.0	00	0.00	0.333	1.112	0.703	0.590
	-1.750	1.520	1.00	30.0	0.0	0.0	0.00	0.333	2.632	1.209	2,110
4	-1.750			10.0	0.0	20	0.00	0.333	2.632	1.209	2,110
]	-2.286	0.536	1.00	30.0	0.0	0.0	0.00	0.333	3.168	1.388	2,646

# [Normal · Passive]

 $q = 0.00 \text{ t/m}^2$ 

Na	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	8.	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
1	0.650	0.000		5.4							
	0.360	0.290									5 1
2	0.360	0.500			A			1111 1111			4 5
	-0.230	0.590					1 1 1				
3	-0.230	1.500			1						0.000
	-1.750	1.520			T					:	1.520
4	-1.750	0.525	1.00	25.0	0.0	Λ.0	0.00	3.690	0.000	0,000	1.520
	-2.286	0.536	1.00	35.0	0.0	0.0	0.00	3.690	0.536	1.978	2.056

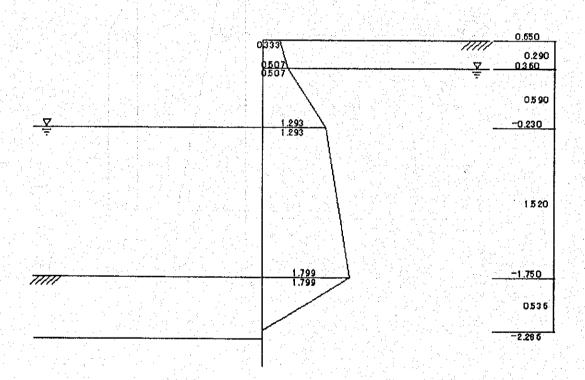


2) Acting Load

## [Normal]

Balancing Pont of Active Force and Passive Force: BL. -2.286 m (0.536 m from Riverbed and 2.936 m from top of Pile).

Nα	Loading Point El. m	Active SP Pa t/m²	Active WP Pwa t/m²	Passive SP Pp t /m²	Passive WP Pwp t/m²	Load Strength Pa+Pwa-Pp-Pwp t/m²
1	0.650	0,333	0.000	0.000	0.000	0.333
1	0.360	0.507	0,000	0.000	0.000	0,507
2	0.360	0.507	0.000	0.000	0,000	0.507
		0.703	0.590	0,000	0.000	1.293
.3		0.703	0.590	0.000	0.000	1.293
	-1.750	1.209	2.110	0.000	1.520	1,799
4	-1.750	1.209	2,110	0.000	1.520	1.799
	-2.286	1.388	2.646	1.978	2.056	0.000



## 3) Loading Point

#### [Normal]

Na	Hi t/m	Yi m	Mi ≕Hi∙yi	A ≕yi/H	B =(3-A)·	Q =B·Hi
			5.7		A <sup>2</sup> /6	
1	1/2×0.333×0.290=0.048	2.646+2/3×0.290=2.839	0.136	0.967	0.317	0.015
	0.290=0.074	2.646+1/3×0.290=2.743	0.203	0.934	0.301	0.022
2	1/2×0.507×0.590=0.150	2.056+2/3×0.590=2,449	0.367	0.834	0.251	0.038
	0.590=0.381	2.056+1/3×0.590=2.253	0,858	0.767	0.219	0.083
3	1/2×1.293×1.520=0.983	0.536+2/3×1,520=1,549	1.523	0.528	0.115	0.113
4171	1.520=1.367	0.536+1/3×1.520=1.043	1.426	0.355	0.056	0.076
4	::1/2×1.799×0.536=0.482	.::-2/3×0.536=0.357	0.172	0,122	0.007	0.003
1. 1.1.1	0.536=0.000	1/3×0.536=0.179	0.000	0.061	0.002	0.000
Total	3.485		4.685		3,002	0.351

Acting Point ho =  $\Sigma \text{Mi} / \Sigma \text{hi} = 4.686/3.485 = 1.344 \text{ m}$ 

## 3. Stress Calculation of PC Sheet Pile

Normal

## 1) Loading Condition (par m)

Derived from 2)Load to be act

Horizontal P <sub>0</sub>	Acting Point h <sub>0</sub>
3.485 t f/m	1.344 m

k Value of Grand	Surface K
1.302	kgf/cm <sup>3</sup>

Coefficient of Reaction (k): Average of "ki" below  $1/\beta$  from the Fulcrum.

$$1/\beta_{i} \leq L_{i}: \qquad K = K_{i}$$

$$1/\beta_{i} > L_{i}:$$

$$K = \frac{K_1 \times L_1 + K_2 \times L_2}{L_1 + L_2}$$

where:

K value at the Fulcrum

K<sub>2</sub>: k value below the Fulcrum

L<sub>1</sub>: Thickness of layer from Fulcrum to boundary of next Layer.

$$L_2 = \frac{1 - \beta_1 \times L_1}{\beta_2}$$

 $\beta_1$ : Specific characteristic of Sheet Pile

 $(\beta)$  calculated by  $K_1$ 

 $\beta_2$ : Specific characteristic of Sheet Pile ( $\beta$ ) calculated by  $K_2$ 

#### 2) Condition of PC Sheet Pile

1	Moment of Area I	Section Module Z	Elastic Module E			
ĺ	88700.0 cm <sup>4</sup> /m	8000.0 cm <sup>3</sup> /m	330000 kg/cm <sup>2</sup>			

Efficiency of Splice	Reduction Ratio of
(a)	Corrosion (λ)
0.80	1.00

Reduction Ratio of Corrosion ( $\lambda$ ) is derived from the graph of "Thickness of Corrosion/Section Module".

Allowable Stress	Allowable Deformation				
σα	$\delta a$				
160.0 kg/cm <sup>2</sup>	5.0 cm				

	Bending Stiffness EI	β	
l	2342 t·m²/m	0.611 m <sup>-1</sup>	

Bending S	Stiffness EI'	β		-
292	7 t·m²/m	0.577	m <sup>-1</sup>	3.5

#### ① Deformation/Max. Moment

 $EI = E \times I \times \alpha \times \lambda$ 

- $=330000\times88700.0\times0.80\times1.00$
- $= 23420000000 \text{ kg/cm}^2/\text{m}$
- $= 2342 t \cdot m^2/m$

$$\beta = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI}} = \sqrt[4]{\frac{1000 \times 1.302}{4 \times 2342}} = 0.611 \, \text{m}^{-1}$$

#### 3) Stress Calculation of Sheet Pile

E	'≐E×I	
	= 330000×88700.0	
	$= 29270000000 \text{ kg/cm}^2/$	n
	$= 2927 \text{ t} \cdot \text{m}^2/\text{m}$	ě,

② Embedding of Sheet Pile

$$\beta' = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI'}}$$
$$= \sqrt[4]{\frac{1000 \times 1.302}{4 \times 2927}}$$
$$= 0.577 \text{ m}^{-1}$$

Max. Bending Moment Mmax		Allowable cracking moment
5.611 t·m/m	≦ :	13.20 tf-m/m ← OK

$$Mmax = M_t + M \times \phi = M_t + (P_0 \times h) + \phi$$

where

M<sub>t</sub>: Moment at Top of Pile h: Wall height = 2.936 m

$$\phi = \sqrt{\frac{(1+2\cdot\beta\cdot h)^2 + 1}{2\cdot\beta\cdot h}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2\cdot\beta\cdot h}\right)$$

$$= \sqrt{\frac{(1+2\times0.611\times1.344)^2 + 1}{2\times0.611\times1.344}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2\times0.611\times1.344}\right)$$

$$= 1.198$$

Mmax = 
$$M_t + M_0 \times \phi$$
  
=  $M_t + (P \times h) \times \phi$   
= 0.000 + 3.485 × 1.344 × 1.198  
= 5.611 t·m/m

#### 4) Deformation at Top of Sheet Pile

Deformation at Top of Sheet Pile $\Sigma \delta$	
2.52 cm	$\leq \delta a = 5.00 \text{cm} \leftarrow \text{OK}$

$$\Sigma \delta = \delta_1 + \delta_2 + \delta_3$$

Where:

 $\delta_1$ : Deformation at Fulcrum.

 $\delta_2$ : Deformation by Deflection Angle Sheet Pile at Fulcrum.

 $\delta_3$ : Deformation of Sheet Pile as Cantilever above Fulcrum.

$$\delta_{1} = \left(\frac{1+\beta \cdot h_{0}}{2 \cdot EI \cdot \beta^{3}} \times P + \frac{Mt}{2 \cdot EI \cdot \beta^{2}}\right) \times 100$$

$$= \left(\frac{1+0.611 \times 1.344}{2 \times 2342 \times 0.611^{3}} \times 3.485 + \frac{0.000}{2 \times 2342 \times 0.611^{2}}\right) \times 100$$

$$= 0.59 \text{ cm}$$

$$\delta_{2} = \left(\frac{1+2 \cdot \beta \cdot h_{0}}{2 \cdot EI \cdot \beta^{2}} \times P + \frac{M_{t}}{2 \cdot EI \cdot \beta}\right) \times H \times 100$$

$$= \left(\frac{1+2 \times 0.704 \times 0.703}{2 \times 2399 \times 0.704^{2}} \times 1.677 + \frac{0.000}{2 \times 2399 \times 0.704}\right) \times 1.805 \times 100$$

$$= 1.55 \text{ cm}$$

$$\delta_{3} = \left(\frac{Q \cdot H^{3}}{EI} + \frac{Mt \cdot H^{2}}{2 \cdot EI}\right) \times 100$$

$$= \left(\frac{0.351 \times 2.9363}{2342} + \frac{0.000 \times 2.936}{2 \times 2342}\right) \times 100$$
  
= 0.38 cm

$$\Sigma \delta = 0.59 + 1.55 + 0.38$$
  
= 2.52 cm

#### 5) Stress Calculation for All Length of Sheet Pile

Tota	Lengt	th of Pile	L
		8.500	m

Driving Depth of Sheet Pile (Dz):

Dz = 
$$\pi / \beta'$$
  
=  $\pi / 0.577$   
= 5.445 m

Required Total Length of Sheet pile (L):

$$L = H + Dz$$
= 2.936 + 5.445
= 8.381 m < 8.500 m

## (3) Load Calculation (Seismic Case)

#### 1) Soil Pressure & Water Pressure

[Earthquake · Active]

 $q = 0.00 \text{ t/m}^2$ 

No	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	δ	Seismie Degree	Ka	Σγh • t/m³	Soil P. t/m³	Water P.
1	0.650	0.000	1 00	20.0	0.0	0.0	0.10	0.397	0.000	0.199	0.000
	-0.230	0.880	1.80	30.0	0.0	0.0	0.10	0.397	1.584	0.827	0.000
- 2	-0.230	500	1.00		0.0	0.0	0.20	0.473	1.584	0.986	.0000
	-1.750	1.520	1.00	30.0	0.0	0.0	0.20	0.473	3.104	1.705	1.520
3	-1.750	0.606	1 00	100 300	0.0	0.0	0.20	0,473	3.104	1.705	1.520
1.2	-2.356	0.606	1.00	30.0	0.0	0.0	0.20	0.473	3.710	1.991	2,126

## [Earthquake · Passive]

 $q = 0.00 \text{ t/m}^2$ 

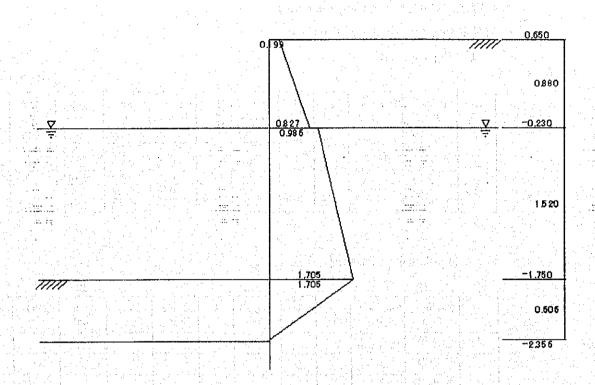
Na	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P.
1	0.650	0.880									11.00
	-0.230				1 1 1	100		- 1,2			
2	-0.230	1.520		10 July 10 Jul						4 7 2	0.000
	-1.750									41.44	1.520
3	-1.750	0.606	1.00	35.0	0,0	0,0	0.20	3.285	0,000	0.000	1.520
i fin	-2.356		, X	1.51	0.0		0.20	3.285	0.606	1.991	2.126
	100					100	3.5			1. H	-

## 2) Acting Load

## [Earthquake]

Balancing Pont of Active Force and Passive Force: BL. -2.356 m (0.606 m from Riverbed and 3.006 m from top of Pile).

No	Loading Point E1. m	Active SP Pa t /m²	Active WP Pwa t/m²	Passive SP Pp t /m²	Passive WP Pwp t/m²	Load Strength Pa+Pwa-Pp-Pwp t/m²
1	0.650	0.199	0.000	0.000	0.000	0.199
1.1	-0.230	0.827	0.000	0.000	0.000	0.827
2	-0.230	0.986	0.000	0.000	0.000	0.986
1.37	-1.750	1.705	1.520	0.000	1.520	1.705
3	-1.750	1.705	1.520	0.000	1.520	1.705
	-2.356	1.991	2.126	1.991	2.126	0,000



# 3) Loading Point

## [Earthquake]

Nα	Hi t/m	Yi m	Mi =Hi∙yi	A =yi/H	$B = (3-A)$ $A^2/6$	Q =B·Hi
1	1/2×0.199×0.880=0.088	2.126+2/3×0.880=2.713	0.239	0.903	0.285	0.025
	0.880=0.364	2.126+1/3×0.880=2.419	0.881	0.805	0.237	0.086
2	1/2×0.986×1.520=0.749	0.606+2/3×1.520=1.619	1,213	0.539	0.119	0.089
	1,520=1.296	0.606+1/3×1.520=1.113	1.442	0,370	0.060	0.078
3	1/2×1.705×0,606=0.517	2/3×0.606=0.404	0.209	0.134	0.009	0.004
	0.606=0.000	1/3×0.606=0.202	0.000	0.067	0.002	0.000
Total	3.014		3.984			0.283

Acting Point ho =  $\Sigma \text{ Mi} / \Sigma \text{ hi} = 3.984/3.014 = 1.322 \text{ m}$ 

## 3. Stress Calculation of PC Sheet Pile

[Earthquake]

## 1) Loading Condition (par m)

Derived from 2)Load to be act

H	orizontal I	Po	Acting Point h <sub>0</sub>
	3.014 t	/m	1.322 m

			•	
k Value o	f Grand	Surface K	ζ	
	1.225	Kg/cm³		. "

Coefficient of Reaction (k): Average of "ki" below  $1 / \beta$  from the Fulcrum.

$$1/\beta_1 \leq L_1: K = K_1$$

$$1/\beta_1 > L_1$$
:

$$K = \frac{K_1 \times L_1 + K_2 \times L_2}{L_1 + L_2}$$

where:

K<sub>i</sub>: k value at the Fulcrum

K<sub>2</sub> k value below the Fulcrum

L<sub>1</sub>: Thickness of layer from Fulcrum to boundary of next

$$L_2 = \frac{1 - \beta_1 \times L_1}{\beta_2}$$

 $\beta_1$ : Specific characteristic of Sheet Pile

( $\beta$ ) calculated by  $K_1$ 

 $\beta_2$ : Specific characteristic of Sheet Pile

(β) calculated by K2

#### 2) Condition of PC Sheet Pile

Moment of Area I	Section Module Z	Elastic Module E
88700.0 cm <sup>4</sup> /m	8000.0 cm <sup>3</sup> /m	330000 kg/cm <sup>2</sup>

Efficiency of Splice	Reduction Ratio of
(α)	Corrosion (λ)
0.80	1.00

Reduction Ratio of Corrosion ( $\lambda$ ) is derived from the graph of "Thickness of Corrosion/Section Module".

	Allowable Stress	Allowable Deformation
-	$\sigma$ a	The state of an electrical
	240.0 kg/cm <sup>2</sup>	7.5 cm

Bending Stiffness EI		β
2342 t·m²/m	7.5	0.601 m <sup>-1</sup>

Bending Sti	ffness EI'	eta ,	
2927	t·m²/m	0.569	m <sup>-1</sup>

① Deformation/Max. Moment

 $EI = E \times I \times \alpha \times \lambda$ 

 $=330000 \times 88700.0 \times 0.80 \times 1.00$ 

 $= 23420000000 \text{ kg/cm}^2/\text{m}$ 

 $= 2342 t \cdot m^2/m$ 

② Embedding of Sheet Pile

 $EI' = E \times I$ 

 $=330000\times88700.0$ 

 $= 29270000000 \text{ kg/cm}^2/\text{m}$ 

 $= 2927 t \cdot m^2/m$ 

$$\beta = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI}}$$

$$= \sqrt[4]{\frac{1000 \times 1.225}{4 \times 2342}}$$

$$= 0.601 \text{ m}^{-1}$$

$$\beta' = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI'}}$$
$$= \sqrt[4]{\frac{1000 \times 1.225}{4 \times 2927}}$$
$$= 0.569 \text{ m}^{-1}$$

#### 3) Stress Calculation of PC Sheet Pile

Max. B	ending l	Moment Mmax	
	4.813	t·m/m	_] ≦

Allowable cracking moment 13.20 tf-m/m ← OK

 $Mmax = M_t + M \times \phi = Mt + (P_0 \times h) + \phi$ 

where

M<sub>t</sub>: Moment at Top of Pile H: Wall height = 3.006 m

$$\phi = \sqrt{\frac{(1+2 \cdot \beta \cdot h)^2 + 1}{2 \cdot \beta \cdot h}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \cdot \beta \cdot h}\right)$$

$$= \sqrt{\frac{(1+2 \times 0.601 \times 1.322)^2 + 1}{2 \times 0.601 \times 1.322}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \times 0.601 \times 1.322}\right)$$

$$= 1.208$$

Mmax = 
$$M_t + M_0 \times \phi$$
  
=  $M_t + (P \times h) \times \phi$   
= 0.000 + 3.014 × 1.322 × 1.208  
= 4.813 t·m/m

#### 4) Deformation at Top of Sheet Pile

Deformation at Top of Sheet Pile 
$$\Sigma \delta$$

$$2.25 \text{ Cm} \leq \delta \text{ a = 7.50cm} \leftarrow \text{OK}$$

$$\Sigma \delta = \delta_1 + \delta_2 + \delta_3$$

Where:

 $\delta_1$ : Deformation at Fulcrum.

 $\delta_{2}$ : Deformation by Deflection Angle Sheet Pile at Fulcrum.

 $\delta_3$ : Deformation of Sheet Pile as Cantilever above Fulcrum.

$$\delta_{1} = \left(\frac{1 + \beta \cdot h_{0}}{2 \cdot EI \cdot \beta^{3}} \times P + \frac{Mt}{2 \cdot EI \cdot \beta^{2}}\right) \times 100$$

$$= \left(\frac{1 + 0.601 \times 1.322}{2 \times 2342 \times 0.601^{3}} \times 3.014 + \frac{0.000}{2 \times 2342 \times 0.601^{2}}\right) \times 100$$

$$= 0.53 \text{ cm}$$

$$\delta_{2} = \left(\frac{1 + 2 \cdot \beta \cdot h_{0}}{2 \cdot EI \cdot \beta^{2}} \times P + \frac{Mt}{2 \cdot EI \cdot \beta}\right) \times H \times 100$$

$$= \left(\frac{1 + 2 \times 0.601 \times 1.322}{2 \times 2342 \times 0.601^{2}} \cdot 3.014 + \frac{0.000}{2 \times 2342 \times 0.601}\right) \times 3.006 \times 100$$

$$= 1.39 \text{ cm}$$

$$\delta_{3} = \left(\frac{Q \cdot H^{3}}{EI} + \frac{Mt \cdot H^{2}}{2 \cdot EI}\right) \times 100$$

$$= \left(\frac{0.283 \times 3.006^{3}}{2342} + \frac{0.000 \times 3.006^{2}}{2 \times 2342}\right) \times 100$$

$$= 0.33 \text{ cm}$$

$$\therefore \Sigma \delta_{3} = 0.53 + 1.39 + 0.33$$

$$= 2.25 \text{ cm}$$

#### 5) Stress Calculation for All Length of Sheet Pile

Ċ.	100		11/2/	10.0	100		<u> </u>	<u> </u>	1.3.3.11
1			Tatal	T'an	gth o	r Di	. خا	'`∴T÷	100
٠		<u> </u>	IQIAI	LUI	Rin o		ic	بإ	10.00
		2		91	M	m	Ţ.Ī.,	127 14	
ı	1.1	30000	177.7	∵ ソ.し	$\mathcal{M}$	111	7		1.17

Driving Depth of Sheet Pile (Dz):

Dz = 
$$\pi / \beta$$
'  
=  $\pi / 0.569$   
= 5.521 m

Required Total Length of Sheet pile (L):

$$L = H + D_z$$
= 3.006 + 5.521
= 8.527 m < 9.000 m

Location: WF.50 L **DESIGN CONDITION** 

(1) Caluculation Method

Loading Combination

: Normal and Earthquake

Position of Support

: Balancing Point of Active and Passive Soil Pressure

Length of Sheet Pile -

: Balancing Point of Moment 1.0 (Normal)

Balancing Point of Moment 1.0 (Earthquake)

Earth Pressure of Cohesive Soil

 $\Sigma \gamma_h - 2C$ 

(Normal)

Port Formula

(Earthquake)

Seismic Coefficient

K,

= 0.12

Κ,

= 0.24 (Constant, Submarged)

Reduction of Seismic Forth under River-bed

: Non

(2) Soil Pressure

**Background Surface** 

: Horizontally

Live Load

	Passive	Active Down	Active Up	
Normal	$0.00 \text{ t/m}^2$	$0.00 \text{ t/m}^2$	1.00 t/m <sup>2</sup>	
Earthquake	$0.00 \text{ t/m}^2$	$0.00 \text{ t/m}^2$	0.50 t/m <sup>2</sup>	

Friction Angle to Wall

Non

Reduction of Water Pressure

: Non (Trapezoid)

(3) Dimensions of Steel Sheet Pile

Туре

: PC Sheet Pile (t=220 mm)

Efficiency of Splice

: 0.800

Thickness of Corrosion

: Non

Allowable Stress

: Normal 160.0 kg/cm<sup>2</sup> 、Earthquake 240 kg/cm<sup>2</sup>

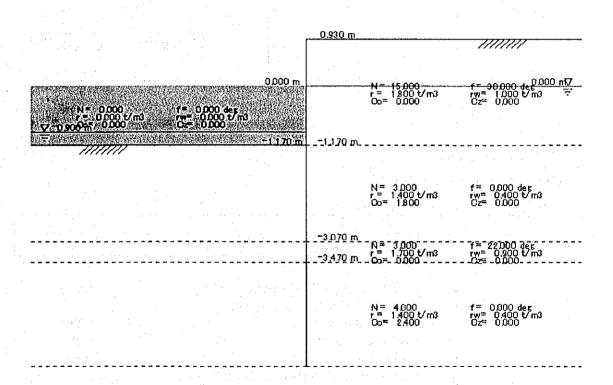
Allowable Deformation at Pile Head

: Normal 5.0 cm, Earthquake 7.5 cm

(4) Soil Condition

	Elevation m	Thick-	Avg. Unit Weight			Internal	Cohesion	Horizontal K			
No		m · · · · · ·	ness m	N- Value	Wet y t/m³	Sat.  7 t/m³	Sub. 7 t/m³	Friction Angle \$ (° )	Со	Value K	
1	0.930	2,100	15.0	1.80	2.00	1.00	30.0	0.000	2.07		
	-1.170	2.100	2.100	2.100	13.0	1.00	2.00	1.00	30.0	0.000	2.07
2	1.170	1.900	3.0	1.40	1.40	0,40	0.0	1.800	0.60		
	-3.070	1.900	3.0	1.40	1.40	0.40	0.0	1.800	0.69		
3	-3,070	0,400	20	1.70	1.00	0.90	22.0	0.000	0.00		
1.	-3.470	0.400	3.0	1.70	1.90	0.90	22.0	0.000	0.92		
4	-3,470	006 500	40	1.40	1.40	0.40	0.0	2,400	2.51		
	-999.999	996.529	4.0	.0 1.40	1.40				. :		

# [Normal Case]



## [Seismic Case]

		, ,	.930 m		
		<u>v.</u>	.300 111		1111111
	0.0	00 m		N= 15.000	
	Star Star Star Star	10002		r = 1 800 t/m3 Oo= 0.000	rw= 1,000 t/m3 = 0.000
0000 €/	10000 des N3.2000 m= 0000 t/m3 Cc= 0000 m	1016			
Δ-0.00m	to the first the same of the s	70 m -1	170 m		
mm					
and the second section of the second				en e	
	en de la companya de La companya de la co		· · · · · · · · · · · · · · · · · · ·	N= 3.000 r= 1.400 t/m3 Oo= 1.800	f≃ 0.000 des rw= 0.400 t/m3 Cz= 0.000
		. "		Oo≖ 1.800	Cz= 0.000
			020		
			030 m	N ≅ 3 5000 r ≈ 1.700 t/m3 . Co.≂. 0.000	f= 22000 der rw= 0.900 t/m3 -Oz= 0.000
		+3.	470 m	. Co= . 0.000	0.2 = 0.000
				N= 4.000	f= 0.000 da#
				r = 1,400 t/m3 Oc= 2,400	f= 0.000 deg rw= 0.400 t/m3 Cz= 0.000
ekan di					

#### 2. Load Calculation

#### (1) Formula of Soil Pressure

## 1) Sandy Soil: Active Soil Pressure

$$P_{a} = K_{a} \cdot \left( \sum \gamma h + \frac{q_{a}}{\cos \beta_{a}} \right) - 2 \cdot C \cdot \sqrt{K_{a}}$$

Where:

P <sub>a</sub>	: Strength of Active Soil Pressure	$(t/m^2)$
K,	: Coefficient of Active Soil Pressure	
γ	: Unit Weight of Soil	$(t/m^3)$
h	: Thickness of Layer	(m)
$q_{\mathbf{a}}$	: Active Load	$(t/m^2)$
ß.	: Angle between G-Surface and Level Surface	(°)
C	: Cohesive	$(t/m^2)$
7.7	$\cos(\phi - \theta) \cos \sigma$	* •
K., =		

$$K_{a} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{a} - \theta)}{\cos(\sigma + \theta) \cdot \cos\beta_{a}}}\right)}$$

Where:

φ : Internal Friction Angle (°)

 $\theta$ : Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_h)$ ,  $K_h$ : Horizontal Seismic Degree

#### 2) Sandy Soil: Passive Soil Pressure

$$P_{p} = K_{p} \cdot \left( \sum \gamma h + \frac{q_{p}}{\cos \beta_{p}} \right) + 2 \cdot C \cdot \sqrt{K_{p}}$$

Where:

Pp	: Strength of Passive Soil Pressure		(t/m²)
$K_{o}$	: Coefficient of Passiv	e Soil Pressure	
γ	: Unit Weight of Soil		$(t/m^3)$
h	: Thickness of Layer		(m)
$q_{\mathfrak{o}}$	: Active Load		$(t/m^2)$
C	: Cohesive		$(t/m^2)$
$\beta_{p}$	: Angle between G-Su	rface and Level Surface	(° )

$$K_{p} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos \theta \cdot \cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{p} - \theta)}{\cos(\sigma + \theta) \cdot \cos \beta_{p}}}\right)}$$

Where:

 $\phi$ : Internal Friction Angle (°)

: Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_h)$ ,

K<sub>h</sub>: Horizontal Seismic Degree

δ : Friction Angle between Soil and Wall (°)

#### (2) Load Calculation (Normal Condition)

#### 1) Soil Pressure & Water Pressure

[Normal · Active]

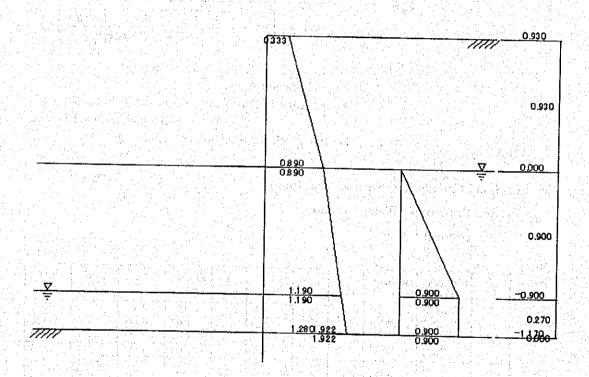
 $q = 0.00 \text{ t/m}^2$ 

Na	Elevation El.m	Thickness m	γ t/m³	ģ	C t/m³	δ	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
1	0.930	0.020	1.00	20.0	0.0 0.0	0.00	0.333	0.000	0.333	0.000	
	0.000	0.930	1.80	30.0		0.0	0.00	0.333	1.674	0.890	0.000
2	0,000	0.900	1.00	30.0	0.0		0.00	0.333	1.674	0.890	0.000
	-0.900	] 0.900	1.00	30.0	0.0	0.0	0.00	0.333	2.574	1.190	0.900
3	-0.900	0,270	1.00	30.0	0.0	0.0	0.00	0.333	2.574	1.190	0.900
	-1.170	0.270	1.00	30.0	0.0	0.0	0.00	0.333	2.844	1.280	1.170
4	-1,170	0.000	0.40	0.0	1.8	0.0	0.00	1.000	2.844	1.922	1,170
	-1.170	0.000	0.40	0.0	1.8	0.0	0.00	1.000	2.844	1.922	1.170

[Normal · Passive]

- 2			00		,	
	~	 11	(1/1	4	1.	. 7

No	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P.
1	0.930	0.930									
	0.000									.,	
2	0.000	0.900		3.	3.54	-					
	-0.900						<b>:</b> 1				13.
3	-0.900	0.270					5 5	4.74			0.000
	-1.170								1 12		0.270
4	-1.170	0.000									0.270
	-1.170										0.270
	1 444 4	275		·							

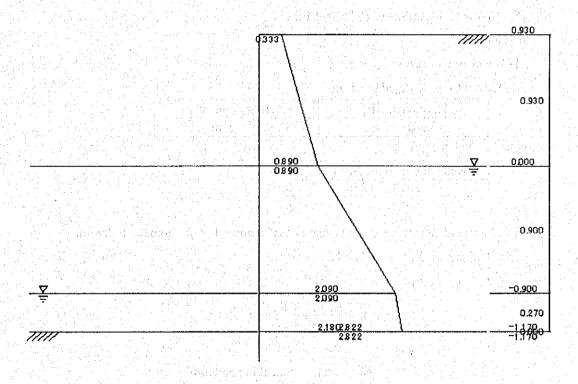


2) Acting Load

## [Normal]

Balancing Pont of Active Force and Passive Force: EL. -1.170 m (0.000 m from Riverbed and 2.100 m from top of Pile).

No	Loading Point El. m	Active SP Pa t/m²	Active WP Pwa t/m²	Passive SP Pp t/m²	Passive WP Pwp t/m²	Load Strength Pa+Pwa-Pp-Pwp t/m²
1	0.930	0.333	0.000	0.000	0.000	0.333
	0.000	0.890	0.000	0.000	0.000	0.890
2	0.000	0.890	0.000	0.000	0.000	0.890
1	-0.900	1.190	0.900	0.000	0,000	2.090
3	-0.900	1,190	0.900	0.000	0.000	2.090
	-1.170	1.280	1.170	0.000	0.270	2.180
4	-1.170	1.922	1.170	0.000	0.270	2,822
	-1.170	1.922	1,170	0.000	0.270	2.822



#### 3) Loading Point

#### [Normal]

No	Hi t/m	Yi m	Mi =Hi∙yi	A ⊨yi/H	B =(3-A)	Q =B·Hi
	1000000000000000		0.077	2 2 2 2	A <sup>2</sup> /6	
1	1/2×0.333×0.930=0.155 0.930=0.414	1,170+2/3×0,930=1,790	0.277	0.852	0.260	0.040
	····	1.170+1/3×0.930=1.480	0.613	0.705	0.190	0.079
2	1/2×0.890×0.900=0.401	0.270+2/3×0.900=0.870	0.349	0.414	0.074	0.030
1.5	0.900=0.941	0.270+1/3×0.900=0.570	0.536	0.271	0.034	0.032
, 3	1/2×2.090×0.270=0.282	0.000+2/3×0.270=0.180	0.051	0.086	0.004	0.001
	0.270=0.294	0.000+1/3×0.270=0.090	0.026	0.043	0.001	0.000
4	.1/2×2.822×0.000=0.000	.2/3×0.000=0.000	0.000	0.000	0.000	0.000
	0.000=0.000	1/3×0.000=0.000	0.000	0.000	0.000	0.000
Total	2.487		1.852		¥11 (1) 44 (411111)	0.181

Acting Point ho =  $\Sigma \text{Mi} / \Sigma \text{hi} = 1.852/2.487 = 0.745 \text{ m}$ 

## 3. Stress Calculation of PC Sheet Pile

[Normal]

#### 1) Loading Condition (par m)

Derived from 2)Load to be act

	Horizontal P <sub>0</sub>	Acting Point h <sub>0</sub>
Ì	2.487 t/m	0.745 m

			24	<u> </u>	
k Valu	e of G	rand S	urfac	e K	
	0.6	93 k	g/cm	3	1

Coefficient of Reaction (k): Average of "ki" below  $1 / \beta$  from the Fulcrum.

$$1/\beta_1 \leq L_1$$
:  $K=K$ 

 $1/\beta_1 > L_i$ :

$$K = \frac{K_1 \times L_1 + K_2 \times L_2}{L_1 + L_2}$$

where:

K<sub>1</sub>: k value at the Fulcrum

K<sub>2</sub>: k value below the Fulcrum

L<sub>1</sub>: Thickness of layer from Fulcrum to boundary of next Layer.

$$L_2 = \frac{1 - \beta_1 \times L_1}{\beta_2}$$

 $\beta_1$ : Specific characteristic of Sheet Pile

 $(\beta)$  calculated by  $K_1$ 

 $\beta_2$ : Specific characteristic of Sheet Pile

( $\beta$ ) calculated by  $K_2$ 

#### 2) Condition of PC Sheet Pile

Moment of Area I	Section Module Z	Elastic Module E
88700.0 cm <sup>4</sup> /m	8000.0 cm <sup>3</sup> /m	330000 kg/cm <sup>2</sup>

Efficiency of Splice	Reduction Ratio of		
(a)	Corrosion (λ)		
0.80	1.00		

Reduction Ratio of Corrosion ( $\lambda$ ) is derived from the graph of "Thickness of Corrosion/Section Module".

Allowable Stress	Allowable Deformation
σα	δa
160.0 kg/cm <sup>2</sup>	5.0 cm

Bending Stiffness EI	β
2342 t·m²/m	0.522 m <sup>-1</sup>

İ	Bending Stiffness EI'	$oldsymbol{eta}$ , $oldsymbol{eta}$
	2927 t·m <sup>2</sup> /m	0.493 m <sup>-1</sup>

#### ① Deformation/Max. Moment

$$EI = E \times I \times \alpha \times \lambda$$

- $=330000\times88700.0\times0.80\times1.00$
- $= 23420000000 \text{ kg/cm}^2/\text{m}$
- $= 2342 \text{ t} \cdot \text{m}^2/\text{m}$

② Embedding of Sheet Pile  
EI' = 
$$E \times I$$
  
= 330000 × 88700.0  
= 29270000000 kg/cm<sup>2</sup>/m

 $= 2927 t \cdot m^2/m$ 

$$\beta = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI}}$$

$$= \sqrt[4]{\frac{1000 \times 0.693}{4 \times 2342}}$$

$$= 0.522 \text{ m}^{-1}$$

# $\beta' = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI'}}$ $= \sqrt[4]{\frac{1000 \times 0.693}{4 \times 2927}}$ $= 0.493 \text{ m}^{-1}$

#### 3) Stress Calculation of Sheet Pile

Max. Bending M	Ioment Mmax		
2.911	t·m/m	≦	-:

Allowable cracking moment	
13.20 tf-m/m ← OK	

$$Mmax = M_1 + M \times \phi = M_1 + (P_0 \times h) + \phi$$

where

M<sub>t</sub>: Moment at Top of Pile h: Wall height = 2.100 m

$$\phi = \sqrt{\frac{(1+2 \cdot \beta \cdot h)^2 + 1}{2 \cdot \beta \cdot h}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \cdot \beta \cdot h}\right)$$

$$= \sqrt{\frac{(1+2 \times 0.522 \times 0.745)^2 + 1}{2 \times 0.522 \times 0.745}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \times 0.522 \times 0.745}\right)$$

$$= 1.571$$

$$= M_{t} + M_{0} \times \phi$$

$$= M_{t} + (P \times h) \times \phi$$

$$= 0.000 + 2.487 \times 0.745 \times 1.571$$

$$= 2.911 \text{ t·m/m}$$

#### 4) Deformation at Top of Sheet Pile

Deformation at Top of Sheet Pile 
$$\Sigma \delta$$
  
1.32 cm  $\leq \delta a = 5.00$ cm  $\leftarrow$  OK

$$\Sigma \delta = \delta_1 + \delta_2 + \delta_3$$

Where:

 $\delta_{-1}$ : Deformation at Fulcrum.

δ<sub>2</sub>: Deformation by Deflection Angle Sheet Pile at Fulcrum.

 $\delta_3$ : Deformation of Sheet Pile as Cantilever above Fulcrum.

$$\delta_{1} = \left(\frac{1+\beta \cdot h_{o}}{2 \cdot EI \cdot \beta^{3}} \times P + \frac{Mt}{2 \cdot EI \cdot \beta^{2}}\right) \times 100$$

$$= \left(\frac{1+0.522 \times 0.745}{2 \times 2342 \times 0.522^{3}} \times 2.487 + \frac{0.000}{2 \times 2342 \times 0.522^{2}}\right) \times 100$$

$$= 0.52 \text{ cm}$$

$$\delta_{2} = \left(\frac{1+2 \cdot \beta \cdot h_{o}}{2 \cdot EI \cdot \beta^{2}} \times P + \frac{M_{i}}{2 \cdot EI \cdot \beta}\right) \times H \times 100$$

$$= \left(\frac{1+2 \times 0.522 \times 0.745}{2 \times 2342 \times 0.522^{2}} \times 2.487 + \frac{0.000}{2 \times 2342 \times 0.522}\right) \times 2.100 \times 100$$

$$= 0.73 \text{ cm}$$

$$\delta_{3} = \left(\frac{Q \cdot H^{3}}{EI} + \frac{Mt \cdot H^{2}}{2 \cdot EI}\right) \times 100$$

$$= \left(\frac{0.181 \times 2.1003}{2342} + \frac{0.000 \times 2.100}{2 \times 2342}\right) \times 100$$

$$= 0.07 \text{ cm}$$

$$\Sigma \delta = 0.52 + 0.73 + 0.07$$
  
= 1.32cm

## 5) Stress Calculation for All Length of Sheet Pile

	5 . 5	7.	1.	:			. 7	<del> </del>
Т	otal	I	eng	gti	ı of	Pile	]	
, a	1		1		- 8.	500	m	. 4

## Driving Depth of Sheet Pile (Dz):

Dz = 
$$\pi / \beta'$$
  
=  $\pi / 0.493$   
= 6.372 m

## Required Total Length of Sheet pile (L):

$$L = H + Dz$$

$$= 2.100 + 6.372$$

$$= 8.472 \text{ m} < 8.500 \text{ m}$$

## (3) Load Calculation (Seismic Case)

## 1) Soil Pressure & Water Pressure

[Earthquake · Active]

 $q = 0.00 \text{ t/m}^2$ 

No	Elevation El.m	Thickness m	γ t/m³	<b>,</b>	c t/m³	ò	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
1	0.930	0.930	1.80	30.0	0.0	^^	0.10	0.397	0.000	0.199	0.000
	0.000	0.930	1.00	30.0	0.0	0.0	0.10	0.397	1.674	0.863	0.000
2	0.000	0.230	1.80	30.0	0.0	0.0	0.10	0.397	1.674	0.863	0.000
-	-0.230	0.230	1.60	30.0	0.0	0.0	0.10	0.397	2.088	1.027	0.000
3	-0.230	0.940	1.00	30.0	0.0	0.0	0.20	0.473	2.088	1.224	0.000
	-1,170	0.740	1.00	30.0	0.0	0.0	0.20	0.473	3.028	1.669	0.940
4	-1.170	0.000	0.40	0.0	1.8	0.0	0.20	1.000	3.028	1.764	0.940
	-1.170	0.000	0.40	0.0	1.8	0.0	0.20	1.000	3.028	1.764	0.940

[Earthquake · Passive]

q = 0.00 t/m

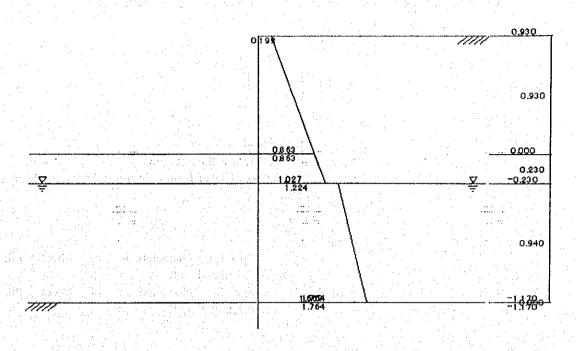
No	Elevation El.m	Thickness m	γ t/m³		c t/m³	8	Seismic Degree	Ka	Σγh Vm³	Soil P. t/m³	Water P.
1	0.930	0.930						42.54			
	0.000							10.00	N. A. V.		
2	0.000	0.230							i, a izan		0.076310
	-0.230			4 14				14400		- J. V.	
3	-0.230	0.940									0.000
14,51	-1.170										0.940
4	-1.170	0.000									0.940
	-1.170			1		4 10 1		. 1 - 12 -			0.940

## 2) Acting Load

[Earthquake]

Balancing Pont of Active Force and Passive Force: EL. -1.170 m (0.000 m from Riverbed and 2.1006 m from top of Pile).

Na	Loading Point E1. m	Active SP Pa t/m²	Active WP Pwa t/m²	Passive SP Pp t/m²	Passive WP Pwp t/m²	Load Strength Pa+Pwa-Pp-Pwp t/m²
1	0.930	0.199	0.000	0.000	0.000	0.199
	0.000	0.863	0.000	0.000	0.000	0.863
2	0.000	0.863	0,000	0.000	0.000	0.863
2.14.11	-0.230	1.027	0.000	0.000	0.000	1.027
3	-0.230	1.224	0.000	0.000	0.000	1.224
- 1	1.170	1.669	0.940	0.000	0.940	1,669
4	-1.170	1.764	0.940	0.000	0.940	1.764
	-1.170	1.764	0.940	0,000	0.940	1.764



## 3) Loading Point

## [Earthquake]

	Hi	Yi	Mi	A	В	Q
Na	t/m	<b>m</b> a salah	=Hi·yi	=yi/H	=(3-A)· A²/6	=B·Hi
1	1/2×0.199×0.930=0.093	1.170+2/3×0.930=1.790	0.166	0.852	0.260	0.024
	0.930=0.401	1,170+1/3×0.930=1.480	0,593	0.705	0.190	0.076
2	1/2×0.863×0.230=0.099	0.940+2/3×0.230=1.093	0.108	0.520	0.112	0.011
	0.230=0.118	0.940+1/3×0.230=1.017	0.120	0.484	0.098	0.012
3	1/2×1.224×0.940=0.575	0.000+2/3×0.940=0.627	0.361	0.299	0.040	0.023
	0.940=0.784	0.000+1/3×0.940=0.313	0.245	0.149	0.011	0,008
4	1/2×1.764×0.000=0.000	2/3×0,000=0,000	0,000	0.000	0,000	0.000
	000,0=000,0	1/3×0.000=0.000	0.000	0.000	0.000	0.000
Total	2.070	and the second s	1.593			0.154

Acting Point ho =  $\Sigma \text{Mi} / \Sigma \text{hi} = 1.593 / 2.070 = 0.770 \text{ m}$ 

## 3. Stress Calculation of PC Sheet Pile

[Earthquake]

## 1) Loading Condition (par m)

Derived from 2)Load to be act

Horizontal P <sub>0</sub>	Acting Point h <sub>0</sub>
2.070 t/m	0.770 m

ſ	k Value of Grand Surface K
ſ	0.693 Kg/cm <sup>3</sup>

Coefficient of Reaction (k): Average of "ki" below  $1 / \beta$  from the Fulcrum.

$$1/\beta_1 \leq L_1: K = K_1$$
$$1/\beta_1 > L_1:$$

$$K = \frac{K_1 \times L_1 + K_2 \times L_2}{L_1 + L_2}$$

where:

 $K_1$ : k value at the Fulcrum

: k value below the Fulcrum

: Thickness of layer from Fulcrum to boundary of next  $L_1$ 

$$L_2 = \frac{1 - \beta_1 \times L}{\beta_2}$$

: Specific Sheet Pile  $\beta_{1}$ characteristic

 $(\beta)$  calculated by  $K_1$ 

Specific characteristic Sheet Pile

(β) calculated by K2

#### 2) Condition of PC Sheet Pile

Moment of Area I	Section Module Z	Elastic Module E
88700.0 cm <sup>4</sup> /m	8000.0 cm <sup>3</sup> /m	330000 kg/cm <sup>2</sup>

E	fficiency of Splice	Reduction Ratio of
	( a )	Corrosion (λ)
	0.80	1.00

Reduction Ratio of Corrosion ( $\lambda$ ) is derived from the graph of "Thickness of Corrosion/Section Module".

Allowable Stress	Allowable Deformation
oa y	$\delta a$
240.0 kg/cm <sup>2</sup>	7.5 cm

Bending Stiffness EI	Ä, i		β	0.7
2342 t·m <sup>2</sup> /m		0.522	2 m <sup>-1</sup>	

Bending Stiffness EI'	$oldsymbol{eta}$ ,
2927 t·m <sup>2</sup> /m	0.493 m <sup>-1</sup>

CARTAGORIA (POTO) JOSEPHO DE LA LIGITA

① Deformation/Max. Moment

 $EI = E \times I \times \alpha \times \lambda$ 

 $=330000 \times 88700.0 \times 0.80 \times 1.00$ 

 $= 23420000000 \text{ kg/cm}^2/\text{m}$ 

 $= 2342 t \cdot m^2/m$ 

② Embedding of Sheet Pile

 $EI' = E \times I$ 

 $=330000 \times 88700.0$ 

 $= 29270000000 \text{ kg/cm}^2/\text{m}$ 

 $= 2927 t \cdot m^2/m$ 

$$\beta = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI}}$$

$$= \sqrt[4]{\frac{1000 \times 0.693}{4 \times 2342}}$$

$$= 0.522 \text{ m}^{-1}$$

$$\beta' = \sqrt[4]{\frac{1000 \cdot K}{4 \cdot EI'}}$$
$$= \sqrt[4]{\frac{1000 \times 0.693}{4 \times 2927}}$$
$$= 0.493 \text{ m}^{-1}$$

#### 3) Stress Calculation of PC Sheet Pile

Max. Bending Moment Mmax		Allowable cracking moment
2.466 t·m/m	≦	13.20 tf-m/m ← OK

$$Mmax = M_t + M \times \phi = Mt + (P_0 \times h) + \phi$$

where

M<sub>t</sub>: Moment at Top of Pile H: Wall height = 2.100 m

$$\phi = \sqrt{\frac{(1+2 \cdot \beta \cdot h)^2 + 1}{2 \cdot \beta \cdot h}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \cdot \beta \cdot h}\right)$$

$$= \sqrt{\frac{(1+2 \times 0.522 \times 0.770)^2 + 1}{2 \times 0.522 \times 0.770}} \cdot \exp\left(-\tan^{-1}\frac{1}{1+2 \times 0.522 \times 0.770}\right)$$

$$= 1.547$$

Mmax = 
$$M_t + M_0 \times \phi$$
  
=  $M_t + (P \times h) \times \phi$   
=  $0.000 + 2.070 \times 0.770 \times 1.547$   
=  $2.466 t \cdot m/m$ 

#### 4) Deformation at Top of Sheet Pile

Deformation at Top of Sheet Pile 
$$\Sigma \delta$$

1.11 Cm  $< \delta a = 7.50 \text{cm} \leftarrow \text{OK}$ 
 $\Sigma \delta = \delta_1 + \delta_2 + \delta_3$ 

Where:

 $\delta_1$ : Deformation at Fulcrum.

 $\delta_2$ : Deformation by Deflection Angle Sheet Pile at Fulcrum.

 $\delta_3$ : Deformation of Sheet Pile as Cantilever above Fulcrum.

$$\delta_{1} = \left(\frac{1+\beta \cdot h_{0}}{2 \cdot \text{EI} \cdot \beta^{3}} \times P + \frac{Mt}{2 \cdot \text{EI} \cdot \beta^{2}}\right) \times 100$$

$$= \left(\frac{1+0.522 \times 0.770}{2 \times 2342 \times 0.522^{3}} \times 2.070 + \frac{0.000}{2 \times 2342 \times 0.522^{2}}\right) \times 100$$

$$= 0.44 \text{ cm}$$

$$\delta_{2} = \left(\frac{1+2 \cdot \beta \cdot h_{0}}{2 \cdot \text{EI} \cdot \beta^{2}} \times P + \frac{Mt}{2 \cdot \text{EI} \cdot \beta}\right) \times H \times 100$$

$$= \left(\frac{1+2 \times 0.522 \times 0.770}{2 \times 2342 \times 0.522^{2}} \cdot 2.070 + \frac{0.000}{2 \times 2342 \times 0.522}\right) \times 2.100 \times 100$$

$$= 0.61 \text{ cm}$$

$$\delta_{3} = \left(\frac{Q \cdot H^{3}}{\text{EI}} + \frac{Mt \cdot H^{2}}{2 \cdot \text{EI}}\right) \times 100$$

$$= \left(\frac{0.154 \times 2.100^{3}}{2342} + \frac{0.000 \times 2.100^{2}}{2 \times 2342}\right) \times 100$$

$$= 0.06 \text{ cm}$$

$$\therefore \Sigma \delta = 0.44 + 0.61 + 0.06$$

$$= 1.11 \text{ cm}$$

#### 5) Stress Calculation for All Length of Sheet Pile

					-		
		Total	Length	of Pile		L	
ļ	2.25%		8.500	m	1.	- ,	

Driving Depth of Sheet Pile (Dz):

Dz = 
$$\pi / \beta'$$
  
=  $\pi / 0.493$   
= 6.372 m

Required Total Length of Sheet pile (L):

$$L = H + D_z$$
= 2.100 + 6.372
= 8.472 m < 8.500 m

Location: WF.117 R

1.DESIGN CONDITION

(1) Calculation Method

Loading Combination

: Normal and Earthquake

Position of Support

: Balancing Point of Active and Passive Soil Pressure

Length of Sheet Pile

: Balancing Point of Moment 1.5 (Normal)

Balancing Point of Moment 1.2 (Earthquake)

Earth Pressure of Cohesive Soil

:  $\Sigma \gamma_h - 2C$ 

(Normal)

Port Formula

(Earthquake)

Seismic Coefficient

K,

= 0.12

K,

= 0.24 (Constant, Submarged)

Reduction of Seismic Forth under River-bed

. Non

(2) Soil Pressure

**Background Surface** 

. Type of berm

Width of berm

B = 4.500 m

Grade of slope

I = 1:2.0

Height of berm

H = 3.360 m

Live Load

Г

		Passive	Active Down	Active Up
	Normal	0.00 t/m <sup>2</sup>	0.00 t/m <sup>2</sup>	1.00 t/m <sup>2</sup>
:	Earthquake	0.00 t/m <sup>2</sup>	$0.00 \text{ t/m}^2$	0.50 t/m <sup>2</sup>

Friction Angle to Wall

Non

Reduction of Water Pressure

. Non (Trapezoid)

(3) Dimensions of Steel Sheet Pile

Type

: PC Sheet Pile (t=220 mm)

Efficiency of Splice

. 0.800

Thickness of Corrosion

: Non

Allowable Stress

: Normal 160.0 kg/cm<sup>2</sup>, Earthquake 240 kg/cm<sup>2</sup>

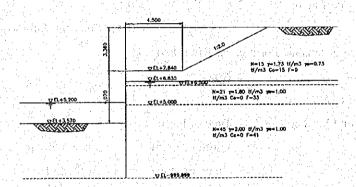
Allowable Deformation at Pile Head

: Normal 5.0 cm, Earthquake 7.5 cm

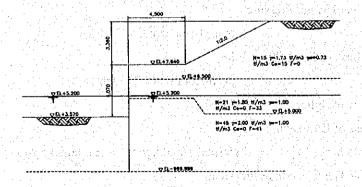
(4) Soil Condition

	Elevation	Thick-	Avg.	1	Unit Weigh	t .	Internal Friction	Cohesion	Horizontal K	
No	m	ness m	N- Value	Wet γ t/m³	Sat. 7 t/m³	Sub. y t/m³	Angle  (°)	Со	Value K	
1	7.640 6.500	1.140	15.0	1,73	1.73	0.73	0.0	15.000	2.07	
2	6.500 5.000	1.500	21.0	1,80	2.00	1.00	33.0	0,000	2.38	
3	5,000 -999,999	1005.000	45.0	2.00	2.00	1.00	41.0	0.000	3.24	

[Normal Case]



[Seismic Case]



#### 2. Load Calculation

#### (1) Formula of Soil Pressure

#### 1) Sandy Soil: Active Soil Pressure

$$P_a = K_a \cdot \left( \sum_i \gamma h + \frac{q_a}{\cos \beta_a} \right) - 2 \cdot C \cdot \sqrt{K_a}$$

Where:

$P_{a}$	: Strength of Active Soil Pressure	$(t/m^2)$
K	· Coefficient of Active Soil Pressure	

: Unit Weight of Soil

$$\gamma$$
: Unit Weight of Soil (t/m<sup>3</sup>)  
h: Thickness of Layer (m)

$$\beta$$
. Angle between G-Surface and Level Surface (°)

$$K_{a} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{a} - \theta)}{\cos(\sigma + \theta) \cdot \cos\beta_{a}}}\right)}$$

Where:

: Internal Friction Angle (°)

: Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_h)$ ,

Kh: Horizontal Seismic Degree

: Friction Angle between Soil and Wall (°)

#### 2) Sandy Soil: Passive Soil Pressure

$$P_{p} = K_{p} \cdot \left( \sum_{\gamma} h + \frac{q_{p}}{\cos \beta_{p}} \right) + 2 \cdot C \cdot \sqrt{K_{p}}$$

Where:

$$\gamma$$
: Unit Weight of Soil (t/m<sup>3</sup>)

$$q_{\rm p}$$
: Active Load (t/m<sup>2</sup>)  
C: Cohesive (t/m<sup>2</sup>)

$$\beta_p$$
: Angle between G-Surface and Level Surface (°)

$$K_{p} = \frac{\cos(\phi - \theta) \cdot \cos \sigma}{\cos(\sigma + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \sigma) \cdot \sin(\phi - \beta_{p} - \theta)}{\cos(\sigma + \theta) \cdot \cos\beta_{p}}}\right)}$$

Where:

: Internal Friction Angle (°)

: Active Angle of Seismic Force (°),  $\theta = \tan^{-1}(K_b)$ ,

K<sub>h</sub>: Horizontal Seismic Degree

: Friction Angle between Soil and Wall (°)

#### (2) Load Calculation (Normal Condition)

Depth of embedment of PC sheet pile shall be installed in depth balancing active pressure side with passive pressure side.

Estimating length of PC sheet pile at 8.5 m, balance of moment is considered.

## 1) Soil Pressure & Water Pressure

[Normal · Active]

 $- q = 0.00 \text{ t/m}^2$ 

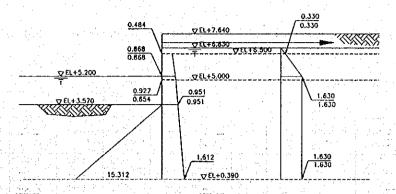
			<del> </del>		1 1 1						
No	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. Vm <sup>3</sup>
1	7.640	0.500	177	0.0	15.0	- ^ ^	0.00	1.000	0.000	0.000	0.000
	- 7.140	0.500	173	0.0	15.0	0.0	0.00	1.000	0.865	0.000	0.000
2	7.140	0.210	1.72	^^	15.0	0.0	0.00	1.000	0.865	0.000	0.000
1.11	6.830	0.310	1,73	0.0	15.0	0.0	0.00	1:000	1,401	0.000	0.000
3	6.830	0.220	0.72	0.0	15.0	0.0	0.00	1.000	1.401	0.000	0.000
1.4.4	6.500	0.330	0.73	0.73 0.0	15.0	5.0	0.00	1.000	1.642	0.000	0.330
4	6.500	1 200	1.00	22.0	0.0	0.0	0.00	0.295	1.642	0.484	0.330
	5.200	1.300	1.00	33.0	0.0	0,0	0.00	0.295	2.942	0.868	1,630
5	5.200	0.200	1.00	33.0	0.0	0.0	0.00	0.295	2.942	0.868	1.630
4 44	5.000	] 0.200	1.00	33.0	0.0	0.0	0.00	0.295	3.142	0.927	1.830
6	5,000	1.430	1.00	41.0	0.0	0.0	0.00	0,208	3.142	0.654	1,830
	3.570	1.450	1.00	71.0	0.0	0,0	0.00	0.208	4.572	0.951	3.260
7	3,570	3.180	1.00	41.0	0.0	0.0	0.00	0.208	4.572	0.951	3.260
<u> </u>	0.390	3.100	1.00	-1.0	0.0	5.0	0.00	0.208	7.752	1.612	6.440

[Normal · Passive]

 $a = 0.00 \text{ t/m}^2$ 

No	Elevation El.m	Thickness m	γ t/m³	<b>\$</b>	c t/m³	δ	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m <sup>3</sup>
1	7.640	0.500		.4]		11.5				Principal Comment	
145	7.140	0.500		4.5	- 1. C. A.		1. 5x 3+ 3	e segualdar		J. J. 18	
2	7.140	0.310			33.35			1 and 13 mm	1 p 2 c		
	6.830	0.310					1 1 1 1		1.0	10.37%	1000
3	6,830	0.220			1.00		3 - 1 - 1 - 3	To a king the first	. <u>N. 1</u>		
- H 1 1 H	6.500	0.330		14.0	14 PM 14 PM	1000	人名 人名	779NK [	.1.		31111111
4	6.500	1.300				1.65	1 3 3 3 4			1.2	
<u> </u>	5.200	1.300	1.0			1.54					- 10 miles
5	5.200	0.200	100	·			4.4 12.1	1.20	4	5 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 -	0.000
<u> </u>	5,000	0.200	AR STA	11. 11. 1			1.18	10. 为6人。			0,200
6	5.000	1,430		- 1.		1.0					0.200
1000	3,570	1,430					11 11 14	mark get	100		1.630
7	3.570	3.180	1.00	41.0	0.0	0,0	0.00	4.815	0,000	0.000	1.630
	0.390	3.700	1.00	71.0	0.0	0.0	0.00	4.815	3.180	15.312	4.810

# [Normal Case]



# Moment around tie rod joint

# [Normal · Active]

No	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint  Mi = Pi Yi (tf-m/m)
1	1/2×0.000×0.500=0.000	-0.500+1/3×0.500=-0.333	0.000×-0.333=0.000
	1/2×0.000×0.500=0.000	-0.500+2/3×0.500=-0.167	0.000×-0.167=0.000
2	1/2×0.000×0.310=0.000	0.000+1/3×0.310=0.000	0.000×0.000=0.000
48	1/2×0.000×0.310=0.000	0.000+2/3×0.310⊨0.000	0.000×0,000≔0.000
3	1/2×0.000×0.330=0.000	0.310+1/3×0.330=0.420	0.000×0.420=0.000
	1/2×0,330×0,330=0.054	0.310+2/3×0.330=0.530	0.054×0.530=0.029
4	1/2×0.814×1.300=0.529	0.640+1/3×1.300=1.073	0.529×1.073=0.568
	1/2×2.498×1.300=1.624	0.640+2/3×1.300=1.507	1.624×1.507=2.446
- 5	1/2×2.498×0.200=0.250	1.940+1/3×0.200=2.007	0.250×2.007=0.501
	1/2×2.757×0.200=0.276	1.940+2/3×0.200=2.073	0.276×2.073=0.572
6	1/2×2,484×1,430=1,776	2.140+1/3×1.430=2.617	1.776×2.617=4.647
	1/2×4.211×1.430=3.011	2.140+2/3×1.430=3.093	3.011×3.093=9.314
7	1/2×4.211×3.180=6.695	3.570+1/3×3.180=4.630	6.695×4.630=31.000
	1/2×8.052×3.180≔12.803	3.570+2/3×3.180=5.690	12.803×5.690=72.847
Total			Ma = 121.924

# [Normal · Passive]

Na	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint Mi = Pi Yi (tf-m/m)
1	1/2×0.000×0.500=0.000	-0.500+1/3×0.500=-0.333	0.000×-0.333=0.000
a Again	1/2×0.000×0.500=0.000	-0.500+2/3×0.500=-0.167	0.000×-0.167=0.000
2	1/2×0.000×0.310=0.000	0.000+1/3×0.310=0.000	0.000.0≃0.000=0.000
75-15-1 <sub>13</sub>	1/2×0.000×0.310=0.000	0.000+2/3×0.310=0.000	0.000×0.000=0.000
3	1/2×0,000×0,330=0,000	0.310+1/3×0.330=0.420	0.000×0.420=0.000
	1/2×0.000×0.330=0.000	0.310+2/3×0.330=0.530	0.000×0.530=0.000
4	1/2×0.000×1.300=0.000	0.640+1/3×1.300=1.073	0.000×1.073=0.000
	1/2×0.000×1.300=0.000	0.640+2/3×1.300=1.507	0.000×1.507=0.000
5	1/2×0.000×0.200=0.000	1.940+1/3×0.200⊨2.007	0.000×2.007=0.000
	1/2×0,200×0,200=0.020	1.940+2/3×0.200=2.073	0.020×2.073=0.041
6	1/2×0.200×1.430=0.143	2.140+1/3×1.430=2.617	0.143×2.617=0.374
	1/2×1.630×1.430=1.165	2.140+2/3×1.430=3.093	1.165×3.093≒3.605
7	1/2×1,630×3.180=2.592	3.570+1/3×3.180⊨4.630	2.592×4.630=12.000
	1/2×20.122×3.180=31.994	3.570+2/3×3.180=5.690	31.994×5.690=182.046
Total			Mp = 198.066

## Consideration of depth of embedment of PC sheet pile

Ma / Mp = 198.066 / 121.924 = 1.62

(Normal case: Safety factor = 1.50) ..........O.K

[Seismic · Active]

 $q = 0.00 \text{ t/m}^2$ 

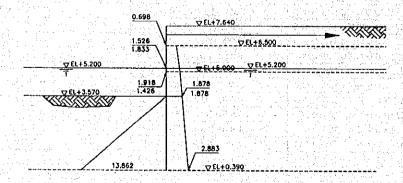
-	No	Elevation El.m	Thickness m	γ t/m³	, \$	c t/m³	δ	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
Ī	1	7.640	0.500	1.72		15.0		0.12	1.000	0.000	0.000	0.000
. [		7,140	0.500	173	0.0	15,0	0.0	0.12	1.000	0.865	0.000	0.000
- [	2	7.140	0.640	1.73	0.0	15.0	0.0	0.12	1.000	0.865	0.000	0.000
÷ [		6.500		1./3		15.0		0.12	1.000	1.972	0.000	0.000
. [	3	6.500	1 200	1.00	1.80 33.0	0.0	0.0	0.12	0.354	1.972	0.698	0.000
L	1 1 11	5.200	1.300	1.60		0.0	0.0	0.12	0.354	4.312	1.526	0.000
٠	4	5.200	0.200	n 100	33.0	0.0	0.0	0.24	0.425	4.312	1.833	0.000
Į		5.000	0.200	1.00		0.0		0.24	0.425	4.512	1.918	0.200
•	5	5.000	1.430	1.00	41.0	0.0	0.0	0.24	0.316	4.512	1.426	0.200
.		3,570	1.430	1.450 1.00	41.0	0.0	0.0	0.24	0.316	5,942	1.878	1.630
١	6	3.570	3.180	1.00	41.0	0.0	0,0	0.24	0.316	5.942	1.878	1.630
l		0.390	3.100	, I	71.0	0.0	0.0	0.24	0.316	9.122	2.883	4.810

[Seismic · Passive]

 $q = 0.00 t / m^2$ 

No	Elevation El.m	Thickness m	γ Vm³	, \$	C Vm³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
1	7.640	0.500		1.10				- 1. 14.			
7.33	7.140	0.500	1.30			100			1 1 1 1 1 1 1 1		are terminal
2	7.140	0.640		. A. 100			21.789		y 155 (485)	154 t 15 t	
78 - 50	6.500	0.640	1.1		C 10 (0 to 20)		18 J. 18		11 4 11 11 11	4 1 2 4 1	2.5 A2.5
3	6.500			- 41		1.41	Sala ella	(			
	5.200	1.300			14 at 15	\$100.00	14 14	117.15	3.1		1 112
4	5.200	0.200	100	14.4				1.14			0,000
	5.000	0.200						F. 365			0,200
. 5	5.000	1.430		1.75			V- 5	1111	14 5 5 5		0.200
- 1 1 2 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	3.570	1,430		15-77					eren Sol		1.630
6	3.570	3.180	1.00	41.0	0.0	0.0	0,24	4.359	0,000	0.000	1.630
1 1	0.390	3.180	1.00	71.0	0.0	0.0	0,24	4.359	3.180	13.862	4.810

[Seismic Case]



#### Moment around tie rod joint

#### [Seismic · Active]

No	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint Mi = Pi · Yi (tf-m/m)
1	1/2×0.000×0.500=0.000	-0.500+1/3×0.500≈-0.333	0.000×-0.333=0.000
y Paris	1/2×0.000×0.500=0.000	-0.500+2/3×0.500=-0.167	0.000×-0.167=0.000
2	1/2×0.000×0.640=0.000	0.000+1/3×0.640=0.213	0.000×0.213=0.000
1.5	1/2×0.000×0.640=0.000	0.000+2/3×0.640=0.427	0.000×0.427=0.000
3	1/2×0.698×1.300=0.454	0.640+1/3×1.300=1.073	0.454×1.073=0.487
	1/2×1.526×1.300=0.992	0.640+2/3×1.300=1.507	0.992×1.507≓1.494
4	1/2×1.833×0.200=0.183	1.940+1/3×0.200=2.007	0.183×2.007=0.368
114.47	1/2×2.118×0.200=0.212	1.940+2/3×0.200=2.073	0.212×2.073=0,439
. 5	1/2×1.626×1.430=1.163	2.140+1/3×1.430=2.617	1.163×2.617=3.042
	1/2×3.508×1.430=2.508	2.140+2/3×1.430=3.093	2.508×3.093=7.759
. 6	1/2×3,508×3,180=5.578	3.570+1/3×3.180=4.630	5.578×4.630=25.825
	1/2×7.693×3.180=12.232	3.570+2/3×3.180=5.690	12.232×5.690=69.599
Total			Ma = 109,013

# [Seismic · Passive]

No	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint Mi = Pi Yi (tf-π/m)
1	1/2×0.000×0.500≔0.000	-0.500+1/3×0.500=-0.333	0.000×-0.333=0.000
	1/2×0.000×0.500=0.000	-0.500+2/3×0,500=-0,167	0.000×-0.167=0.000
2	1/2×0.000×0.640=0.000	0.000+1/3×0.640=0.213	0.000×0.213=0.000
2414,04	1/2×0.000×0.640=0.000	0.000+2/3×0.640=0.427	0.000×0.427=0.000
3	1/2×0,000×1,300=0,000	0.640+1/3×1.300=1.073	0.000×1.073=0.000
	1/2×0.000×1.300⇒0.000	0.640+2/3×1.300=1.507	0.000×1.507=0.000
4	1/2×0.000×0.200=0.000	1.940+1/3×0.200=2,007	0.000×2.007=0.000
	1/2×0.200×0.200=0.020	1.940+2/3×0.200=2.073	0.020×2.073=0.041
5	1/2×0.200×1.430=0.143	2.140+1/3×1.430=2.617	0.143×2.617=0.374
	1/2×1.630×1.430=1.165	2.140+2/3×1.430=3.093	1.165×3.093=3.605
6	1/2×1.630×3.180=2.592	3.570+1/3×3.180=4.630	2.592×4.630=12.000
	1/2×18.672×3.180=29.689	3.570+2/3×3.180=5.690	29.689×5.690=168.927
Total			Mp = 184.948

## Consideration of depth of embedment of PC sheet pile

Ma / Mp = 184.948 / 109.013 = 1.70 (Seismic case: Safety factor = 1.20)...........O.K

Therefore, it is adopted that length of PC sheet pile is 8.50 m.

### 3. Stress Calculation of PC Sheet Pile

Balance point that depend on active pressure and passive pressure is estimated fulcurm.

In structural calculation of PC sheet pile, sheet pile is regarded as simple beam between tie rod joint and estimated fulcrum.

## Calculations of earth pressure and water pressure

# [Normal · Active]

 $q = 0.00 \text{ t/m}^2$ 

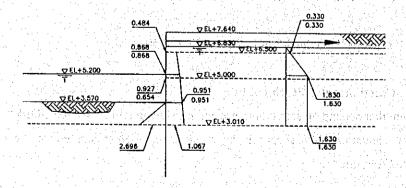
No	Elevation El,m	Thickness m	γ t/m³	ø	ç t/m³	δ	Seismic Degree	Ka	Σγh t/m³	Soil P, t/m³	Water P.		
1	7.640	0.500	173	0.0	15.0	0.0	0.00	1.000	0.000	0.000	0.000		
	7.140	0.500	1//3	0.0	15.0	0.0	0.00	1.000	0.865	0.000	0,000		
2	7.140	0.310	1 72	0.0	15.0	0.0	0.00	1.000	0.865	0.000	0.000		
1	6.830	0.310	1.73	73 0.0	15.0	0.0	0.00	- 1:000	1.401	0.000	0.000		
3	6.830	0.330	0.73		15.0	0.0	0.00	1.000	1.401	0.000	0.000		
1 1 10	6.500	0.550	0.73	0.0	15.0	15.0	15.0	0.0	0.00	1.000	1.642	0.000	0.330
4	6.500	1 200	1.00	22.0	0.0	2.0	0.00	0.295	1.642	0.484	0.330		
	5.200	1.300	1.00	33.0	0.0	0.00	0.295	2.942	0.868	1.630			
5	5.200	0,200	1.00	32.0	0.0	0.0	0.00	- 0.295	2.942	0.868	1.630		
	5.000	0.200	1.00	1.00 33.0	0.0	0.0	0.00	0.295	3.142	0.927	1.830		
6	5.000	1.430	1.00	41.0	0.0	0.0	0.00	0.208	3.142	0.654	1.830		
4 - 502	3.570	1.430	2.00	7.00 41.0	0.0	0.00	0.208	4.572	0.951	3.260			
7	3,570	0.560	1.00	41.0	0.0	0.0	0.00	0.208	4.572	0.951	3.260		
	3.010		1.00	71.0	0.0	0.0	0.00	0.208	5.132	1.067	3.820		

#### [Normal · Passive]

 $q = 0.00 t/m^2$ 

Na	Elevation El.m	Thickness m	γ t/m³	,\$	c t/m³	\$	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m <sup>3</sup>
1	7.640 7,140	0.500									
2	7.140 6.830	0,310									
3	6.830 6.500	0.330			Arriva						
4	6.500 5.200	1.300									
5	5.200 5.000	0.200								ere e Willey W	0.000
7	5,000 3,570	1.430			<b>A</b> 0	1 (A)					0.200 1.630
	3.570 3.010	0.560	1.00	41.0	0.0	0.0	0.00	4.815 4.815	0.000	0.000 2.696	1.630 2.190

# [Normal Case]



Load calculation

No	Elevation El.m	Thickness m	Passive earth pressure tf/m <sup>2</sup>	Passive water pressure tf/m <sup>2</sup>	Active earth pressure tf/m <sup>2</sup>	Active water pressure tf/m <sup>2</sup>	Load tf/m²
1	7.640	0.500	0,000	0.000	0.000	0,000	0,000
	7.140	0.500	0.000	0.000	0.000	0.000	0.000
2	7.140	0.310	0.000	0.000	0.000	0.000	0.000
	6,830	0.310	0.000	0.000	0.000	0.000	0.000
3	6.830	0.220	0.000	0.000	0.000	0.000	0.000
	6.500	0.330	0.000	0.000	0.000	0.330	0.330
4	6.500	1.300	0.000	0.000	0.484	0.330	. 0.814
	5.200	1.300	0.000	0.000	0.868	1.630	2.498
5	5.200	0.200	0.000	0.000	0.868	1.630	2.498
	5.000	0.200	0.000	0.200	0.927	1.830	2.557
6	5.000	1,430	0.000	0.200	0.654	1.830	2.284
1 1 1 1 1	3.570	1,430	0.000	1.630	0.951	3.260	2.581
7	3.570	0,560	0.000	1.630	0.951	3.260	2.581
L	3.010	0.500	2.696	2.190	1.067	3,820	7 0.001

Load = (Active earth pressure + Active water Pressure) - (Passive earth pressure + Passive water pressure)

## Maximum bending moment of PC sheet pile

No	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint  Mi = Pi · Yi (tf-m/m)
2	1/2×0,000×0.310=0.000	0.000+1/3×0.310=0.000	0.000×0.000=0.000
1 14 1 2	1/2×0.000×0.310≠0.000	0.000+2/3×0.310=0.000	0.000×0.000=0.000
3	1/2×0.000×0.330=0.000	0.310+1/3×0.330=0.420	0,000×0,420=0,000
	1/2×0.330×0.330=0.054	0.310+2/3×0.330=0.530	0.054×0.530=0.029
4	1/2×0,814×1,300=0.529	0.640+1/3×1.300=1.073	0.529×1.073=0.568
	1/2×2,498×1,300=1,624	0.640+2/3×1.300=1.507	1.624×1.507=2.446
5	1/2×2.498×0.200=0.250	1.940+1/3×0.200=2.007	0.250×2.007=0.501
	1/2×2.557×0.200=0.256	1.940+2/3×0.200=2.073	0.256×2.073=0.530
6	1/2×2,284×1.430=1.633	2.140+1/3×1.430=2.617	1.633×2.617=4,273
	1/2×2,581×1,430=1,845	2.140+2/3×1.430=3.093	1.845×3.093=5.708
7	1/2×2.581×0.560=0.723	3.570+1/3×0.560=3.757	0.723×3.757=2.715
	1/2×0.001×0.560=0.000	3.570+2/3×0.560=3.943	0.000×3.943=0.001
Total	$\Sigma Q = 6.914$		Ma = 16.772

Ma = 16.772 tf-m/m

Rb = Ma/L

Where

Rb: Reaction force at estimated fulcrum (tf/m)

Ma: Moment at estimated fulcrum (tf-m/m)

L: Length of beam between tie rod joint and estimated fulcrum (m)

$$Rb = 16.772 / 4.130 = 4.061 \text{ tf/m}$$

 $Ra = \Sigma Q - Rb$ 

Where

Ra: Reaction force at the rod joint (tf/m)

Q: Resultant Force (tf/m)

Rb: Reaction force at estimated fulcrum (tf/m)

Ra = 6.914 - 4.061 = 2.853 tf/m

Position (X0) where shearing stress is 0 tf/m is calculated. And bending moment is the maximum about PC sheet pile in the position.

X0 = EL+4.939 m

Mmax = 4.379 tf-m/m

Max. Bending Moment Mmax 4.379 - ( · m/m

Al	lowable cracking	moment	
	13.20 tf-m/m ←	OK	

# Calculations of earth pressure and water pressure

[Seismic · Active]

 $q = 0.00 \text{ t/m}^2$ 

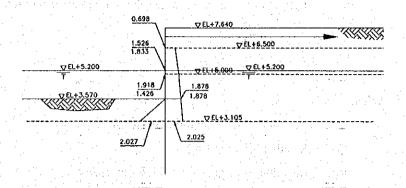
No	Elevation El.m	Thickness m	γ t/m³	ģ	c t/m³	8	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P.			
1	7.640	0.500	1772	00	15.0	0.0	0.12	1.000	0.000	0.000	, 0,000			
1,24 5	7.140	0.300	173	0.0	15.0	0.0	0.12	1.000	0.865	0.000	0.000			
2	7.140	0.640	1 77	0.0	15.0		0.12	1.000	0.865	0.000	0.000			
	6,500	0.640	1./3	1.73	1.73	1.73	3 0.0	15.0	0.0	0.12	1.000	1.972	0.000	0.000
3	6.500	1 200		100	0.0	ું દેવ કહે છે.	0.12	0.354	1.972	0.698	0.000			
1	5.200	1.300	1.80	33.0	0.0	0.0	0.12	0.354	4.312	1.526	0.000			
4	5.200	0,200	1.00	33.0	0.0	0.0	0.24	0.425	4.312	1.833	0.000			
	5.000	0,200	1.00	٠.٠٥	0.0		0.24	0.425	4.512	1.918	0.200			
5	5.000	1,430	1.00	41.0	0.0	0.0	0.24	0.316	4.512	1.426	0.200			
	3.570	1.450	1.00	71.0	0.0	0.0	0.24	0.316	5.942	1.878	1.630			
6	3.570	0,465	1.00	41.0	0.0	0.0	0.24	0.316	5.942	1.878	1.630			
	3.105	0.403	1.00	41.0	0.0	0.0	0.24	0.316	6.407	2.025	2.095			

#### [Seismic · Passive]

 $q = 0.00 t / m^2$ 

No	Elevation El.m	Thickness m	γ t/m³	ø	c t/m³	δ	Seismic Degree	Ka	Σγh t/m³	Soil P. t/m³	Water P. t/m³
1	7.640	0.500				1 1					7. 1
	7.140	0.500				10.00		1.1	- 1 1 N		40.5
2	7.140	0.640				27.4	7 17 1		188734		
	6.500	0.640	ali Iri	1,11	0.544		11,77	1.151	4 4 5	1.0	1.144.44
3	6.500	1.200		1111	3 5 6 6 6			5.55		1000	
	5.200	1.300		1911 12:	9-912-41			1944	77,11		1.2
4	5.200	0.200									0.000
	5.000	0.200		## 43 E 4		To de Maria	10 mg 1 mg				0,200
5	5.000	1.430		- 24.			44.44				0.200
	3.570	1.430		1.45		19.5	1 UNI 0	7.1	3. A. P. L.		1.630
6	3.570	0.465	1.00	41.0	0.0	0.0	0.24	4.359	0.000	0.000	1.630
<u> </u>	3.105	0,403	1.00	71.0	0.0	0.0	0.24	4.359	0.465	2.027	2.095

#### Seismic Case



#### Load calculation

No	Elevation El.m	Thickness m	Passive earth pressure tf/m <sup>2</sup>	Passive water pressure tf/m <sup>2</sup>	Active earth pressure tf/m <sup>2</sup>	Active water pressure tf/m²	Load tf/m²
1	7.640	0.500	0.000	0.000	0.000	0.000	0.000
	7.140	0.500	0.000	0.000	0.000	0.000	0.000
2	7.140	0.010	0.000	0.000	0,000	0.000	0.000
	6.500	0.640	0.000	0.000	0.000	0.000	0,000
3	6.500	1.200	0.000	0.000	0.698	0.000	0.698
	5.200	1.300	0.000	0.000	1.526	0.000	1.526
4	5.200	0.200	0.000	0.000	1,833	0.000	1.833
1.11	5.000	0.200	0.000	0.200	1.918	0.200	1.918
5	5.000	1.430	0.000	0.200	1.426	0.200	1.426
	3.570	1.430	0.000	1.630	1.878	1.630	1.878
6	3.570	0.465	0.000	1.630	1.878	1.630	1.878
100	3.105	0.403	2.027	2.095	2.025	2.095	-0.002

Load = (Active earth pressure + Active water Pressure) - (Passive earth pressure + Passive water pressure)

## Maximum bending moment of PC sheet pile

Nα	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m.	Moment around tie rod joint  Mi = Pi · Yi (tf-m/m)
2	1/2×0.000×0.640=0.000	0.000+1/3×0.640=0.213	0,000×0.213=0.000
	1/2×0,000×0,640=0,000	0.000+2/3×0.640=0,427	0.000×0.640=0.000
3	1/2×0.698×1.300=0.454	0.640+1/3×1.300=1.073	0.454×1.073=0.487
1.365	1/2×1,526×1,300=0,992	0.640+2/3×1.300=1.507	0.992×1.507=1.494
4	1/2×1.833×0.200=0.183	1.940+1/3×0.200=2.007	0.183×2.007=0.368
	1/2×1.918×0.200=0.192	1.940+2/3×0.200=2.073	0.192×2.073=0.398
5	1/2×1,426×1,430=1,020	2.140+1/3×1.430=2.617	1.020×2.617=2.668
	√1/2×1.878×1.430≐1.343	2.140+2/3×1.430=3.093	1.343×3.093=4.154
6	1/2×1.878×0.465=0.437	3.570+1/3×0.465=3.725	0.437×3.725=1.626
	1/2×-0.002×0.465=0.000	3.570+2/3×0.465=3.880	0.000×3.880=0.000
Total	$\Sigma Q = 4.621$		Ma = 11.193

Ma = 11.193 tf-m/m

Rb = Ma / L

Where

Rb: Reaction force at estimated fulcrum (tf/m)

Ma: Moment at estimated fulcrum (tf-m/m)

L: Length of beam between tie rod joint and estimated fulcrum (m)

$$Rb = \frac{11.193}{4.035} = 2.774 \text{ tf/m}$$

 $Ra = \Sigma Q - Rb$ 

Where

Ra: Reaction force at tie rod joint (tf/m)

Q: Resultant Force (tf/m)

Rb: Reaction force at estimated fulcrum (tf/m)

Ra = 4.619 - 2.774 = 1.845 tf/m

Position (X0) where shearing stress is 0 tf/m is calculated. And bending moment is the maximum about PC sheet pile in the position.

X0 = EL + 4.983 m

Mmax = 2.800 tf-m/m

14	Max. Bending Moment Mmax	
i	2.800 t·m/m	>

A	lowab	le crac	king	mome	ent
	13.20	tf-m/r	n ←	OK	

#### 4. Stress Calculation of Tie Rod Rope

Extension of tie rod rope

[Normal Case]

Na	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint  Mi = Pi · Yi (tf-m/m)
1	1/2×0.000×0.500=0.000	-0.500+1/3×0.500=4.463	0.000×4.463=0.000
	1/2×0.000×0.500=0.000	-0.500+2/3×0.500=4.297	0.000×4.297=0.000
2	1/2×0.000×0.310=0.000	0.000+1/3×0.310≐4.027	0,000×4.027=0.000
	1/2×0.000×0.310=0.000	0.000+2/3×0.310=3,923	0,000×3,923=0,000
3	1/2×0.000×0.330=0.000	0.310+1/3×0.330=3.710	0.000×3.710=0.000
	1/2×0.330×0.330=0.054	0.310+2/3×0.330=3.600	0.054×3.600=0.196
4	1/2×0.814×1.300=0.529	0.640+1/3×1.300=3.057	0.529×3.057=1.617
	1/2×2.498×1.300=1.624	0.640+2/3×1.300=2.623	1.624×2.623=4.260
5	1/2×2.498×0.200=0.250	1,940+1/3×0,200=2,123	0,250×2,123≒0.530
1.00	1/2×2.557×0.200=0.256	1.940+2/3×0.200=2.057	0.256×2.057=0.526
6	1/2×2.284×1.430=1.633	2.140+1/3×1,430=1.513	1.633×2.617⊨2.471
1,1	1/2×2.581×1.430=1.845	2.140+2/3×1.430=1.037	1.845×3.093=1.913
7.	1/2×2.581×0.560=0.723	3.570+1/3×0.560=0.373	0.723×0.373=0.270
	1/2×0.001×0.560=0.000	3.570+2/3×0.560=0.187	0.000×0.187=0.000
Total	$\Sigma Q = 6.914$		Ma = 11.783

Ma = 11.783 tf-m/m

Ra = Ma / L

Where

Ra: Reaction force at estimated fulcrum (tf/m)

Ma: Moment at estimated fulcrum (tf-m/m)

L: Length of beam between tie rod joint and estimated fulcrum (m)

$$Ra = 11.783/4.130 = 2.853 \text{ tf/m}$$

 $T = Ra \times L$ 

Where

Ra: Reaction force at tie rod joint (tf/m)

L: Interval of tie rod (m)

 $T = 2.853 \times 2.00 = 5.706$  tf/rope

Stress calculation of tie rod rope

Tie rod rope (SS41 or equivalent  $\phi = 32$  mm) is adopted.

$$\sigma = \frac{T}{\frac{\pi}{4} \times (\Phi - 2 \times t)^2}$$

Where

T: Extension force of tie rod (kgf/rope)

 $\phi$ : Diameter of tie rod rope (cm)

: Corrosion width (cm)

$$\sigma = \frac{5706}{\frac{\pi}{4} \times (3.2 - 2 \times 0.1)^2} = 807 \text{ kgf/cm}^2$$

≤ 900 kgf/cm² (Allowable capacity)......O.K

#### [Seismic Case]

No	Resultant Force Pi t f/m	Distance from joint point to Pi Yi m	Moment around tie rod joint  Mi = Pi Yi (tf-m/m)
1	1/2×0.000×0.500=0.000	-0.500+1/3×0.500=4.368	0.000×4.368=0.000
	1/2×0.000×0.500=0.000	-0.500+2/3×0.500=4.202	0.000×4.202=0.000
2	1/2×0.000×0.640=0.000	0.000+1/3×0.640=3.822	0.000×3.822=0.000
	1/2×0,000×0,640=0,000	0.000+2/3×0.640=3.608	0.000×3.608=0.000
3	1/2×0.698×1.300=0.454	0.640+1/3×1.300=2.962	0.454×2.962=1.344
	1/2×1,526×1.300=0.992	0.640+2/3×1.300=2.528	0.992×2.528=2.508
4	1/2×1.833×0.200=0.183	1.940+1/3×0.200=2.028	0.183×2.028=0.372
	1/2×1.918×0.200=0.192	1.940+2/3×0.200=1.962	0.192×1.962=0.376
5	1/2×1.426×1.430=1.020	2.140+1/3×1.430=1.418	1.020×1.418=1.446
	1/2×1.878×1.430=1.343	2.140+2/3×1.430=0.942	1.343×0.942=1.264
6	1/2×1.878×0.465=0.437	3.570+1/3×0.465=0.310	0.437×0.310=0.135
	1/2×-0.002×0.465=0.000	3.570+2/3×0.465=0.155	0.000×3.155=0.000
Total	$\Sigma Q = 4.621$		Ma = 7.445

Ma = 7.445 tf-m/m

Ra = Ma / L

Where

Ra: Reaction force at estimated fulcrum (tf/m)

Ma: Moment at estimated fulcrum (tf-m/m)

L: Length of beam between tie rod joint and estimated fulcrum (m)

$$Ra = \frac{7.445}{4.035} = 1.845 \text{ tf/m}$$

$$T = Ra \times L$$

Where

-Ra: Reaction force at tie rod joint (tf/m)

L: Interval of tie rod (m)

 $T = 1.845 \times 2.00 = 3.690 \text{ tf/rope}$ 

Stress calculation of tie rod rope

Tie rod rope (SS41 or equivalent  $\phi = 32$  mm) is adopted.

$$\sigma = \frac{T}{\pi/4 \times (\Phi - 2 \times t)^2}$$

Where

T: Extension force of tie rod (kgf/rope)

 $\phi$ : Diameter of tie rod rope (cm)

t: Corrosion width (cm)

$$\sigma = \frac{3690}{\pi/4 \times (3.2 - 2 \times 0.1)^2} = 522 \text{ kgf/cm}^2$$

≤ 1350 kgf/cm² (Allowable capacity)......O.K

#### 5. Stress Calculation of Waling

Waling is regarded as single beam between tie rod joint and estimated fulcrum.

$$M = \frac{T}{4} \times L$$

Where

M: Bending moment toward waling (tf-m)

T: Extension stress of tie rod (tf/rope)

L: Interval tie rod rope (m)

[Normal Case]

$$M = 5.706/4 \times 2.00 = 2.853 \text{ tf-m}$$

$$\sigma = M/Z$$

Where

σ Bending stress of waling (kgf/cm²)

M: Bending moment toward waling (tf-m)

Z: modulus of elasticity (cm<sup>3</sup>)

U shape steel beam (2[  $150 \times 75 \times 6.5 \times 10$ ) for waling is adopted.

$$\sigma = 2.853 \times 100000 / 230$$
 = 1240 kgf/cm<sup>2</sup>

≤ 1400 kgf/cm² (Allowable capacity)......O.K

Seismic Case

$$M = 3.690/4 \times 2.00 = 1.845 \text{ tf-m}$$

$$\sigma = M/Z$$

Where

 $\sigma$  Bending stress of waling (kgf/cm<sup>2</sup>)

M: Bending moment toward waling (tf-m)

Z: Section modulus (cm<sup>3</sup>)

U shape steel beam (2[  $150 \times 75 \times 6.5 \times 10$ ) for waling is adopted.

$$\sigma = 1.845 \times 100000 / 230$$
 = 802 kgf/cm<sup>2</sup>

≤ 2100 kgf/cm² (Allowable capacity) O.K

#### 6. Consideration of Anchor work

Length of counterfort

Depth of embedment of counterfort is calculated with specific volume ( $\beta$ ).

$$\beta = \sqrt[4]{\frac{kh \times B}{4EI}}$$

Where

kh: Coefficient of horizontal subgrade reaction (kgf/cm³)

B: Width of member (cm)

kh = 
$$0.691 \text{ N}^{0.406}$$
 =  $0.691 (18)^{0.406}$  =  $2.234 \text{ kgf/cm}^3$ 

N: N-volume 
$$N = 18$$

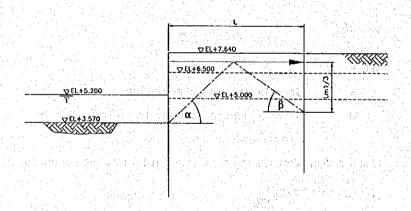
$$\beta = \sqrt{\frac{2.234 \times 50}{4 \times 330000 \times 44000}} = 6.622 \times 10^{-3} \text{ cm}^3 = 0.662 \text{ m}^{-1}$$

Lm1 = 
$$\frac{\pi}{\beta}$$
 = 3.14 / 0.662 = 4.74 m

$$L = 4.74 + 0.25 = 4.99 = 5.00 \text{ m}$$

#### Consideration of distance for counterfort

Internal friction angle	Wall friction angle δ°			Collapse angle	
φ°	Active side	Passive side	Kh .	Active side $\alpha$ °	Passive side β°
			0.00	60,00	30.00
33	0.00	0.00	0.10	55.30	28.43
			0.20	49.60	26.50



#### [Normal Case]

$$Lm1/3 = 4.74/3 = 1.58 m$$

$$L1 = 3.570 \times \tan 30 = 2.06$$

$$L2 = 1.580 \times \tan 60^{\circ} = 2.74$$

$$Ln = L1 + L2 = 2.06 + 2.74 = 4.80 = 5.00 \text{ m}$$

#### [Seismic Case]

$$Lm1/3 = 1.58 m$$

$$L1-w = 1.630 \times \tan 40.4$$
° = 1.39

$$L1-L = 1.940 \times \tan 34.7$$
 ° = 1.34

$$L2 = 1.580 \times \tan 61.57$$
 ° = 2.92

Ls = L1-w + L1-L + L2 = 
$$1.39 + 1.34 + 2.92 = 5.65 = 6.00 \text{ m}$$

Therefore,

It is adopted that distance for counterfort is above 6.00 m.

-Stress calculation of counterfort -

$$M \max = 0.322 \times \frac{T}{\beta}$$

Where

T: Extension stress of tie rod (tf/rope)

B: Specific volume (m<sup>-1</sup>)

Normal Case

$$M \max = 0.322 \times \frac{5.706}{0.662} = 2.775 \text{ tf-m}$$

# Max. Bending Moment Mmax 2.775 t·m/m

Allowable cracking moment
13.20 tf-m/m ← OK

[Seismic Case]

$$M \text{ max} = 0.322 \times \frac{3.690}{0.662} = 1.795 \text{ tf-m}$$

# Max. Bending Moment Mmax 1.795 t·m/m

Allowable cracking moment
13.20 tf-m/m ← OK

Displacement of counterfort

$$\delta = \frac{T}{2 \times E \times I \times \beta^3}$$

Where

T: Extension stress of tie rod (tf/rope)

 $\beta$ : Specific volume (m<sup>-1</sup>)

E: modulus of elasticity (kgf/cm²)

I: Geometrical moment of inertia (cm<sup>4</sup>)

[Normal Case]

$$\delta = \frac{5.706 \times 1000}{2 \times 330000 \times 44000 \times 0.006622^3} = 0.68 \text{ cm}$$

Deformation at Top of Sheet Pile 
$$\Sigma \delta$$
 0.68 cm

$$\leq \delta a = 5.00 \text{cm} \leftarrow \text{OK}$$

[Seismic Case]

....

$$\delta = \frac{3.690 \times 1000}{2 \times 330000 \times 44000 \times 0.006622^3}$$

= 0.44 cm

Deformation at Top of Sheet Pile  $\Sigma \delta$ 0.44 cm

 $\leq \delta a = 5.00 \text{cm} \leftarrow \text{OK}$