DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
REPUBLIC OF THE PHILIPPINES

JAPAN INTERNATIONAL COOPERATION AGENCY

THE DETAILED DESIGN OF PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III)

FINAL REPORT

VOLUME-III-2

STRUCTURAL CALCULATION OF LOWER MARIKINA RIVER

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CTI Engineering International Co., Ltd.
Consulting Engineers

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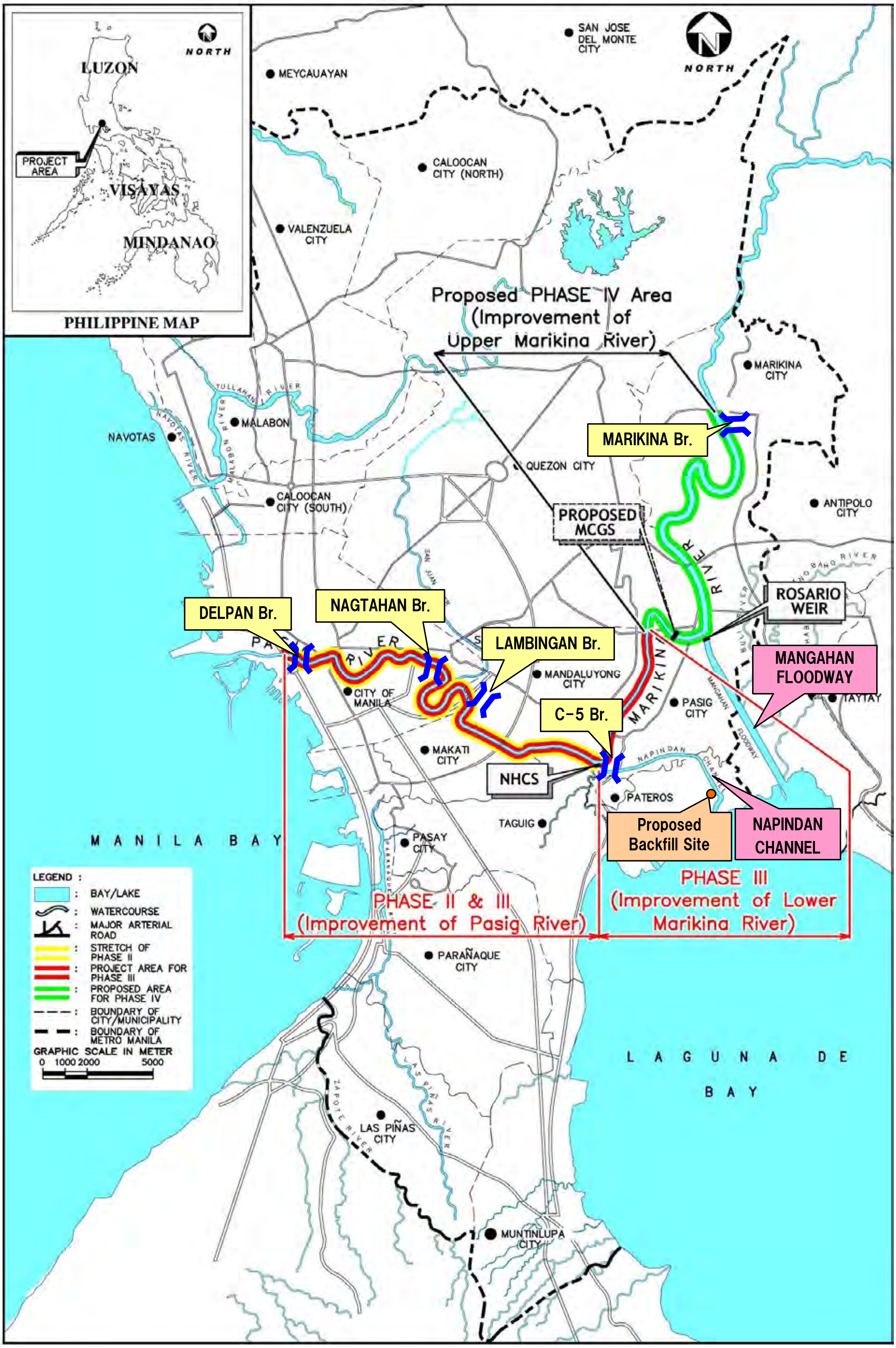

PROJECT LOCATION MAP

THE DETAILED DESIGN
OF
PASIG-MARIKINA RIVER CHANNEL
IMPROVEMENT PROJECT (PHASE III)

FINAL REPORT
Vol.-III-2 STRUCTURAL CALCULATION
OF LOWER MARIINA RIVER

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ABBREVIATIONS AND ACRONYMS

Units of Measurement

mm	:	millimeter
cm	:	centimeter
m	:	meter
km	:	kilometer
g, gr	:	gram
kg	:	kilogram
t, ton	:	metric ton
m^2	:	square meter
ha, has	:	hectare, hectares
km^2	:	square kilometer
m^3	:	cubic meter
s, sec	:	second
m, min.	:	minute
h, hr	:	hour
y, yr	:	year
MW	:	megawatt
mm/hr	:	millimeter per hour
m/s	:	meter per second
km/hr	:	kilometer per hour
mg/l	:	milligram per liter
m^3/s	:	cubic meter per second
$m^3/s/km^2$	:	cubic meter per second per square kilometer
%	:	percent
ppm	:	parts per million
x x	:	symbol of multiplication (times)
≤ , ≥	:	Inequality sign (e.g. A≤B means that value A is less than or equal to value B.)
< , >	:	Inequality sign (e.g. A<B means that value A is less than value B.)
Y, Y, JPY	:	Japanese Yen
P, P, PHP	:	Philippine Peso

CHAPTER 1 CALCULATION OF CONSOLIDATION SETTLEMENT

The detailed calculation of consolidation settlement is indicated from the following page.

<u>New Calculation of Settlement and Side Deformation (1)</u>

Section 1 – STA 1+100

1. Design Condition (Form of Embankment)

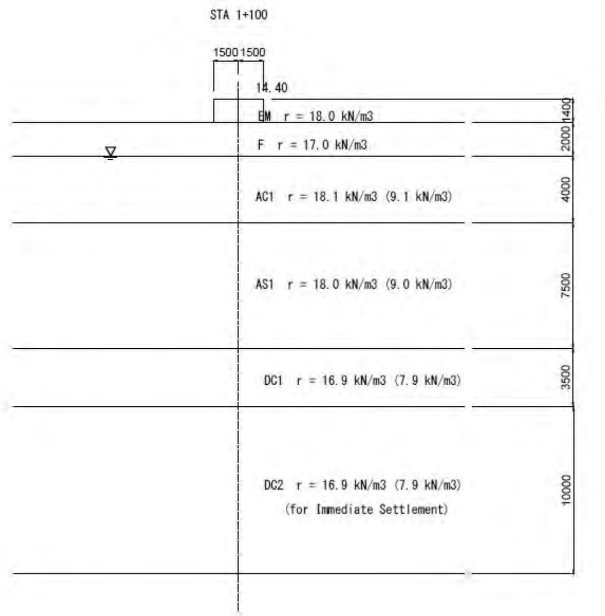


Fig.1 Embankment and Geological Condition

2. Calculation of Instant Settlement and Side deformation

2-1 Geological Condition

Layer	Geology	Thickness	Average N-Value	Modulus of Deformation of Soil Ei 700N (kN/m^3)	Remarks
1	F	2.00	7	4,900	
2	AC1	4.00	5	3,500	
3	AS1	7.50	12	8,400	
4	DC1	3.50	5	3,500	
5	DC2	10.00	20	14,000	

2-2　Calculation Points

1	Center	0m
2	Shoulder	1.50m

2-3　Load

$q = 18.0 * 1.40 = 25.2$ kN/m^2

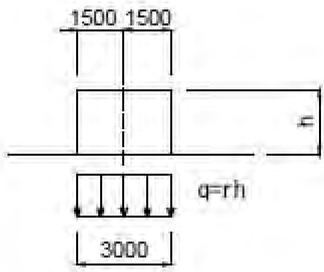


Fig.2　Load

2-4　Immediate Settlement

$$S_{ix} = \sum_{i=1}^{n} \frac{-3 a_i \cdot q_i}{E_m \cdot \pi} \log\sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left[1.0 - \frac{0.75}{\pi} \cdot \left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right| \right] \right]$$

Where,　S_{ix} : immediate settlement of foundation at the point of xm in lateral direction (m)

qi : surcharge of embankment (=25.2 kN/m^2)

Em: converted coefficient of reaction of soil (kN/m^2)

2ai: width of load (=3.00m)

H : Depth of influence of instant settlement (m)

n : number of distribution loads

x : distance from a center of embankment (m)

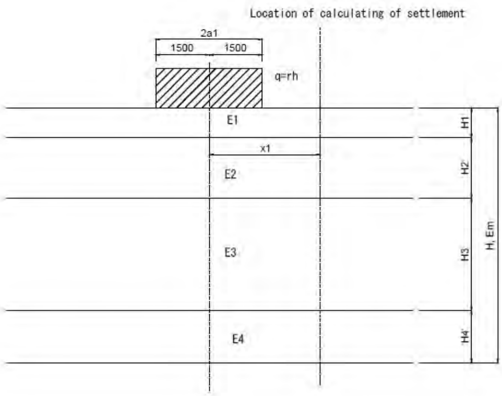


Fig.3 Location of Calculating

2-5 Coefficient of Reaction of Soil

Converted coefficient of reaction of soil is obtained by the following equation.

B=L

$$E_m = \frac{-\dfrac{1}{B+2h_n \cdot \tan\theta} + \dfrac{1}{B}}{\Sigma \dfrac{1}{E_i}\left(-\dfrac{1}{B+2h_i \cdot \tan\theta} + \dfrac{1}{B+2h_{i-1} \cdot \tan\theta}\right)}$$

Where, Em: converted coefficient of reaction of soil considering the change of soil (kN/m^2)

B : loading width (=3.00m)

L : depth of loading (=3.00m)

h_n : infected depth, 3 times loading width or more (27.00m)

Ei : coefficient of reaction of soil at i^{th} layer

θ : distribution angle of load (=30°)

Layer	Thickness (m)	Depth (m)	Ei (kN/m^2)
1	2.00	2.00	4,900
2	4.00	6.00	3,500
3	7.50	13.50	8,400
4	3.50	17.00	3,500
5	10.00	27.00	14,000

2-6 Converted Coefficient of Reaction of Soil

Layer	Layer	Depth (m)	Equation	Total
Numerator			0.30405	0.30405
Denominator	1	2.00	0.0000296	
	2	6.00	0.0000250	
	3	13.50	0.0000056	
	4	17.00	0.0000027	
	5	27.00	0.0000011	0.0000640

Em = 0.30405 / 0.0000640 = 4,751 kN/m^2

2-7 Immediate Settlement

Location	Distance	Immediate Settlement (m)
Center	0.00m	0.019

2-8 Side Deformation

$$R_{ix}=\sum_{i=1}^{n}-\frac{(1+\nu)(1-2\nu)\cdot q_i\cdot a_i}{E_m\cdot \pi}\left(\frac{b_i}{2a_i}\log\frac{(a_i-x)^2+b_i^2}{(a_i+x)^2+b_i^2}+\frac{a_i-x}{a_i}\tan^{-1}\frac{b_i}{a_i-x}-\frac{a_i+x}{a_i}\tan^{-1}\frac{b_i}{a_i+x}\right)$$

Where, Rix: side deformation at x m from the center (m)

qi : surcharge of embankment (=25.2 kN/m^2)

Em: converted coefficient of reaction of soil considering the change of soil (= 4,751 kN/m^2)

ν : Poison's ratio (=0.30)

2a1: loading width (= 3.00m)

2b1: depth of loading (=3.00m)

n : number of loads (=1)

x: distance from the center (m)

Location	Distance	Side Deformation (m)
Center	0.00m	0
Shoulder	1.50m	-0.0017

3. Calculation of Consolidation Settlement

3-1 Method of Calculation

We'd like to use Cc method.

$$S = \frac{C_c}{1+e_o}\times \log_{10}\frac{P_o+\Delta P}{P_o+q_o}\times H$$

Where S : consolidation settlement (m)

e0: initial void ratio

e1: void ratio after consolidation

Cc: compression index of clay

H : total depth of clay layer (m)

P0: effective earth covering due to pre-consolidation pressure(kN/m^2)

q0: Increment of vertical stress due to Pre-consolidation pressure (kN/m^2)

Δ p: stress increment due to surcharge of embankment (kN/m^2)

3-2 Calculation of Stress Increment

q = 18.0 * 1.40 = 25.2 kN/m^2

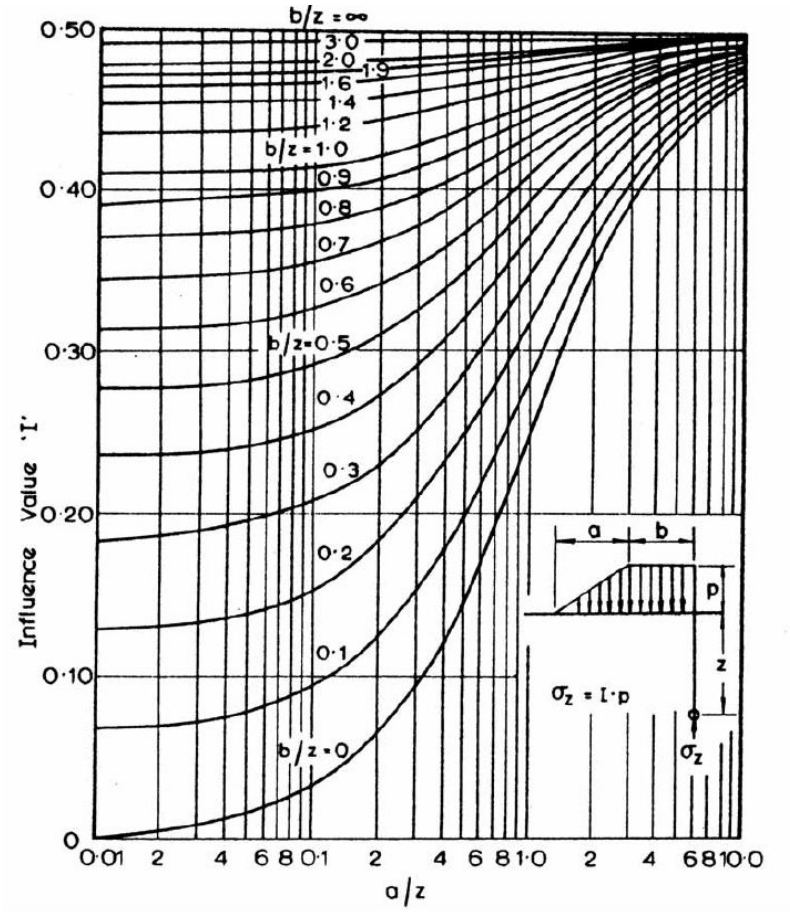


Fig.4 Osterberg Method

Layer	Geology	Depth (z)	a(m)	b(m)	a/z	b/z	I	Δ p (kN/m^2)
2	AC1	4.00	0	1.50	0	0.375	0.22	5.54
3	AS1	-	-	-				-
4	DC	15.25	0	1.50	0	0.098	0.07	1.76

3-3 Pre-consolidation Pressure

Layer	Geology	Calculation	p0 (kN/m^2)	po (kgf/cm^2)
2	AC1	2.00*17.0+2.00*9.1	52.20	0.532
4	DC	2.00*17.0+4.00*9.1+7.50*9.0+1.75*7.9	151.73	1.548

3-5 Calculation of Cc

Layer	pa (kgf/cm2)	ea	pb (kgf/cm2)	eb	Cc = (ea-eb)/Log(Pb/Pa)	Remarks
2	1.6	0.8644	12.8	0.5898	0.3041	
4	3.2	1.2400	12.8	0.8800	0.5979	

3-4 Consolidation Settlement

Layer	H(m)	Cc	eo	Po (kN/m^2)	Δ p (kN/m^2)	q0 (kN/m^2)	S (m)
2	4.00	0.3041	0.9934	52.20	5.54	0	0.027
4	3.50	0.5979	1.495	151.73	1.76	0	0.004
Total							0.031

3-5 Consolidation Time

(1) Calculation of Cv

Layer	Pre-consolidation Pressure (kgf/cm^2)	Cv ($10^{-3}cm^2/sec$)	Cv (m^2/day)	d (m)	Settlement
2	0.532	2.332	0.02015	H/2 =2.00	0.027
4	1.548	3.348	0.02893	H=3.50	0.004

(2) Consolidation Time

$$t = Tv*d^2 / Cv$$

Where, t : time until consolidation index U

TV: time factor as shown below in response to consolidation index U

U(%)	10	20	30	40	50	60	70	80	90	100
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848	∞

Cv : coefficient of consolidation (m^2/day)

d : maximum discharge distance, both sides discharge d =H/2, one side discharge (m)

H : thickness of layer (m)

U : consolidation index (%), U = St / Sc

Layer	Consoli-dation (%)	10	20	30	40	50	60	70	80	90	100
2	Day	2	6	14	25	39	57	80	113	168	
	Settle-ment	0.003	0.006	0.009	0.012	0.015	0.018	0.021	0.024	0.027	0.030
4	Day	3	13	30	53	83	122	171	240	359	
	Settle-ment	0.000	0.001	0.001	0.002	0.002	0.002	0.003	0.003	0.004	0.004
Total	Settle-ment	0.003	0.007	0.010	0.014	0.017	0.020	0.024	0.027	0.031	0.034

3-6 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.019 + 0.031 = 0.050 m < 0.50 m….OK

New Calculation of Settlement and Side Deformation (2)

Section 2 – STA 3+170

1. Design Condition (Form of Embankment)

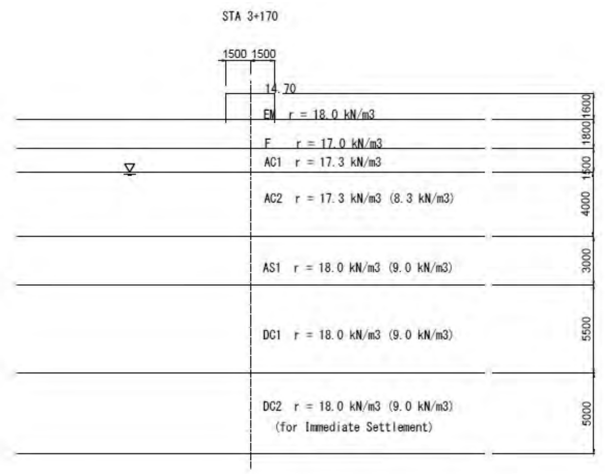


Fig.1 Embankment and Geological Condition

2. Calculation of Instant Settlement and Side deformation

2-1 Geological Condition

Layer	Geology	Thickness	Average N-Value	Modulus of Deformation of Soil Ei 700N (kN/m^3)	Remarks
1	F	1.80	7	4,900	
2	AC1	1.50	6	4,200	
3	AC2	4.00	3	2,100	
4	AS1	3.00	12	8,400	
5	DC1	5.50	5	3,500	
6	DC2	5.00	10	7,000	

2-2 Calculation Points

1	Center	0m
2	Shoulder	1.50m

2-3 Load

$$q = 18.0 * 1.60 = 28.8 \text{ kN/m}^2$$

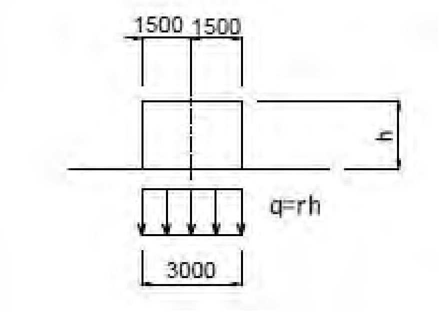


Fig.2 Load

2-4 Immediate Settlement

$$S_{ix} = \sum_{i=1}^{n} \frac{-3 a_i \cdot q_i}{E_m \cdot \pi} \log\sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left[1.0 - \frac{0.75}{\pi} \cdot \left\{ \left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right| \right\} \right]$$

Where, S_{ix} : immediate settlement of foundation at the point of xm in lateral direction (m)

qi : surcharge of embankment (=28.8 kN/m^2)

Em: converted coefficient of reaction of soil (kN/m^2)

2ai: width of load (m)

H : Depth of influence of instant settlement (m)

n : number of distribution loads

x : distance from a center of embankment (m)

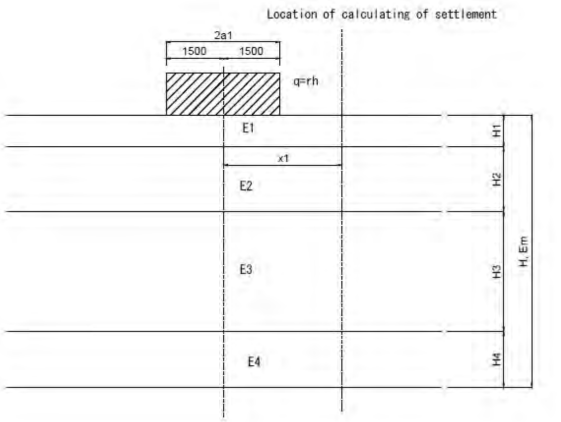


Fig.3 Location of Calculating

2-5 Coefficient of Reaction of Soil

Converted coefficient of reaction of soil is obtained by the following equation.

B=L

$$E_m = \frac{-\frac{1}{B+2h_n \cdot \tan\theta} + \frac{1}{B}}{\Sigma \frac{1}{E_i}\left(-\frac{1}{B+2h_i \cdot \tan\theta} + \frac{1}{B+2h_{i-1} \cdot \tan\theta}\right)}$$

Where, Em: converted coefficient of reaction of soil considering the change of soil (kN/m^2)

B : loading width (=3.00m)

L : depth of loading (=3.00m)

h_n : infected depth, 3 times loading width or more (20.8m)

Ei : coefficient of reaction of soil at ith layer

θ : distribution angle of load (=30°)

Layer	Thickness (m)	Depth (m)	Ei (kN/m^2)
1	1.80	1.80	4,900
2	1.50	3.30	4,200
3	4.00	7.30	2,100
4	3.00	10.30	8,400
5	5.50	15.80	3,500
6	5.00	20.80	7,000

2-6 Converted Coefficient of Reaction of Soil

Layer	Layer	Depth (m)	Equation	Total
Numerator			0.29630	0.29630
Denominator	1	1.80	0.0000278	
	2	3.30	0.0000119	
	3	7.30	0.0000283	
	4	10.30	0.0000024	
	5	15.80	0.0000057	
	6	20.80	0.0000014	0.0000775

Em = 0.29630 / 0.0000775 = 3,823 kN/m^2

2-7 Immediate Settlement

Location	Distance	Immediate Settlement (m)
Center	0.00m	0.012

2-8 Side Deformation

$$R_{ix} = \sum_{i=1}^{n} - \frac{(1+\nu)(1-2\nu)\cdot q_i \cdot a_i}{E_m \cdot \pi} \left(\frac{b_i}{2a_i} \log \frac{(a_i - x)^2 + b_i^2}{(a_i + x)^2 + b_i^2} + \frac{a_i - x}{a_i} \tan^{-1} \frac{b_i}{a_i - x} - \frac{a_i + x}{a_i} \tan^{-1} \frac{b_i}{a_i + x} \right)$$

Where, Rix: side deformation at x m from the center (m)

qi : surcharge of embankment (=28.8 kN/m^2)

Em: converted coefficient of reaction of soil considering the change of soil (= 3,823 kN/m^2)

ν : Poison's ratio (=0.30)

2a1: loading width (=3.00m)

2b1: depth of loading (=3.00m)

n : number of loads (=1)

x: distance from the center (m)

Location	Distance	Side Deformation (m)
Center	0.00m	0
Shoulder	1.50m	-0.0024

3. Calculation of Consolidation Settlement

3-1 Method of Calculation

We'd like to use Cc method.

$$S = \frac{C_c}{1+e_o} \times \log_{10} \frac{P_o + \Delta P}{P_o + q_o} \times H$$

Where S : consolidation settlement (m)

e0: initial void ratio

e1: void ratio after consolidation

Cc: compression index of clay

H : total depth of clay layer (m)

P0: effective earth covering due to pre-consolidation pressure(kN/m^2)

q0: Increment of vertical stress due to Pre-consolidation pressure (kN/m^2)

Δ p: stress increment due to surcharge of embankment (kN/m^2)

3-2 Calculation of Stress Increment

q = 18.0 * 1.60 = 28.8 kN/m^2

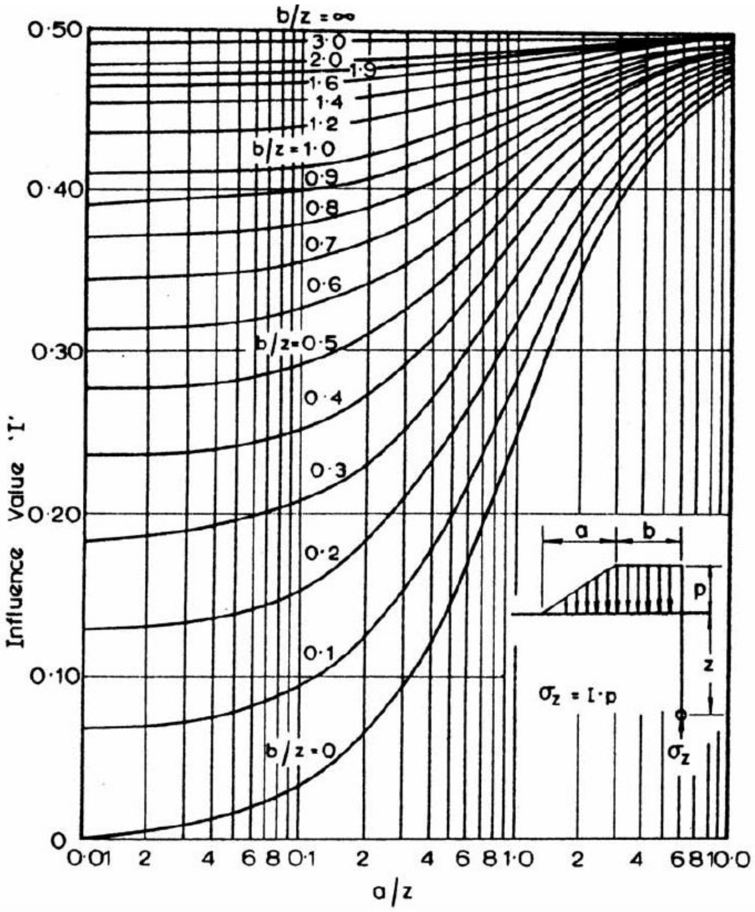


Fig.4 Osterberg Method

Layer	Geology	Depth (z)	a(m)	b(m)	a/z	b/z	I	Δ p (kN/m^2)
1	F	-	-	-	-	-	-	-
2	AC1	2.55	0	1.50	0	0.588	0.30	8.64
3	AC2	5.30	0	1.50	0	0283	0.18	5.18
4	AS1	-	-	-				-
5	DC1	13.05	0	1.50	0	0.115	0.08	2.30

3-3 Pre-consolidation Pressure

Layer	Geology	Calculation	p0 (kN/m^2)	po (kgf/cm^2)
2	AC1	0.90*17.0+0.75*17.3	28.28	0.288
3	AC2	0.90*17.0+1.50*17.3+2.00*8.3	57.85	0.590
5	DC1	0.90*17.0+1.50*17.3+4.00*8.3+3.00*9.0 +2.75*9.0	126.2	1.287

3-5 Calculation of Cc

Layer	pa (kgf/cm2)	ea	pb (kgf/cm2)	eb	Cc = (ea-eb)/Log(Pb/Pa)	Remarks
2	1.6	1.0319	12.8	0.6490	0.5412	
3	1.6	1.0319	12.8	0.6490	0.5412	
5	1.6	0.9299	12.8	0.6381	0.3231	

3-4 Consolidation Settlement

Layer	H(m)	Cc	eo	Po (kN/m^2)	Δ p (kN/m^2)	q0 (kN/m^2)	S (m)
2	1.50	0.5412	1.2576	28.28	8.64	0	0.042
3	4.00	0.5412	1.1847	57.85	5.18	0	0.036
5	5.50	0.3231	0.9505	126.2	2.30	0	0.007
Total							0.085

3-5 Consolidation Time

(1) Calculation of Cv

Converted Height

$$H' = 1.50 * \sqrt{(0.02044 / 0.03166)} + 4.00 = 5.21 \text{ m}$$

Layer	Pre-consolidation Pressure (kgf/cm^2)	Cv ($10^{-3} cm^2/sec$)	Cv (m^2/day)	d (m)	Settlement (m)
2	0.288	3.664	0.03166		

3	0.590	2.366	0.02044		
2+3			0.02044	H/2=2.61	0.078
5	1.287	11.718	0.10124	H=5.50	0.007

(2) Consolidation Time

$t = Tv*d^2 / Cv$

Where, t : time until consolidation index U

TV: time factor as shown below in response to consolidation index U

U(%)	10	20	30	40	50	60	70	80	90	100
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848	∞

Cv : coefficient of consolidation (m^2/day)

d : maximum discharge distance, both sides discharge d =H/2, one side discharge (m)

H : thickness of layer (m)

U : consolidation index (%), U = St / Sc

Layer	Consoli-dation (%)	10	20	30	40	50	60	70	80	90	100
2+3	Day	3	10	24	42	66	96	134	189	283	
	Settle-ment	0.009	0.017	0.026	0.035	0.044	0.052	0.061	0.070	0.078	0.087
5	Day	2	9	21	38	59	86	120	169	253	
	Settle-ment	0.001	0.002	0.002	0.003	0.004	0.005	0.006	0.006	0.007	0.008
Total	Settle-ment	0.010	0.019	0.028	0.038	0.048	0.057	0.067	0.076	0.085	

3-6 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.012 + 0.085 = 0.097 m < 0.50 m….OK

New Calculation of Settlement and Side Deformation (3)

Section 2 – STA 3+450

1. Design Condition (Form of Embankment)

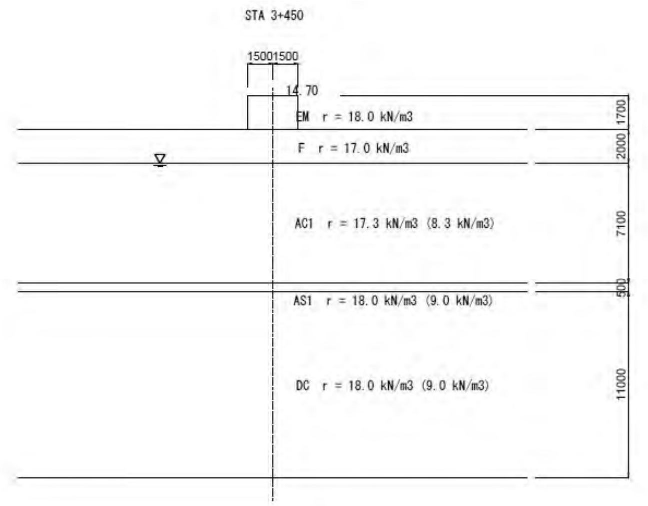


Fig.1 Embankment and Geological Condition

2. Calculation of Instant Settlement and Side deformation

2-1 Geological Condition

Layer	Geology	Thickness	Average N-Value	Modulus of Deformation of Soil Ei 700N (kN/m^3)	Remarks
1	F	2.00	3	2,100	Assumption
2	AC1	7.10	3	2,100	Assumption
3	AS1	0.50	9	6,300	
4	DC	11.00	6	4,200	

2-2 Calculation Points

1	Center	0m
2	Shoulder	1.50m

2-3　Load

$$q = 18.0 * 1.70 = 30.6 \text{ kN/m}^2$$

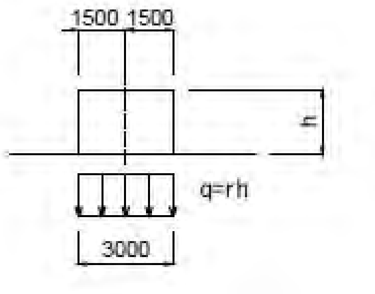


Fig.2　Load

2-4　Immediate Settlement

$$S_{ix} = \sum_{i=1}^{n} \frac{-3\, a_i \cdot q_i}{E_m \cdot \pi} \log\sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left[\, 1.0 - \frac{0.75}{\pi} \cdot \left[\, \left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right| \,\right]\,\right]$$

Where,　S_{ix} : immediate settlement of foundation at the point of xm in lateral direction (m)

qi : surcharge of embankment (=30.6 kN/m^2)

Em: converted coefficient of reaction of soil (kN/m^2)

2ai: width of load (=3.00m)

H : Depth of influence of instant settlement (m)

n : number of distribution loads

x : distance from a center of embankment (m)

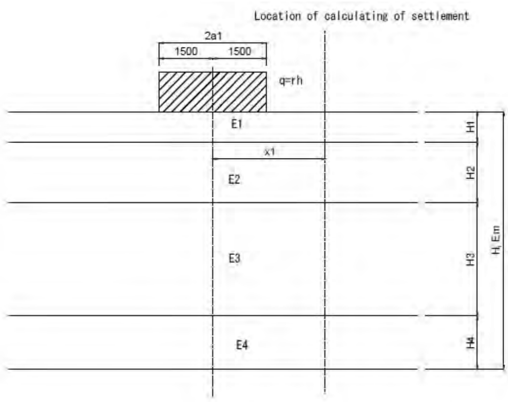


Fig.3 Location of Calculating

2-5 Coefficient of Reaction of Soil

Converted coefficient of reaction of soil is obtained by the following equation.

B=L

$$E_m = \frac{-\dfrac{1}{B+2h_n \cdot \tan\theta} + \dfrac{1}{B}}{\Sigma \dfrac{1}{E_i}\left(-\dfrac{1}{B+2h_i \cdot \tan\theta} + \dfrac{1}{B+2h_{i-1} \cdot \tan\theta}\right)}$$

Where, Em: converted coefficient of reaction of soil considering the change of soil (kN/m^2)

B : loading width (=3.0m)

L : depth of loading (=3.00m)

h_n : infected depth, 3 times loading width or more (20.60m)

Ei : coefficient of reaction of soil at i^{th} layer

θ : distribution angle of load (=30°)

Layer	Thickness (m)	Depth (m)	Ei (kN/m^2)
1	2.00	2.00	2,100
2	7.10	9.10	2,100
3	0.50	9.60	6,300
4	11.00	20.60	4,200

2-6 Converted Coefficient of Reaction of Soil

Layer	Layer	Depth (m)	Equation	Total
Numerator			0.29598	0.29598
Denominator	1	2.00	0.0000690	
	2	9.10	0.0000544	
	3	9.60	0.0000005	
	4	20.60	0.0000080	0.0001319

Em = 0.29598 / 0.0001319 = 2,244 kN/m^2

2-7 Immediate Settlement

Location	Distance	Immediate Settlement (m)
Center	0.00m	0.022

2-8 Side Deformation

$$R_{ix} = \sum_{i=1}^{n} - \frac{(1+\nu)(1-2\nu) \cdot q_i \cdot a_i}{E_m \cdot \pi} \left(\frac{b_i}{2a_i} \log \frac{(a_i - x)^2 + b_i^2}{(a_i + x)^2 + b_i^2} + \frac{a_i - x}{a_i} \tan^{-1} \frac{b_i}{a_i - x} - \frac{a_i + x}{a_i} \tan^{-1} \frac{b_i}{a_i + x} \right)$$

Where, Rix: side deformation at x m from the center (m)

qi : surcharge of embankment (=30.6 kN/m^2)

Em: converted coefficient of reaction of soil considering the change of soil (= 2,244 kN/m^2)

ν : Poison's ratio (=0.30)

2a1: loading width (= 3.00m)

2b1: depth of loading (=3.00m)

n : number of loads (=1)

x: distance from the center (m)

Location	Distance	Side Deformation (m)
Center	0.00m	0
Shoulder	1.50m	-0.0043

3. Calculation of Consolidation Settlement

3-1 Method of Calculation

We'd like to use Cc method.

$$S = \frac{C_c}{1 + e_o} \times \log_{10} \frac{P_o + \Delta P}{P_o + q_o} \times H$$

Where S : consolidation settlement (m)

e0: initial void ratio

e1: void ratio after consolidation

Cc: compression index of clay

H : total depth of clay layer (m)

P0: effective earth covering due to pre-consolidation pressure(kN/m^2)

q0: Increment of vertical stress due to Pre-consolidation pressure (kN/m^2)

Δ p: stress increment due to surcharge of embankment (kN/m^2)

3-2 Calculation of Stress Increment

q = 18.0 * 1.70 = 30.6 kN/m^2

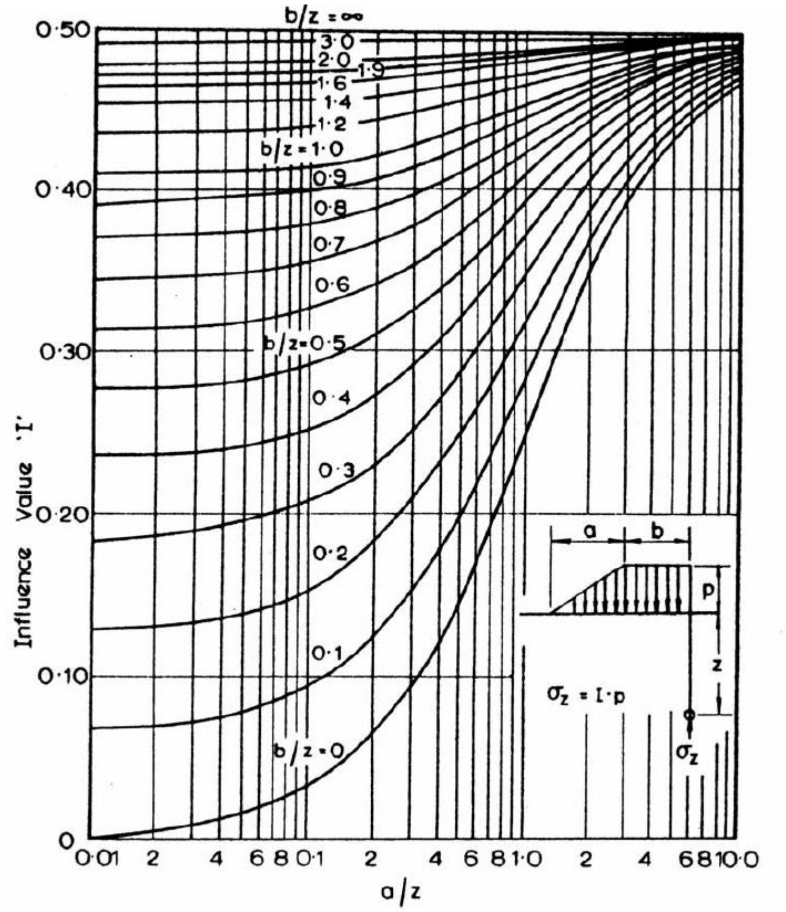


Fig.4 Osterberg Method

Layer	Geology	Depth (z)	a(m)	b(m)	a/z	b/z	I	Δ p (kN/m^2)
1	F							
2	AC	5.55	0	1.50	0	0.270	0.17	5.20
3	AS1	-	-	-	-	-	-	-
4	DC	15.10	0	1.50	0	0.099	0.07	2.14

3-3 Pre-consolidation Pressure

Layer	Geology	Calculation	p0 (kN/m^2)	po (kgf/cm^2)
2	AC1	2.00*17.0 + 3.55*8.3	63.47	0.647
4	DC	2.00*17.0+7.10*8.3+0.50*9.0+5.5*9.0	146.93	1.499

3-5 Calculation of Cc

Layer	pa (kgf/cm^2)	ea	pb (kgf/cm^2)	eb	Cc = (ea-eb)/Log(Pb/Pa)	Remarks
2	1.6	1.2443	12.8	0.7159	0.5851	
4	1.6	0.9299	12.8	0.6381	0.3231	

3-4 Consolidation Settlement

Layer	H(m)	Cc	eo	Po (kN/m^2)	Δ p (kN/m^2)	q0 (kN/m^2)	S (m)
2	7.10	0.5851	1.4353	63.47	5.20	0	0.063
4	11.0	0.3231	0.9366	146.93	2.14	0	0.012
Total							0.075

3-5 Consolidation Time

(1) Calculation of Cv

Layer	Pre-consolidation Pressure (kgf/cm^2)	Cv ($10^{-3}cm^2$/sec)	Cv (m^2/day)	d (m)	Settlement (m)
2	0.647	1.736	0.01500	H/2=3.55	0.063
4	1.499	11.981	0.10352	H=11.00	0.012

(2) Consolidation Time

$$t = Tv*d^2 / Cv$$

Where, t : time until consolidation index U

TV: time factor as shown below in response to consolidation index U

U(%)	10	20	30	40	50	60	70	80	90	100
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848	∞

Cv : coefficient of consolidation (m^2/day)

d : maximum discharge distance, both sides discharge d =H/2, one side discharge (m)

H : thickness of layer (m)

U : consolidation index (%), U = St / Sc

Layer	Consoli-dation (%)	10	20	30	40	50	60	70	80	90	100
2	Day	7	26	60	106	166	241	339	476	712	
	Settle-ment	0.007	0.014	0.021	0.028	0.035	0.042	0.049	0.056	0.063	0.070
4	Day	9	36	83	147	230	335	471	663	991	
	Settle-ment	0.001	0.003	0.004	0.005	0.007	0.008	0.009	0.010	0.012	0.013
Total	Settle-ment	0.008	0.017	0.025	0.033	0.042	0.050	0.058	0.066	0.075	

3-6 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.022 + 0.075 = 0.097 m < 0.50 m….OK

New Calculation of Settlement and Side Deformation (4)

Section 3 – STA 4+050

1. Design Condition (Form of Embankment)

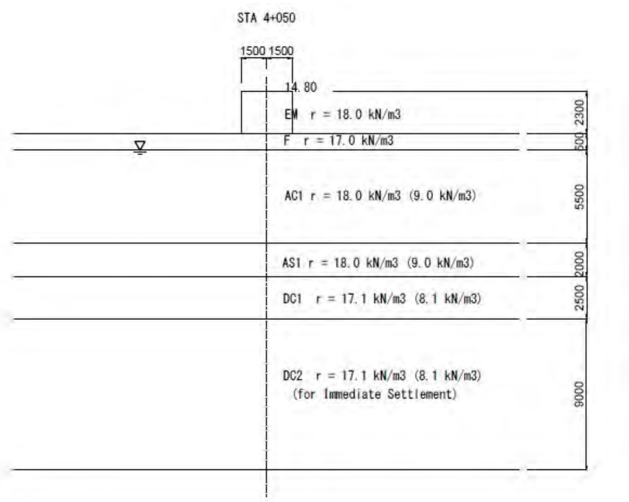


Fig.1 Embankment and Geological Condition

2. Calculation of Instant Settlement and Side deformation

2-1 Geological Condition

Layer	Geology	Thickness	Average N-Value	Modulus of Deformation of Soil Ei 700N (kN/m^3)	Remarks
1	F	0.50	3	2,100	
2	AC1	4.50	3	2,100	
3	AC2	1.00	12	8,400	
4	AS1	2.00	22	15,400	
5	DC1	2.50	9	6,300	
6	DC2	9.00	26	18,200	

2-2 Calculation Points

1	Center	0m
2	Shoulder	1.50m

2-3 Load

q = 18.0 * 2.30 = 41.4 kN/m^2

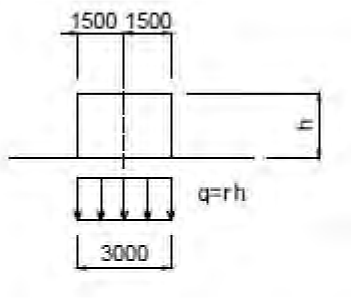


Fig.2 Load

2-4 Immediate Settlement

$$S_{ix} = \sum_{i=1}^{n} \frac{-3 a_i \cdot q_i}{E_m \cdot \pi} \log\sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left[1.0 - \frac{0.75}{\pi} \cdot \left\{ \left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right| \right\} \right]$$

Where, S_{ix} : immediate settlement of foundation at the point of xm in lateral direction (m)

qi : surcharge of embankment (=41.4 kN/m^2)

Em: converted coefficient of reaction of soil (kN/m^2)

2ai: width of load (=3.0m)

H : Depth of influence of instant settlement (m)

n : number of distribution loads

x : distance from a center of embankment (m)

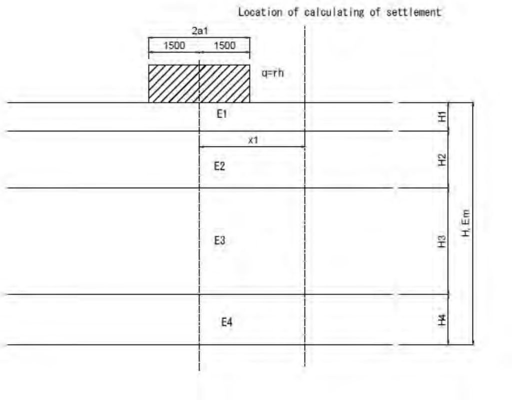


Fig.3 Location of Calculating

2-5 Coefficient of Reaction of Soil

Converted coefficient of reaction of soil is obtained by the following equation.

B=L

$$E_m = \frac{-\dfrac{1}{B+2h_n \cdot \tan\theta} + \dfrac{1}{B}}{\Sigma \dfrac{1}{E_i}\left(-\dfrac{1}{B+2h_i \cdot \tan\theta} + \dfrac{1}{B+2h_{i-1} \cdot \tan\theta}\right)}$$

Where, Em: converted coefficient of reaction of soil considering the change of soil (kN/m^2)

B : loading width (=3.00m)

L : depth of loading (=3.00m)

h_n : infected depth, 3 times loading width or more (19.50m)

Ei : coefficient of reaction of soil at i^{th} layer

θ : distribution angle of load (=30°)

Layer	Thickness (m)	Depth (m)	Ei (kN/m^2)
1	0.50	0.50	2,100
2	4.50	5.00	2,100
3	1.00	6.00	8,400
4	2.00	8.00	15,400
5	2.50	10.50	6,300
6	9.00	19.50	18,200

2-6 Converted Coefficient of Reaction of Soil

Layer	Layer	Depth (m)	Equation	Total
Numerator			0.05689	0.055689
Denominator	1	0.50	0.0000019	
	2	5.00	0.0000117	
	3	6.00	0.0000004	
	4	8.00	0.0000004	
	5	10.50	0.0000009	
	6	19.50	0.0000007	0.0000160

Em = 0.05569 / 0.0000160 =3,481 kN/m^2

2-7 Immediate Settlement

Location	Distance	Immediate Settlement (m)
Center	0.00m	0.019

2-8 Side Deformation

$$R_{ix}=\sum_{i=1}^{n}-\frac{(1+\nu)(1-2\nu)\cdot q_i\cdot a_i}{E_m\cdot \pi}\left(\frac{b_i}{2a_i}\log\frac{(a_i-x)^2+b_i^2}{(a_i+x)^2+b_i^2}+\frac{a_i-x}{a_i}\tan^{-1}\frac{b_i}{a_i-x}-\frac{a_i+x}{a_i}\tan^{-1}\frac{b_i}{a_i+x}\right)$$

Where, Rix: side deformation at x m from the center (m)

qi : surcharge of embankment (=41.4 kN/m^2)

Em: converted coefficient of reaction of soil considering the change of soil (= 3,481 kN/m^2)

ν : Poison’s ratio (=0.30)

2a1: loading width (=3.00m)

2b1: depth of loading (=3.00m)

n : number of loads (=1)

x: distance from the center (m)

Location	Distance	Side Deformation (m)
Center	0.00m	0
Shoulder	1.50m	-0.0038

3. Calculation of Consolidation Settlement

3-1 Method of Calculation

We'd like to use Cc method.

$$S = \frac{C_c}{1+e_o} \times \log_{10} \frac{P_o + \Delta P}{P_o + q_o} \times H$$

Where S : consolidation settlement (m)

e0: initial void ratio

e1: void ratio after consolidation

Cc: compression index of clay

H : total depth of clay layer (m)

P0: effective earth covering due to pre-consolidation pressure(kN/m^2)

q0: Increment of vertical stress due to Pre-consolidation pressure (kN/m^2)

Δp: stress increment due to surcharge of embankment (kN/m^2)

3-2 Calculation of Stress Increment

q = 18.0 * 2.30 = 41.4 kN/m^2

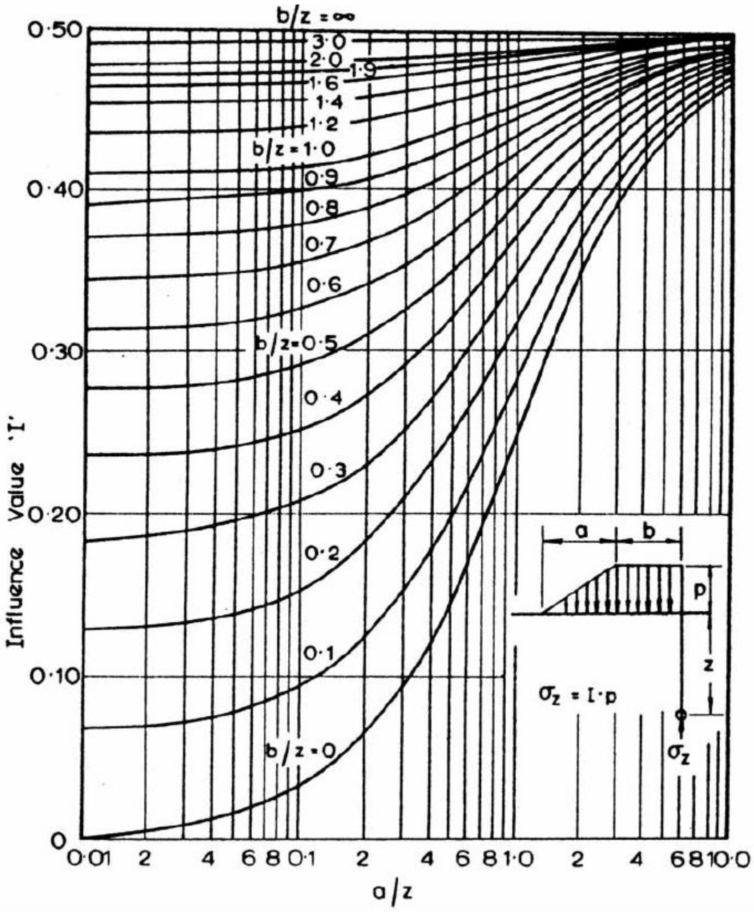


Fig.4 Osterberg Method

Layer	Geology	Depth (z)	a(m)	b(m)	a/z	b/z	I	Δp (kN/m^2)
1	F	-	-	-	-	-	-	-
2	AC1	3.25	0	1.50	0	0.462	0.27	11.18
3	AC2	5.50	0	1.50	0	0.273	0.17	7.04
4	AS1	-	-	-				-
5	DC1	9.25	0	1.50	0	0.162	0.11	4.55

3-3 Pre-consolidation Pressure

Layer	Geology	Calculation	p0 (kN/m^2)	po (kgf/cm^2)
2	AC1	0.50*17.0+2.25*9.0	28.75	0.293
3	AC2	0.50*17.0+4.50*9.0+0.50*9.0	53.50	0.546
5	DC1	0.50*17.0+4.50*9.0+1.00*9.0+2.00*9.0 +1.25*8.1	86.13	0.879

3-5 Calculation of Cc

Layer	pa (kgf/cm2)	ea	pb (kgf/cm^2)	eb	Cc = (ea-eb)/Log(Pb/Pa)	Remarks
2	1.6	1.1346	12.8	0.7160	0.4635	
3	1.6	1.1346	12.8	0.7160	0.4635	
5	1.6	1.2378	12.8	0.8404	0.4400	

3-4 Consolidation Settlement

Layer	H(m)	Cc	eo	Po (kN/m^2)	Δ p (kN/m^2)	q0 (kN/m^2)	S (m)
2	4.50	0.4635	1.4139	28.75	11.18	0	0.123
3	1.00	0.4635	1.3434	53.50	7.04	0	0.011
5	2.50	0.4400	1.2825	86.13	4.55	0	0.011
Total							0.145

3-5 Consolidation Time

(1) Calculation of Cv

Converted Height

$$H' = 4.50 * \sqrt{(0.04218 / 0.05775)} + 1.00 = 4.85 \text{ m}$$

Layer	Pre-consolidation Pressure (kgf/cm^2)	Cv ($10^{-3}cm^2$/sec)	Cv (m^2/day)	d (m)	Settlement (m)
2	0.293	6.684	0.05775		

3	0.546	4.882	0.04218		
2+3			0.044218	H/2=2.43	0.134
5	0.879	6.615	0.05715	H=3.50	0.011

(2) Consolidation Time

$t = Tv*d^2 / Cv$

Where, t : time until consolidation index U

TV: time factor as shown below in response to consolidation index U

U(%)	10	20	30	40	50	60	70	80	90	100
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848	∞

Cv : coefficient of consolidation (m^2/day)

d : maximum discharge distance, both sides discharge d =H/2, one side discharge (m)

H : thickness of layer (m)

U : consolidation index (%), U = St / Sc

Layer	Consoli-dation (%)	10	20	30	40	50	60	70	80	90	100
2+3	Day	1	4	9	17	26	38	54	76	113	
	Settle-ment	0.015	0.030	0.045	0.060	0.075	0.089	0.104	0.119	0.134	0.149
5	Day	2	7	15	27	42	62	86	122	182	
	Settle-ment	0.001	0.002	0.004	0.005	0.006	0.007	0.008	0.010	0.011	0.012
Total	Settle-ment	0.016	0.032	0.049	0.065	0.081	0.096	0.112	0.129	0.145	

3-6 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.019 + 0.145 = 0.164 m < 0.50 m….OK

New Calculation of Settlement and Side Deformation (5)

Section 3 – STA 4+400

1. Design Condition (Form of Embankment)

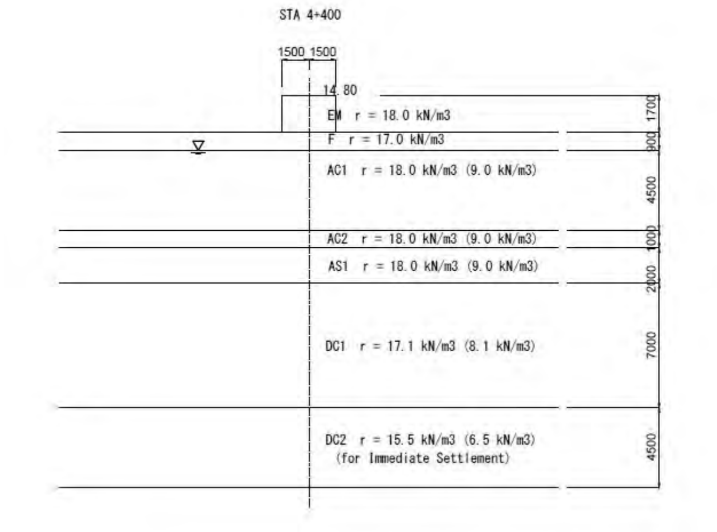


Fig.1 Embankment and Geological Condition

2. Calculation of Instant Settlement and Side deformation

2-1 Geological Condition

Layer	Geology	Thickness	Average N-Value	Modulus of Deformation of Soil Ei 700N (kN/m^3)	Remarks
1	F	0.90	3	2,100	
2	AC1	4.50	4	2,800	
3	AC2	1.00	10	7,000	
4	AS1	2.00	9	6,300	
5	DC1	7.00	11	7,700	
6	DC2	4.50	21	14,700	

2-2 Calculation Points

1	Center	0m
2	Shoulder	1.50m

2-3 Load

$q = 18.0 * 1.70 = 30.6\ kN/m^2$

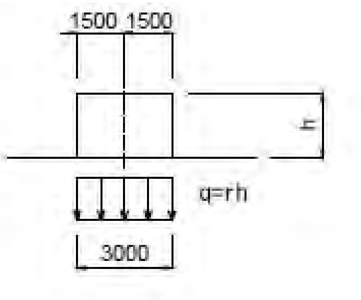


Fig.2 Load

2-4 Immediate Settlement

$$S_{ix} = \sum_{i=1}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log\sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left[1.0 - \frac{0.75}{\pi} \cdot \left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right| \right] \right]$$

Where, S_{ix} : immediate settlement of foundation at the point of xm in lateral direction (m)

qi : surcharge of embankment (=30.6 kN/m^2)

Em: converted coefficient of reaction of soil (kN/m^2)

2ai: width of load (=3.00m)

H : Depth of influence of instant settlement (m)

n : number of distribution loads

x : distance from a center of embankment (m)

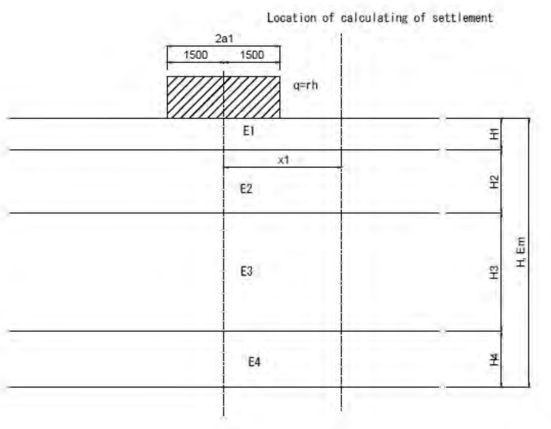


Fig.3 Location of Calculating

2-5 Coefficient of Reaction of Soil

Converted coefficient of reaction of soil is obtained by the following equation.

B=L

$$E_m = \frac{-\dfrac{1}{B+2h_n \cdot \tan\theta} + \dfrac{1}{B}}{\Sigma \dfrac{1}{E_i}\left(-\dfrac{1}{B+2h_i \cdot \tan\theta} + \dfrac{1}{B+2h_{i-1} \cdot \tan\theta}\right)}$$

Where, Em: converted coefficient of reaction of soil considering the change of soil (kN/m^2)

B : loading width (=3.00m)

L : depth of loading (=3.00m)

h_n : infected depth, 3 times loading width or more (19.90m)

Ei : coefficient of reaction of soil at i^{th} layer

θ : distribution angle of load (=30°)

Layer	Thickness (m)	Depth (m)	Ei (kN/m^2)
1	0.90	0.90	2,100
2	4.50	5.40	2,800
3	1.00	6.40	7,000
4	2.00	8.40	6,300
5	7.00	15.40	7,700
6	4.50	19.90	14,700

2-6 Converted Coefficient of Reaction of Soil

Layer	Layer	Depth (m)	Equation	Total
Numerator			0.29482	0.29482
Denominator	1	0.90	0.0000408	
	2	5.40	0.0000497	
	3	6.40	0.0000017	
	4	8.40	0.0000028	
	5	15.40	0.0000040	
	6	19.90	0.0000007	0.0000997

Em = 0.29482 / 0.0000997 =2,957 kN/m^2

2-7 Immediate Settlement

Location	Distance	Immediate Settlement (m)
Center	0.00m	0.017

2-8 Side Deformation

$$R_{ix}=\sum_{i=1}^{n}-\frac{(1+\nu)(1-2\nu)\cdot q_i\cdot a_i}{E_m\cdot\pi}\left(\frac{b_i}{2a_i}\log\frac{(a_i-x)^2+b_i^2}{(a_i+x)^2+b_i^2}+\frac{a_i-x}{a_i}\tan^{-1}\frac{b_i}{a_i-x}-\frac{a_i+x}{a_i}\tan^{-1}\frac{b_i}{a_i+x}\right)$$

Where, Rix: side deformation at x m from the center (m)

qi : surcharge of embankment (=30.6 kN/m^2)

Em: converted coefficient of reaction of soil considering the change of soil (= 2,957 kN/m^2)

ν : Poison's ratio (=0.30)

2a1: loading width (=3.00m)

2b1: depth of loading (=3.00m)

n : number of loads (=1)

x: distance from the center (m)

Location	Distance	Side Deformation (m)
Center	0.00m	0
Shoulder	1.50m	-0.0033

3. Calculation of Consolidation Settlement

3-1 Method of Calculation

We'd like to use Cc method.

$$S = \frac{C_c}{1+e_o} \times \log_{10} \frac{P_o + \Delta P}{P_o + q_o} \times H$$

Where S : consolidation settlement (m)

e0: initial void ratio

e1: void ratio after consolidation

Cc: compression index of clay

H : total depth of clay layer (m)

P0: effective earth covering due to pre-consolidation pressure(kN/m^2)

q0: Increment of vertical stress due to Pre-consolidation pressure (kN/m^2)

Δ p: stress increment due to surcharge of embankment (kN/m^2)

3-2 Calculation of Stress Increment

q = 18.0 * 1.70 = 30.6 kN/m^2

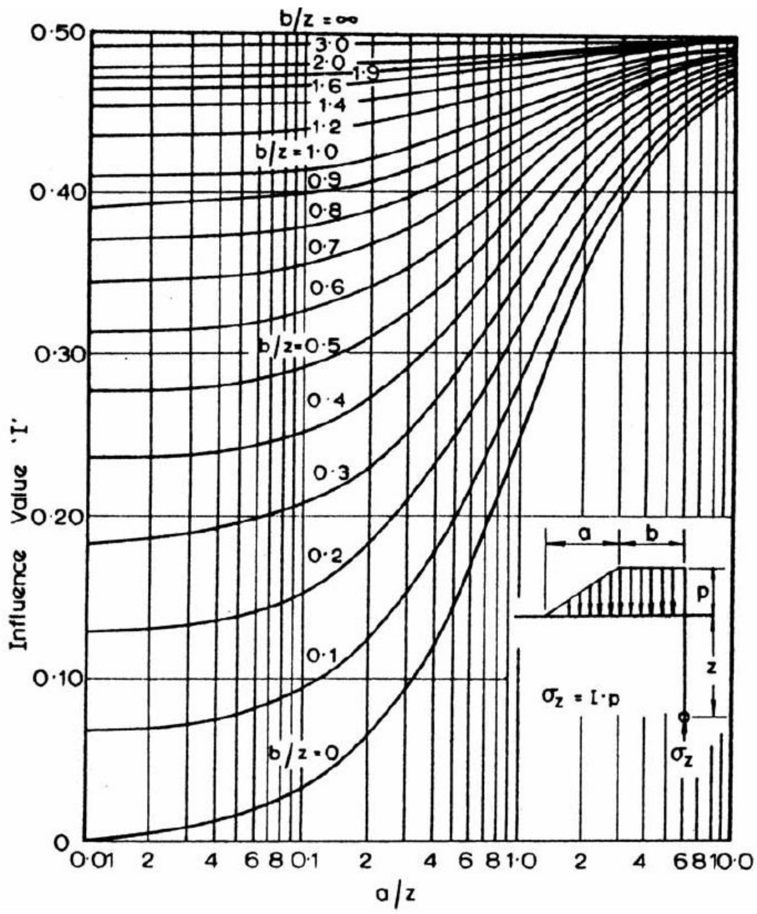


Fig.4 Osterberg Method

Layer	Geology	Depth (z)	a(m)	b(m)	a/z	b/z	I	Δ p (kN/m^2)
1	F	-	-	-	-	-	-	-
2	AC1	3.15	0	1.50	0	0.476	0.27	8.26
3	AC2	5.90	0	1.50	0	0.254	0.16	4.90
4	AS1	-	-	-				-
5	DC1	11.90	0	1.50	0	0.126	0.08	2.45

3-3 Pre-consolidation Pressure

Layer	Geology	Calculation	p0 (kN/m^2)	po (kgf/cm^2)
2	AC1	0.90*17.0+2.25*9.0	35.55	0.363
3	AC2	0.90*17.0+4.50*9.0+0.50*9.0	60.30	0.615
5	DC1	0.90*17.0+4.50*9.0+1.0*9.0+2.0*9.0 +3.50*8.10	111.15	1.134

3-5 Calculation of Cc

Layer	pa (kgf/cm^2)	ea	pb (kgf/cm^2)	eb	Cc = (ea-eb)/Log(Pb/Pa)	Remarks
2	1.6	1.108	12.8	0.687	0.4662	
3	1.6	1.108	12.8	0.687	0.4662	
5	1.6	0.4008	12.8	0.332	0.0762	

3-4 Consolidation Settlement

Layer	H(m)	Cc	eo	Po (kN/m^2)	Δ p (kN/m^2)	q0 (kN/m^2)	S (m)
2	4.50	0.4662	1.327	35.55	8.26	0	0.082
3	1.00	0.4662	1.271	60.30	4.90	0	0.007
5	7.00	0.0762	0.411	111.15	2.45	0	0.004
Total							0.093

3-5 Consolidation Time

(1) Calculation of Cv

Converted Height

$$H' = 4.50* \sqrt{(0.06351 / 0.03811)} + 1.00 = 6.81 \text{ m}$$

Layer	Pre-consolidation Pressure (kgf/cm^2)	Cv ($10^{-3} cm^2/sec$)	Cv (m^2/day)	d (m)	Settlement (m)
2	0.363	4.411	0.03811		
3	0.615	7.351	0.06351		
2+3			0.06351	H/2=3.41	0.089
5	1.134	10.171	0.08788	H=7.00	0.004

(2) Consolidation Time

$t = Tv*d^2 / Cv$

Where, t : time until consolidation index U

TV: time factor as shown below in response to consolidation index U

U(%)	10	20	30	40	50	60	70	80	90	100
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848	∞

Cv : coefficient of consolidation (m^2/day)

d : maximum discharge distance, both sides discharge d =H/2, one side discharge (m)

H : thickness of layer (m)

U : consolidation index (%), U = St / Sc

Layer	Consoli-dation (%)	10	20	30	40	50	60	70	80	90	100
2+3	Day	1	6	13	23	36	53	74	104	155	
	Settle-ment	0.010	0.020	0.030	0.040	0.050	0.059	0.069	0.079	0.089	0.099
5	Day	4	17	40	70	110	160	225	316	473	
	Settle-ment	0	0.001	0.001	0.002	0.002	0.002	0.003	0.003	0.004	0.004
Total	Settle-ment	0.010	0.021	0.031	0.042	0.052	0.061	0.072	0.082	0.093	

3-6 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.017 + 0.093 = 0.110 m < 0.50 m….OK

CHAPTER 2 CALCULATION OF CONCRETE BLOCK RETAINING WALL

The detailed calculation of concrete block wall is indicated from the following page.

Calculation of Concrete Block Wall H = 1.00 m

1. Design Condition

    Height: H=1.00m

    Calculation Height: h = 1.00 + 0.30 (Base) = 1.30m

    Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

    Inclination: 0.5 : 1

    Uniform Load: 10 kN/m^2

    Backfill Earth: r = 18 kN/m^3, ϕ =30°

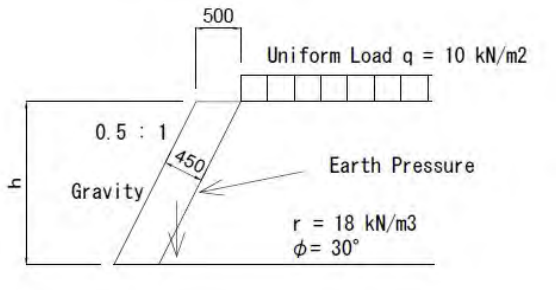


Fig.1 Design Condition

2. Study of Stability against Turn Over

    A resultant force has to be located out behind of a 1/3 range of wall area of center.

    A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma\cos\theta}{6\gamma_h b}h^2 + \left\{\frac{K_a q\cos^2\theta/\cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2}\right\}h$$

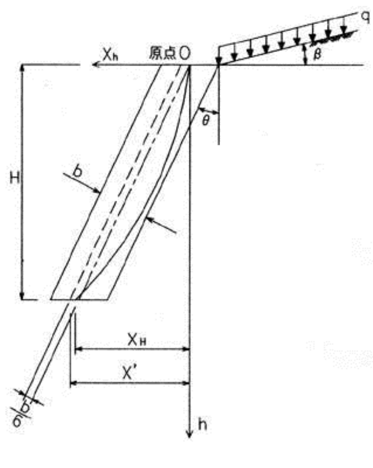


Fig.2 Diagram of Force Line

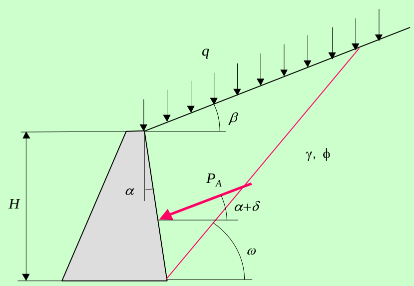
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	$H=$	4	m	
壁面傾斜角	$\alpha=$	-26.6	度	-0.46426 rad
地表載荷重	$q=$	10	kN/m^2	
設計水平震度	$k_H=$	0		
盛土傾斜角	$\beta=$	0	度	0 rad
単位体積重量	$\gamma=$	18	kN/m^3	
せん断抵抗角	$\phi=$	30	度	0.523599 rad
壁面摩擦角	$\delta=$	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{\frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\frac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}}-\sin(\phi+\delta+\alpha-\beta)}\right\}+\beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, $\omega=$ 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + qK_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m^3)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m^2)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

$Ka*r*\cos\theta*h^2 / (6*r*b) = 0.139*18.0*\cos 26.6*1.30^2 / (6*18.0*0.45) = 0.078$ m

$(Ka*q*\cos\theta/(2*r*b)+\tan\theta/2)*h$

$= (0.139*10.0*\cos 26.6/ (2*18.0*0.45)+\tan 26.6 / 2)*1.30 = 0.425$m

$Xh = 0.078 + 0.425 = 0.503$m $< 1.30 / 2 + 1/6 *0.50 = 0.733$m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60

Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)

$PH = 1/2 * r * H * Ka$

$= 1/2 * 18.0 * 1.30^2 * 0.139$

$= 2.11$ kN/m

3.3 Weight of Wall

(1) Weight of Wall

$W1 = 0.50 * 1.00 * 23.5$

$= 11.75$ kN/m

(2) Base Concrete

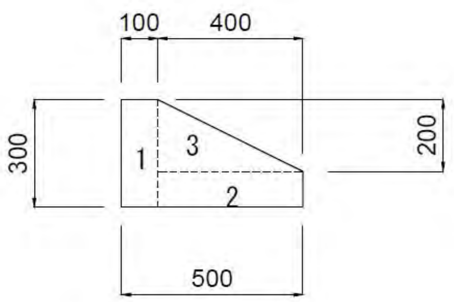


Fig.4 Base Concrete

W2 = (0.10*0.30 + 0.40*0.10 + 1/2 * 0.40*0.20) * 23.5 = 2.59 kN/m

(3) Total Weight

W = W1 + W2 = 11.75 + 2.59 = 14.34 kN/m

3.1 Safety Factor for Sliding

Fs = 14.34 * 0.60 / 2.11 = 4.08 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 0.503 m

E = 0.503 – 1.30 / 2 = -0.147 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 14.34 kN/m ＜ 31.5 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1.0 kN/m^2)
q	=	ground surface surcharge (kN/m^2)
	=	$\gamma_2 \cdot$ Df = 0.60m * 17.00 = 10.20 kN/m^2
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)
k	=	coefficient (1+0.3 x Df’/B = 1+0.3*0.60 / 0.50 = 1.36)

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Minimum N value in F's layer = 5

$\phi = (15 * N)^{1/2} + 15 = 24.0^{\circ}$

Nc = 19.5

Nq = 9.8

Nr = 6.0

$Qu = A' (\alpha * k * c * Nc + k * q * Nq + 1/2 * r1 * \beta * B * Nr)$

$= 0.50 (1.0*1.36*1.0*19.5 + 1.36 * 10.20*9.8 + 1/2 * 17.0*1.0*0.50*6.0)$

$= 94.48$ kN/m

$Qa = 1/3 * Qu = 1/3 * 94.48 = 31.5$ kN/m

Calculation of Concrete Block Wall H = 1.25 m

1. Design Condition

   Height: H=1.25m

   Calculation Height: h = 1.25 + 0.30 (Base) = 1.55m

   Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

   Inclination: 0.5 : 1

   Uniform Load: 10 kN/m^2

   Backfill Earth: r = 18 kN/m^3, ϕ =30°

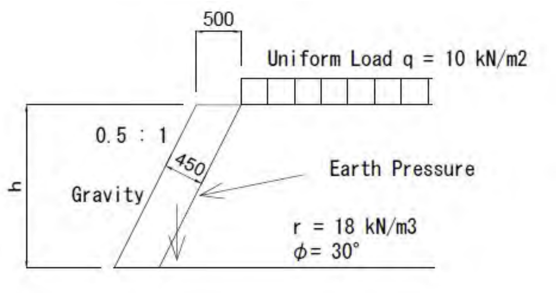


Fig.1　Design Condition

2. Study of Stability against Turn Over

   A resultant force has to be located out behind of a 1/3 range of wall area of center.

   A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma \cos\theta}{6\gamma_h b} h^2 + \left\{ \frac{K_a q \cos^2\theta / \cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2} \right\} h$$

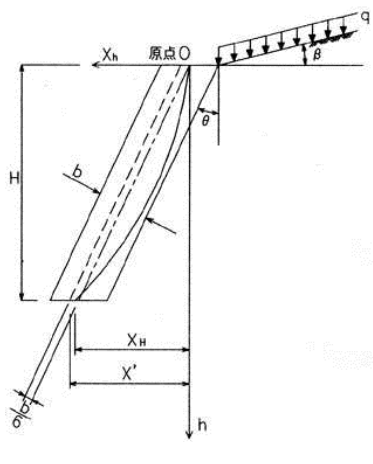


Fig.2　Diagram of Force Line

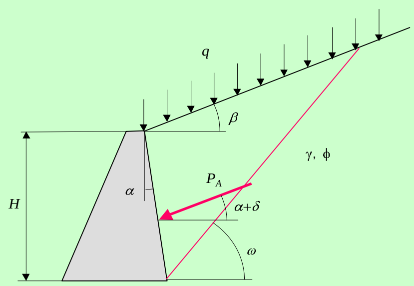
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	H=	4	m	
壁面傾斜角	α =	-26.6	度	-0.46426 rad
地表載荷重	q=	10	kN/m^2	
設計水平震度	k_H =	0		
盛土傾斜角	β =	0	度	0 rad
単位体積重量	γ =	18	kN/m^3	
せん断抵抗角	ϕ =	30	度	0.523599 rad
壁面摩擦角	δ =	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{ \frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\dfrac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}} - \sin(\phi+\delta+\alpha-\beta)} \right\} + \beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)$= 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)$= 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)$= 0.447077

$\sin(\phi+\delta+\alpha-\beta)$= 0.397148

したがって, ω= 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)$= 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)$= 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)$= 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)$= 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + qK_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m^3)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m^2)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

$Ka*r*\cos\theta*h^2 / (6*r*b) = 0.139*18.0*\cos 26.6 *1.55^2 / (6*18.0*0.45) = 0.110$ m

$(Ka*q*\cos\theta/(2*r*b)+\tan\theta/2)*h$

$= (0.139*10.0*\cos 26.6/ (2*18.0*0.45)+\tan 26.6 / 2)*1.55 = 0.506$m

$Xh = 0.110 + 0.506 = 0.616$m $< 1.55 / 2 + 1/6 *0.50 = 0.858$m....OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60

Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)

$PH = 1/2 * r * H * Ka$

$= 1/2 * 18.0 * 1.55^2 * 0.139$

$= 3.01$ kN/m

3.3 Weight of Wall

(1) Weight of Wall

$W1 = 0.50 * 1.25 * 23.5$

$= 14.68$ kN/m

(2) Base Concrete

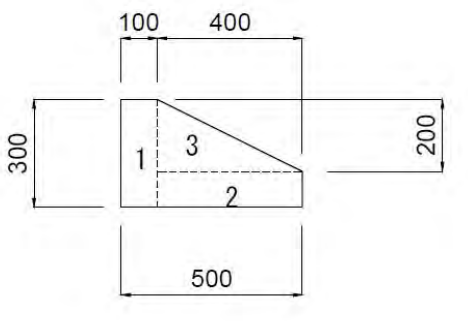


Fig.4 Base Concrete

$W2 = (0.10*0.30 + 0.40*0.10 + 1/2 * 0.40*0.20) * 23.5 = 2.59$ kN/m

(3) Total Weight

W = W1 + W2 = 14.68 + 2.59 = 17.27 kN/m

3.4 Safety Factor for Sliding

Fs = 17.27 * 0.60 / 3.01 = 3.44 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 0.737 m

E = 0.616 – 1.55 / 2 = -0.159 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 17.27 kN/m ＜ 31.5 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1.0 kN/m^2)
q	=	ground surface surcharge (kN/m^2)
	=	$\gamma_2 \cdot$ Df = 0.60m * 17.00 = 10.20 kN/m^2
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)
k	=	coefficient (1+0.3 x Df’/B = 1+0.3*0.60 / 0.50 = 1.36)

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^{\circ}$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.50 (1.0*1.36*1.0*19.5 + 1.36 * 10.20*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 94.48 kN/m

Qa = 1/3 * Qu = 1/3 * 94.48 = 31.5 kN/m

Calculation of Concrete Block Wall H = 1.50 m

1. Design Condition

    Height: H=1.50m

    Calculation Height: h = 1.50 + 0.30 (Base) = 1.80m

    Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

    Inclination: 0.5 : 1

    Uniform Load: 10 kN/m^2

    Backfill Earth: r = 18 kN/m^3, ϕ =30°

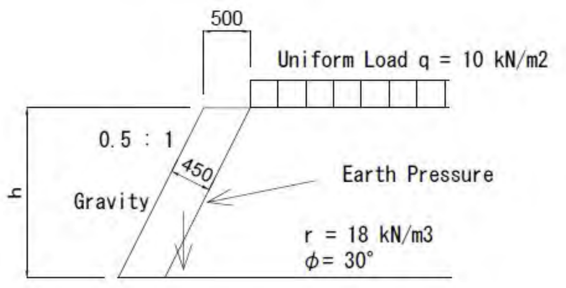


Fig.1 Design Condition

2. Study of Stability against Turn Over

    A resultant force has to be located out behind of a 1/3 range of wall area of center.

    A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma\cos\theta}{6\gamma_h b}h^2 + \left\{\frac{K_a q\cos^2\theta/\cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2}\right\}h$$

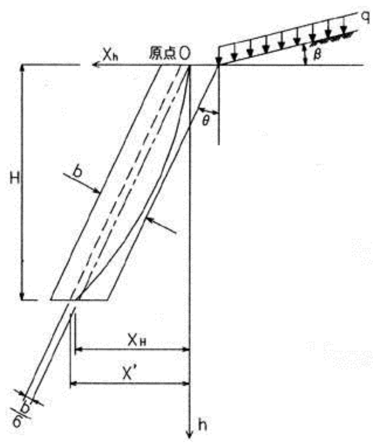


Fig.2 Diagram of Force Line

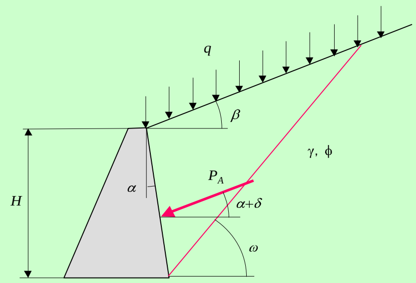
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	H=	4	m	
壁面傾斜角	α =	-26.6	度	-0.46426 rad
地表載荷重	q=	10	kN/m^2	
設計水平震度	k_H =	0		
盛土傾斜角	β =	0	度	0 rad
単位体積重量	γ =	18	kN/m^3	
せん断抵抗角	ϕ =	30	度	0.523599 rad
壁面摩擦角	δ =	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{ \frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\frac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}} - \sin(\phi+\delta+\alpha-\beta)} \right\} + \beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, ω = 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + q K_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m^3)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m^2)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

$Ka*r*\cos\theta*h^2 / (6*r*b) = 0.139*18.0*\cos 26.6*1.80^2 / (6*18.0*0.45) = 0.149$ m

$(Ka*q*\cos\theta/(2*r*b)+\tan\theta/2)*h$

$= (0.139*10.0*\cos 26.6/ (2*18.0*0.45)+\tan 26.6 / 2)*1.80 = 0.588$m

$Xh = 0.149 + 0.588 = 0.737\text{m} < 1.80 / 2 + 1/6 *0.50 = 0.983$m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60

Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)

$PH = 1/2 * r * H * Ka$

$= 1/2 * 18.0 * 1.80^2 * 0.139$

$= 4.05$ kN/m

3.3 Weight of Wall

(1) Weight of Wall

$W1 = 0.50 * 1.50 * 23.5$

$= 17.63$ kN/m

(2) Base Concrete

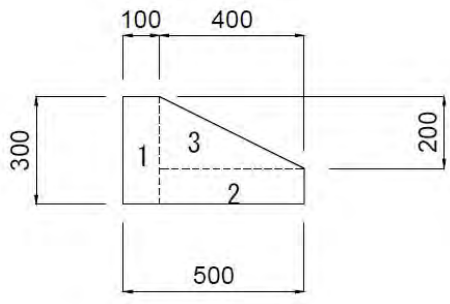


Fig.4 Base Concrete

$W2 = (0.10*0.30 + 0.40*0.10 + 1/2 * 0.40*0.20) * 23.5 = 2.59$ kN/m

(3) Total Weight

W = W1 + W2 = 17.63 + 2.59 = 20.22 kN/m

3.4 Safety Factor for Sliding

Fs = 20.22 * 0.60 / 4.05 = 3.00 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 0.737 m

E = 0.737 – 1.80 / 2 = -0.163 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 20.22 kN/m ＜ 31.5 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α , β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1.0 kN/m^2)
q	=	ground surface surcharge (kN/m^2)
	=	$\gamma_2 \cdot$ Df = 0.60m * 17.00 = 10.20 kN/m^2
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)
k	=	coefficient (1+0.3 x Df’/B = 1+0.3*0.60 / 0.50 = 1.36)

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^\circ$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.50 (1.0*1.36*1.0*19.5 + 1.36 * 10.20*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 94.48 kN/m

Qa = 1/3 * Qu = 1/3 * 94.48 = 31.5 kN/m

Calculation of Concrete Block Wall H = 1.75 m

1. Design Condition

   Height: H=1.75m

   Calculation Height: h = 1.75 + 0.30 (Base) = 2.05m

   Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

   Inclination: 0.5 : 1

   Uniform Load: 10 kN/m^2

   Backfill Earth: r = 18 kN/m^3, ϕ =30°

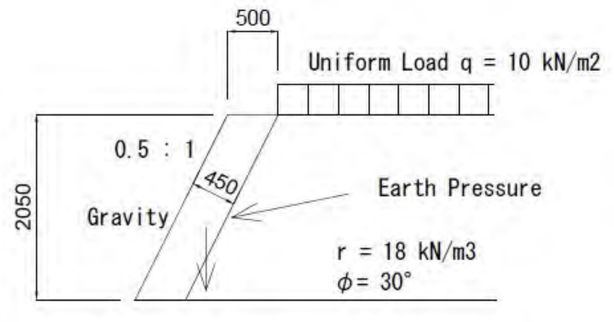


Fig.1 Design Condition

2. Study of Stability against Turn Over

   A resultant force has to be located out behind of a 1/3 range of wall area of center.

   A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma\cos\theta}{6\gamma_h b}h^2 + \left\{\frac{K_a q\cos^2\theta/\cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2}\right\}h$$

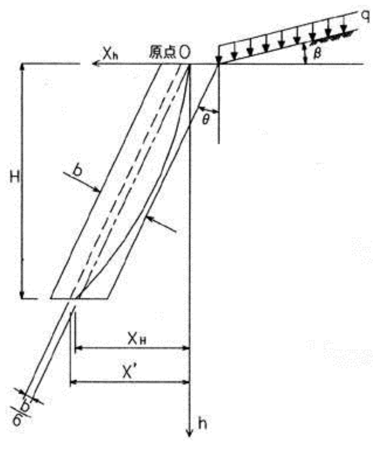


Fig.2 Diagram of Force Line

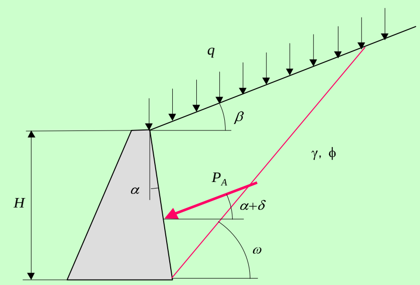
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	H=	4	m	
壁面傾斜角	α =	-26.6	度	-0.46426 rad
地表載荷重	q=	10	kN/m^2	
設計水平震度	k_H=	0		
盛土傾斜角	β =	0	度	0 rad
単位体積重量	γ =	18	kN/m^3	
せん断抵抗角	ϕ =	30	度	0.523599 rad
壁面摩擦角	δ =	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{\frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\frac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}}-\sin(\phi+\delta+\alpha-\beta)}\right\}+\beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, ω= 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + qK_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m^3)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m^2)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

$Ka*r*\cos\theta*h^2 / (6*r*b) = 0.139*18.0*\cos 26.6 *2.05^2 / (6*18.0*0.45) = 0.193$ m

$(Ka*q*\cos\theta/(2*r*b)+\tan\theta/2)*h$

$= (0.139*10.0*\cos 26.6/ (2*18.0*0.45)+\tan 26.6 / 2)*2.05 = 0.670$m

Xh = 0.193 + 0.670 = 0.863m < 2.05 / 2 + 1/6 *0.50 = 1.108m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60

Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)

PH = 1/2 * r * H * Ka

$= 1/2 * 18.0 * 2.05^2 * 0.139$

= 5.26 kN/m

3.3 Weight of Wall

(1) Weight of Wall

W1 = 0.50 * 1.75 * 23.5

= 20.56 kN/m

(2) Base Concrete

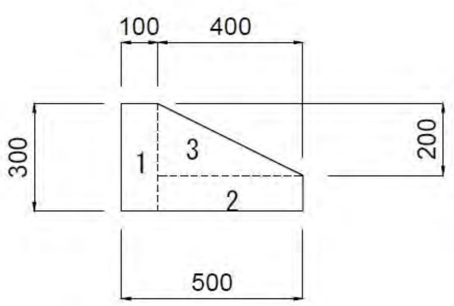


Fig.4 Base Concrete

W2 = (0.10*0.30 + 0.40*0.10 + 1/2 * 0.40*0.20) * 23.5 = 2.59 kN/m

(3) Total Weight

W = W1 + W2 = 20.56 + 2.59 = 23.15 kN/m

3.4 Safety Factor for Sliding

Fs = 23.15 * 0.60 / 5.26 = 2.64 > 1.50.....OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 0.863 m

E = 0.863 - 2.05 / 2 = -0.162 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 23.15 kN/m ＜ 31.5 kN/m....OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table (=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1.0 kN/m^2 1)
q	=	ground surface surcharge (kN/m^2)
	=	$\gamma_2 \cdot$ Df = 0.60m * 17.00 = 10.20 kN/m^2
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)

k = coefficient (1+0.3 x Df'/B = 1+0.3*0.60 / 0.50 = 1.36)

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^\circ$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.50 (1.0*1.36*1.0*19.5 + 1.36 * 10.20*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 94.48 kN/m

Qa = 1/3 * Qu = 1/3 * 94.48 = 31.5 kN/m

Calculation of Concrete Block Wall H = 2.00m

1. Design Condition

Height: H=2.00m

Calculation Height: h = 2.00 + 0.30 (Base) = 2.30m

Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

Inclination: 0.5 : 1

Uniform Load: 10 kN/m^2

Backfill Earth: r = 18 kN/m^3, ϕ =30°

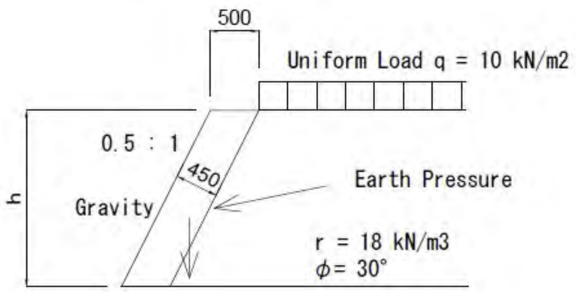


Fig.1 Design Condition

2. Study of Stability against Turn Over

A resultant force has to be located out behind of a 1/3 range of wall area of center.

A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{K a \gamma \cos\theta}{6\gamma_h b} h^2 + \left\{ \frac{K_a q \cos^2\theta / \cos(\theta + \beta)}{2\gamma_h b} + \frac{\tan\theta}{2} \right\} h$$

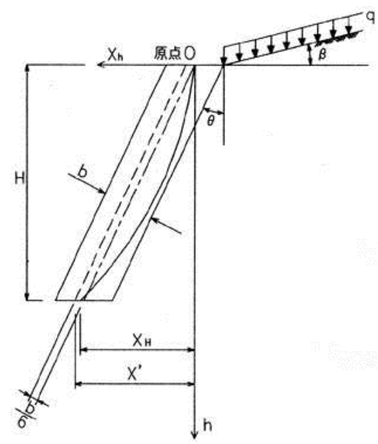


Fig.2 Diagram of Force Line

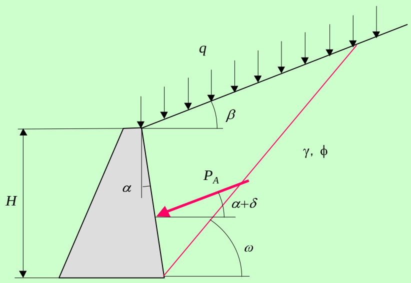
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	$H=$	4	m	
壁面傾斜角	$\alpha=$	-26.6	度	-0.46426 rad
地表載荷重	$q=$	10	kN/m²	
設計水平震度	$k_H=$	0		
盛土傾斜角	$\beta=$	0	度	0 rad
単位体積重量	$\gamma=$	18	kN/m³	
せん断抵抗角	$\phi=$	30	度	0.523599 rad
壁面摩擦角	$\delta=$	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{\frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\frac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}}-\sin(\phi+\delta+\alpha-\beta)}\right\}+\beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, $\omega=$ 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + q K_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m³)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m²)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

$Ka*r*\cos\theta*h^2 / (6*r*b) = 0.139*18.0*\cos 26.6*2.30^2 / (6*18.0*0.45) = 0.244$ m

$(Ka*q*\cos\theta/(2*r*b)+\tan\theta/2)*h$

$= (0.139*10.0*\cos 26.6/ (2*18.0*0.45)+\tan 26.6 / 2)*2.30 = 0.752$m

$Xh = 0.244 + 0.752 = 0.996$m $< 2.30 / 2 + 1/6 *0.50 = 1.233$m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: $\mu = 0.60$

Safety factor : $Fs = 1.50$

3.2 Horizontal Earth Pressure (PH)

$PH = 1/2 * r * H * Ka$

$= 1/2 * 18.0 * 2.30^2 * 0.139$

$= 6.62$ kN/m

3.3 Weight of Wall

(1) Weight of Wall

$W1 = 0.50 * 2.00 * 23.5$

$= 23.50$ kN/m

(2) Base Concrete

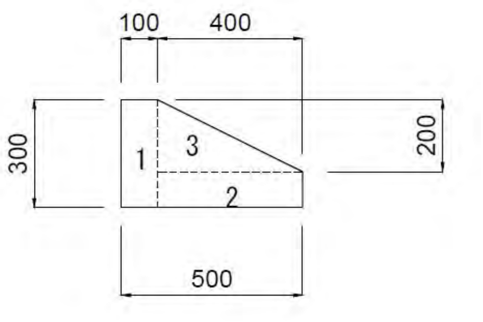


Fig.4 Base Concrete

$W2 = (0.10*0.30 + 0.40*0.10 + 1/2 * 0.40*0.20) * 23.5 = 2.59$ kN/m

(3) Total Weight

W = W1 + W2 = 23.50 + 2.59 = 26.09 kN/m

3.4 Safety Factor for Sliding

Fs = 26.09* 0.60 / 6.62 = 2.36 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 0.996 m

E = 0.996 - 2.30 / 2 = -0.154 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 26.09 kN/m ＜ 31.5 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1.0 kN/m^2)
q	=	ground surface surcharge 上載荷重(kN/m^2)
	=	$\gamma_2 \cdot Df$ = 0.60m * 17.00 = 10.20 kN/m^2
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)
k	=	coefficient (1+0.3 x Df’/B = 1+0.3*0.60 / 0.50 = 1.36)

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^{\circ}$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.50 (1.0*1.36*1.0*19.5 + 1.36 * 10.20*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 94.48 kN/m

Qa = 1/3 * Qu = 1/3 * 94.48 = 31.5 kN/m

Calculation of Concrete Block Wall H = 2.25 m in Section 3

1. Design Condition

   Height: H=2.25m

   Calculation Height: h = 2.25 + 0.70 (Base) = 2.95m

   Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

   Inclination: 0.5 : 1

   Uniform Load: 10 kN/m^2

   Backfill Earth: r = 18 kN/m^3, ϕ =30°

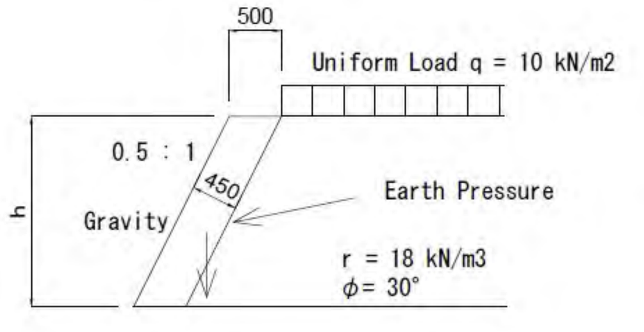


Fig.1 Design Condition

2. Study of Stability against Turn Over

   A resultant force has to be located out behind of a 1/3 range of wall area of center.

   A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma\cos\theta}{6\gamma_h b}h^2 + \left\{\frac{K_a q\cos^2\theta/\cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2}\right\}h$$

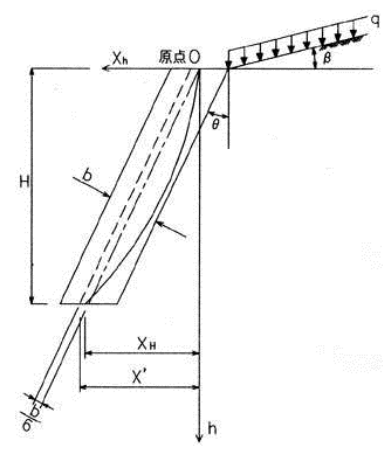


Fig.2 Diagram of Force Line

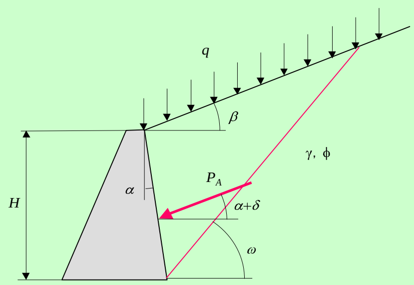
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	H=	4	m	
壁面傾斜角	α =	-26.6	度	-0.46426 rad
地表載荷重	q=	10	kN/m^2	
設計水平震度	k_H=	0		
盛土傾斜角	β =	0	度	0 rad
単位体積重量	γ =	18	kN/m^3	
せん断抵抗角	ϕ =	30	度	0.523599 rad
壁面摩擦角	δ =	20	度	0.349066 rad



地震合成角

$\theta = \tan^{-1} k_H$ = 0 rad 0 度

主働すべり角

$$\omega = \tan^{-1}\left\{ \frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\dfrac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}} - \sin(\phi+\delta+\alpha-\beta)} \right\} + \beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, ω= 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A = \frac{1}{2}\gamma K_A H^2 + qK_A H$ = 25.576 kN/m

r : unit weight of soil (18 kN/m^3)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m^2)

β : embankment inclination angle to the wall face (=0°)

h: height of wall (=2.70 m, including concrete base)

Ka*r*cos θ *h^2 / (6*r*b) = 0.139*18.0*cos 26.6 *2.95^2 / (6*18.0*0.45) = 0.401 m

(Ka*q*cos θ /(2*r*b)+tan θ /2)*h

= (0.139*10.0*cos 26.6/ (2*18.0*0.45)+tan26.6 / 2)*2.95 = 0.964m

Xh = 0.401 + 0.964 = 1.365m < 2.95 / 2 + 1/6 *0.50 = 1.558m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60

Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)

PH = 1/2 * r * H * Ka

= 1/2 * 18.0 * 2.95^2 * 0.139

= 10.89 kN/m

3.3 Weight of Wall

(1) Weight of Wall

W1 = 0.50 * 2.25 * 23.5

= 26.44 kN/m

(2) Base Concrete

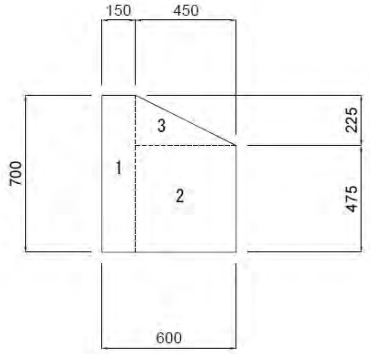


Fig.4 Base Concrete

W2 = (0.10*0.70 + 0.50*0.475 + 1/2 * 0.45*0.225) * 23.5 = 8.42 kN/m

(3) Total Weight

W = W1 + W2 = 26.44 + 8.42 = 34.86 kN/m

3.4 Safety Factor for Sliding

Fs = 34.86 * 0.60 / 10.89 = 1.92 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 1.365 m

E = 1.365 — 2.95 / 2 = -0.110 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 34.86 kN/m < 64.7 kN/m….OK

Situation of Resultant Force and Reaction

Xh = 1.516 m

E = 1.516 — 3.20 / 2 = -0.084 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 37.80 kN/m < 52.6 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table (=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1 .0 kN/m^2)
q	=	ground surface surcharge 上載荷重(kN/m^2)

= $\gamma_2 \cdot$ Df = 1.00m * 17.00 = 17.00 kN/m^2 (for Section 3)

γ_1, γ_2 = unit weight of soil of ground foundation (= 17.0 kN/m^3)

B',L' = width and length of effective loading areas

e = distance from center of footing to acting of resultant force on footing as illustrated in following figure

Df = depth from ground surface to bottom of footing (=0.60m)

k = coefficient

1+0.3 x Df'/B = 1+0.30 * 1.00 / 0.50 = 1.60

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Df = 1.00m

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^\circ$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.60 (1.0*1.60*1.0*19.5 + 1.60 * 17.00*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 193.96 kN/m

Qa = 1/3 * Qu = 1/3 * 193.96 = 64.7 kN/m

Calculation of Concrete Block Wall H = 2.50 m in Section 3

1. Design Condition

   Height: H=2.50m

   Calculation Height: h = 2.50 + 0.70 (Base) = 3.20m

   Thickness: 0.50m (=Block + Backfill Concrete = 0.35+0.10 = 0.45)

   Inclination: 0.5 : 1

   Uniform Load: 10 kN/m^2

   Backfill Earth: r = 18 kN/m^3, ϕ =30°

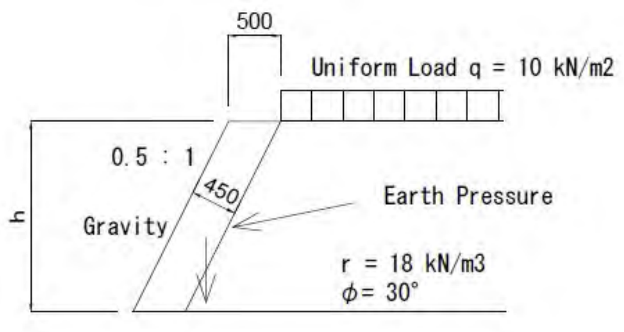


Fig.1 Design Condition

2. Study of Stability against Turn Over

   A resultant force has to be located out behind of a 1/3 range of wall area of center.

   A distance from a original point of the resultant force is described as Xh.

$$Xh = \frac{Ka\gamma\cos\theta}{6\gamma_h b}h^2 + \left\{\frac{K_a q\cos^2\theta/\cos(\theta+\beta)}{2\gamma_h b} + \frac{\tan\theta}{2}\right\}h$$

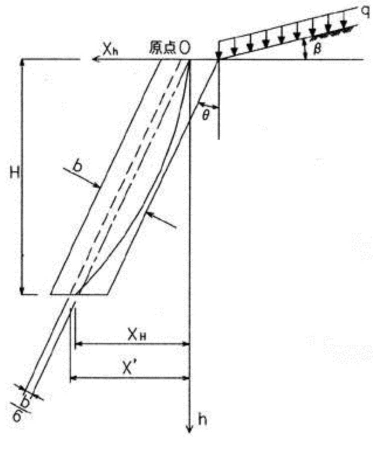


Fig.2 Diagram of Force Line

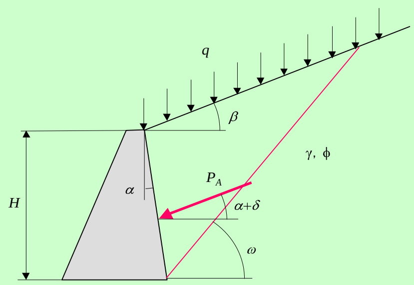
Where,

Xh: point of force line at depth h (m)

Ka: coefficient of earth pressure (=0.139)

クーロンの主働土圧

擁壁高さ	$H=$	4	m	
壁面傾斜角	$\alpha=$	-26.6	度	-0.46426 rad
地表載荷重	$q=$	10	kN/m²	
設計水平震度	$k_H=$	0		
盛土傾斜角	$\beta=$	0	度	0 rad
単位体積重量	$\gamma=$	18	kN/m³	
せん断抵抗角	$\phi=$	30	度	0.523599 rad
壁面摩擦角	$\delta=$	20	度	0.349066 rad



地震合成角

$\theta=\tan^{-1}k_H$ = 0 rad 0 度

主働すべり角

$$\omega=\tan^{-1}\left\{\frac{\cos(\phi+\delta+\alpha-\beta)}{\sqrt{\frac{\cos(\alpha+\delta+\theta)\sin(\phi+\delta)}{\cos(\alpha-\beta)\sin(\phi-\beta-\theta)}}-\sin(\phi+\delta+\alpha-\beta)}\right\}+\beta$$

ここで,

$\cos(\phi+\delta+\alpha-\beta)=$ 0.917755

$\cos(\alpha+\delta+\theta)\sin(\phi+\delta)=$ 0.760968

$\cos(\alpha-\beta)\sin(\phi-\beta-\theta)=$ 0.447077

$\sin(\phi+\delta+\alpha-\beta)=$ 0.397148

したがって, $\omega=$ 0.791019 rad 45.32203 度

主働土圧係数 KA

$$K_A=\frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)}}\right\}^2}$$

ここで,

$\cos^2(\phi-\alpha-\theta)=$ 0.303029 $\cos\theta\cos^2\alpha\cos(\alpha+\delta+\theta)=$ 0.794213

$\sin(\phi+\delta)\sin(\phi-\beta-\theta)=$ 0.383022 $\cos(\alpha+\delta+\theta)\cos(\alpha-\beta)=$ 0.888228

したがって, KA= 0.139

主働土圧合力

$P_A=\frac{1}{2}\gamma K_A H^2+qK_AH$ = 25.576 kN/m

r : unit weight of soil (18 kN/m³)

θ : angle of inclination of wall (26.6°)

b: thickness of wall (= 0.50m)

q: surcharge (=10 kN/m²)

β : embankment inclination angle to the wall face (=0°)
h: height of wall (=2.70 m, including concrete base)

Ka*r*cos θ *h^2 / (6*r*b) = 0.139*18.0*cos 26.6 *3.20^2 / (6*18.0*0.45) = 0.471 m
(Ka*q*cos θ /(2*r*b)+tan θ /2)*h
= (0.139*10.0*cos 26.6/ (2*18.0*0.45)+tan26.6 / 2)*3.20 = 1.045m
Xh = 0.471 + 1.045 = 1.516m < 3.20 / 2 + 1/6 *0.50 = 1.683m….OK

3. Stability of Sliding

3.1 Calculation Condition

Friction factor on the ground: μ =0.60
Safety factor : Fs = 1.50

3.2 Horizontal Earth Pressure (PH)
PH = 1/2 * r * H * Ka
= 1/2 * 18.0 * 3.20^2 * 0.139
= 12.81 kN/m

3.3 Weight of Wall

(1) Weight of Wall
W1 = 0.50 * 2.50 * 23.5
= 29.38 kN/m

(2) Base Concrete

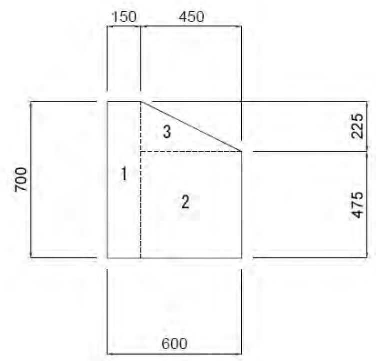


Fig.4 Base Concrete

W2 = (0.10*0.70 + 0.50*0.475 + 1/2 * 0.45*0.225) * 23.5 = 8.42 kN/m

(3) Total Weight

W = W1 + W2 = 29.38 + 8.42 = 37.80 kN/m

3.4 Safety Factor for Sliding

Fs = 37.80 * 0.60 / 12.81 = 1.77 > 1.50…..OK

4. Bearing Capacity

4.1 Situation of Resultant Force and Reaction

Xh = 1.516 m

E = 1.516 — 3.20 / 2 = -0.084 < 0

Therefore, a resultant force locates at the middle third of base concrete.

Reaction force = 37.80 kN/m < 64.7 kN/m….OK

4.2 Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α, β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (= 1 .0 kN/m^2)
q	=	ground surface surcharge 上載荷重(kN/m^2)
	=	γ_2 · Df = 1.00m * 17.00 = 17.00 kN/m^2 (for Section 3)
γ_1, γ_2	=	unit weight of soil of ground foundation (= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure

Df = depth from ground surface to bottom of footing (=0.60m)

k = coefficient

1+0.3 x Df'/B = 1+0.30 * 1.00 / 0.50 = 1.60

Df' = structure embedded depth into base (= 0.60m)

Nc, Nq and Nr = bearing capacity factors

Df = 1.00m

Minimum N value in F's layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^{\circ}$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A' (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=0.60 (1.0*1.60*1.0*19.5 + 1.60 * 17.00*9.8 + 1/2 * 17.0*1.0*0.50*6.0)

= 193.96 kN/m

Qa = 1/3 * Qu = 1/3 * 193.96 = 64.7 kN/m

Height of Concrete Railing

1. Risk of Head due to Drop

U.S. Code (ASTM F 1292:1999): Standard Specification for Impact Attenuation of Surfacing Materials within the Use Zone of Playground Equipment shows that an impact exceeding HIC 1000 has a possibility to cause injury to a human head.

HIC (Head Injury Criterion) score is an impact score describing the relation ship between the magnitude and duration of impact accelerations and the risk of head trauma. This HIC establishes minimum performance requirements for the impact attenuation of playground surfacing materials installed.

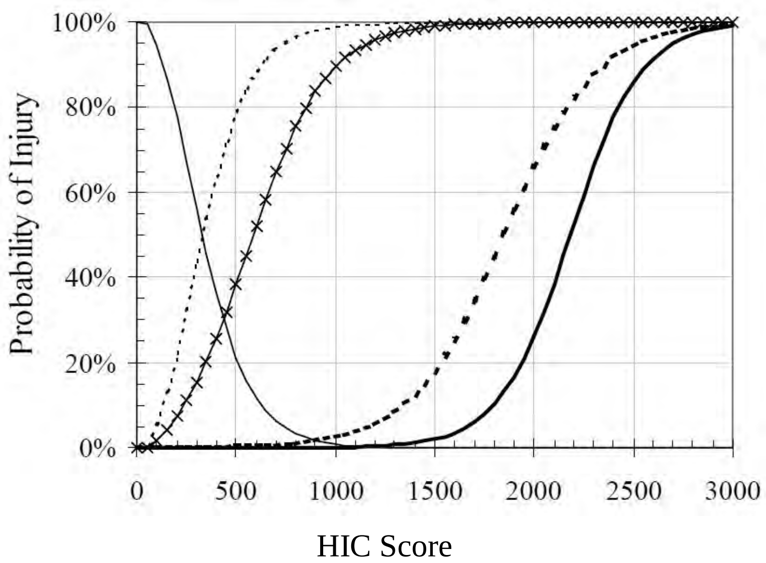


Fig. 1 Probability of Specific Head Injury Level for a Given HIC Score

No Injury   Minor   Moderate   Critical   Fatal

We would like to apply this HIC score to the safety of dike structures. With the review of dike structures, some effective facilities shall be required to be installed on the places where there are some possibilities of exceeding HIC 1000.

2. Limitation of Drop Height

Some Japanese company is making available the data of laboratory tests on critical drop height followed ASTM F1292 to the public as shown below.

Table 1 Comparison of Materials Adopted to Drop Height of HIC 1000

Category of Buffer Materials	Depth of Buffer Materials Managed softly (m)			Depth of Buffer Materials Compacted (m)
	16cm	23cm	31cm	23cm
Wood Chip	2.1	3.0	3.3	3.0
Bark	1.8	3.0	3.3	2.1
Sawdust	1.8	2.1	3.6 or more	1.8
Fine Sand	1.5	1.5	2.7	1.5
Coarse Sand	1.5	1.5	1.8	1.2
Fine Gravel	1.8	2.1	3.0	1.8
Medium Gravel	1.5	1.5	1.8	1.5
Synthetic Rubber Matte	3~3.6	-	-	-

Grass sodding with 10cm thick is a material used on backfill soil in front of concrete block retaining walls. The backfill soils are compacted materials, and the grass sodding don't have enough thickness as buffer material. So a buffering effect would be as same as the one of medium gravel as shown above.

From this view point, a limitation of drop height shall be 1.50m, and it shows clearly that retaining walls shall provide handrails for prevention of drop accident, especially by children, if a difference of height between top of wall and design ground exceeds 1.50m.

CHAPTER 3 CALCULATION OF GRAVITY WALL

The detailed calculation of gravity wall is indicated from the following page.

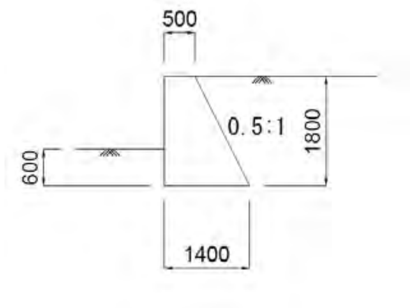
Calculation of Gravity Wall at around STA 4+420

1. Design Condition

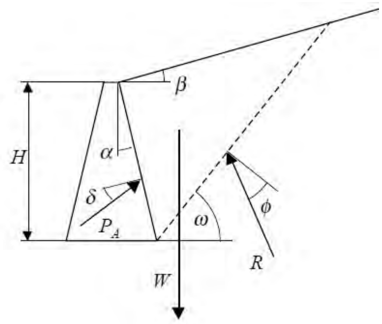
   Unit weight of Soil : r = 18 kN/m^3

   Internal friction of angle : ϕ =30°

   Surcharge : q = 10 kN/m^2



2. Earth Pressure



$$K_A = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha\cos(\alpha+\delta)\left\{1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\alpha+\delta)\cos(\alpha-\beta)}}\right\}^2}$$

From ϕ =30° , α =26.6° , β =0, δ =20°

KA = 0.566

KAH = 0.566 * cos (26.6° + 20°) = 0.389

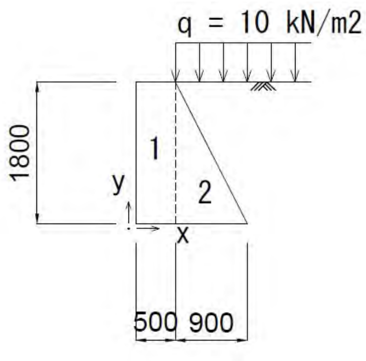
KAV = 0.566 * sin (26.6° + 20°) = 0.411

Ph = 0.389 * (10.0*1.80 + 1/2 *18.0 * 1.80^2) = 18.35 kN/m

Pv = 0.411 * (10.0*1.80 + 1/2 *18.0 * 1.80^2) = 19.38 kN/m

3. Stability Analysis

(1) Weight of Wall



	Equation	W (kN/m)	x (m)	Wx (kN/m)	H (kN/m)	y (m)	Hy (kN)
1	0.50*1.80*23.5	21.15	0.25	5.29			
2	0.90*1.80*1/2*23.5	19.04	0.80	15.23			
	Total	40.19		20.52			

(2) Total Forces

	Forces	W (kN/m)	x (m)	Wx (kN/m)	H (kN/m)	y (m)	Hy (kN)
1	Wall Weight	40.19		20.52			
2	Earth Pressure	19.38	1.10	21.32	18.35	0.60	11.01
	Total	59.57		41.84	18.35		11.01

(3) Stability Analysis

Turn Over

x = 0.70 - (41.84 – 11.01) / 59.57 = 0.18m

=B/6 = 1.40 / 6 = 0.23m….OK

Sliding

Fs = 59.57 * 0.60 / 18.39 = 1.95 >1.50…..OK

Bearing Capacity

q = 59.57 / 1.40 (1 ±6 * 0.18 / 1.40) = 75.37 , 9.73 kN/m^2 < 91.29…OK

(4) Ultimate Bering capacity

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu	=	ultimate bearing capacity (kN)
A’	=	effective loading area on footing (m^2)
α , β	=	coefficient depending on shape of footing as shown in the following table
		(=1.0 in shape of excessively long rectangle)
C	=	cohesion of foundation ground (=　1 .0 kN/m^2)
q	=	ground surface surcharge 上載荷重(kN/m^2)
	=	γ_2・Df = 0.60m * 17.00 = 10.20 kN/m^2 (for Section 3)
γ_1, γ_2	=	unit weight of soil of ground foundation　(= 17.0 kN/m^3)
B’,L’	=	width and length of effective loading areas
		B’ = 1.40 – 2*0.18 = 1.04m
e	=	distance from center of footing to acting of resultant
		force on footing as illustrated in following figure
Df	=	depth from ground surface to bottom of footing (=0.60m)
k	=	coefficient
		1+0.3 x Df’/B = 1+0.30 * 0.60 / 1.40 = 1.13
Df’	=	structure embedded depth into base (= 0.60m)
Nc, Nq and Nr	=	bearing capacity factors

Df = 1.00m

Minimum N value in F’s layer = 5

$\phi = (15*N)^{1/2} + 15 = 24.0^\circ$

Nc = 19.5

Nq = 9.8

Nr = 6.0

Qu = A’ (α *k*c*Nc + k*q*Nq + 1/2 * r1 * β *B*Nr)

=1.04 (1.0*1.13*1.0*19.5 + 1.13 * 17.00*9.8 + 1/2 * 17.0*1.0*1.04*6.0)

= 273.87 kN/m

Qa = 1/3 * Qu = 1/3 * 273.87 = 91.29 kN/m

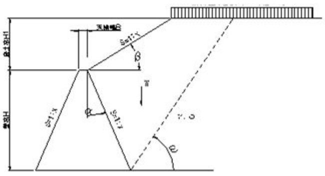
Stability Analysis of Gravity Wall
Earth Pressure Calculation with Trial Wedge Method

1. Guideline

	Earthwork Guideline	1

2. Design Condition

Unit Weight of Earth	γ(kN/m3)		18.0
Internal Friction Angle	φ(°)		30.0
Surcharge	q(kN/㎡)	Others	10.00



3. Shape

Top Width B(m)		0.300
Front Slope	S=1:x	0.50
Back Slope	S=1:x	0.00
Height of Wall H(m)		1.00
Gradient of Slope	S=1:x	2.00
Height of Embankment H1(m)		0.00
Unit Weight of Concrete (kN/m3)		23.5
Foundation Soil Layer	2.Sandy	2
Allowable Bearing Capacity (kN/㎡)		50.0
Friction Factor		0.6

	x	y
1	0	0
2	0.5	1.00
3	0.8	1.00
5	0.80	1.00
6	2.47	1.00
3	0.80	1.00
4	0.8	0
1	0	0
6	1.47450852	1.00
7	0.80	0.00

Result of Stability Analusis

Turn Over	OK
Sliding	OK
Bearing Capacity	OK

4. Earth Pressure

$$Pa = \frac{\sin(\omega - \phi)}{\cos(\omega - \phi - \delta - \alpha)}(W + Q)$$

$$W = \frac{1}{2}\gamma\left\{(H + H_1)^2 \frac{\cos(\omega - \alpha)}{\sin \omega} - H_1{}^2 \frac{\cos(\alpha - \beta)}{\sin \beta}\right\}\frac{1}{\cos \alpha}$$

$$l = (H + H_1)\frac{\cos(\omega - \alpha)}{\sin \omega \cdot \cos \alpha} - H_1 \frac{\cos(\beta - \alpha)}{\sin \beta \cdot \cos \alpha}$$

$$Q = q \cdot l$$

Sliding Angle	56.000	degree	Q=	6.745 kN/m
Maximum Earth Pressure	5.649	kN/m	L=	0.675 m
			W=	6.071 kN/m

Horizontal Earth Pressure
Ph= PA*COS(α+δ)
= 5.649*COS(0.00+20.00)
= 5.308 (kN/m)

Vertical Earth Pressure
Pv= PA*SIN(α+δ)
= 5.649*SIN(0.00+20.00)
= 1.932 (kN/m)

角度	Pa
51.0	5.515
52.0	5.564
53.0	5.602
54.0	5.628
55.0	5.644
56.0	5.649
57.0	5.644
58.0	5.629
59.0	5.604
60.0	5.569
61.0	5.526

5. Stability Analysis

Computation of Body Weight

Front slope
W1= 1/2*S1*H*H*γCON
= 1/2*0.500*1.000*1.000*23.500
= 5.875

Body
W2= B*H*γCON
= 0.300*1.000*23.500
= 7.050

Back Slope
W3= 1/2*S2*H*H*γCON
= 1/2*0.000*1.000*1.000*23.500
= 0.000

Forces

Horizontal Earth Pressure (kN/m)	5.308
Vertical Earth Pressure (kN/m)	1.932
Pressure Height (m)	0.333

Stability Analysis Sheet

Number	X(m)	Y(m)	V(kN)	H(kN)	X･V(kN･m)	Y･H(kN･m)
W1	0.333		5.875		1.958	
W2	0.650		7.050		4.583	
W3	0.800		0.000		0.000	
Ph		0.333		-5.308		-1.768
Pv	0.800		1.932		1.546	
Total			14.857	-5.308	8.087	-1.768

Stability Analysis

Turn Over

X=(X*V+Y*H)/ Σ V
=(8.087 -1.768)/14.857
=0.425

e=BB/2-X
=0.800/2-0.425
=-0.025
<B/6 OK
(B/6=0.133)

Bearing (Bearing Ground 2.Sandy)

Resultant force locates at core of center
q1=V/BB+6*V*e/BB^2
=14.857/0.800+6*14.857*-0.025/0.800^2
=15.045 kN/m^2
q1<qa OK
q2=V/BB-6*V*e/BB^2
=14.857/0.800-6*14.857*-0.025/0.800^2
=22.098 kN/m^2
q2<qa OK

(qa=50.000)

Sliding

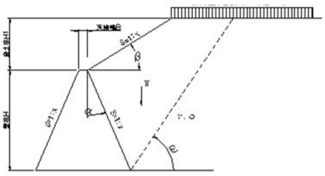
Fs= Σ V* μ / Σ H
=14.857*0.600/5.308
=1.679
>1.5 OK

Stability Analysis of Gravity Wall
Earth Pressure Calculation with Trial Wedge Method

1. Guideline

Earthwork Guideline	1

2. Design Condition



Unit Weight of Earth γ(kN/m3)		18.0
Internal Friction Angle φ(°)		30.0
Surcharge q(kN/㎡)	Others	10.00

3. Shape

Top Width B(m)		0.500
Front Slope	S=1:x	0.50
Back Slope	S=1:x	0.00
Height of Wall H(m)		1.50
Gradient of Slope	S=1:x	2.00
Height of Embankment H1(m)		0.00
Unit Weight of Concrete (kN/m3)		23.5
Foundation Soil Layer	2.Sandy	2
Allowable Bearing Capacity (kN/㎡)		50.0
Friction Factor		0.6

	x	y
1	0	0
2	0.75	1.50
3	1.3	1.50
5	1.25	1.50
6	3.26	1.50
3	1.25	1.50
4	1.25	0
1	0	0

6	2.26176278	1.50
7	1.25	0.00

Result of Stability Analusis

Turn Over	OK
Sliding	OK
Bearing Capacity	OK

4. Earth Pressure

$$Pa = \frac{\sin(\omega-\phi)}{\cos(\omega-\phi-\delta-\alpha)}(W+Q)$$

$$W = \frac{1}{2}\gamma\left\{(H+H_1)^2\frac{\cos(\omega-\alpha)}{\sin\omega} - H_1{}^2\frac{\cos(\alpha-\beta)}{\sin\beta}\right\}\frac{1}{\cos\alpha}$$

$$l = (H+H_1)\frac{\cos(\omega-\alpha)}{\sin\omega\cdot\cos\alpha} - H_1\frac{\cos(\beta-\alpha)}{\sin\beta\cdot\cos\alpha}$$

$$Q = q\cdot l$$

Sliding Angle	56.000	degree	Q=	10.118 kN/m
Maximum Earth Pressure	10.480	kN/m	L=	1.012 m
			W=	13.659 kN/m

角度	Pa
51.0	10.231
52.0	10.323
53.0	10.393
54.0	10.442
55.0	10.471
56.0	10.480
57.0	10.471
58.0	10.443
59.0	10.396
60.0	10.333
61.0	10.252

Horizontal Earth Pressure

Ph= PA*COS(α+δ)

= 10.480*COS(0.00+20.00)

= 9.848 (kN/m)

Vertical Earth Pressure

Pv= PA*SIN(α+δ)

= 10.480*SIN(0.00+20.00)

= 3.584 (kN/m)

5. Stability Analysis

Computation of Body Weight

Front slope

W1= 1/2*S1*H*H*γCON

= 1/2*0.500*1.500*1.500*23.500

= 13.219

Body

W2= B*H*γCON

= 0.500*1.500*23.500

= 17.625

Back Slope

W3= 1/2*S2*H*H*γCON

= 1/2*0.000*1.500*1.500*23.500

= 0.000

Forces

Horizontal Earth Pressure (kN/m)	9.848
Vertical Earth Pressure (kN/m)	3.584
Pressure Height (m)	0.500

Stability Analysis Sheet

Number	X(m)	Y(m)	V(kN)	H(kN)	X･V(kN･m)	Y･H(kN･m)
W1	0.500		13.219		6.609	
W2	1.000		17.625		17.625	
W3	1.250		0.000		0.000	
Ph		0.500		-9.848		-4.924
Pv	1.250		3.584		4.480	
Total			34.428	-9.848	28.714	-4.924

Stability Analysis

Turn Over

X=(X*V+Y*H)/ΣV
=(28.714 -4.924)/34.428
=0.691

e=BB/2-X
=1.250/2-0.691
=-0.066
＜B/6 OK
(B/6=0.208)

Bearing (Bearing Ground 2.Sandy)

Resultant force locates at core of center
q1=V/BB+6*V*e/BB^2
=34.428/1.250+6*34.428*-0.066/1.250^2
=18.816 kN/m^2
q1＜qa OK
q2=V/BB-6*V*e/BB^2
=34.428/1.250-6*34.428*-0.066/1.250^2
=36.269 kN/m^2
q2＜qa OK

(qa=50.000)

Sliding

Fs=ΣV*μ/ΣH
=34.428*0.600/9.848
=2.098
＞1.5 OK

CHAPTER 4 CALCULATION OF STEEL SHEET PILE REVETMENT

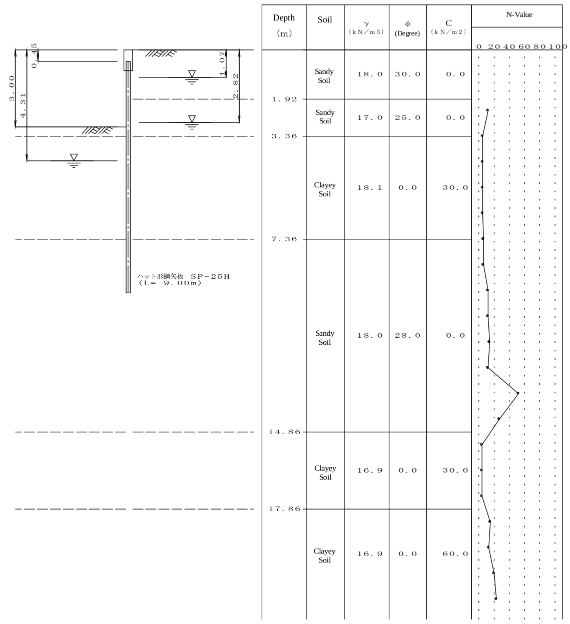
The detailed calculation of steel sheet pile revetment is indicated from the following page.

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 1,1 + 100

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_{lt}	=	0.40 m
Landside WL	L_{wa}	=	1.07 m (Normal Condition)
	L_{wa}'	=	2.82 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.31 m (Normal Condition)
	L_{wp}'	=	4.31 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \dfrac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1 - \dfrac{\Sigma \gamma h + 2Q}{2C}} \cdot \tan\theta$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value
1	2.36	12
2	3.36	5
3	4.36	5
4	5.36	5
5	6.36	5
6	7.36	6
7	8.36	6
8	9.36	12
9	10.36	12
10	11.36	14

No	Depth (m)	N-Value
11	12.36	12
12	13.36	52
13	14.36	27
14	15.36	4
15	16.36	4
16	17.36	4
17	18.36	15
18	19.36	13
19	20.36	20
20	21.36	23

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	ϕ	C kN/m^2	a	k'	ζ (degree) normal	ζ (degree) seismic	kh(kN/m^3) normal	kh(kN/m^3) seismic
1	1.92	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	3.36	S	9.0	17.00	8.00	25.0	0.0	0.0	0.200	auto	auto	——	——
3	7.36	C	5.0	18.10	9.10	0.0	30.0	0.0	0.200	auto	auto	——	——
4	14.86	S	12.0	18.00	9.00	28.0	0.0	0.0	0.200	auto	auto	——	——
5	17.86	C	5.0	16.90	7.90	0.0	30.0	0.0	0.200	auto	auto	——	——
6	21.36	C	20.0	16.90	7.90	0.0	60.0	0.0	0.200	auto	auto	——	——

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- ϕ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.50 m

Allowable stress　　σ_a = 180 N/mm (Normal)
σ_a' = 270 N/mm (Seismic)

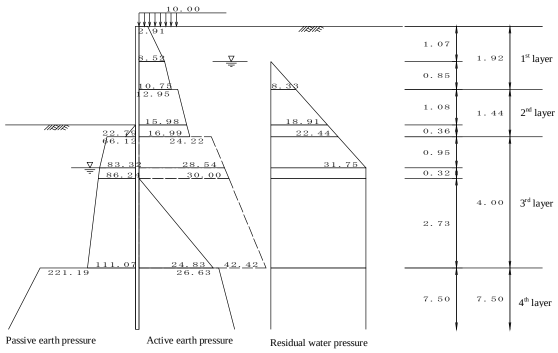
Allowable displacement　　δ_a = 50.0 mm (Normal)
δ_a' = 75.0 mm (Seismic)

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.07	Sandy soil	18.0	30.0	― ―	10.000 29.260	0.30142 0.30142	0.29115 0.29115
2	1.07～ 1.92	Sandy soil	9.0	30.0	― ―	29.260 36.910	0.30142 0.30142	0.29115 0.29115
3	1.92～ 3.00	Sandy soil	8.0	25.0	― ―	36.910 45.550	0.36312 0.36312	0.35074 0.35074
4	3.00～ 3.36	Sandy soil	8.0	25.0	― ―	45.550 48.430	0.36312 0.36312	0.35074 0.35074
5	3.36～ 4.31	Clayey soil	9.1	―	30.0 30.0	48.430 57.075	― ―	― ―
6	4.31～ 4.63	Clayey soil	9.1	―	30.0 30.0	57.075 60.000	― ―	― ―
7	4.63～ 7.36	Clayey soil	9.1	―	30.0 30.0	60.000 84.830	― ―	― ―
8	7.36～ 14.86	Sandy soil	9.0	28.0	― ―	84.830 152.330	0.32506 0.32506	0.31398 0.31398
9	14.86～ 17.86	Clayey soil	7.9	―	30.0 30.0	152.330 176.030	― ―	― ―
10	17.86～ 21.36	Clayey soil	7.9	―	60.0 60.0	176.030 203.680	― ―	― ―

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta =$ 15.00、$\beta =$ 0.00、$\theta =$ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

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2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
4	3.00～ 3.36	Sandy soil	17.0	25.0	― ―	0.000 6.120	3.85477 3.85477	3.72343 3.72343
5	3.36～ 4.31	Clayey soil	18.1	0.0	30.0 30.0	6.120 23.315	― ―	― ―
6	4.31～ 4.63	Clayey soil	9.1	0.0	30.0 30.0	23.315 26.240	― ―	― ―
7	4.63～ 7.36	Clayey soil	9.1	0.0	30.0 30.0	26.240 51.070	― ―	― ―
8	7.36～ 14.86	Sandy soil	9.0	28.0	― ―	51.070 118.570	4.48391 4.48391	4.33112 4.33112
9	14.86～ 17.86	Clayey soil	7.9	0.0	30.0 30.0	118.570 142.270	― ―	― ―
10	17.86～ 21.36	Clayey soil	7.9	0.0	60.0 60.0	142.270 169.920	― ―	― ―

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta =$ -15.00、$\beta =$ 0.00、$\theta =$ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.07	2.91 8.52	― ―	2.91 8.52	― ―	― ―
2	1.07～ 1.92	8.52 10.75	― ―	8.52 10.75	0.00 8.33	― ―
3	1.92～ 3.00	12.95 15.98	― ―	12.95 15.98	8.33 18.91	― ―
4	3.00～ 3.36	15.98 16.99	― ―	15.98 16.99	18.91 22.44	0.00 22.79
5	3.36～ 4.31	-11.57 -2.92	24.22 28.54	24.22 28.54	22.44 31.75	66.12 83.32
6	4.31～ 4.63	-2.92 0.00	28.54 30.00	28.54 30.00	31.75 31.75	83.32 86.24
7	4.63～ 7.36	0.00 24.83	30.00 42.42	30.00 42.42	31.75 31.75	86.24 111.07
8	7.36～ 14.86	26.63 47.83	― ―	26.63 47.83	31.75 31.75	221.19 513.54
9	14.86～ 17.86	92.33 116.03	76.17 88.02	92.33 116.03	31.75 31.75	178.57 202.27
10	17.86～ 21.36	56.03 83.68	88.02 101.84	88.02 101.84	31.75 31.75	262.27 289.92

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- Formula for active earth pressure

Sandy soil $P_{a1}=K_a\cdot[\Sigma\gamma h+\dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil $P_{a1}=\Sigma\gamma h+Q-2C$
$P_{a2}=K_c\cdot(\Sigma\gamma h+Q)$
K_c : Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1}=[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

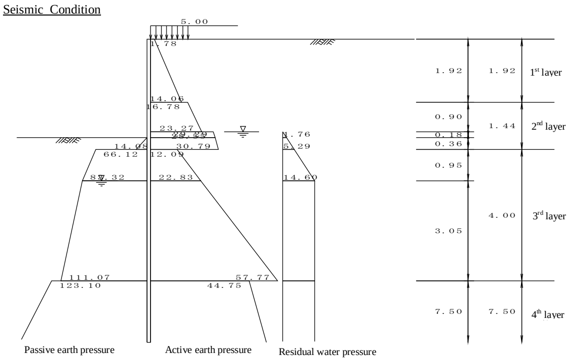
- Formula for passive earth pressure

Sandy soil $P_p=K_p\cdot[\Sigma\gamma h+\dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil $P_p=\Sigma\gamma h+Q+2C$

Mixed soil $P_p=[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 1.92	Sandy Soil	18.0	30.0	― ―	5.00 39.56	0.00 0.00	0.100 0.100	5.71 5.71	0.36790 0.36790	0.35537 0.35537	― ―
2	1.92～ 2.82	Sandy Soil	17.0	25.0	― ―	39.56 54.86	0.00 0.00	0.100 0.100	5.71 5.71	0.43909 0.43909	0.42413 0.42413	― ―
3	2.82～ 3.00	Sandy Soil	8.0	25.0	― ―	54.86 56.30	0.00 1.76	0.200 0.200	11.31 11.31	0.53868 0.53868	0.52033 0.52033	― ―
4	3.00～ 3.36	Sandy Soil	8.0	25.0	― ―	56.30 59.18	1.76 5.29	0.200 0.200	11.31 11.31	0.53868 0.53868	0.52033 0.52033	― ―
5	3.36～ 4.31	Clayey Soil	9.1	―	30.0 30.0	59.18 67.83	5.29 14.60	0.200 0.200	11.31 11.31	― ―	― ―	41.56 41.03
6	4.31～ 7.36	Clayey Soil	9.1	―	30.0 30.0	67.83 95.58	14.60 44.49	0.200 0.200	11.31 11.31	― ―	― ―	41.03 39.19

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	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
7	7.36～ 14.86	Sandy Soil	9.0	28.0	― ―	95.58 163.08	44.49 117.99	0.200 0.200	11.31 11.31	0.48473 0.48473	0.46822 0.46822	― ―
8	14.86～ 17.86	Clayey Soil	7.9	―	30.0 30.0	163.08 186.78	117.99 147.39	0.200 0.200	11.31 11.31	― ―	― ―	33.55 30.99
9	17.86～ 21.36	Clayey Soil	7.9	―	60.0 60.0	186.78 214.43	147.39 181.69	0.200 0.200	11.31 11.31	― ―	― ―	39.52 38.53

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta =$ 15.00、$\beta =$ 0.00、$\theta =$ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
4	3.00～ 3.36	Sandy soil	17.00	25.0	― ―	0.000 6.120	0.00 0.00	0.100 0.100	5.71 5.71	2.30029 2.30029	2.30029 2.30029
5	3.36～ 4.31	Clayey Soil	18.10	0.0	30.0 30.0	6.120 23.315	0.00 0.00	0.100 0.100	5.71 5.71	― ―	― ―
6	4.31～ 7.36	Clayey Soil	9.10	0.0	30.0 30.0	23.315 51.070	0.00 29.89	0.200 0.200	11.31 11.31	― ―	― ―
7	7.36～ 14.86	Sandy soil	9.00	28.0	― ―	51.070 118.570	29.89 103.39	0.200 0.200	11.31 11.31	2.41037 2.41037	2.41037 2.41037
8	14.86～ 17.86	Clayey Soil	7.90	0.0	30.0 30.0	118.570 142.270	103.39 132.79	0.200 0.200	11.31 11.31	― ―	― ―
9	17.86～ 21.36	Clayey Soil	7.90	0.0	60.0 60.0	142.270 169.920	132.79 167.09	0.200 0.200	11.31 11.31	― ―	― ―

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
$\delta =$ -0.00、$\beta =$ 0.00、$\theta = \tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 1.92	1.78 14.06	― ―	― ―
2	1.92～ 2.82	16.78 23.27	― ―	― ―
3	2.82～ 3.00	28.55 29.29	0.00 1.76	― ―

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No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
4	3.00～ 3.36	29.29 30.79	1.76 5.29	0.00 14.08
5	3.36～ 4.31	12.09 22.83	5.29 14.60	66.12 83.32
6	4.31～ 7.36	22.83 57.77	14.60 14.60	83.32 111.07
7	7.36～ 14.86	44.75 76.36	14.60 14.60	123.10 285.80
8	14.86～ 17.86	147.13 181.01	14.60 14.60	178.57 202.27
9	17.86～ 21.36	109.84 145.16	14.60 14.60	262.27 289.92

- Formula for active earth pressure

Sandy soil $P_{a1}=K_a\cdot[\Sigma\gamma h+\dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil $P_{a1}=\Sigma\gamma h+Q-2C$

$P_{a2}=K_c\cdot(\Sigma\gamma h+Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1}=[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p=K_p\cdot[\Sigma$ [illegible] Q [illegible] Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

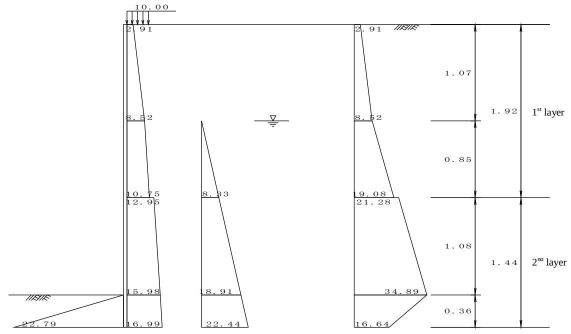
Clayey soil $P_p=\Sigma\gamma h$ [illegible]

Mixed soil $P_p=[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.07	2.91 8.52	—— ——	—— ——	2.91 8.52
2	1.07～ 1.92	8.52 10.75	0.00 8.33	—— ——	8.52 19.08
3	1.92～ 3.00	12.95 15.98	8.33 18.91	—— ——	21.28 34.89
4	3.00～ 3.36	15.98 16.99	18.91 22.44	0.00 22.79	34.89 16.64
5	3.36～ 4.31	24.22 28.54	22.44 31.75	66.12 83.32	-19.46 -23.03

P_a : Active earth pressure

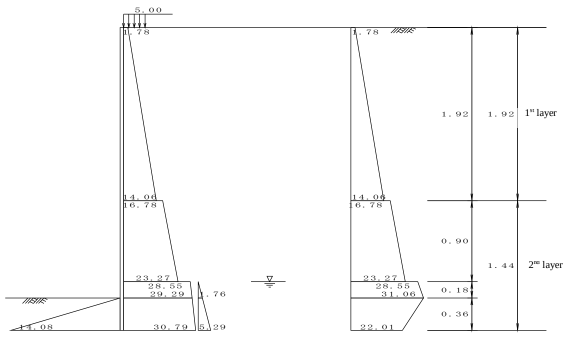
P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.36 m (GL -3.36 m)

3-2 Seismic Condition



Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.92	1.78 14.06	—— ——	—— ——	1.78 14.06
2	1.92～ 2.82	16.78 23.27	—— ——	—— ——	16.78 23.27
3	2.82～ 3.00	28.55 29.29	0.00 1.76	—— ——	28.55 31.06
4	3.00～ 3.36	29.29 30.79	1.76 5.29	0.00 14.08	31.06 22.01
5	3.36～ 4.31	12.09 22.83	5.29 14.60	66.12 83.32	-48.73 -45.88

P_a : Active earth pressure

P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.36 m (GL -3.36 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to $1/\beta$ depth. The modules are calculated by the formula below;

$K_h = 6910\times N^{0.406}$

where,

$$\beta = \sqrt[4]{\frac{Kh\cdot B}{4EI}}$$

Unit width	B	= 1.0000 m
Corrosion margin	t_1	= 1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	= 0.82
Section efficiency	μ	= 1.00
Young's modulus	E	= 200000 N/mm²
Inertia sectional moment	I_0	= 24400 cm⁴(original condition)
	I	= 20008 cm⁴(after reduction by corrosion and section)
Inertia sectional moment	EI	= 200000 × 10^3 × 20008 × 10^{-8} = 4.002 × 10^4

	Depth (m)	N-value		Depth (m)	N-value
1	2.36	12	11	12.36	12
2	3.36	5	12	13.36	52
3	4.36	5	13	14.36	27
4	5.36	5	14	15.36	4
5	6.36	5	15	16.36	4
6	7.36	6	16	17.36	4
7	8.36	6	17	18.36	15
8	9.36	12	18	19.36	13
9	10.36	12	19	20.36	20
10	11.36	14	20	21.36	23

4-2 Normal Condition

K_h = 13282 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh\cdot B}{4EI}}$$

$= 0.537 \quad m^{-1}$

$L = \dfrac{1}{\beta} = 1.86$ m

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.36 m) to 1.86 m depth (GL -5.22 m).

	Depth (m)	N-value
1	3.36	5.00
2	4.36	5.00
3	5.22	5.00
	Σh =	15.00

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{15.00}{3}$

$= 5.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 5.00^{0.406} = 13282$ kN/m³

Kh (normal condition) = 13282 kN/m³

4-3 Seismic Condition

K_h = 13282 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.537$ m⁻¹

$$L = \frac{1}{\beta} = 1.86 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.36 m) and 1.86 m depth (GL -5.22 m).

	Depth (m)	N-value
1	3.36	5.00
2	4.36	5.00
3	5.22	5.00
	Σh = 15.00	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{15.00}{3}$

$= 5.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 5.00^{0.406} = 13282$ kN/m³

Kh (seismic condition) = 13282 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.07	1.07	2.91 8.52	1.56 4.56	3.00 2.65	4.68 12.06
2	1.07～ 1.92	0.85	8.52 19.08	3.62 8.11	2.01 1.72	7.27 13.97
3	1.92～ 3.00	1.08	21.28 34.89	11.49 18.84	1.08 0.72	12.41 13.57
4	3.00～ 3.36	0.36	34.89 16.64	6.28 3.00	0.24 0.12	1.51 0.36
				ΣP = 57.45		ΣM = 65.82

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.36 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{65.82}{57.45} = 1.15 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.92	1.92	1.78 14.06	1.71 13.50	2.72 2.08	4.64 28.07
2	1.92～ 2.82	0.90	16.78 23.27	7.55 10.47	1.14 0.84	8.61 8.80
3	2.82～ 3.00	0.18	28.55 31.06	2.57 2.80	0.48 0.42	1.23 1.17
4	3.00～ 3.36	0.36	31.06 22.01	5.59 3.96	0.24 0.12	1.34 0.48
				ΣP = 48.14		ΣM = 54.34

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.36 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{54.34}{48.14} = 1.13 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	13282 kN/m³
calculated value	β	=	0.53671 m⁻¹
resultant earth force (lateral)	P_0	=	57.45 kN/m
height of acting position of load	h_0	=	1.15 m
moment	M_0	=	65.82 kN·m/m

in consideration of ψ_m = 1.304,

maximum moment	M_{max}	=	85.80 kN·m/m
depth of generated position of M_{max}	l_m	=	0.785 m
depth of 1st fixed point	l_i	=	2.249 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	13282 kN/m³
calculated value	β	=	0.53671 m⁻¹
resultant earth force (lateral)	P_0	=	48.14 kN/m
height of acting position of load	h_0	=	1.13 m
moment	M_0	=	54.34 kN·m/m

in consideration of ψ_m = 1.310,

maximum moment	M_{max}	=	71.19 kN·m/m
depth of generated position of M_{max}	l_m	=	0.791 m
depth of 1st fixed point	l_i	=	2.255 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{85.80\times10^6}{1320\times10^3} = 65\ N/mm^2 \leqq \sigma_a = 180\ N/mm^2 \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{71.19\times10^6}{1320\times10^3} = 54\ N/mm^2 \leqq \sigma_a = 270\ N/mm^2 \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.07	3.00 2.65	0.894 0.788	0.280 0.229	1.56 4.56	0.437 1.043
2	1.07～ 1.92	2.01 1.72	0.597 0.513	0.143 0.109	3.62 8.11	0.517 0.884
3	1.92～ 3.00	1.08 0.72	0.321 0.214	0.046 0.021	11.49 18.84	0.530 0.402
4	3.00～ 3.36	0.24 0.12	0.071 0.036	0.002 0.001	6.28 3.00	0.016 0.002
						ΣQ = 3.830

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

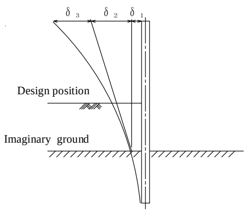
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5367\times1.15)\times57.45}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5367^3} = 0.00750\ m$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5367\times1.15)\times57.45}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5367^2}\times(3.00+0.36) = 0.01867\ m$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.83\times(3.00+0.36)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00363\ m$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00750 + 0.01867 + 0.00363$
$= 0.02980$ m
$= 29.80 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.92	2.72 2.08	0.810 0.619	0.239 0.152	1.71 13.50	0.408 2.052
2	1.92～ 2.82	1.14 0.84	0.339 0.250	0.051 0.029	7.55 10.47	0.385 0.300
3	2.82～ 3.00	0.48 0.42	0.143 0.125	0.010 0.007	2.57 2.80	0.025 0.021
4	3.00～ 3.36	0.24 0.12	0.071 0.036	0.002 0.001	5.59 3.96	0.014 0.003
						ΣQ = 3.208

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

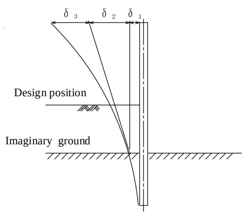
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5367\times1.13)\times48.14}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5367^3} = 0.00625\ m$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5367\times1.13)\times48.14}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5367^2}\times(3.00+0.36) = 0.01552\ m$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.21\times(3.00+0.36)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00304\ m$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00625 + 0.01552 + 0.00304$
$= 0.02481$ m
$= 24.81 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction $K_h = 13282 kN/m^3$

Calculated value $\beta = 0.51073 m^{-1}$

Penetration length of SSP $D = 0.36 + \dfrac{3}{0.511} = 6.23m$

Whole length of SSP $L = 3.00 - 0.40 + 6.23 = 8.83m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction $K_h = 13282 kN/m^3$

Calculated value $\beta = 0.51073 m^{-1}$

Penetration length of SSP $D = 0.36 + \dfrac{3}{0.511} = 6.23m$

Whole length of SSP $L = 3.00 - 0.40 + 6.23 = 8.83m$

Therefore, whole length of SSP is set as 8.90 m in consideration of round unit of SSP length.

7 Calculation Result

			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		85.80	71.19
Stress intensity	σ (N/mm^2)		65 (180)	54 (270)
Lateral displacement	δ (mm)		29.80 (50.0)	24.81 (75.0)
Penetration depth	D (m)		6.23	6.23
Whole length of SSP	L (m)	8.90		

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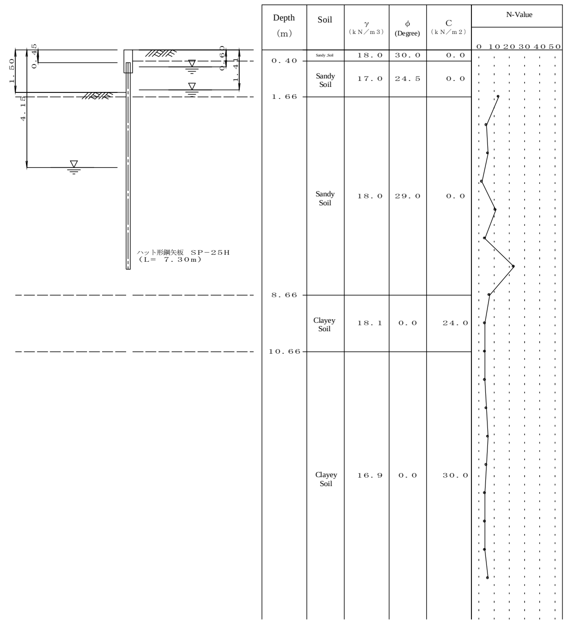
－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 1+325 D=1.50 NJ

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	1.50 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	0.60 m (Normal Condition)
	L_{wa}'	=	1.41 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.15 m (Normal Condition)
	L_{wp}'	=	4.15 m (Seismic Condition)

Imaginary riverbed　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　Chang's formula

Penetration depths　$L = \frac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　γ_w = 9.8 kN/m^3

Type of water pressure　trapezoidal water pressure

Lateral pressure　calculated in consideration of site conditions

Study case
- Normal Condition
- Seismic Condition

Design earthquake intensity　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　not considered

Minimum angle of rupture　ζ_0 = 10 degrees

Rear side angle of slope　not considered

Angle of rupture (clayey soil)　$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$

Equilibrium factor of compression　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　$K_h = 6910 \times N'^{0.406}$

Average N-value　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value
1	1.66	13
2	2.66	5
3	3.66	6
4	4.66	2
5	5.66	11
6	6.66	4
7	7.66	23
8	8.66	7
9	9.66	4
10	10.66	4

No	Depth (m)	N-Value
11	11.66	4
12	12.66	5
13	13.66	6
14	14.66	5
15	15.66	4
16	16.66	4
17	17.66	4
18	18.66	6
19	19.66	6
20	20.66	50

1-6 Vertical Load

Vertical load on landside　calculated in consideration of embankment shape on landside

Vertical load on riverside　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	ϕ	C kN/m^2	a	k'	ζ (degree) normal	ζ (degree) seismic	kh(kN/m^3) normal	kh(kN/m^3) seismic
1	0.40	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	1.66	S	13.0	17.00	8.00	24.5	0.0	0.0	0.200	auto	auto	——	——
3	8.66	S	6.0	18.00	9.00	29.0	0.0	0.0	0.200	auto	auto	——	——
4	10.66	C	4.0	18.10	9.10	0.0	24.0	0.0	0.200	auto	auto	——	——
5	19.40	C	5.0	16.90	7.90	0.0	30.0	0.0	0.200	auto	auto	——	——

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- ϕ : internal friction angle of soil
- C_0 : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　t_1 = 1.00 mm (riverside)　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　0.10 m

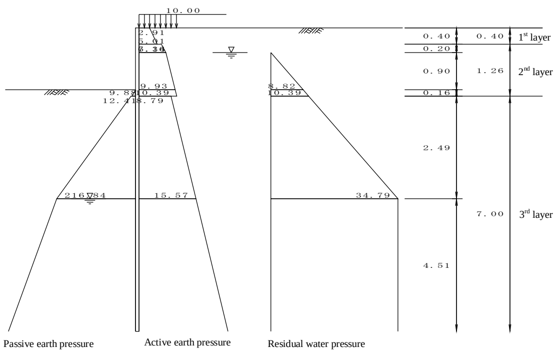
Allowable stress　σ_a = 180 N/mm (Normal)
σ_a' = 270 N/mm (Seismic)

Allowable displacement　δ_a = 50.0 mm (Normal)
δ_a' = 75.0 mm (Seismic)

Bending of cantilever beam　calculated as distributed load of each layer

Reduction of material modulus　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure



2-1 Normal Condition

2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 0.40	Sandy soil	18.0	30.0	—— ——	10.000 17.200	0.30142 0.30142	0.29115 0.29115
2	0.40～ 0.60	Sandy soil	17.0	24.5	—— ——	17.200 20.600	0.36978 0.36978	0.35718 0.35718
3	0.60～ 1.50	Sandy soil	8.0	24.5	—— ——	20.600 27.800	0.36978 0.36978	0.35718 0.35718
4	1.50～ 1.66	Sandy soil	8.0	24.5	—— ——	27.800 29.080	0.36978 0.36978	0.35718 0.35718
5	1.66～ 4.15	Sandy soil	9.0	29.0	—— ——	29.080 51.490	0.31307 0.31307	0.30240 0.30240
6	4.15～ 8.66	Sandy soil	9.0	29.0	—— ——	51.490 92.080	0.31307 0.31307	0.30240 0.30240
7	8.66～ 9.09	Clayey soil	9.1	—	24.0 24.0	92.080 96.000	—— ——	—— ——
8	9.09～ 10.66	Clayey soil	9.1	—	24.0 24.0	96.000 110.280	—— ——	—— ——
9	10.66～ 11.89	Clayey soil	7.9	—	30.0 30.0	110.280 120.000	—— ——	—— ——
10	11.89～ 19.40	Clayey soil	7.9	—	30.0 30.0	120.000 179.326	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
4	1.50～ 1.66	Sandy soil	17.0	24.5	—— ——	0.000 2.720	3.76087 3.76087	3.63272 3.63272
5	1.66～ 4.15	Sandy soil	18.0	29.0	—— ——	2.720 47.540	4.72202 4.72202	4.56112 4.56112
6	4.15～ 8.66	Sandy soil	9.0	29.0	—— ——	47.540 88.130	4.72202 4.72202	4.56112 4.56112
7	8.66～ 9.09	Clayey soil	9.1	0.0	24.0 24.0	88.130 92.050	—— ——	—— ——
8	9.09～ 10.66	Clayey soil	9.1	0.0	24.0 24.0	92.050 106.330	—— ——	—— ——
9	10.66～ 11.89	Clayey soil	7.9	0.0	30.0 30.0	106.330 116.050	—— ——	—— ——
10	11.89～ 19.40	Clayey soil	7.9	0.0	30.0 30.0	116.050 175.376	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 0.40	2.91 5.01	—— ——	2.91 5.01	—— ——	—— ——
2	0.40～ 0.60	6.14 7.36	—— ——	6.14 7.36	—— ——	—— ——
3	0.60～ 1.50	7.36 9.93	—— ——	7.36 9.93	0.00 8.82	—— ——
4	1.50～ 1.66	9.93 10.39	—— ——	9.93 10.39	8.82 10.39	0.00 9.88
5	1.66～ 4.15	8.79 15.57	—— ——	8.79 15.57	10.39 34.79	12.41 216.84
6	4.15～ 8.66	15.57 27.85	—— ——	15.57 27.85	34.79 34.79	216.84 401.97
7	8.66～ 9.09	44.08 48.00	46.04 48.00	46.04 48.00	34.79 34.79	136.13 140.05
8	9.09～ 10.66	48.00 62.28	48.00 55.14	48.00 62.28	34.79 34.79	140.05 154.33
9	10.66～ 11.89	50.28 60.00	55.14 60.00	55.14 60.00	34.79 34.79	166.33 176.05
10	11.89～ 19.40	60.00 119.33	60.00 89.66	60.00 119.33	34.79 34.79	176.05 235.38

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

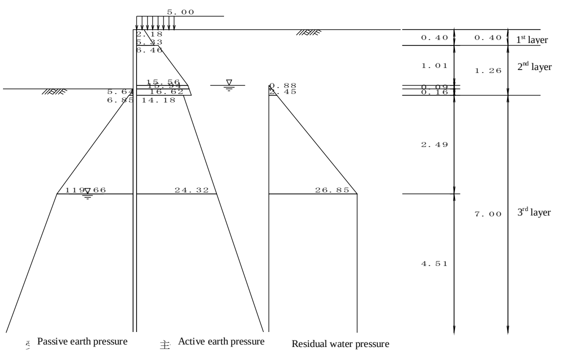
- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 0.40	Sandy Soil	18.0	30.0	—— ——	5.00 12.20	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	0.40～ 1.41	Sandy Soil	17.0	24.5	—— ——	12.20 29.37	0.00 0.00	0.200 0.200	11.31 11.31	0.54834 0.54834	0.52966 0.52966	—— ——
3	1.41～ 1.50	Sandy Soil	8.0	24.5	—— ——	29.37 30.09	0.00 0.88	0.200 0.200	11.31 11.31	0.54834 0.54834	0.52966 0.52966	—— ——
4	1.50～ 1.66	Sandy Soil	8.0	24.5	—— ——	30.09 31.37	0.88 2.45	0.200 0.200	11.31 11.31	0.54834 0.54834	0.52966 0.52966	—— ——
5	1.66～ 4.15	Sandy Soil	9.0	29.0	—— ——	31.37 53.78	2.45 26.85	0.200 0.200	11.31 11.31	0.46809 0.46809	0.45214 0.45214	—— ——
6	4.15～ 8.66	Sandy Soil	9.0	29.0	—— ——	53.78 94.37	26.85 71.05	0.200 0.200	11.31 11.31	0.46809 0.46809	0.45214 0.45214	—— ——

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
7	8.66～ 10.66	Clayey Soil	9.1	—	24.0 24.0	94.37 112.57	71.05 90.65	0.200 0.200	11.31 11.31	—— ——	—— ——	37.43 35.54
8	10.66～ 19.40	Clayey Soil	7.9	—	30.0 30.0	112.57 181.62	90.65 176.30	0.200 0.200	11.31 11.31	—— ——	—— ——	37.95 31.58

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
4	1.50～ 1.66	Sandy Soil	17.00	24.5	—— ——	0.000 2.720	0.00 0.00	0.200 0.200	11.31 11.31	2.07455 2.07455	2.07455 2.07455
5	1.66～ 4.15	Sandy Soil	18.00	29.0	—— ——	2.720 47.540	0.00 0.00	0.200 0.200	11.31 11.31	2.51705 2.51705	2.51705 2.51705
6	4.15～ 8.66	Sandy Soil	9.00	29.0	—— ——	47.540 88.130	0.00 44.20	0.200 0.200	11.31 11.31	2.51705 2.51705	2.51705 2.51705
7	8.66～ 10.66	Clayey soil	9.10	0.0	24.0 24.0	88.130 106.330	44.20 63.80	0.200 0.200	11.31 11.31	—— ——	—— ——
8	10.66～ 19.40	Clayey soil	7.90	0.0	30.0 30.0	106.330 175.376	63.80 149.45	0.200 0.200	11.31 11.31	—— ——	—— ——

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;

δ = -0.00、β = 0.00、θ = $\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 0.40	2.18 5.33	—— ——	—— ——
2	0.40～ 1.41	6.46 15.56	—— ——	—— ——
3	1.41～ 1.50	15.56 15.94	0.00 0.88	—— ——
4	1.50～ 1.66	15.94 16.62	0.88 2.45	0.00 5.64
5	1.66～ 4.15	14.18 24.32	2.45 26.85	6.85 119.66

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
6	4.15～ 8.66	24.32 42.67	26.85 26.85	119.66 221.83
7	8.66～ 10.66	69.30 93.35	26.85 26.85	136.13 154.33
8	10.66～ 19.40	79.58 173.46	26.85 26.85	166.33 235.38

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot \left[\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)} \right] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$
$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$
K_c : Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [\, K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}\,] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot \left[\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)} \right] \cdot \cos\delta$

Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

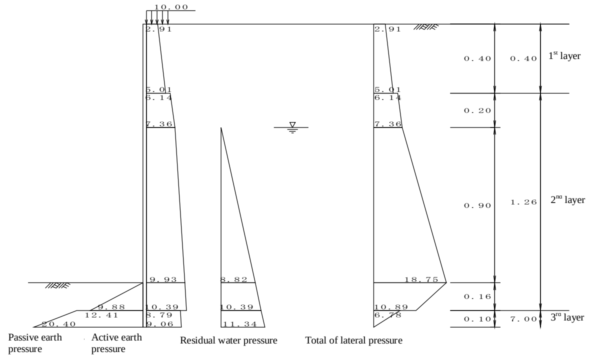
Mixed soil $P_p = [\, K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}\,] \cdot \cos\delta$

Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition

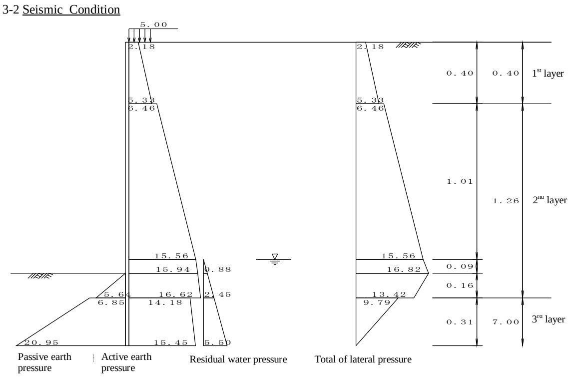


	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 0.40	2.91 5.01	—— ——	—— ——	2.91 5.01
2	0.40～ 0.60	6.14 7.36	—— ——	—— ——	6.14 7.36
3	0.60～ 1.50	7.36 9.93	0.00 8.82	—— ——	7.36 18.75
4	1.50～ 1.66	9.93 10.39	8.82 10.39	0.00 9.88	18.75 10.89
5	1.66～ 1.76	8.79 9.06	10.39 11.34	12.41 20.40	6.78 0.00
6	1.76～ 4.15	9.06 15.57	11.34 34.79	20.40 216.84	0.00 -166.47

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.26 m (GL -1.76 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 0.40	2.18 5.33	—— ——	—— ——	2.18 5.33
2	0.40～ 1.41	6.46 15.56	—— ——	—— ——	6.46 15.56
3	1.41～ 1.50	15.56 15.94	0.00 0.88	—— ——	15.56 16.82
4	1.50～ 1.66	15.94 16.62	0.88 2.45	0.00 5.64	16.82 13.42
5	1.66～ 1.97	14.18 15.45	2.45 5.50	6.85 20.95	9.79 0.00
6	1.97～ 4.15	15.45 24.32	5.50 26.85	20.95 119.66	0.00 -68.49

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.47 m (GL -1.97 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{\,0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)
	I	=	20008 cm⁴(after reduction by corrosion and section)
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$

	Depth (m)	N-value
1	1.66	13
2	2.66	5
3	3.66	6
4	4.66	2
5	5.66	11
6	6.66	4
7	7.66	23
8	8.66	7
9	9.66	4
10	10.66	4

	Depth (m)	N-value
11	11.66	4
12	12.66	5
13	13.66	6
14	14.66	5
15	15.66	4
16	16.66	4
17	17.66	4
18	18.66	6
19	19.66	6
20	20.66	50

4-2 Normal Condition

K_h = 15827 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.561 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.78 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -1.76 m) to 1.78 m depth (GL -3.54 m).

	Depth (m)	N-value
1	1.76	12.22
2	2.66	5.00
3	3.54	5.88
	Σh = 23.10	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{23.10}{3}$

$= 7.70$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 7.70^{0.406} = 15827$ kN/m³

Kh (normal condition) = 15827 kN/m^3

4-3 Seismic Condition

K_h = 15011 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.553 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.81 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -1.97 m) and 1.81 m depth (GL -3.78 m).

	Depth (m)	N-value
1	1.97	10.51
2	2.66	5.00
3	3.66	6.00
4	3.78	5.53
	Σh = 27.04	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{27.04}{4}$

$= 6.76$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 6.76^{0.406} = 15011$ kN/m³

Kh (seismic condition) = 15011 kN/m^3

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 0.40	0.40	2.91 5.01	0.58 1.00	1.62 1.49	0.95 1.49
2	0.40～ 0.60	0.20	6.14 7.36	0.61 0.74	1.29 1.22	0.79 0.90
3	0.60～ 1.50	0.90	7.36 18.75	3.31 8.44	0.86 0.56	2.84 4.70
4	1.50～ 1.66	0.16	18.75 10.89	1.50 0.87	0.20 0.15	0.31 0.13
5	1.66～ 1.76	0.10	6.78 0.00	0.33 0.00	0.06 0.03	0.02 0.00
				ΣP = 17.38		ΣM = 12.13

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	1.50 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.26 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{12.13}{17.38} = 0.70 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 0.40	0.40	2.18 5.33	0.44 1.07	1.84 1.70	0.80 1.82
2	0.40～ 1.41	1.01	6.46 15.56	3.26 7.86	1.23 0.90	4.03 7.05
3	1.41～ 1.50	0.09	15.56 16.82	0.70 0.76	0.53 0.50	0.37 0.38
4	1.50～ 1.66	0.16	16.82 13.42	1.35 1.07	0.42 0.36	0.56 0.39
5	1.66～ 1.97	0.31	9.79 0.00	1.52 0.00	0.21 0.10	0.32 0.00
				ΣP = 18.02		ΣM = 15.72

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	1.50 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.47 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{15.72}{18.02} = 0.87 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	15827 kN/m³
calculated value	β	=	0.56076 m⁻¹
resultant earth force (lateral)	P_0	=	17.38 kN/m
height of acting position of load	h_0	=	0.70 m
moment	M_0	=	12.13 kN·m/m
in consideration of ψ_m = 1.566,			
maximum moment	M_{max}	=	19.00 kN·m/m
depth of generated position of M_{max}	l_m	=	0.912 m
depth of 1st fixed point	l_i	=	2.312 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	15011 kN/m³
calculated value	β	=	0.55338 m⁻¹
resultant earth force (lateral)	P_0	=	18.02 kN/m
height of acting position of load	h_0	=	0.87 m
moment	M_0	=	15.72 kN·m/m
in consideration of ψ_m = 1.427,			
maximum moment	M_{max}	=	22.43 kN·m/m
depth of generated position of M_{max}	l_m	=	0.850 m
depth of 1st fixed point	l_i	=	2.270 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{19.00\times10^6}{1320\times10^3} = 14\ \mathrm{N/mm^2} \leqq \sigma_a = 180\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{22.43\times10^6}{1320\times10^3} = 17\ \mathrm{N/mm^2} \leqq \sigma_a = 270\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 0.40	1.62 1.49	0.924 0.848	0.295 0.258	0.58 1.00	0.172 0.258
2	0.40～ 0.60	1.29 1.22	0.734 0.697	0.204 0.186	0.61 0.74	0.125 0.137
3	0.60～ 1.50	0.86 0.56	0.488 0.317	0.100 0.045	3.31 8.44	0.330 0.380
4	1.50～ 1.66	0.20 0.15	0.116 0.086	0.006 0.004	1.50 0.87	0.010 0.003
5	1.66～ 1.76	0.06 0.03	0.037 0.018	0.001 0.000	0.33 0.00	0.000 0.000
						ΣQ = 1.415

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

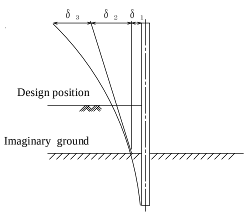
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5608\times0.70)\times17.38}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5608^3} = 0.00171\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5608\times0.70)\times17.38}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5608^2}\times(1.50+0.26) = 0.00216\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{1.42\times(1.50+0.26)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00019\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00171 + 0.00216 + 0.00019$
$= 0.00407$ m
$= 4.07 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 0.40	1.84 1.70	0.932 0.865	0.300 0.266	0.44 1.07	0.131 0.284
2	0.40～ 1.41	1.23 0.90	0.626 0.456	0.155 0.088	3.26 7.86	0.506 0.691
3	1.41～ 1.50	0.53 0.50	0.270 0.254	0.033 0.030	0.70 0.76	0.023 0.022
4	1.50～ 1.66	0.42 0.36	0.212 0.185	0.021 0.016	1.35 1.07	0.028 0.017
5	1.66～ 1.97	0.21 0.10	0.105 0.053	0.005 0.001	1.52 0.00	0.008 0.000
						ΣQ = 1.711

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

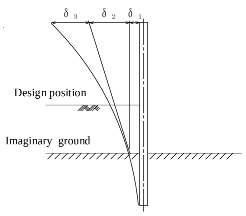
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5534\times0.87)\times18.02}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5534^3} = 0.00197\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5534\times0.87)\times18.02}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5534^2}\times(1.50+0.47) = 0.00285\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{1.71\times(1.50+0.47)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00033\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00197 + 0.00285 + 0.00033$
$= 0.00515$ m
$= 5.15 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction	$K_h = 15827 kN/m^3$
Calculated value	$\beta = 0.53362 m^{-1}$
Penetration length of SSP	$D = 0.26 + \dfrac{3}{0.534} = 5.88 m$
Whole length of SSP	$L = 1.50 - 0.40 + 5.88 = 6.98 m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction	$K_h = 15011 kN/m^3$
Calculated value	$\beta = 0.52660 m^{-1}$
Penetration length of SSP	$D = 0.47 + \dfrac{3}{0.527} = 6.17 m$
Whole length of SSP	$L = 1.50 - 0.40 + 6.17 = 7.27 m$

Therefore, whole length of SSP is set as 7.30 m in consideration of round unit of SSP length.

7 <u>Calculation Result</u>

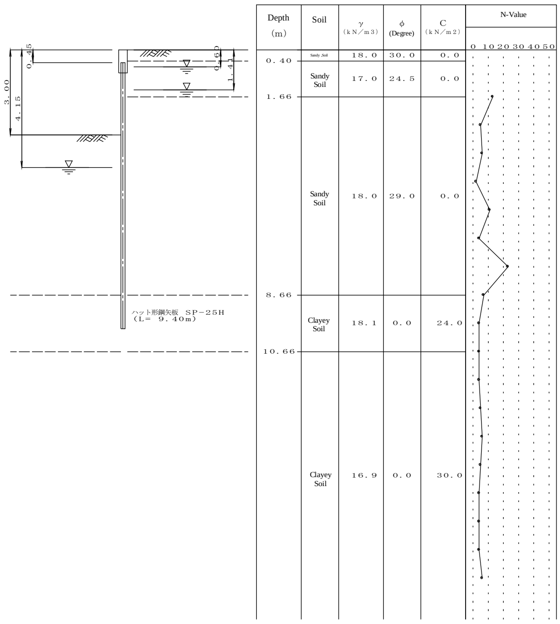
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		19.00	22.43
Stress intensity	σ (N/mm^2)		14 (180)	17 (270)
Lateral displacement	δ (mm)		4.07 (50.0)	5.15 (75.0)
Penetration depth	D (m)		5.88	6.17
Whole length of SSP	L (m)	7.30		

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 1+325 NJ

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	0.60 m (Normal Condition)
	L_{wa}'	=	1.41 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.15 m (Normal Condition)
	L_{wp}'	=	4.15 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang’s formula

Penetration depths　　$L = \frac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}}\cdot\tan\theta$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value
1	1.66	13
2	2.66	5
3	3.66	6
4	4.66	2
5	5.66	11
6	6.66	4
7	7.66	23
8	8.66	7
9	9.66	4
10	10.66	4

No	Depth (m)	N-Value
11	11.66	4
12	12.66	5
13	13.66	6
14	14.66	5
15	15.66	4
16	16.66	4
17	17.66	4
18	18.66	6
19	19.66	6
20	20.66	50

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	ϕ	C kN/m^2	a	k'	ζ (degree)		kh(kN/m^3)	
										normal	seismic	normal	seismic
1	0.40	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	――	――
2	1.66	S	13.0	17.00	8.00	24.5	0.0	0.0	0.200	auto	auto	――	――
3	8.66	S	6.0	18.00	9.00	29.0	0.0	0.0	0.200	auto	auto	――	――
4	10.66	C	4.0	18.10	9.10	0.0	24.0	0.0	0.200	auto	auto	――	――
5	19.40	C	5.0	16.90	7.90	0.0	30.0	0.0	0.200	auto	auto	――	――

Note)
depth : from top of coping to bottom of the layer
soil : sandy(S), clayey(C), mixed (M)
N-value : average N-value in the layer
γ : wet unit weight of soil
γ' : saturated unit weight of soil
ϕ : internal friction angle of soil
C_o : soil adhesion
a : slope of soil adhesion
k’ : design seismic coefficient (underwater)
ζ : angle of active rupture
kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young’s modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

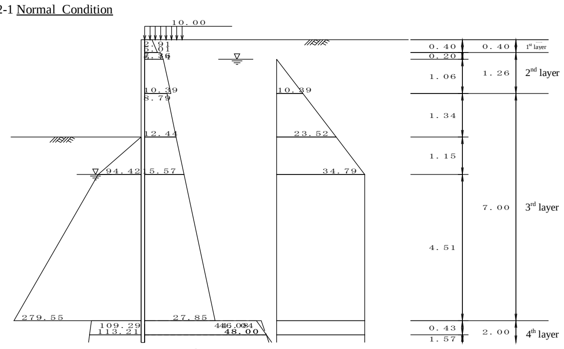
Allowable stress	σ_a	=	180 N/mm（Normal）
	σ_a'	=	270 N/mm（Seismic）
Allowable displacement	δ_a	=	50.0 mm（Normal）
	δ_a'	=	75.0 mm（Seismic）

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 0.40	Sandy soil	18.0	30.0	―― ――	10.000 17.200	0.30142 0.30142	0.29115 0.29115
2	0.40～ 0.60	Sandy soil	17.0	24.5	―― ――	17.200 20.600	0.36978 0.36978	0.35718 0.35718
3	0.60～ 1.66	Sandy soil	8.0	24.5	―― ――	20.600 29.080	0.36978 0.36978	0.35718 0.35718
4	1.66～ 3.00	Sandy soil	9.0	29.0	―― ――	29.080 41.140	0.31307 0.31307	0.30240 0.30240
5	3.00～ 4.15	Sandy soil	9.0	29.0	―― ――	41.140 51.490	0.31307 0.31307	0.30240 0.30240
6	4.15～ 8.66	Sandy soil	9.0	29.0	―― ――	51.490 92.080	0.31307 0.31307	0.30240 0.30240
7	8.66～ 9.09	Clayey soil	9.1	―	24.0 24.0	92.080 96.000	―― ――	―― ――
8	9.09～ 10.66	Clayey soil	9.1	―	24.0 24.0	96.000 110.280	―― ――	―― ――
9	10.66～ 11.89	Clayey soil	7.9	―	30.0 30.0	110.280 120.000	―― ――	―― ――
10	11.89～ 19.40	Clayey soil	7.9	―	30.0 30.0	120.000 179.326	―― ――	―― ――

Case: Lower Marikina Section 1+325 NJ

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
5	3.00～ 4.15	Sandy soil	18.0	29.0	―― ――	0.000 20.700	4.72202 4.72202	4.56112 4.56112
6	4.15～ 8.66	Sandy soil	9.0	29.0	―― ――	20.700 61.290	4.72202 4.72202	4.56112 4.56112
7	8.66～ 9.09	Clayey soil	9.1	0.0	24.0 24.0	61.290 65.210	―― ――	―― ――
8	9.09～ 10.66	Clayey soil	9.1	0.0	24.0 24.0	65.210 79.490	―― ――	―― ――
9	10.66～ 11.89	Clayey soil	7.9	0.0	30.0 30.0	79.490 89.210	―― ――	―― ――
10	11.89～ 19.40	Clayey soil	7.9	0.0	30.0 30.0	89.210 148.536	―― ――	―― ――

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 0.40	2.91 5.01	―― ――	2.91 5.01	―― ――	―― ――
2	0.40～ 0.60	6.14 7.36	―― ――	6.14 7.36	―― ――	―― ――
3	0.60～ 1.66	7.36 10.39	―― ――	7.36 10.39	0.00 10.39	―― ――
4	1.66～ 3.00	8.79 12.44	―― ――	8.79 12.44	10.39 23.52	―― ――
5	3.00～ 4.15	12.44 15.57	―― ――	12.44 15.57	23.52 34.79	0.00 94.42
6	4.15～ 8.66	15.57 27.85	―― ――	15.57 27.85	34.79 34.79	94.42 279.55
7	8.66～ 9.09	44.08 48.00	46.04 48.00	46.04 48.00	34.79 34.79	109.29 113.21
8	9.09～ 10.66	48.00 62.28	48.00 55.14	48.00 62.28	34.79 34.79	113.21 127.49
9	10.66～ 11.89	50.28 60.00	55.14 60.00	55.14 60.00	34.79 34.79	139.49 149.21
10	11.89～ 19.40	60.00 119.33	60.00 89.66	60.00 119.33	34.79 34.79	149.21 208.54

Case: Lower Marikina Section 1+325 NJ

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$
$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$
K_c : Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [\,K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}\,] \cdot \cos\delta$

- Formula for passive earth pressure

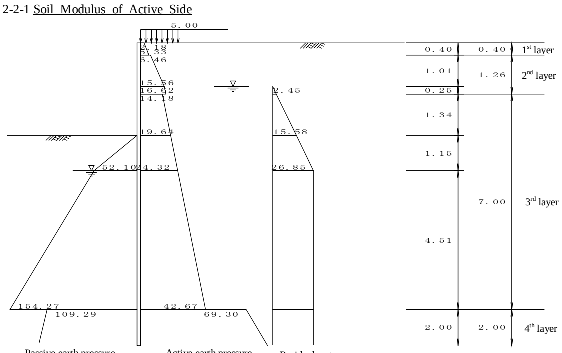
Sandy soil $P_p = K_p \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = [\,K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}\,] \cdot \cos\delta$

2-2 Seismic Condition

2-2-1 Soil Modulus of Active Side



	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 0.40	Sandy Soil	18.0	30.0	―― ――	5.00 12.20	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	―― ――
2	0.40～ 1.41	Sandy Soil	17.0	24.5	―― ――	12.20 29.37	0.00 0.00	0.200 0.200	11.31 11.31	0.54834 0.54834	0.52966 0.52966	―― ――
3	1.41～ 1.66	Sandy Soil	8.0	24.5	―― ――	29.37 31.37	0.00 2.45	0.200 0.200	11.31 11.31	0.54834 0.54834	0.52966 0.52966	―― ――
4	1.66～ 3.00	Sandy Soil	9.0	29.0	―― ――	31.37 43.43	2.45 15.58	0.200 0.200	11.31 11.31	0.46809 0.46809	0.45214 0.45214	―― ――
5	3.00～ 4.15	Sandy Soil	9.0	29.0	―― ――	43.43 53.78	15.58 26.85	0.200 0.200	11.31 11.31	0.46809 0.46809	0.45214 0.45214	―― ――
6	4.15～ 8.66	Sandy Soil	9.0	29.0	―― ――	53.78 94.37	26.85 71.05	0.200 0.200	11.31 11.31	0.46809 0.46809	0.45214 0.45214	―― ――

Case: Lower Marikina Section 1+325 NJ

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
7	8.66～ 10.66	Clayey Soil	9.1	―	24.0 24.0	94.37 112.57	71.05 90.65	0.200 0.200	11.31 11.31	―― ――	―― ――	37.43 35.54
8	10.66～ 19.40	Clayey Soil	7.9	―	30.0 30.0	112.57 181.62	90.65 176.30	0.200 0.200	11.31 11.31	―― ――	―― ――	37.95 31.58

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
5	3.00～ 4.15	Sandy Soil	18.00	29.0	―― ――	0.000 20.700	0.00 0.00	0.200 0.200	11.31 11.31	2.51705 2.51705	2.51705 2.51705
6	4.15～ 8.66	Sandy Soil	9.00	29.0	―― ――	20.700 61.290	0.00 44.20	0.200 0.200	11.31 11.31	2.51705 2.51705	2.51705 2.51705
7	8.66～ 10.66	Clayey Soil	9.10	0.0	24.0 24.0	61.290 79.490	44.20 63.80	0.200 0.200	11.31 11.31	―― ――	―― ――
8	10.66～ 19.40	Clayey soil	7.90	0.0	30.0 30.0	79.490 148.536	63.80 149.45	0.200 0.200	11.31 11.31	―― ――	―― ――

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
δ = -0.00、β = 0.00、θ = $\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 0.40	2.18 5.33	―― ――	―― ――
2	0.40～ 1.41	6.46 15.56	―― ――	―― ――
3	1.41～ 1.66	15.56 16.62	0.00 2.45	―― ――
4	1.66～ 3.00	14.18 19.64	2.45 15.58	―― ――
5	3.00～ 4.15	19.64 24.32	15.58 26.85	0.00 52.10
6	4.15～ 8.66	24.32 42.67	26.85 26.85	52.10 154.27

No	Depth (m)	Active side		
		Pa1 kN/m^2	Pa2 kN/m^2	Pa kN/m^2
7	8.66～ 10.66	69.30 93.35	26.85 26.85	109.29 127.49
8	10.66～ 19.40	79.58 173.46	26.85 26.85	139.49 208.54

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

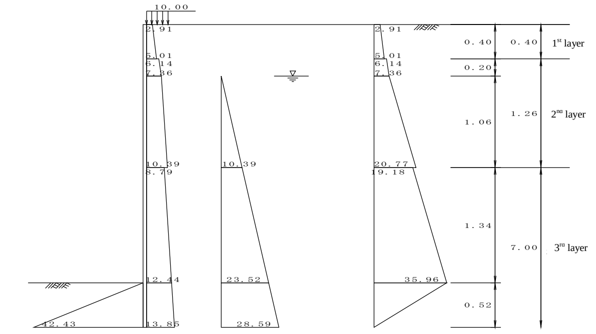
Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



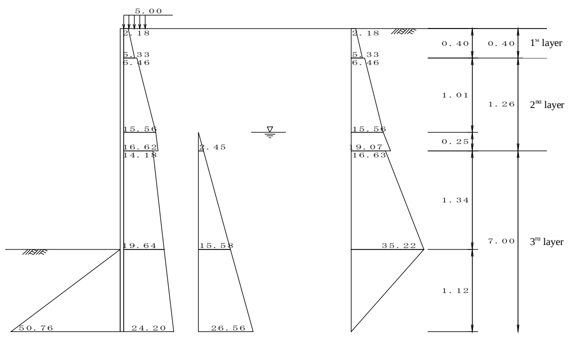
Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 0.40	2.91 5.01	—	—	2.91 5.01
2	0.40～ 0.60	6.14 7.36	—	—	6.14 7.36
3	0.60～ 1.66	7.36 10.39	0.00 10.39	—	7.36 20.77
4	1.66～ 3.00	8.79 12.44	10.39 23.52	—	19.18 35.96
5	3.00～ 3.52	12.44 13.85	23.52 28.59	0.00 42.43	35.96 0.00
6	3.52～ 4.15	13.85 15.57	28.59 34.79	42.43 94.42	0.00 -44.05

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.52 m (GL -3.52 m)

3-2 Seismic Condition



Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 0.40	2.18 5.33	—	—	2.18 5.33
2	0.40～ 1.41	6.46 15.56	—	—	6.46 15.56
3	1.41～ 1.66	15.56 16.62	0.00 2.45	—	15.56 19.07
4	1.66～ 3.00	14.18 19.64	2.45 15.58	—	16.63 35.22
5	3.00～ 4.12	19.64 24.20	15.58 26.56	0.00 50.76	35.22 0.00
6	4.12～ 4.15	24.20 24.32	26.56 26.85	50.76 52.10	0.00 -0.93

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 1.12 m (GL -4.12 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m			
Corrosion margin	t_1	=	1.00 mm (active side)	t_2	=	1.00 mm (passive side)
Corrosion rate	η	=	0.82			
Section efficiency	μ	=	1.00			
Young's modulus	E	=	200000 N/mm^2			
Inertia sectional moment	I_0	=	24400 cm^4(original condition)			
	I	=	20008 cm^4(after reduction by corrosion and section)			
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$			

	Depth (m)	N-value		Depth (m)	N-value
1	1.66	13	11	11.66	4
2	2.66	5	12	12.66	5
3	3.66	6	13	13.66	6
4	4.66	2	14	14.66	5
5	5.66	11	15	15.66	4
6	6.66	4	16	16.66	4
7	7.66	23	17	17.66	4
8	8.66	7	18	18.66	6
9	9.66	4	19	19.66	6
10	10.66	4	20	20.66	50

4-2 Normal Condition

K_h = 13847 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$= 0.542 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.84 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.52 m) to 1.84m depth (GL -5.36 m).

	Depth (m)	N-value
1	3.52	5.86
2	3.66	6.00
3	4.66	2.00
4	5.36	8.31
	Σh = 22.16	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{22.16}{4}$

$= 5.54$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 5.54^{0.406} = 13847$ kN/m³
Kh (normal condition) = 13847 kN/m³

4-3 Seismic Condition

K_h = 14832 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.552 \quad m^{-1}$

$$L = \frac{1}{\beta} = 1.81 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -4.12 m) and 1.81 m depth (GL -5.93 m).

	Depth (m)	N-value
1	4.12	4.16
2	4.66	2.00
3	5.66	11.00
4	5.93	9.09
	Σh = 26.25	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{26.25}{4}$

$= 6.56$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 6.56^{0.406} = 14832$ kN/m³
Kh (seismic condition) = 14832 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 0.40	0.40	2.91 5.01	0.58 1.00	3.38 3.25	1.97 3.26
2	0.40～ 0.60	0.20	6.14 7.36	0.61 0.74	3.05 2.98	1.87 2.20
3	0.60～ 1.66	1.06	7.36 20.77	3.90 11.01	2.56 2.21	10.00 24.34
4	1.66～ 3.00	1.34	19.18 35.96	12.85 24.09	1.41 0.96	18.12 23.21
5	3.00～ 3.52	0.52	35.96 0.00	9.29 0.00	0.34 0.17	3.20 0.00
				ΣP = 64.08		ΣM = 88.17

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.52 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m
h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$
$$= \frac{88.17}{64.08} = 1.38 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 0.40	0.40	2.18 5.33	0.44 1.07	3.99 3.85	1.74 4.11
2	0.40～ 1.41	1.01	6.46 15.56	3.26 7.86	3.38 3.05	11.04 23.94
3	1.41～ 1.66	0.25	15.56 19.07	1.94 2.38	2.63 2.54	5.11 6.06
4	1.66～ 3.00	1.34	16.63 35.22	11.14 23.60	2.01 1.57	22.44 36.97
5	3.00～ 4.12	1.12	35.22 0.00	19.73 0.00	0.75 0.37	14.73 0.00
				ΣP = 71.42		ΣM = 126.14

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	1.12 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$
$$= \frac{126.14}{71.42} = 1.77 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} (\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	13847	kN/m³
calculated value	β	=	0.54234	m^{-1}
resultant earth force (lateral)	P_0	=	64.08	kN/m
height of acting position of load	h_0	=	1.38	m
moment	M_0	=	88.17	kN·m/m

in consideration of ψ_m = 1.229,

maximum moment	M_{max}	=	108.33	kN·m/m
depth of generated position of M_{max}	l_m	=	0.704	m
depth of 1st fixed point	l_i	=	2.152	m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	14832	kN/m³
calculated value	β	=	0.55173	m^{-1}
resultant earth force (lateral)	P_0	=	71.42	kN/m
height of acting position of load	h_0	=	1.77	m
moment	M_0	=	126.14	kN·m/m

in consideration of ψ_m = 1.152,

maximum moment	M_{max}	=	145.34	kN·m/m
depth of generated position of M_{max}	l_m	=	0.593	m
depth of 1st fixed point	l_i	=	2.016	m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{108.33\times10^6}{1320\times10^3} = 82\ \text{N/mm}^2 \leqq \sigma_a = 180\ \text{N/mm}^2 \quad \text{(ok)}$$

$$\sigma = \frac{M_{max}}{Z} = \frac{145.34\times10^6}{1320\times10^3} = 110\ \text{N/mm}^2 \leqq \sigma_a = 270\ \text{N/mm}^2 \quad \text{(ok)}$$

5-4 Displacement

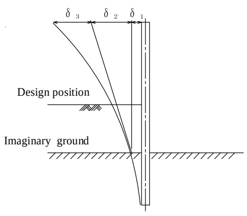
5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 0.40	3.38 3.25	0.962 0.924	0.314 0.295	0.58 1.00	0.183 0.296
2	0.40～ 0.60	3.05 2.98	0.867 0.848	0.267 0.258	0.61 0.74	0.164 0.190
3	0.60～ 1.66	2.56 2.21	0.729 0.628	0.201 0.156	3.90 11.01	0.784 1.719
4	1.66～ 3.00	1.41 0.96	0.401 0.274	0.070 0.034	12.85 24.09	0.895 0.822
5	3.00～ 3.52	0.34 0.17	0.098 0.049	0.005 0.001	9.29 0.00	0.043 0.000
						ΣQ = 5.096

Y : Height from imaginary riverbed to acting position
α : $\alpha = \dfrac{Y}{H+L_k}$
ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$
Q : $\zeta \times P$
P : Lateral force
H : Depth to design position
L_k : Depth from design position to imaginary ground

Displacement



Case: Lower Marikina Section 1+325 NJ

$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5423\times1.38)\times64.08}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5423^3} = 0.00877\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5423\times1.38)\times64.08}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5423^2}\times(3.00+0.52) = 0.02386\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{5.10\times(3.00+0.52)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00554\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00877 + 0.02386 + 0.00554$
$= 0.03817$ m
$= 38.17 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

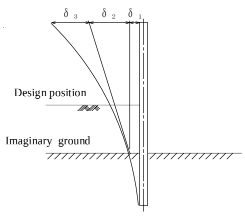
Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 0.40	3.99 3.85	0.968 0.935	0.317 0.301	0.44 1.07	0.138 0.321
2	0.40～ 1.41	3.38 3.05	0.821 0.740	0.245 0.206	3.26 7.86	0.799 1.619
3	1.41～ 1.66	2.63 2.54	0.638 0.617	0.160 0.151	1.94 2.38	0.311 0.361
4	1.66～ 3.00	2.01 1.57	0.489 0.380	0.100 0.063	11.14 23.60	1.114 1.490
5	3.00～ 4.12	0.75 0.37	0.181 0.091	0.015 0.004	19.73 0.00	0.304 0.000
						ΣQ = 6.457

Y : Height from imaginary riverbed to acting position
α : $\alpha = \dfrac{Y}{H+L_k}$
ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$
Q : $\zeta \times P$
P : Lateral force
H : Depth to design position
L_k : Depth from design position to imaginary ground

Case: Lower Marikina Section 1+325 NJ

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5517\times1.77)\times71.42}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5517^3} = 0.01049\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5517\times1.77)\times71.42}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5517^2}\times(3.00+1.12) = 0.03562\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{6.46\times(3.00+1.12)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.01129\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.01049 + 0.03562 + 0.01129$
$= 0.05740$ m
$= 57.40 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

Case: Lower Marikina Section 1+325 NJ

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP (Chang)

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$
$$L = H - H_{1t} + D$$
$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction	$K_h = 13847 kN/m^3$
Calculated value	$\beta = 0.51609 m^{-1}$
Penetration length of SSP	$D = 0.52 + \dfrac{3}{0.516} = 6.33 m$
Whole length of SSP	$L = 3.00 - 0.40 + 6.33 = 8.93 m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction	$K_h = 14832 kN/m^3$
Calculated value	$\beta = 0.52503 m^{-1}$
Penetration length of SSP	$D = 1.12 + \dfrac{3}{0.526} = 6.83 m$
Whole length of SSP	$L = 3.00 - 0.40 + 6.83 = 9.43 m$

Therefore, whole length of SSP is set as 9.50 m in consideration of round unit of SSP length.

7 Calculation Result

			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		108.33	145.34
Stress intensity	σ (N/mm^2)		82 (180)	110 (270)
Lateral displacement	δ (mm)		38.17 (50.0)	57.40 (75.0)
Penetration depth	D (m)		6.33	6.83
Whole length of SSP	L (m)	9.50		

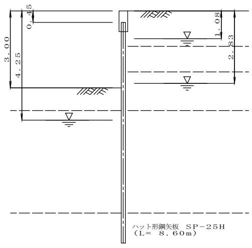
s

－Steel Sheet Pile　Design Calculation－

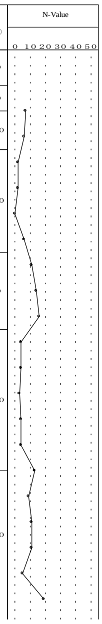
Lower Marikina Section 3 + 170 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



Depth (m)	Soil	γ (kN/m3)	φ (Degree)	C (kN/m2)
1.38	Sandy Soil	18.0	30.0	0.0
2.38	Sandy Soil	17.0	30.0	0.0
3.88	Clayey Soil	17.3	0.0	36.0
7.88	Clayey Soil	17.3	0.0	12.0
10.88	Sandy Soil	18.0	26.0	0.0
16.38	Clayey Soil	18.0	0.0	24.0
	Clayey Soil	18.0	0.0	66.0



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	1.08 m (Normal Condition)
	L_{wa}'	=	2.83 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.25 m (Normal Condition)
	L_{wp}'	=	4.25 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang’s formula

Penetration depths　　$L = \dfrac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1-\dfrac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{\,0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value	No	Depth (m)	N-Value
1	2.38	7	11	12.38	4
2	3.38	6	12	13.38	3
3	4.38	2	13	14.38	4
4	5.38	2	14	15.38	4
5	6.38	0	15	16.38	13
6	7.38	6	16	17.38	9
7	8.38	11	17	18.38	11
8	9.38	14	18	19.38	11
9	10.38	16	19	20.38	5
10	11.38	4	20	21.38	19

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree) normal	ζ (degree) seismic	kh(kN/m^3) normal	kh(kN/m^3) seismic
1	1.38	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	――	――
2	2.38	S	9.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	――	――
3	3.88	C	6.0	17.30	8.30	0.0	36.0	0.0	0.200	auto	auto	――	――
4	7.88	C	2.0	17.30	8.30	0.0	12.0	0.0	0.200	auto	auto	――	――
5	10.88	S	12.0	18.00	9.00	26.0	0.0	0.0	0.200	auto	auto	――	――
6	16.38	C	4.0	18.00	9.00	0.0	24.0	0.0	0.200	auto	auto	――	――
7	21.38	C	11.0	18.00	9.00	0.0	66.0	0.0	0.200	auto	auto	――	――

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- φ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k’ : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young’s modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm（Normal）
σ_a' = 270 N/mm（Seismic）

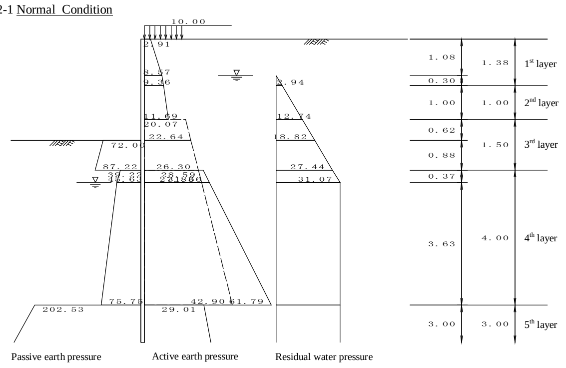
Allowable displacement　　δ_a = 50.0 mm（Normal）
δ_a' = 75.0 mm（Seismic）

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.08	Sandy soil	18.0	30.0	—— ——	10.000 29.440	0.30142 0.30142	0.29115 0.29115
2	1.08～ 1.38	Sandy soil	9.0	30.0	—— ——	29.440 32.140	0.30142 0.30142	0.29115 0.29115
3	1.38～ 2.38	Sandy soil	8.0	30.0	—— ——	32.140 40.140	0.30142 0.30142	0.29115 0.29115
4	2.38～ 3.00	Clayey soil	8.3	—	36.0 36.0	40.140 45.286	—— ——	—— ——
5	3.00～ 3.88	Clayey soil	8.3	—	36.0 36.0	45.286 52.590	—— ——	—— ——
6	3.88～ 4.25	Clayey soil	8.3	—	12.0 12.0	52.590 55.661	—— ——	—— ——
7	4.25～ 7.88	Clayey soil	8.3	—	12.0 12.0	55.661 85.790	—— ——	—— ——
8	7.88～ 10.88	Sandy soil	9.0	26.0	—— ——	85.790 112.790	0.35007 0.35007	0.33814 0.33814
9	10.88～ 16.38	Clayey soil	9.0	—	24.0 24.0	112.790 162.290	—— ——	—— ——
10	16.38～ 21.38	Clayey soil	9.0	—	66.0 66.0	162.290 207.290	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
5	3.00～ 3.88	Clayey soil	17.3	0.0	36.0 36.0	0.000 15.224	—— ——	—— ——
6	3.88～ 4.25	Clayey soil	17.3	0.0	12.0 12.0	15.224 21.625	—— ——	—— ——
7	4.25～ 7.88	Clayey soil	8.3	0.0	12.0 12.0	21.625 51.754	—— ——	—— ——
8	7.88～ 10.88	Sandy soil	9.0	26.0	—— ——	51.754 78.754	4.05145 4.05145	3.91340 3.91340
9	10.88～ 16.38	Clayey soil	9.0	0.0	24.0 24.0	78.754 128.254	—— ——	—— ——
10	16.38～ 21.38	Clayey soil	9.0	0.0	66.0 66.0	128.254 173.254	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.08	2.91 8.57	—— ——	2.91 8.57	—— ——	—— ——
2	1.08～ 1.38	8.57 9.36	—— ——	8.57 9.36	0.00 2.94	—— ——
3	1.38～ 2.38	9.36 11.69	—— ——	9.36 11.69	2.94 12.74	—— ——
4	2.38～ 3.00	-31.86 -26.71	20.07 22.64	20.07 22.64	12.74 18.82	—— ——
5	3.00～ 3.88	-26.71 -19.41	22.64 26.30	22.64 26.30	18.82 27.44	72.00 87.22
6	3.88～ 4.25	28.59 31.66	26.30 27.83	28.59 31.66	27.44 31.07	39.22 45.63
7	4.25～ 7.88	31.66 61.79	27.83 42.90	31.66 61.79	31.07 31.07	45.63 75.75
8	7.88～ 10.88	29.01 38.14	—— ——	29.01 38.14	31.07 31.07	202.53 308.20
9	10.88～ 16.38	64.79 114.29	56.40 81.14	64.79 114.29	31.07 31.07	126.75 176.25
10	16.38～ 21.38	30.29 75.29	81.14 103.64	81.14 103.64	31.07 31.07	260.25 305.25

- Formula for active earth pressure

Sandy soil $P_{a1}=K_a \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil $P_{a1}=\Sigma\gamma h+Q-2C$

$P_{a2}=K_c \cdot (\Sigma\gamma h+Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1}=[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

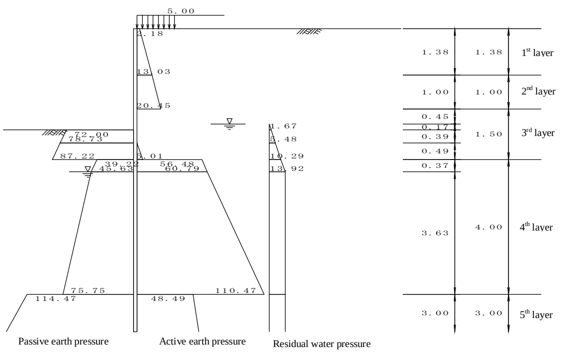
- Formula for passive earth pressure

Sandy soil $P_p=K_p \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil $P_p=\Sigma\gamma h+Q+2C$

Mixed soil $P_p=[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 1.38	Sandy Soil	18.0	30.0	—— ——	5.00 29.84	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	1.38～ 2.38	Sandy Soil	17.0	30.0	—— ——	29.84 46.84	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	2.38～ 2.83	Clayey Soil	17.3	—	36.0 36.0	46.84 54.63	0.00 0.00	0.200 0.200	11.31 11.31	—— ——	—— ——	42.78 42.41
4	2.83～ 3.00	Clayey Soil	8.3	—	36.0 36.0	54.63 56.04	0.00 1.67	0.200 0.200	11.31 11.31	—— ——	—— ——	42.41 42.34
5	3.00～ 3.39	Clayey Soil	8.3	—	36.0 36.0	56.04 59.27	1.67 ——	0.200 ——	11.31 ——	—— ——	—— ——	42.34 ——
6	3.39～ 3.88	Clayey Soil	8.3	—	36.0 36.0	59.27 63.34	—— 10.29	—— 0.200	—— 11.31	—— ——	—— ——	—— 41.99

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
7	3.88～ 4.25	Clayey Soil	8.3	—	12.0 12.0	63.34 66.41	10.29 13.92	0.200 0.200	11.31 11.31	—— ——	—— ——	33.27 32.47
8	4.25～ 7.88	Clayey Soil	8.3	—	12.0 12.0	66.41 96.54	13.92 49.49	0.200 0.200	11.31 11.31	—— ——	—— ——	32.47 21.42
9	7.88～ 10.88	Sandy Soil	9.0	26.0	—— ——	96.54 123.54	49.49 78.89	0.200 0.200	11.31 11.31	0.51997 0.51997	0.50225 0.50225	—— ——
10	10.88～ 16.38	Clayey Soil	9.0	—	24.0 24.0	123.54 173.04	78.89 132.79	0.200 0.200	11.31 11.31	—— ——	—— ——	34.27 26.94
11	16.38～ 21.38	Clayey Soil	9.0	—	66.0 66.0	173.04 218.04	132.79 181.79	0.200 0.200	11.31 11.31	—— ——	—— ——	40.52 39.13

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
5	3.00～ 3.39	Clayey soil	17.30	0.0	36.0 36.0	0.000 6.733	0.00 ——	0.200 ——	11.31 ——	—— ——	—— ——
6	3.39～ 3.88	Clayey Soil	17.30	0.0	36.0 36.0	6.733 15.224	—— 0.00	—— 0.200	—— 11.31	—— ——	—— ——
7	3.88～ 4.25	Clayey Soil	17.30	0.0	12.0 12.0	15.224 21.625	0.00 0.00	0.200 0.200	11.31 11.31	—— ——	—— ——
8	4.25～ 7.88	Clayey soil	8.30	0.0	12.0 12.0	21.625 51.754	0.00 35.57	0.200 0.200	11.31 11.31	—— ——	—— ——
9	7.88～ 10.88	Sandy Soil	9.00	26.0	—— ——	51.754 78.754	35.57 64.97	0.200 0.200	11.31 11.31	2.21182 2.21182	2.21182 2.21182
10	10.88～ 16.38	Clayey Soil	9.00	0.0	24.0 24.0	78.754 128.254	64.97 118.87	0.200 0.200	11.31 11.31	—— ——	—— ——
11	16.38～ 21.38	Clayey soil	9.00	0.0	66.0 66.0	128.254 173.254	118.87 167.87	0.200 0.200	11.31 11.31	—— ——	—— ——

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
δ = -0.00、β = 0.00、θ = $\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta-\theta) \cdot \left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 1.38	2.18 13.03	—— ——	—— ——
2	1.38～ 2.38	13.03 20.45	—— ——	—— ——
3	2.38～ 2.83	0.00 0.00	—— ——	—— ——
4	2.83～ 3.00	0.00 0.00	0.00 1.67	—— ——
5	3.00～ 3.39	0.00 0.00	1.67 5.48	72.00 78.73
6	3.39～ 3.88	0.00 5.01	5.48 10.29	78.73 87.22
7	3.88～ 4.25	56.48 60.79	10.29 13.92	39.22 45.63
8	4.25～ 7.88	60.79 110.47	13.92 13.92	45.63 75.75
9	7.88～ 10.88	48.49 62.05	13.92 13.92	114.47 174.19
10	10.88～ 16.38	108.22 181.72	13.92 13.92	126.75 176.25
11	16.38～ 21.38	79.90 136.82	13.92 13.92	260.25 305.25

- Formula for active earth pr Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

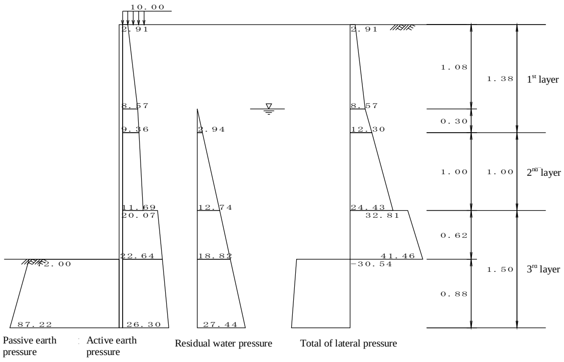
Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.08	2.91 8.57	—— ——	—— ——	2.91 8.57
2	1.08～ 1.38	8.57 9.36	0.00 2.94	—— ——	8.57 12.30
3	1.38～ 2.38	9.36 11.69	2.94 12.74	—— ——	12.30 24.43
4	2.38～ 3.00	20.07 22.64	12.74 18.82	—— ——	32.81 41.46
5	3.00～ 3.88	22.64 26.30	18.82 27.44	72.00 87.22	-30.54 -33.49

P_a : Active earth pressure

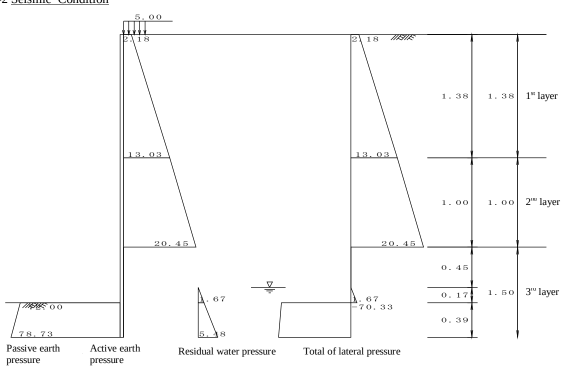
P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.00 m (GL -3.00 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.38	2.18 13.03	—— ——	—— ——	2.18 13.03
2	1.38～ 2.38	13.03 20.45	—— ——	—— ——	13.03 20.45
3	2.38～ 2.83	0.00 0.00	—— ——	—— ——	0.00 0.00
4	2.83～ 3.00	0.00 0.00	0.00 1.67	—— ——	0.00 1.67
5	3.00～ 3.39	0.00 0.00	1.67 5.48	72.00 78.73	-70.33 -73.25

P_a : Active earth pressure

P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.00 m (GL -3.00 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)
	I	=	20008 cm⁴(after reduction by corrosion and section)
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$

	Depth (m)	N-value
1	2.38	7
2	3.38	6
3	4.38	2
4	5.38	2
5	6.38	0
6	7.38	6
7	8.38	11
8	9.38	14
9	10.38	16
10	11.38	4

	Depth (m)	N-value
11	12.38	4
12	13.38	3
13	14.38	4
14	15.38	4
15	16.38	13
16	17.38	9
17	18.38	11
18	19.38	11
19	20.38	5
20	21.38	19

4-2 Normal Condition

K_h = 12248 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.526 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.90 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.00 m) to 1.90 m depth (GL -4.90 m).

	Depth (m)	N-value
1	3.00	6.38
2	3.38	6.00
3	4.38	2.00
4	4.90	2.00
	Σh = 16.38	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{16.38}{4}$

$= 4.10$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 4.10^{0.406} = 12248$ kN/m³
Kh (normal condition) = 12248 kN/m³

4-3 Seismic Condition

K_h = 12248 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.526$ m⁻¹

$$L = \frac{1}{\beta} = 1.90 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.00 m) and 1.90 m depth (GL -4.90 m).

	Depth (m)	N-value
1	3.00	6.38
2	3.38	6.00
3	4.38	2.00
4	4.90	2.00
	Σh = 16.38	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{16.38}{4}$

$= 4.10$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 4.10^{0.406} = 12248$ kN/m³
Kh (seismic condition) = 12248 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.08	1.08	2.91 8.57	1.57 4.63	2.64 2.28	4.15 10.55
2	1.08～ 1.38	0.30	8.57 12.30	1.29 1.84	1.82 1.72	2.34 3.17
3	1.38～ 2.38	1.00	12.30 24.43	6.15 12.21	1.29 0.95	7.91 11.64
4	2.38～ 3.00	0.62	32.81 41.46	10.17 12.85	0.41 0.21	4.20 2.66
				ΣP = 50.72		ΣM = 46.63

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.00 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m
h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{46.63}{50.72} = 0.92 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.38	1.38	2.18 13.03	1.51 8.99	2.54 2.08	3.83 18.70
2	1.38～ 2.38	1.00	13.03 20.45	6.51 10.23	1.29 0.95	8.38 9.75
3	2.38～ 2.83	0.45	0.00 0.00	0.00 0.00	0.47 0.32	0.00 0.00
4	2.83～ 3.00	0.17	0.00 1.67	0.00 0.14	0.11 0.06	0.00 0.01
				ΣP = 27.38		ΣM = 40.66

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.00 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{40.66}{27.38} = 1.49 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
EI = 200000 × 10³ × 20008 × 10⁻⁸		=	4.002 × 10⁴

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	12248 kN/m³
calculated value	β	=	0.52594 m⁻¹
resultant earth force (lateral)	P_0	=	50.72 kN/m
height of acting position of load	h_0	=	0.92 m
moment	M_0	=	46.63 kN·m/m

in consideration of ψ_m = 1.426,

maximum moment	M_{max}	=	66.48 kN·m/m
depth of generated position of M_{max}	l_m	=	0.894 m
depth of 1st fixed point	l_i	=	2.388 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	12248 kN/m³
calculated value	β	=	0.52594 m⁻¹
resultant earth force (lateral)	P_0	=	27.38 kN/m
height of acting position of load	h_0	=	1.49 m
moment	M_0	=	40.66 kN·m/m

in consideration of ψ_m = 1.214,

maximum moment	M_{max}	=	49.35 kN·m/m
depth of generated position of M_{max}	l_m	=	0.707 m
depth of 1st fixed point	l_i	=	2.201 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{66.48\times10^6}{1320\times10^3} = 50\ N/mm^2 \leqq \sigma_a = 180\ N/mm^2 \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{49.35\times10^6}{1320\times10^3} = 37\ N/mm^2 \leqq \sigma_a = 270\ N/mm^2 \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.08	2.64 2.28	0.880 0.760	0.274 0.216	1.57 4.63	0.430 0.998
2	1.08～ 1.38	1.82 1.72	0.607 0.573	0.147 0.133	1.29 1.84	0.189 0.245
3	1.38～ 2.38	1.29 0.95	0.429 0.318	0.079 0.045	6.15 12.21	0.485 0.551
4	2.38～ 3.00	0.41 0.21	0.138 0.069	0.009 0.002	10.17 12.85	0.092 0.030
						ΣQ = 3.020

Y : Height from imaginary riverbed to acting position

α : $\alpha = \frac{Y}{H+L_k}$

ζ : $\zeta = \frac{(3-\alpha)\times\alpha^2}{6}$

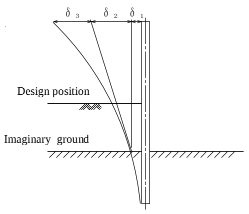
Q : ζ × P

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5259\times0.92)\times50.72}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5259^3} = 0.00646\ m$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5259\times0.92)\times50.72}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5259^2}\times(3.00+0.00) = 0.01352\ m$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.02\times(3.00+0.00)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00204\ m$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00646 + 0.01352 + 0.00204$
$= 0.02202$ m
$= 22.02 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.38	2.54 2.08	0.847 0.693	0.257 0.185	1.51 8.99	0.388 1.661
2	1.38～ 2.38	1.29 0.95	0.429 0.318	0.079 0.045	6.51 10.23	0.514 0.462
3	2.38～ 2.83	0.47 0.32	0.157 0.107	0.012 0.005	0.00 0.00	0.000 0.000
4	2.83～ 3.00	0.11 0.06	0.038 0.019	0.001 0.000	0.00 0.14	0.000 0.000
						ΣQ = 3.024

Y : Height from imaginary riverbed to acting position

α : $\alpha = \frac{Y}{H+L_k}$

ζ : $\zeta = \frac{(3-\alpha)\times\alpha^2}{6}$

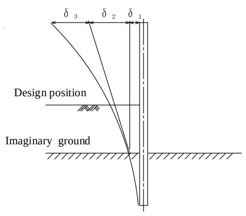
Q : ζ × P

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5259\times1.49)\times27.38}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5259^3} = 0.00419\ m$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5259\times1.49)\times27.38}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5259^2}\times(3.00+0.00) = 0.00951\ m$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.02\times(3.00+0.00)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00204\ m$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00419 + 0.00951 + 0.00204$
$= 0.01574$ m
$= 15.74 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
$EI = 200000\times10^3\times24400\times10^{-8}$		=	4.880×10^4

6-1 Penetration Depth and Whole Length of SSP (Chang)

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h\cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction	$K_h = 12248 kN/m^3$
Calculated value	$\beta = 0.50049 m^{-1}$
Penetration length of SSP	$D = 0.00 + \frac{3}{0.500} = 5.99m$
Whole length of SSP	$L = 3.00 - 0.40 + 5.99 = 8.59m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction	$K_h = 12248 kN/m^3$
Calculated value	$\beta = 0.50049 m^{-1}$
Penetration length of SSP	$D = 0.00 + \frac{3}{0.500} = 5.99m$
Whole length of SSP	$L = 3.00 - 0.40 + 5.99 = 8.59m$

Therefore, whole length of SSP is set as 8.60 m in consideration of round unit of SSP length.

7 Calculation Result

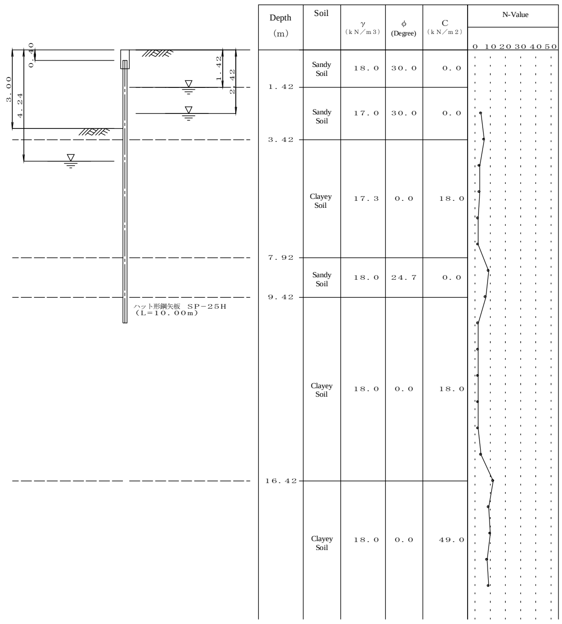
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		66.48	49.35
Stress intensity	σ (N/mm^2)		50 (180)	37 (270)
Lateral displacement	δ (mm)		22.02 (50.0)	15.74 (75.0)
Penetration depth	D (m)		5.99	5.99
Whole length of SSP	L (m)	8.60		

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 3 + 240 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	1.42 m (Normal Condition)
	L_{wa}'	=	2.42 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.24 m (Normal Condition)
	L_{wp}'	=	4.24 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \dfrac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　$\gamma_w = 9.8$ kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　$\zeta_0 = 10$ degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1 - \dfrac{\Sigma\gamma h + 2Q}{2C}\cdot\tan\theta}$

Equilibrium factor of compression　　$K_c = 0.50$ (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{\,0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as $1/\beta$

N-value distribution

No	Depth (m)	N-Value
1	2.42	4
2	3.42	6
3	4.42	3
4	5.42	3
5	6.42	2
6	7.42	2
7	8.42	9
8	9.42	7
9	10.42	2
10	11.42	2

No	Depth (m)	N-Value
11	12.42	2
12	13.42	2
13	14.42	2
14	15.42	4
15	16.42	12
16	17.42	9
17	18.42	10
18	19.42	8
19	20.42	9
20	21.42	17

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree)		kh(kN/m^3)	
										normal	seismic	normal	seismic
1	1.42	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	3.42	S	4.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	——	——
3	7.92	C	3.0	17.30	9.30	0.0	18.0	0.0	0.200	auto	auto	——	——
4	9.42	S	8.0	18.00	9.00	24.7	0.0	0.0	0.200	auto	auto	——	——
5	16.42	C	2.0	18.00	6.00	0.0	18.0	0.0	0.200	auto	auto	——	——
6	20.92	C	10.0	18.00	9.00	0.0	49.0	0.0	0.200	auto	auto	——	——

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- φ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm (Normal)
σ_a' = 270 N/mm (Seismic)

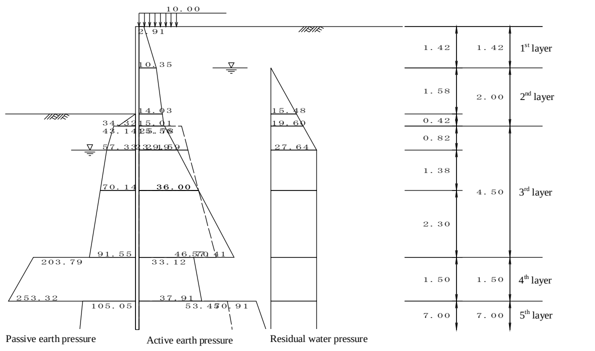
Allowable displacement　　δ_a = 50.0 mm (Normal)
δ_a' = 75.0 mm (Seismic)

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus
- Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
- Not reduced: I_0 applied to calculation of penetration depth
- Reduced: I_0 applied to calculation of section forces and displacement
- Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.42	Sandy soil	18.0	30.0	—— ——	10.000 35.560	0.30142 0.30142	0.29115 0.29115
2	1.42～ 3.00	Sandy soil	8.0	30.0	—— ——	35.560 48.200	0.30142 0.30142	0.29115 0.29115
3	3.00～ 3.42	Sandy soil	8.0	30.0	—— ——	48.200 51.560	0.30142 0.30142	0.29115 0.29115
4	3.42～ 4.24	Clayey soil	9.3	—	18.0 18.0	51.560 59.186	—— ——	—— ——
5	4.24～ 5.62	Clayey soil	9.3	—	18.0 18.0	59.186 72.000	—— ——	—— ——
6	5.62～ 7.92	Clayey soil	9.3	—	18.0 18.0	72.000 93.410	—— ——	—— ——
7	7.92～ 9.42	Sandy soil	9.0	24.7	—— ——	93.410 106.910	0.36710 0.36710	0.35460 0.35460
8	9.42～ 16.42	Clayey soil	6.0	—	18.0 18.0	106.910 148.910	—— ——	—— ——
9	16.42～ 20.92	Clayey soil	9.0	—	49.0 49.0	148.910 189.410	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
3	3.00～ 3.42	Sandy soil	17.0	30.0	—— ——	0.000 7.140	4.97650 4.97650	4.80693 4.80693
4	3.42～ 4.24	Clayey soil	17.3	0.0	18.0 18.0	7.140 21.326	—— ——	—— ——
5	4.24～ 5.62	Clayey soil	9.3	0.0	18.0 18.0	21.326 34.140	—— ——	—— ——
6	5.62～ 7.92	Clayey soil	9.3	0.0	18.0 18.0	34.140 55.550	—— ——	—— ——
7	7.92～ 9.42	Sandy soil	9.0	24.7	—— ——	55.550 69.050	3.79809 3.79809	3.66867 3.66867
8	9.42～ 16.42	Clayey soil	6.0	0.0	18.0 18.0	69.050 111.050	—— ——	—— ——
9	16.42～ 20.92	Clayey soil	9.0	0.0	49.0 49.0	111.050 151.550	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.42	2.91 10.35	—— ——	2.91 10.35	—— ——	—— ——
2	1.42～ 3.00	10.35 14.03	—— ——	10.35 14.03	0.00 15.48	—— ——
3	3.00～ 3.42	14.03 15.01	—— ——	14.03 15.01	15.48 19.60	0.00 34.32
4	3.42～ 4.24	15.56 23.19	25.78 29.59	25.78 29.59	19.60 27.64	43.14 57.33
5	4.24～ 5.62	23.19 36.00	29.59 36.00	29.59 36.00	27.64 27.64	57.33 70.14
6	5.62～ 7.92	36.00 57.41	36.00 46.70	36.00 57.41	27.64 27.64	70.14 91.55
7	7.92～ 9.42	33.12 37.91	—— ——	33.12 37.91	27.64 27.64	203.79 253.32
8	9.42～ 16.42	70.91 112.91	53.45 74.46	70.91 112.91	27.64 27.64	105.05 147.05
9	16.42～ 20.92	50.91 91.41	74.46 94.71	74.46 94.71	27.64 27.64	209.05 249.55

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

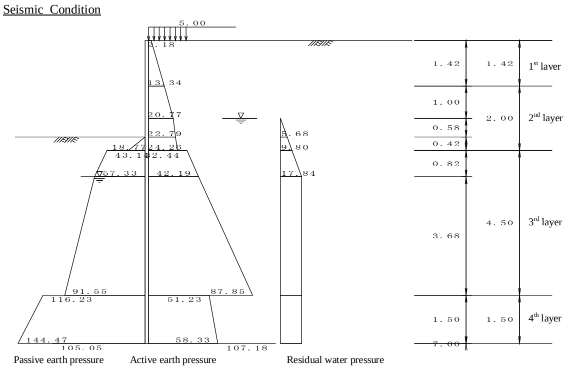
- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 1.42	Sandy Soil	18.0	30.0	—— ——	5.00 30.56	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	1.42～ 2.42	Sandy Soil	17.0	30.0	—— ——	30.56 47.56	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	2.42～ 3.00	Sandy Soil	8.0	30.0	—— ——	47.56 52.20	0.00 5.68	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
4	3.00～ 3.42	Sandy Soil	8.0	30.0	—— ——	52.20 55.56	5.68 9.80	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663 3rd layer	—— ——
5	3.42～ 4.24	Clayey Soil	9.3	—	18.0 18.0	55.56 63.19	9.80 17.84	0.200 0.200	11.31 11.31	—— ——	—— ——	39.17 38.24

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
6	4.24～ 7.92	Clayey Soil	9.3	—	18.0 18.0	63.19 97.41	17.84 53.90	0.200 0.200	11.31 11.31	—— ——	—— ——	38.24 33.29
7	7.92～ 9.42	Sandy Soil	9.0	24.7	—— ——	97.41 110.91	53.90 68.60	0.200 0.200	11.31 11.31	0.54445 0.54445	0.52590 0.52590	—— ——
8	9.42～ 16.42	Clayey Soil	6.0	—	18.0 18.0	110.91 152.91	68.60 137.20	0.200 0.200	11.31 11.31	—— ——	—— ——	30.82 19.31
9	16.42～ 20.92	Clayey Soil	9.0	—	49.0 49.0	152.91 193.41	137.20 181.30	0.200 0.200	11.31 11.31	—— ——	—— ——	39.46 37.65

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
4	3.00～ 3.42	Sandy soil	17.00	30.0	—— ——	0.000 7.140	0.00 0.00	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
5	3.42～ 4.24	Clayey Soil	17.30	0.0	18.0 18.0	7.140 21.326	0.00 0.00	0.200 0.200	11.31 11.31	—— ——	—— ——
6	4.24～ 7.92	Clayey Soil	9.30	0.0	18.0 18.0	21.326 55.550	0.00 36.06	0.200 0.200	11.31 11.31	—— ——	—— ——
7	7.92～ 9.42	Sandy soil	9.00	24.7	—— ——	55.550 69.050	36.06 50.76	0.200 0.200	11.31 11.31	2.09232 2.09232	2.09232 2.09232
8	9.42～ 16.42	Clayey Soil	6.00	0.0	18.0 18.0	69.050 111.050	50.76 119.36	0.200 0.200	11.31 11.31	—— ——	—— ——
9	16.42～ 20.92	Clayey Soil	9.00	0.0	49.0 49.0	111.050 151.550	119.36 163.46	0.200 0.200	11.31 11.31	—— ——	—— ——

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
δ = -0.00、β = 0.00、θ = $\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta-\theta) \cdot \left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 1.42	2.18 13.34	—— ——	—— ——
2	1.42～ 2.42	13.34 20.77	—— ——	—— ——
3	2.42～ 3.00	20.77 22.79	0.00 5.68	—— ——

No	Depth (m)	Active side		
		Pa1 kN/m^2	Pa2 kN/m^2	Pa kN/m^2
4	3.00～ 3.42	22.79 24.26	5.68 9.80	0.00 18.77
5	3.42～ 4.24	32.44 42.19	9.80 17.84	43.14 57.33
6	4.24～ 7.92	42.19 87.85	17.84 17.84	57.33 91.55
7	7.92～ 9.42	51.23 58.33	17.84 17.84	116.23 144.47
8	9.42～ 16.42	107.18 182.52	17.84 17.84	105.05 147.05
9	16.42～ 20.92	90.20 142.24	17.84 17.84	209.05 249.55

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [$ Passive earth pressure Q Active earth pressure Residual water pressure Total of lateral pressure

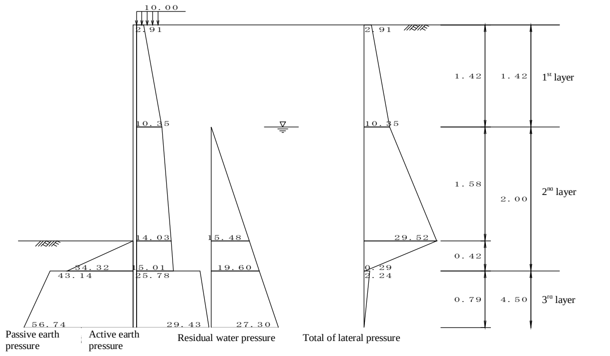
Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition

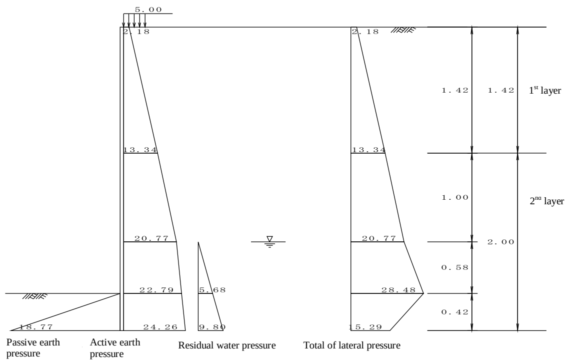


	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 1.42	2.91 10.35	—— ——	—— ——	2.91 10.35
2	1.42～ 3.00	10.35 14.03	0.00 15.48	—— ——	10.35 29.52
3	3.00～ 3.42	14.03 15.01	15.48 19.60	0.00 34.32	29.52 0.29
4	3.42～ 4.21	25.78 29.43	19.60 27.30	43.14 56.74	2.24 0.00
5	4.21～ 4.24	29.43 29.59	27.30 27.64	56.74 57.33	0.00 -0.10

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 1.21 m (GL -4.21 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 1.42	2.18 13.34	—— ——	—— ——	2.18 13.34
2	1.42～ 2.42	13.34 20.77	—— ——	—— ——	13.34 20.77
3	2.42～ 3.00	20.77 22.79	0.00 5.68	—— ——	20.77 28.48
4	3.00～ 3.42	22.79 24.26	5.68 9.80	0.00 18.77	28.48 15.29
5	3.42～ 4.24	32.44 42.19	9.80 17.84	43.14 57.33	-0.90 2.70

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.42 m (GL -3.42 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4(original condition)
	I	=	20008 cm^4(after reduction by corrosion and section)
Inertia sectional moment	EI	=	200000 × 10^3 × 20008 × 10^{-8} = 4.002 × 10^4

	Depth (m)	N-value		Depth (m)	N-value
1	2.42	4	11	12.42	2
2	3.42	6	12	13.42	2
3	4.42	3	13	14.42	2
4	5.42	3	14	15.42	4
5	6.42	2	15	16.42	12
6	7.42	2	16	17.42	9
7	8.42	9	17	18.42	10
8	9.42	7	18	19.42	8
9	10.42	2	19	20.42	9
10	11.42	2	20	21.42	17

4-2 Normal Condition

K_h = 10755 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.509 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.96 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -4.21 m) to 1.96 m depth (GL -6.17 m).

	Depth (m)	N-value
1	4.21	3.64
2	4.42	3.00
3	5.42	3.00
4	6.17	2.25
	Σh = 11.89	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{11.89}{4}$

$= 2.97$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 2.97^{0.406} = 10755$ kN/m³

Kh (normal condition) = 10755 kN/m^3

4-3 Seismic Condition

K_h = 12131 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.525$ m^{-1}

$$L = \frac{1}{\beta} = 1.91 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.42 m) and 1.91 m depth (GL -5.33 m).

	Depth (m)	N-value
1	3.42	6.00
2	4.42	3.00
3	5.33	3.00
	Σh = 12.00	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{12.00}{3}$

$= 4.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 4.00^{0.406} = 12131$ kN/m³

Kh (seismic condition) = 12131 kN/m^3

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m^2)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.42	1.42	2.91 10.35	2.07 7.35	3.73 3.26	7.72 23.96
2	1.42～ 3.00	1.58	10.35 29.52	8.18 23.32	2.26 1.73	18.48 40.40
3	3.00～ 3.42	0.42	29.52 0.29	6.20 0.06	1.07 0.93	6.61 0.06
4	3.42～ 4.21	0.79	2.24 0.00	0.88 0.00	0.52 0.26	0.46 0.00
				ΣP = 48.06		ΣM = 97.68

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	1.21 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{97.68}{48.06} = 2.03 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m^2)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.42	1.42	2.18 13.34	1.55 9.47	2.95 2.47	4.57 23.43
2	1.42～ 2.42	1.00	13.34 20.77	6.67 10.38	1.67 1.33	11.12 13.84
3	2.42～ 3.00	0.58	20.77 28.48	6.02 8.26	0.81 0.61	4.86 5.06
4	3.00～ 3.42	0.42	28.48 15.29	5.98 3.21	0.28 0.14	1.67 0.45
				ΣP = 51.55		ΣM = 65.01

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.42 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{65.01}{51.55} = 1.26 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	20008 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 20008 × 10^{-8}		=	4.002 × 10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	10755 kN/m^3
calculated value	β	=	0.50913 m^{-1}
resultant earth force (lateral)	P_0	=	48.06 kN/m
height of acting position of load	h_0	=	2.03 m
moment	M_0	=	97.68 kN·m/m

in consideration of ψ_m = 1.138,

maximum moment	M_{max}	=	111.21 kN·m/m
depth of generated position of M_{max}	l_m	=	0.619 m
depth of 1st fixed point	l_i	=	2.161 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	12132 kN/m^3
calculated value	β	=	0.52469 m^{-1}
resultant earth force (lateral)	P_0	=	51.55 kN/m
height of acting position of load	h_0	=	1.26 m
moment	M_0	=	65.01 kN·m/m

in consideration of ψ_m = 1.273,

maximum moment	M_{max}	=	82.76 kN·m/m
depth of generated position of M_{max}	l_m	=	0.775 m
depth of 1st fixed point	l_i	=	2.271 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm^3 (original condition)
	Z	=	1320 cm^3 (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{111.21\times10^6}{1320\times10^3} = 84\ \mathrm{N/mm^2} \leqq \sigma_a = 180\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{82.76\times10^6}{1320\times10^3} = 63\ \mathrm{N/mm^2} \leqq \sigma_a = 270\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00〜 1.42	3.73 3.26	0.887 0.775	0.277 0.223	2.07 7.35	0.573 1.637
2	1.42〜 3.00	2.26 1.73	0.537 0.412	0.118 0.073	8.18 23.32	0.969 1.707
3	3.00〜 3.42	1.07 0.93	0.253 0.220	0.029 0.022	6.20 0.06	0.182 0.001
4	3.42〜 4.21	0.52 0.26	0.125 0.062	0.007 0.002	0.88 0.00	0.007 0.000
						ΣQ = 5.076

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

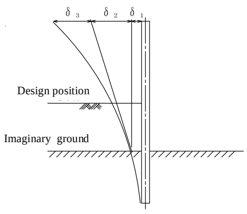
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5091\times2.03)\times48.06}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5091^3} = 0.00926\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5091\times2.03)\times48.06}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5091^2}\times(3.00+1.21) = 0.02991\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{5.08\times(3.00+1.21)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00944\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00926 + 0.02991 + 0.00944$
$= 0.04861$ m
$= 48.61 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00〜 1.42	2.95 2.47	0.862 0.723	0.265 0.198	1.55 9.47	0.410 1.880
2	1.42〜 2.42	1.67 1.33	0.487 0.390	0.099 0.066	6.67 10.38	0.664 0.687
3	2.42〜 3.00	0.81 0.61	0.236 0.179	0.026 0.015	6.02 8.26	0.154 0.125
4	3.00〜 3.42	0.28 0.14	0.082 0.041	0.003 0.001	5.98 3.21	0.019 0.003
						ΣQ = 3.942

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

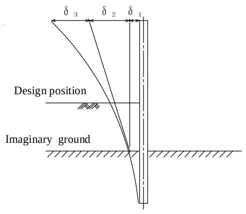
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5247\times1.26)\times51.55}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5247^3} = 0.00741\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5247\times1.26)\times51.55}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5247^2}\times(3.00+0.42) = 0.01859\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.94\times(3.00+0.42)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00394\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00741 + 0.01859 + 0.00394$
$= 0.02994$ m
$= 29.94 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h\cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction $K_h = 10755\ \mathrm{kN/m^3}$
Calculated value $\beta = 0.48448\ \mathrm{m^{-1}}$

Penetration length of SSP $D = 1.21 + \dfrac{3}{0.484} = 7.40$ m

Whole length of SSP $L = 3.00 - 0.40 + 7.40 = 10.00$ m

6-1-2 Seismic Condition

Modules of lateral subgrade reaction $K_h = 12131\ \mathrm{kN/m^3}$
Calculated value $\beta = 0.49930\ \mathrm{m^{-1}}$

Penetration length of SSP $D = 0.42 + \dfrac{3}{0.499} = 6.43$ m

Whole length of SSP $L = 3.00 - 0.40 + 6.43 = 9.03$ m

Therefore, whole length of SSP is set as 10.00 m in consideration of round unit of SSP length.

7 Calculation Result

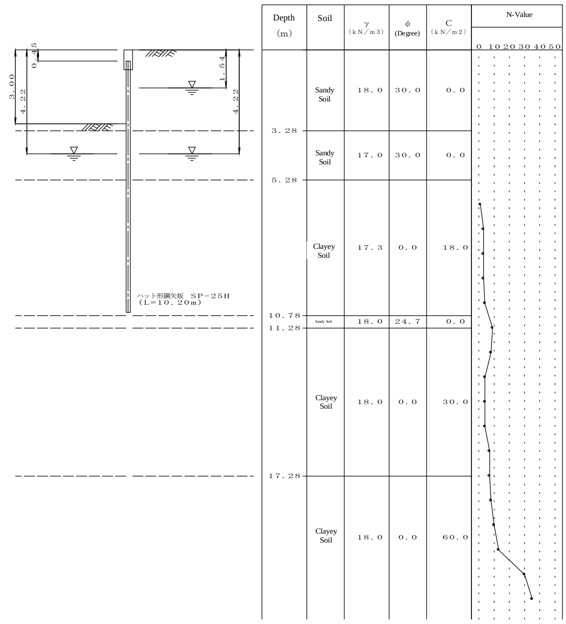
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		111.21	82.76
Stress intensity	σ (N/mm^2)		84 (180)	63 (270)
Lateral displacement	δ (mm)		48.61 (50.0)	29.94 (75.0)
Penetration depth	D (m)		7.40	6.43
Whole length of SSP	L (m)	10.00		

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 3 + 450 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	1.54 m (Normal Condition)
	L_{wa}'	=	4.22 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.22 m (Normal Condition)
	L_{wp}'	=	4.22 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \dfrac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water	γ_w = 9.8 kN/m^3
Type of water pressure	trapezoidal water pressure
Lateral pressure	calculated in consideration of site conditions
Study case	- Normal Condition - Seismic Condition
Design earthquake intensity	k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load	not considered
Minimum angle of rupture	ζ_0 = 10 degrees
Rear side angle of slope	not considered
Angle of rupture (clayey soil)	$\zeta = \tan^{-1}\sqrt{1-\dfrac{\Sigma\gamma h+2Q}{2C}}\cdot\tan\theta$
Equilibrium factor of compression	K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value	No	Depth (m)	N-Value
1	6.28	1	11	16.28	7
2	7.28	3	12	17.28	7
3	8.28	3	13	18.28	8
4	9.28	3	14	19.28	10
5	10.28	4	15	20.28	13
6	11.28	9	16	21.28	30
7	12.28	8	17	22.28	35
8	13.28	4	18	23.28	35
9	14.28	4	19	24.28	35
10	15.28	4	20	25.28	35

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree) normal	ζ (degree) seismic	kh(kN/m^3) normal	kh(kN/m^3) seismic
1	3.28	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	5.28	S	9.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	——	——
3	10.78	C	3.0	17.30	9.30	0.0	18.0	0.0	0.200	auto	auto	——	——
4	11.28	S	9.0	18.00	9.00	24.7	0.0	0.0	0.200	auto	auto	——	——
5	17.28	C	5.0	18.00	9.00	0.0	30.0	0.0	0.200	auto	auto	——	——
6	22.28	C	15.0	18.00	9.00	0.0	60.0	0.0	0.200	auto	auto	——	——

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- φ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm（Normal）
　　σ_a' = 270 N/mm（Seismic）

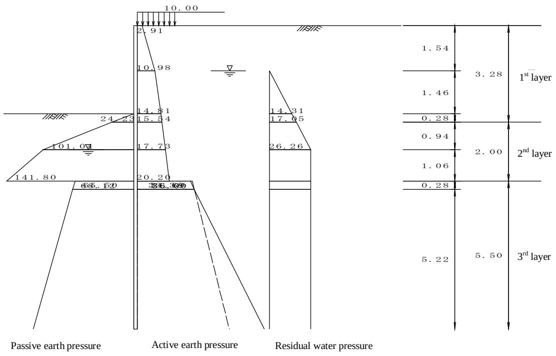
Allowable displacement　　δ_a = 50.0 mm（Normal）
　　δ_a' = 75.0 mm（Seismic）

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus
- Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
- Not reduced: I_0 applied to calculation of penetration depth
- Reduced: I_0 applied to calculation of section forces and displacement
- Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.54	Sandy soil	18.0	30.0	—— ——	10.000 37.720	0.30142 0.30142	0.29115 0.29115
2	1.54～ 3.00	Sandy soil	9.0	30.0	—— ——	37.720 50.860	0.30142 0.30142	0.29115 0.29115
3	3.00～ 3.28	Sandy soil	9.0	30.0	—— ——	50.860 53.380	0.30142 0.30142	0.29115 0.29115
4	3.28～ 4.22	Sandy soil	8.0	30.0	—— ——	53.380 60.900	0.30142 0.30142	0.29115 0.29115
5	4.22～ 5.28	Sandy soil	8.0	30.0	—— ——	60.900 69.380	0.30142 0.30142	0.29115 0.29115
6	5.28～ 5.56	Clayey soil	9.3	—	18.0 18.0	69.380 72.000	—— ——	—— ——
7	5.56～ 10.78	Clayey soil	9.3	—	18.0 18.0	72.000 120.530	—— ——	—— ——
8	10.78～ 11.28	Sandy soil	9.0	24.7	—— ——	120.530 125.030	0.36710 0.36710	0.35460 0.35460
9	11.28～ 17.28	Clayey soil	9.0	—	30.0 30.0	125.030 179.030	—— ——	—— ——
10	17.28～ 22.28	Clayey soil	9.0	—	60.0 60.0	179.030 224.030	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
3	3.00～ 3.28	Sandy soil	18.0	30.0	—— ——	0.000 5.040	4.97650 4.97650	4.80693 4.80693
4	3.28～ 4.22	Sandy soil	17.0	30.0	—— ——	5.040 21.020	4.97650 4.97650	4.80693 4.80693
5	4.22～ 5.28	Sandy soil	8.0	30.0	—— ——	21.020 29.500	4.97650 4.97650	4.80693 4.80693
6	5.28～ 5.56	Clayey soil	9.3	0.0	18.0 18.0	29.500 32.120	—— ——	—— ——
7	5.56～ 10.78	Clayey soil	9.3	0.0	18.0 18.0	32.120 80.650	—— ——	—— ——
8	10.78～ 11.28	Sandy soil	9.0	24.7	—— ——	80.650 85.150	3.79809 3.79809	3.66867 3.66867
9	11.28～ 17.28	Clayey soil	9.0	0.0	30.0 30.0	85.150 139.150	—— ——	—— ——
10	17.28～ 22.28	Clayey soil	9.0	0.0	60.0 60.0	139.150 184.150	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.54	2.91 10.98	—— ——	2.91 10.98	—— ——	—— ——
2	1.54～ 3.00	10.98 14.81	—— ——	10.98 14.81	0.00 14.31	—— ——
3	3.00～ 3.28	14.81 15.54	—— ——	14.81 15.54	14.31 17.05	0.00 24.23
4	3.28～ 4.22	15.54 17.73	—— ——	15.54 17.73	17.05 26.26	24.23 101.04
5	4.22～ 5.28	17.73 20.20	—— ——	17.73 20.20	26.26 26.26	101.04 141.80
6	5.28～ 5.56	33.38 36.00	34.69 36.00	34.69 36.00	26.26 26.26	65.50 68.12
7	5.56～ 10.78	36.00 84.53	36.00 60.26	36.00 84.53	26.26 26.26	68.12 116.65
8	10.78～ 11.28	42.74 44.34	—— ——	42.74 44.34	26.26 26.26	295.88 312.39
9	11.28～ 17.28	65.03 119.03	62.51 89.52	65.03 119.03	26.26 26.26	145.15 199.15

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
10	17.28～ 22.28	59.03 104.03	89.52 112.02	89.52 112.02	26.26 26.26	259.15 304.15

- Formula for active earth pressure

Sandy soil $P_{a1}=K_a\cdot\left[\Sigma\gamma h+\dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

Clayey soil $P_{a1}=\Sigma\gamma h+Q-2C$

$P_{a2}=K_c\cdot(\Sigma\gamma h+Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1}=\left[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}\right]\cdot\cos\delta$

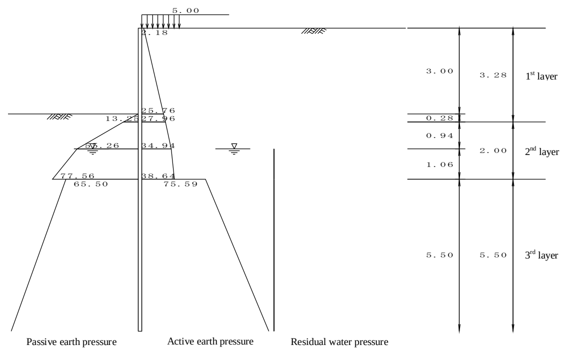
- Formula for passive earth pressure

Sandy soil $P_p=K_p\cdot\left[\Sigma\gamma h+\dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

Clayey soil $P_p=\Sigma\gamma h+Q+2C$

Mixed soil $P_p=\left[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}\right]\cdot\cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 3.00	Sandy Soil	18.0	30.0	—— ——	5.00 59.00	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	3.00～ 3.28	Sandy Soil	18.0	30.0	—— ——	59.00 64.04	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	3.28～ 4.22	Sandy Soil	17.0	30.0	—— ——	64.04 80.02	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
4	4.22～ 5.28	Sandy Soil	8.0	30.0	—— ——	80.02 88.50	0.00 10.39	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
5	5.28～ 10.78	Clayey Soil	9.3	—	18.0 18.0	88.50 139.65	10.39 64.29	0.200 0.200	11.31 11.31	—— ——	—— ——	34.73 23.90
6	10.78～ 11.28	Sandy Soil	9.0	24.7	—— ——	139.65 144.15	64.29 69.19	0.200 0.200	11.31 11.31	0.54445 0.54445	0.52590 0.52590	—— ——
7	11.28～ 17.28	Clayey Soil	9.0	—	30.0 30.0	144.15 198.15	69.19 127.99	0.200 0.200	11.31 11.31	—— ——	—— ——	35.34 29.60
8	17.28～ 22.28	Clayey Soil	9.0	—	60.0 60.0	198.15 243.15	127.99 176.99	0.200 0.200	11.31 11.31	—— ——	—— ——	39.12 37.44

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta=\tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
2	3.00～ 3.28	Sandy soil	18.00	30.0	—— ——	0.000 5.040	0.00 0.00	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
3	3.28～ 4.22	Sandy soil	17.00	30.0	—— ——	5.040 21.020	0.00 0.00	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
4	4.22～ 5.28	Sandy soil	8.00	30.0	—— ——	21.020 29.500	0.00 10.39	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
5	5.28～ 10.78	Clayey soil	9.30	0.0	18.0 18.0	29.500 80.650	10.39 64.29	0.200 0.200	11.31 11.31	—— ——	—— ——
6	10.78～ 11.28	Sandy Soil	9.00	24.7	—— ——	80.650 85.150	64.29 69.19	0.200 0.200	11.31 11.31	2.09232 2.09232	2.09232 2.09232
7	11.28～ 17.28	Clayey Soil	9.00	0.0	30.0 30.0	85.150 139.150	69.19 127.99	0.200 0.200	11.31 11.31	—— ——	—— ——
8	17.28～ 22.28	Clayey Soil	9.00	0.0	60.0 60.0	139.150 184.150	127.99 176.99	0.200 0.200	11.31 11.31	—— ——	—— ——

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;

δ = -0.00、β = 0.00、θ = $\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 3.00	2.18 25.76	—— ——	—— ——
2	3.00～ 3.28	25.76 27.96	—— ——	0.00 13.25
3	3.28～ 4.22	27.96 34.94	—— ——	13.25 55.26
4	4.22～ 5.28	34.94 38.64	0.00 0.00	55.26 77.56
5	5.28～ 10.78	75.59 154.08	0.00 0.00	65.50 116.65
6	10.78～ 11.28	73.44 75.81	0.00 0.00	168.75 178.16
7	11.28～ 17.28	121.23 198.05	0.00 0.00	145.15 199.15
8	17.28～ 22.28	124.31 182.36	0.00 0.00	259.15 304.15
9	16.42～ 20.92	90.20 142.24	17.84 17.84	209.05 249.55

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma$ h … Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

$P_{a2} = K_c \cdot$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

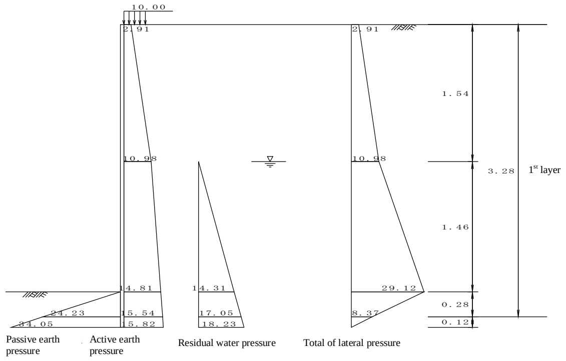
Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition

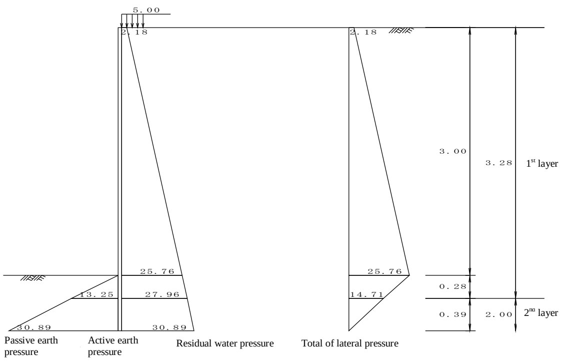


	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.54	2.91 10.98	—— ——	—— ——	2.91 10.98
2	1.54～ 3.00	10.98 14.81	0.00 14.31	—— ——	10.98 29.12
3	3.00～ 3.28	14.81 15.54	14.31 17.05	0.00 24.23	29.12 8.37
4	3.28～ 3.40	15.54 15.82	17.05 18.23	24.23 34.05	8.37 0.00
5	3.40～ 4.22	15.82 17.73	18.23 26.26	34.05 101.04	0.00 -57.05

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.40 m (GL -3.40 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 3.00	2.18 25.76	—— ——	—— ——	2.18 25.76
2	3.00～ 3.28	25.76 27.96	—— ——	0.00 13.25	25.76 14.71
3	3.28～ 3.68	27.96 30.89	—— ——	13.25 30.89	14.71 0.00
4	3.68～ 4.22	30.89 34.94	—— ——	30.89 55.26	0.00 -20.33

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.68 m (GL -3.68 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)
	I	=	20008 cm⁴(after reduction by corrosion and section)
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$

	Depth (m)	N-value		Depth (m)	N-value
1	6.28	1	11	16.28	7
2	7.28	3	12	17.28	7
3	8.28	3	13	18.28	8
4	9.28	3	14	19.28	10
5	10.28	4	15	20.28	13
6	11.28	9	16	21.28	30
7	12.28	8	17	22.28	35
8	13.28	4	18	23.28	35
9	14.28	4	19	24.28	35
10	15.28	4	20	25.28	35

4-2 Normal Condition

K_h = 6910 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.456 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 2.19 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.40 m) to 2.19 m depth (GL -5.59 m).

	Depth (m)	N-value
1	3.40	1.00
2	5.59	1.00
Σh = 2.00		

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{2.00}{2}$

$= 1.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 1.00^{0.406} = 6910$ kN/m³

Kh (normal condition) = 6910 kN/m^3

4-3 Seismic Condition

K_h = 6910 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.456$ m^{-1}

$$L = \frac{1}{\beta} = 2.19 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.67 m) and 2.19 m depth (GL -5.87 m).

	Depth (m)	N-value
1	3.67	1.00
2	5.87	1.00
	Σh = 2.00	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{2.00}{2}$

$= 1.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 1.00^{0.406} = 6910$ kN/m³

Kh (seismic condition) = 6910 kN/m^3

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m^2)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.54	1.54	2.91 10.98	2.24 8.46	2.89 2.37	6.47 20.07
2	1.54～ 3.00	1.46	10.98 29.12	8.02 21.25	1.37 0.89	11.01 18.85
3	3.00～ 3.28	0.28	29.12 8.37	4.08 1.17	0.31 0.21	1.25 0.25
4	3.28～ 3.40	0.12	8.37 0.00	0.50 0.00	0.08 0.04	0.04 0.00
				ΣP = 45.72		ΣM = 57.95

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.40 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t} = \frac{57.95}{45.72} = 1.27 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m^2)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 3.00	3.00	2.18 25.76	3.27 38.64	2.67 1.67	8.76 64.71
2	3.00～ 3.28	0.28	25.76 14.71	3.61 2.06	0.58 0.49	2.10 1.01
3	3.28～ 3.67	0.39	14.71 0.00	2.90 0.00	0.26 0.13	0.76 0.00
				ΣP = 50.49		ΣM = 77.34

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.67 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t} = \frac{77.34}{50.49} = 1.53 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	20008 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 20008 × 10^{-8}		=	4.002 × 10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	6910 kN/m^3
calculated value	β	=	0.45582 m^{-1}
resultant earth force (lateral)	P_0	=	45.72 kN/m
height of acting position of load	h_0	=	1.27 m
moment	M_0	=	57.95 kN·m/m

in consideration of ψ_m = 1.332,

maximum moment	M_{max}	=	77.18 kN·m/m
depth of generated position of M_{max}	l_m	=	0.953 m
depth of 1st fixed point	l_i	=	2.676 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	6910 kN/m^3
calculated value	β	=	0.45582 m^{-1}
resultant earth force (lateral)	P_0	=	50.49 kN/m
height of acting position of load	h_0	=	1.53 m
moment	M_0	=	77.34 kN·m/m

in consideration of ψ_m = 1.252,

maximum moment	M_{max}	=	96.85 kN·m/m
depth of generated position of M_{max}	l_m	=	0.867 m
depth of 1st fixed point	l_i	=	2.590 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm^3 (original condition)
	Z	=	1320 cm^3 (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{77.18\times10^6}{1320\times10^3} = 58\ \mathrm{N/mm^2} \leqq \sigma_a = 180\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{96.85\times10^6}{1320\times10^3} = 73\ \mathrm{N/mm^2} \leqq \sigma_a = 270\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.54	2.89 2.37	0.849 0.698	0.258 0.187	2.24 8.46	0.579 1.581
2	1.54～ 3.00	1.37 0.89	0.404 0.261	0.071 0.031	8.02 21.25	0.566 0.660
3	3.00～ 3.28	0.31 0.21	0.090 0.063	0.004 0.002	4.08 1.17	0.016 0.002
4	3.28～ 3.40	0.08 0.04	0.024 0.012	0.000 0.000	0.50 0.00	0.000 0.000
						ΣQ = 3.405

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

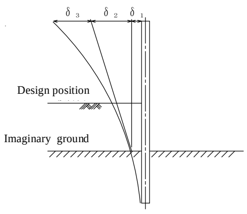
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4558\times1.27)\times45.72}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^3} = 0.00952\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4558\times1.27)\times45.72}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^2}\times(3.00+0.40) = 0.02015\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.40\times(3.00+0.40)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00335\ \mathrm{m}$$

Additional displacement δ_3’ generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00952 + 0.02015 + 0.00335$
$= 0.03301$ m
$= 33.01 \leqq \delta a = 50.00$ mm (ok)

Where,

Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 3.00	2.67 1.67	0.728 0.456	0.201 0.088	3.27 38.64	0.657 3.403
2	3.00～ 3.28	0.58 0.49	0.158 0.133	0.012 0.008	3.61 2.06	0.043 0.017
3	3.28～ 3.67	0.26 0.13	0.072 0.036	0.003 0.001	2.90 0.00	0.007 0.000
						ΣQ = 4.128

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

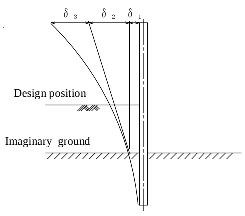
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4558\times1.53)\times50.49}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^3} = 0.01131\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4558\times1.53)\times50.49}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^2}\times(3.00+0.67) = 0.02674\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{4.13\times(3.00+0.67)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00512\ \mathrm{m}$$

Additional displacement δ_3’ generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.01131 + 0.02674 + 0.00512$
$= 0.04317$ m
$= 43.17 \leqq \delta_a = 75.00$ mm (ok)

Where,

δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young’s modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h\cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction $K_h = 6190 kN/m^3$
Calculated value $\beta = 0.43376 m^{-1}$

Penetration length of SSP $D = 0.40 + \dfrac{3}{0.434} = 7.32m$

Whole length of SSP $L = 3.00 - 0.40 + 7.32 = 9.92m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction $K_h = 6190 kN/m^3$
Calculated value $\beta = 0.43376 m^{-1}$

Penetration length of SSP $D = 0.67 + \dfrac{3}{0.434} = 7.59m$

Whole length of SSP $L = 3.00 - 0.40 + 7.59 = 10.19m$

Therefore, whole length of SSP is set as 10.20 m in consideration of round unit of SSP length.

7 Calculation Result

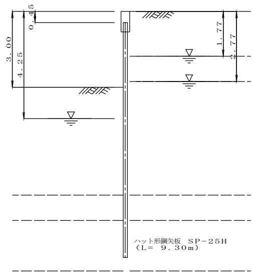
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN・m/m)		77.18	96.85
Stress intensity	σ (N/mm^2)		58 (180)	73 (270)
Lateral displacement	δ (mm)		33.01 (50.0)	43.17 (75.0)
Penetration depth	D (m)		7.32	7.59
Whole length of SSP	L (m)	10.20		

－Steel Sheet Pile　Design Calculation－

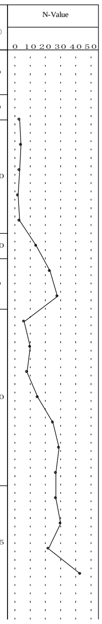
Lower Marikina Section 4 + 050 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



Depth (m)	Soil	γ (kN/m3)	φ (Degree)	C (kN/m2)
1.77	Sandy Soil	18.0	30.0	0.0
2.77	Sandy Soil	17.0	30.0	0.0
7.27	Clayey Soil	18.0	0.0	18.0
8.27	Clayey Soil	18.0	0.0	84.0
10.27	砂質土	18.0	26.0	0.0
17.27	Sandy Soil 粘性土	17.1	0.0	54.0
	Clayey Soil	17.1	0.0	71.5



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_{lt}	=	0.40 m
Landside WL	L_{wa}	=	1.77 m (Normal Condition)
	L_{wa}'	=	2.77 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.25 m (Normal Condition)
	L_{wp}'	=	4.25 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \frac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake
considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C}} \cdot \tan\theta$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value	No	Depth (m)	N-Value
1	2.77	3	11	12.77	8
2	3.77	4	12	13.77	15
3	4.77	3	13	14.77	25
4	5.77	2	14	15.77	29
5	6.77	3	15	16.77	27
6	7.77	14	16	17.77	27
7	8.77	23	17	18.77	30
8	9.77	28	18	19.77	22
9	10.77	6	19	20.77	43
10	11.77	10	20	21.77	46

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree) normal	ζ (degree) seismic	kh(kN/m^3) normal	kh(kN/m^3) seismic
1	1.77	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	2.77	S	3.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	——	——
3	7.27	C	3.0	18.00	9.00	0.0	18.0	0.0	0.200	auto	auto	——	——
4	8.27	C	14.0	18.00	9.00	0.0	84.0	0.0	0.200	auto	auto	——	——
5	10.27	S	25.0	18.00	9.00	26.0	0.0	0.0	0.200	auto	auto	——	——
6	17.27	C	9.0	17.10	8.10	0.0	54.0	0.0	0.200	auto	auto	——	——
7	21.77	C	25.0	17.10	8.10	0.0	71.5	0.0	0.200	auto	auto	——	——

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- φ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm (Normal)
σ_a' = 270 N/mm (Seismic)

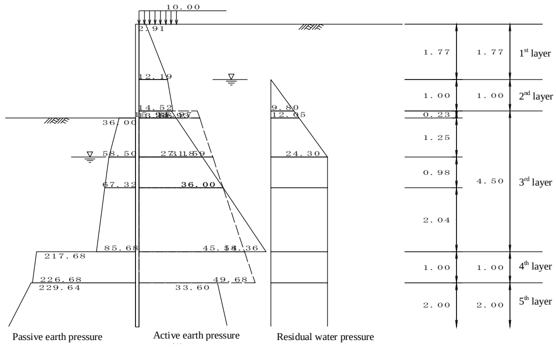
Allowable displacement　　δ_a = 50.0 mm (Normal)
δ_a' = 75.0 mm (Seismic)

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.77	Sandy soil	18.0	30.0	—— ——	10.000 41.860	0.30142 0.30142	0.29115 0.29115
2	1.77～ 2.77	Sandy soil	8.0	30.0	—— ——	41.860 49.860	0.30142 0.30142	0.29115 0.29115
3	2.77～ 3.00	Clayey soil	9.0	—	18.0 18.0	49.860 51.930	—— ——	—— ——
4	3.00～ 4.25	Clayey soil	9.0	—	18.0 18.0	51.930 63.180	—— ——	—— ——
5	4.25～ 5.23	Clayey soil	9.0	—	18.0 18.0	63.180 72.000	—— ——	—— ——
6	5.23～ 7.27	Clayey soil	9.0	—	18.0 18.0	72.000 90.360	—— ——	—— ——
7	7.27～ 8.27	Clayey soil	9.0	—	84.0 84.0	90.360 99.360	—— ——	—— ——
8	8.27～ 10.27	Sandy soil	9.0	26.0	—— ——	99.360 117.360	0.35007 0.35007	0.33814 0.33814
9	10.27～ 17.27	Clayey soil	8.1	—	54.0 54.0	117.360 174.060	—— ——	—— ——
10	17.27～ 21.77	Clayey soil	8.1	—	71.5 71.5	174.060 210.510	—— ——	—— ——

Case: Lower Marikina Section 4 + 050 J

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta = 15.00$、$\beta = 0.00$、$\theta = 0.00$

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
4	3.00～ 4.25	Clayey soil	18.0	0.0	18.0 18.0	0.000 22.500	—— ——	—— ——
5	4.25～ 5.23	Clayey soil	9.0	0.0	18.0 18.0	22.500 31.320	—— ——	—— ——
6	5.23～ 7.27	Clayey soil	9.0	0.0	18.0 18.0	31.320 49.680	—— ——	—— ——
7	7.27～ 8.27	Clayey soil	9.0	0.0	84.0 84.0	49.680 58.680	—— ——	—— ——
8	8.27～ 10.27	Sandy soil	9.0	26.0	—— ——	58.680 76.680	4.05145 4.05145	3.91340 3.91340
9	10.27～ 17.27	Clayey soil	8.1	0.0	54.0 54.0	76.680 133.380	—— ——	—— ——
10	17.27～ 21.77	Clayey soil	8.1	0.0	71.5 71.5	133.380 169.830	—— ——	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta = -15.00$、$\beta = 0.00$、$\theta = 0.00$

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.77	2.91 12.19	—— ——	2.91 12.19	—— ——	—— ——
2	1.77～ 2.77	12.19 14.52	—— ——	12.19 14.52	0.00 9.80	—— ——
3	2.77～ 3.00	13.86 15.93	24.93 25.97	24.93 25.97	9.80 12.05	—— ——
4	3.00～ 4.25	15.93 27.18	25.97 31.59	25.97 31.59	12.05 24.30	36.00 58.50
5	4.25～ 5.23	27.18 36.00	31.59 36.00	31.59 36.00	24.30 24.30	58.50 67.32
6	5.23～ 7.27	36.00 54.36	36.00 45.18	36.00 54.36	24.30 24.30	67.32 85.68
7	7.27～ 8.27	-77.64 -68.64	45.18 49.68	45.18 49.68	24.30 24.30	217.68 226.68
8	8.27～ 10.27	33.60 39.68	—— ——	33.60 39.68	24.30 24.30	229.64 300.08
9	10.27～ 17.27	9.36 66.06	58.68 87.03	58.68 87.03	24.30 24.30	184.68 241.38
10	17.27～ 21.77	31.06 67.51	87.03 105.26	87.03 105.26	24.30 24.30	276.38 312.83

Case: Lower Marikina Section 4 + 050 J

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$
$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$
K_c: Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

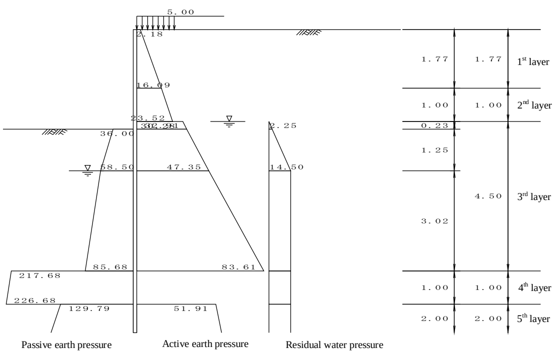
- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

2-2 Seismic Condition



2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 1.77	Sandy Soil	18.0	30.0	—— ——	5.00 36.86	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	1.77～ 2.77	Sandy Soil	17.0	30.0	—— ——	36.86 53.86	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	2.77～ 3.00	Clayey Soil	9.0	—	18.0 18.0	53.86 55.93	0.00 2.25	0.200 0.200	11.31 11.31	—— ——	—— ——	39.36 39.12
4	3.00～ 4.25	Clayey Soil	9.0	—	18.0 18.0	55.93 67.18	2.25 14.50	0.200 0.200	11.31 11.31	—— ——	—— ——	39.12 37.74
5	4.25～ 7.27	Clayey Soil	9.0	—	18.0 18.0	67.18 94.36	14.50 44.10	0.200 0.200	11.31 11.31	—— ——	—— ——	37.74 33.80
6	7.27～ 8.27	Clayey Soil	9.0	—	84.0 84.0	94.36 103.36	44.10 53.90	0.200 0.200	11.31 11.31	—— ——	—— ——	43.20 43.02

Case: Lower Marikina Section 4 + 050 J

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
7	8.27～ 10.27	Sandy Soil	9.0	26.0	—— ——	103.36 121.36	53.90 73.50	0.200 0.200	11.31 11.31	0.51997 0.51997	0.50225 0.50225	—— ——
8	10.27～ 17.27	Clayey Soil	8.1	—	54.0 54.0	121.36 178.06	73.50 142.10	0.200 0.200	11.31 11.31	—— ——	—— ——	41.19 39.11
9	17.27～ 21.77	Clayey Soil	8.1	—	71.5 71.5	178.06 214.51	142.10 186.20	0.200 0.200	11.31 11.31	—— ——	—— ——	40.78 39.78

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
$\delta = 15.00$、$\beta = 0.00$、$\theta = 0.00$

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	ϕ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
4	3.00～ 4.25	Clayey soil	18.00	0.0	18.0 18.0	0.000 22.500	0.00 0.00	0.200 0.200	11.31 11.31	—— ——	—— ——
5	4.25～ 7.27	Clayey soil	9.00	0.0	18.0 18.0	22.500 49.680	0.00 29.60	0.200 0.200	11.31 11.31	—— ——	—— ——
6	7.27～ 8.27	Clayey soil	9.00	0.0	84.0 84.0	49.680 58.680	29.60 39.40	0.200 0.200	11.31 11.31	—— ——	—— ——
7	8.27～ 10.27	Sandy soil	9.00	26.0	—— ——	58.680 76.680	39.40 59.00	0.200 0.200	11.31 11.31	2.21182 2.21182	2.21182 2.21182
8	10.27～ 17.27	Clayey soil	8.10	0.0	54.0 54.0	76.680 133.380	59.00 127.60	0.200 0.200	11.31 11.31	—— ——	—— ——
9	17.27～ 21.77	Clayey soil	8.10	0.0	71.5 71.5	133.380 169.830	127.60 171.70	0.200 0.200	11.31 11.31	—— ——	—— ——

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
$\delta = -0.00$、$\beta = 0.00$、$\theta = \tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta-\theta) \cdot \left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 1.77	2.18 16.09	—— ——	—— ——
2	1.77～ 2.77	16.09 23.52	—— ——	—— ——
3	2.77～ 3.00	30.28 32.91	0.00 2.25	—— ——

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
4	3.00～ 4.25	32.91 47.35	2.25 14.50	36.00 58.50
5	4.25～ 7.27	47.35 83.61	14.50 14.50	58.50 85.68
6	7.27～ 8.27	0.00 0.00	14.50 14.50	217.68 226.68
7	8.27～ 10.27	51.91 60.95	14.50 14.50	129.79 169.60
8	10.27～ 17.27	40.13 111.54	14.50 14.50	184.68 241.38
9	17.27～ 21.77	74.78 120.64	14.50 14.50	276.38 312.83

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot \left[\Sigma \gamma h + \frac{Q}{\cos(-\beta)} \right] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [\ K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}\ ] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma$ ... Q ... Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

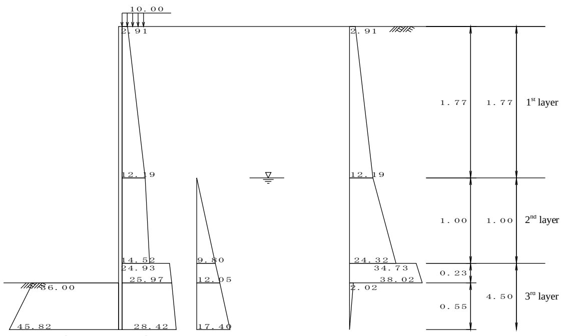
Clayey soil $P_p = \Sigma \gamma h$ ...

Mixed soil $P_p = [\ K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}\ ] \cdot \cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



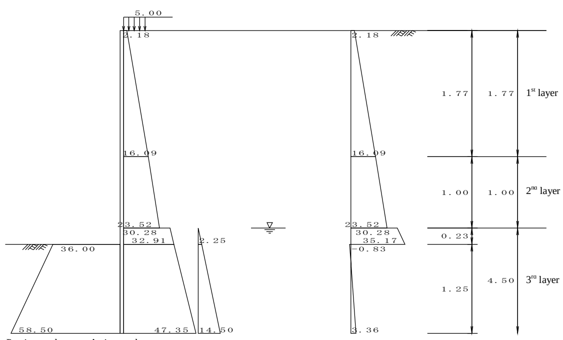
Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.77	2.91 12.19	—— ——	—— ——	2.91 12.19
2	1.77～ 2.77	12.19 14.52	0.00 9.80	—— ——	12.19 24.32
3	2.77～ 3.00	24.93 25.97	9.80 12.05	—— ——	34.73 38.02
4	3.00～ 3.55	25.97 28.42	12.05 17.40	36.00 45.82	2.02 0.00
5	3.55～ 4.25	28.42 31.59	17.40 24.30	45.82 58.50	0.00 -2.61

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.55 m (GL -3.55 m)

3-2 Seismic Condition



Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.77	2.18 16.09	—— ——	—— ——	2.18 16.09
2	1.77～ 2.77	16.09 23.52	—— ——	—— ——	16.09 23.52
3	2.77～ 3.00	30.28 32.91	0.00 2.25	—— ——	30.28 35.17
4	3.00～ 4.25	32.91 47.35	2.25 14.50	36.00 58.50	-0.83 3.36

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.00 m (GL -3.00 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m	
Corrosion margin	t_1	=	1.00 mm (active side)	t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82	
Section efficiency	μ	=	1.00	
Young's modulus	E	=	200000 N/mm²	
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)	
	I	=	20008 cm⁴(after reduction by corrosion and section)	
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$	

	Depth (m)	N-value
1	2.77	3
2	3.77	4
3	4.77	3
4	5.77	2
5	6.77	3
6	7.77	14
7	8.77	23
8	9.77	28
9	10.77	6
10	11.77	10

	Depth (m)	N-value
11	12.77	8
12	13.77	15
13	14.77	25
14	15.77	29
15	16.77	27
16	17.77	27
17	18.77	30
18	19.77	22
19	20.77	43
20	21.77	46

4-2 Normal Condition

K_h = 11170 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.514 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.95 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.55 m) to 1.95 m depth (GL -5.49 m).

	Depth (m)	N-value
1	3.55	3.78
2	3.77	4.00
3	4.77	3.00
4	5.49	2.28
Σh = 13.05		

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{13.05}{4}$

$= 3.26$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 3.26^{0.406} = 11170$ kN/m³

Kh (normal condition) = 11170 kN/m^3

4-3 Seismic Condition

K_h = 11170 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.514 \quad m^{-1}$

$$L = \frac{1}{\beta} = 1.95 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.00 m) and 1.95 m depth (GL -4.95 m).

	Depth (m)	N-value
1	3.00	3.23
2	3.77	4.00
3	4.77	3.00
4	4.95	2.82
Σh = 13.05		

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{13.05}{4}$

$= 3.26$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 3.26^{0.406} = 11170$ kN/m³

Kh (seismic condition) = 11170 kN/m^3

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.77	1.77	2.91 12.19	2.58 10.79	2.96 2.37	7.62 25.52
2	1.77～ 2.77	1.00	12.19 24.32	6.09 12.16	1.44 1.11	8.79 13.48
3	2.77～ 3.00	0.23	34.73 38.02	3.99 4.37	0.70 0.62	2.79 2.72
4	3.00～ 3.55	0.55	2.02 0.00	0.55 0.00	0.36 0.18	0.20 0.00
				ΣP = 40.53		ΣM = 61.12

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.55 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{61.12}{40.53} = 1.51 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.77	1.77	2.18 16.09	1.93 14.24	2.41 1.82	4.66 25.92
2	1.77～ 2.77	1.00	16.09 23.52	8.05 11.76	0.90 0.56	7.22 6.62
3	2.77～ 3.00	0.23	30.28 35.17	3.48 4.04	0.15 0.08	0.53 0.31
				ΣP = 43.51		ΣM = 45.26

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.00 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{45.26}{43.51} = 1.04 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	11170	kN/m³
calculated value	β	=	0.51397	m^{-1}
resultant earth force (lateral)	P_0	=	40.53	kN/m
height of acting position of load	h_0	=	1.51	m
moment	M_0	=	61.12	kN·m/m

in consideration of ψ_m = 1.216,

maximum moment	M_{max}	=	74.32	kN·m/m
depth of generated position of M_{max}	l_m	=	0.727	m
depth of 1st fixed point	l_i	=	2.255	m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	11170	kN/m³
calculated value	β	=	0.51397	m^{-1}
resultant earth force (lateral)	P_0	=	43.51	kN/m
height of acting position of load	h_0	=	1.04	m
moment	M_0	=	45.26	kN·m/m

in consideration of ψ_m = 1.370,

maximum moment	M_{max}	=	62.02	kN·m/m
depth of generated position of M_{max}	l_m	=	0.876	m
depth of 1st fixed point	l_i	=	2.404	m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{74.32\times10^6}{1320\times10^3} = 56\ \text{N/mm}^2 \leqq \sigma_a = 180\ \text{N/mm}^2 \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{62.02\times10^6}{1320\times10^3} = 47\ \text{N/mm}^2 \leqq \sigma_a = 270\ \text{N/mm}^2 \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.77	2.96 2.37	0.834 0.667	0.251 0.173	2.58 10.79	0.646 1.867
2	1.77～ 2.77	1.44 1.11	0.407 0.313	0.072 0.044	6.09 12.16	0.436 0.533
3	2.77～ 3.00	0.70 0.62	0.197 0.176	0.018 0.015	3.99 4.37	0.073 0.063
4	3.00～ 3.55	0.36 0.18	0.103 0.051	0.005 0.001	0.55 0.00	0.003 0.000
						ΣQ = 3.621

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

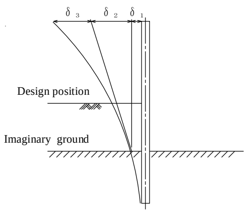
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5140\times1.51)\times40.53}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5140^3} = 0.00662\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5140\times1.51)\times40.53}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5140^2}\times(3.00+0.55) = 0.01733\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.62\times(3.00+0.55)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00403\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00662 + 0.01733 + 0.00403$
$= 0.02799$ m
$= 27.99 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.77	2.41 1.82	0.803 0.607	0.236 0.147	1.93 14.24	0.456 2.091
2	1.77～ 2.77	0.90 0.56	0.299 0.188	0.040 0.017	8.05 11.76	0.324 0.194
3	2.77～ 3.00	0.15 0.08	0.051 0.026	0.001 0.000	3.48 4.04	0.004 0.001
						ΣQ = 3.071

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

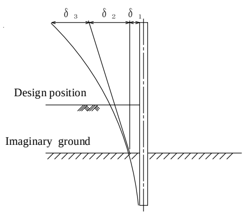
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5140\times1.04)\times43.51}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5140^3} = 0.00614\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5140\times1.04)\times43.51}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5140^2}\times(3.00+0.00) = 0.01278\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.07\times(3.00+0.00)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00207\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00614 + 0.01278 + 0.00207$
$= 0.02099$ m
$= 20.99 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction $K_h = 11170 kN/m^3$
Calculated value $\beta = 0.48909 m^{-1}$

Penetration length of SSP $D = 0.55 + \dfrac{3}{0.489} = 6.68m$

Whole length of SSP $L = 3.00 - 0.40 + 6.68 = 9.28m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction $K_h = 11170 kN/m^3$
Calculated value $\beta = 0.48909 m^{-1}$

Penetration length of SSP $D = 0.00 + \dfrac{3}{0.489} = 6.13m$

Whole length of SSP $L = 3.00 - 0.40 + 6.13 = 8.68m$

Therefore, whole length of SSP is set as 9.30 m in consideration of round unit of SSP length.

7 Calculation Result

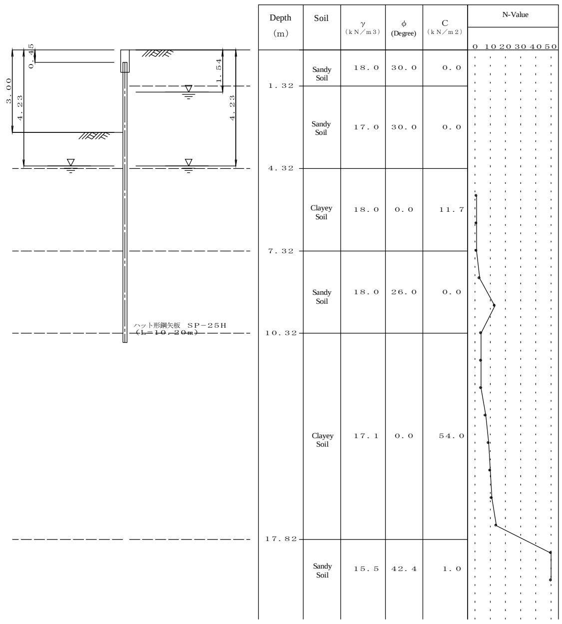
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		74.32	62.02
Stress intensity	σ (N/mm^2)		56 (180)	47 (270)
Lateral displacement	δ (mm)		27.99 (50.0)	20.99 (75.0)
Penetration depth	D (m)		6.68	6.13
Whole length of SSP	L (m)	9.30		

－Steel Sheet Pile Design Calculation－

Lower Marikina Section 4 + 250 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_{lt}	=	0.40 m
Landside WL	L_{wa}	=	1.54 m (Normal Condition)
	L_{wa}'	=	4.23 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.23 m (Normal Condition)
	L_{wp}'	=	4.23 m (Seismic Condition)

Imaginary riverbed calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress Chang's formula

Penetration depths $L = \dfrac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water γ_w = 9.8 kN/m^3

Type of water pressure trapezoidal water pressure

Lateral pressure calculated in consideration of site conditions

Study case
- Normal Condition
- Seismic Condition

Design earthquake intensity k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load not considered

Minimum angle of rupture ζ_0 = 10 degrees

Rear side angle of slope not considered

Angle of rupture (clayey soil) $\zeta = \tan^{-1}\sqrt{1 - \dfrac{\Sigma\gamma h + 2Q}{2C}} \cdot \tan\theta$

Equilibrium factor of compression K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula $K_h = 6910 \times N'^{0.406}$

Average N-value calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value
1	5.32	1
2	6.32	1
3	7.32	1
4	8.32	3
5	9.32	13
6	10.32	4
7	11.32	4
8	12.32	4
9	13.32	7
10	14.32	9

No	Depth (m)	N-Value
11	15.32	10
12	16.32	11
13	17.32	14
14	18.32	50
15	19.32	50
16	20.32	50
17	21.32	50
18	22.32	50
19	23.32	50
20	24.32	50

1-6 Vertical Load

Vertical load on landside calculated in consideration of embankment shape on landside

Vertical load on riverside not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree)		kh(kN/m^3)	
										normal	seismic	normal	seismic
1	1.32	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	――	――
2	4.32	S	3.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	――	――
3	7.32	C	1.0	18.00	9.00	0.0	11.7	0.0	0.200	auto	auto	――	――
4	10.32	S	7.0	18.00	9.00	26.0	0.0	0.0	0.200	auto	auto	――	――
5	17.82	C	9.0	17.10	8.10	0.0	54.0	0.0	0.200	auto	auto	――	――
6	20.00	S	50.0	15.50	6.50	42.4	1.0	0.0	0.200	auto	auto	――	――

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- φ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin t_1 = 1.00 mm (riverside) t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length 0.10 m

Allowable stress σ_a = 180 N/mm (Normal)
σ_a' = 270 N/mm (Seismic)

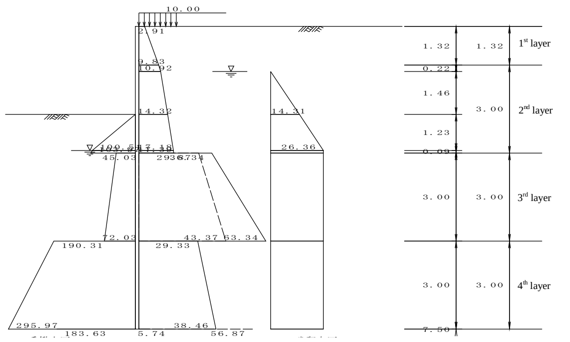
Allowable displacement δ_a = 50.0 mm (Normal)
δ_a' = 75.0 mm (Seismic)

Bending of cantilever beam calculated as distributed load of each layer

Reduction of material modulus
Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



Passive earth pressure　　Active earth pressure　　Residual water pressure

2-1-1 Soil Modulus of Active Side

No	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～1.32	Sandy soil	18.0	30.0	—— ——	10.000 33.760	0.30142 0.30142	0.29115 0.29115
2	1.32～1.54	Sandy soil	17.0	30.0	—— ——	33.760 37.500	0.30142 0.30142	0.29115 0.29115
3	1.54～3.00	Sandy soil	8.0	30.0	—— ——	37.500 49.180	0.30142 0.30142	0.29115 0.29115
4	3.00～4.23	Sandy soil	8.0	30.0	—— ——	49.180 59.020	0.30142 0.30142	0.29115 0.29115
5	4.23～4.32	Sandy soil	8.0	30.0	—— ——	59.020 59.740	0.30142 0.30142	0.29115 0.29115
6	4.32～7.32	Clayey soil	9.0	—	11.7 11.7	59.740 86.740	—— ——	—— ——
7	7.32～10.32	Sandy soil	9.0	26.0	—— ——	86.740 113.740	0.35007 0.35007	0.33814 0.33814
8	10.32～17.82	Clayey soil	8.1	—	54.0 54.0	113.740 174.490	—— ——	—— ——
9	17.82～20.00	Sandy soil	6.5	42.4	—— ——	174.490 188.660	0.18084 0.18084	0.17468 0.17468

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ= 15.00、β＝ 0.00、θ＝ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

No	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
4	3.00～4.23	Sandy soil	17.0	30.0	—— ——	0.000 20.910	4.97650 4.97650	4.80693 4.80693
5	4.23～4.32	Sandy soil	8.0	30.0	—— ——	20.910 21.630	4.97650 4.97650	4.80693 4.80693
6	4.32～7.32	Clayey soil	9.0	0.0	11.7 11.7	21.630 48.630	—— ——	—— ——
7	7.32～10.32	Sandy soil	9.0	26.0	—— ——	48.630 75.630	4.05145 4.05145	3.91340 3.91340
8	10.32～17.82	Clayey soil	8.1	0.0	54.0 54.0	75.630 136.380	—— ——	—— ——
9	17.82～20.00	Sandy soil	6.5	42.4	—— ——	136.380 150.550	10.38845 10.38845	10.03447 10.03447

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ= -15.00、β＝ 0.00、θ＝ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

No	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～1.32	2.91 9.83	——	2.91 9.83	——	——
2	1.32～1.54	9.83 10.92	——	9.83 10.92	——	——
3	1.54～3.00	10.92 14.32	——	10.92 14.32	0.00 14.31	——
4	3.00～4.23	14.32 17.18	——	14.32 17.18	14.31 26.36	0.00 100.51
5	4.23～4.32	17.18 17.39	——	17.18 17.39	26.36 26.36	100.51 103.97
6	4.32～7.32	36.34 63.34	29.87 43.37	36.34 63.34	26.36 26.36	45.03 72.03
7	7.32～10.32	29.33 38.46	——	29.33 38.46	26.36 26.36	190.31 295.97
8	10.32～17.82	5.74 66.49	56.87 87.25	56.87 87.25	26.36 26.36	183.63 244.38
9	17.82～20.00	30.48 32.95	——	30.48 32.95	26.36 26.36	1368.50 1510.69

- Formula for active earth pressure

Sandy soil　$P_{a1} = K_a\cdot\left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

Clayey soil　$P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c\cdot(\Sigma\gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil　$P_{a1} = [K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}]\cdot\cos\delta$

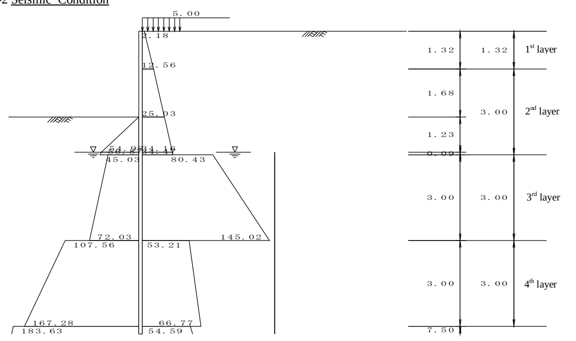
- Formula for passive earth pressure

Sandy soil　$P_p = K_p\cdot\left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

Clayey soil　$P_p = \Sigma\gamma h + Q + 2C$

Mixed soil　$P_p = [K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}]\cdot\cos\delta$

2-2 Seismic Condition



Passive earth pressure　　Active earth pressure　　Residual water pressure

2-2-1 Soil Modulus of Active Side

No	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～1.32	Sandy Soil	18.0	30.0	—— ——	5.00 28.76	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	1.32～3.00	Sandy Soil	17.0	30.0	—— ——	28.76 57.32	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	3.00～4.23	Sandy Soil	17.0	30.0	—— ——	57.32 78.23	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
4	4.23～4.32	Sandy Soil	8.0	30.0	—— ——	78.23 78.95	0.00 0.88	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
5	4.32～7.32	Clayey Soil	9.0	—	11.7 11.7	78.95 105.95	0.88 30.28	0.200 0.200	11.31 11.31	—— ——	—— ——	27.99 12.81
6	7.32～10.32	Sandy Soil	9.0	26.0	—— ——	105.95 132.95	30.28 59.68	0.200 0.200	11.31 11.31	0.51997 0.51997	0.50225 0.50225	—— ——
7	10.32～17.82	Clayey Soil	8.1	—	54.0 54.0	132.95 193.70	59.68 133.18	0.200 0.200	11.31 11.31	—— ——	—— ——	40.79 38.48
8	17.82～20.00	Sandy Soil	6.5	42.4	—— ——	193.70 207.87	133.18 154.55	0.200 0.200	11.31 11.31	0.28983 0.28983	0.27995 0.27995	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;

δ= 15.00、β＝ 0.00、θ＝ 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

No	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
3	3.00～4.23	Sandy soil	17.00	30.0	—— ——	0.000 20.910	0.00 0.00	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
4	4.23～4.32	Sandy soil	8.00	30.0	—— ——	20.910 21.630	0.00 0.88	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
5	4.32～7.32	Clayey soil	9.00	0.0	11.7 11.7	21.630 48.630	0.88 30.28	0.200 0.200	11.31 11.31	—— ——	—— ——
6	7.32～10.32	Sandy soil	9.00	26.0	—— ——	48.630 75.630	30.28 59.68	0.200 0.200	11.31 11.31	2.21182 2.21182	2.21182 2.21182
7	10.32～17.82	Clayey soil	8.10	0.0	54.0 54.0	75.630 136.380	59.68 133.18	0.200 0.200	11.31 11.31	—— ——	—— ——
8	17.82～20.00	Sandy soil	6.50	42.4	—— ——	136.380 150.550	133.18 154.55	0.200 0.200	11.31 11.31	4.67049 4.67049	4.67049 4.67049

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;

δ= -0.00、β＝ 0.00、θ＝ $\tan^{-1}$k

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～1.32	2.18 12.56	—— ——	—— ——
2	1.32～3.00	12.56 25.03	—— ——	—— ——
3	3.00～4.23	25.03 34.16	—— ——	0.00 54.98
4	4.23～4.32	34.16 34.47	0.00 0.00	54.98 56.87
5	4.32～7.32	80.43 145.02	0.00 0.00	45.03 72.03
6	7.32～10.32	53.21 66.77	0.00 0.00	107.56 167.28
7	10.32～17.82	54.59 131.57	0.00 0.00	183.63 244.38
8	17.82～20.00	54.23 58.19	0.00 0.00	636.96 703.14

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

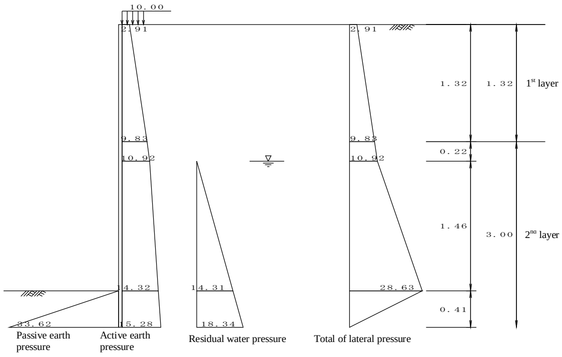
Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

Passive earth pressure Active earth pressure Residual water pressure Total of lateral pressure

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 1.32	2.91 9.83	—— ——	—— ——	2.91 9.83
2	1.32～ 1.54	9.83 10.92	—— ——	—— ——	9.83 10.92
3	1.54～ 3.00	10.92 14.32	0.00 14.31	—— ——	10.92 28.63
4	3.00～ 3.41	14.32 15.28	14.31 18.34	0.00 33.62	28.63 0.00
5	3.41～ 4.23	15.28 17.18	18.34 26.36	33.62 100.51	0.00 -56.97

P_a : Active earth pressure

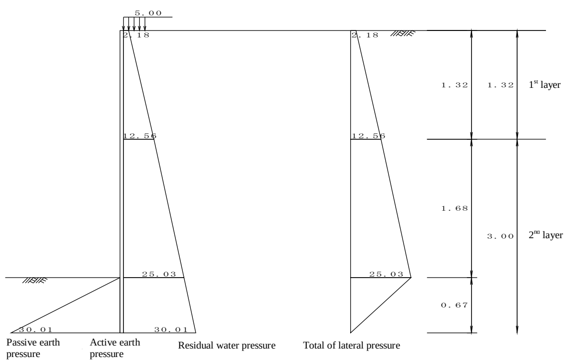
P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.41 m (GL -3.41 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2	Ps kN/m^2
1	0.00～ 1.32	2.18 12.56	—— ——	—— ——	2.18 12.56
2	1.32～ 3.00	12.56 25.03	—— ——	—— ——	12.56 25.03
3	3.00～ 3.67	25.03 30.01	—— ——	0.00 30.01	25.03 0.00
4	3.67～ 4.23	30.01 34.16	—— ——	30.01 54.98	0.00 -20.82

P_a : Active earth pressure

P_w : Residual water pressure

P_p : Passive earth pressure

P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.67 m (GL -3.67 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m		
Corrosion margin	t_1	=	1.00 mm (active side)	t_2 =	1.00 mm (passive side)
Corrosion rate	η	=	0.82		
Section efficiency	μ	=	1.00		
Young's modulus	E	=	200000 N/mm^2		
Inertia sectional moment	I_0	=	24400 cm^4(original condition)		
	I	=	20008 cm^4(after reduction by corrosion and section)		
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$		

	Depth (m)	N-value
1	5.32	1
2	6.32	1
3	7.32	1
4	8.32	3
5	9.32	13
6	10.32	4
7	11.32	4
8	12.32	4
9	13.32	7
10	14.32	9

	Depth (m)	N-value
11	15.32	10
12	16.32	11
13	17.32	14
14	18.32	50
15	19.32	50
16	20.32	50
17	21.32	50
18	22.32	50
19	23.32	50
20	24.32	50

4-2 Normal Condition

K_h = 6910 kN/m^3 is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.456 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 2.19 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.41 m) to 2.19 m depth (GL -5.61 m).

	Depth (m)	N-value
1	3.41	1.00
2	5.32	1.00
3	5.61	1.00
Σh = 3.00		

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{3.00}{3}$

$= 1.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 1.00^{0.406} = 6910$ kN/m³
Kh (normal condition) = 6910 kN/m³

4-3 Seismic Condition

K_h = 6910 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.456$ m⁻¹

$$L = \frac{1}{\beta} = 2.19 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.67 m) and 2.19 m depth (GL -5.87 m).

	Depth (m)	N-value
1	3.67	1.00
2	5.32	1.00
3	5.87	1.00
	Σh = 3.00	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{3.00}{3}$

$= 1.00$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:
$K_h = 6910 \times N'^{0.406} = 6910 \times 1.00^{0.406} = 6910$ kN/m³
Kh (seismic condition) = 11170 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.32	1.32	2.91 9.83	1.92 6.49	2.97 2.53	5.71 16.42
2	1.32～ 1.54	0.22	9.83 10.92	1.08 1.20	2.02 1.94	2.18 2.34
3	1.54～ 3.00	1.46	10.92 28.63	7.97 20.90	1.38 0.90	11.04 18.77
4	3.00～ 3.41	0.41	28.63 0.00	5.89 0.00	0.27 0.14	1.61 0.00
				ΣP = 45.45		ΣM = 58.07

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load P_t = 0.0 kN/m
depth to acting position H_t = 0.00 m
moment M_m = 0.0 kN·m/m
depth to acting position H_m = 0.00 m
Height from riverbed to top of coping H = 3.00 m
Depth of Imaginary riverbed from riverbed L_k = 0.41 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m
h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$
$$= \frac{58.07}{45.45} = 1.28 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.32	1.32	2.18 12.56	1.44 8.29	3.23 2.79	4.66 23.14
2	1.32～ 3.00	1.68	12.56 25.03	10.55 21.02	1.79 1.23	18.90 25.89
3	3.00～ 3.67	0.67	25.03 0.00	8.40 0.00	0.45 0.22	3.76 0.00
				ΣP = 49.70		ΣM = 76.34

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load P_t = 0.0 kN/m
depth to acting position H_t = 0.00 m
moment M_m = 0.0 kN·m/m
depth to acting position H_m = 0.00 m
Height from riverbed to top of coping H = 3.00 m
Depth of Imaginary riverbed from riverbed L_k = 0.67 m

Moment M_t by arbitrary load is as below
$M_t = P_t \cdot (H + L_k - H_t) + M_m$ = 00.00 kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$
$$= \frac{76.34}{49.70} = 1.54 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width B = 1.0000 m
Corrosion margin t_1 = 1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate η = 0.82
Section efficiency μ = 1.00
Young's modulus E = 200000 N/mm²
Inertia sectional moment I_0 = 24400 cm⁴ (original condition)
I = 20008 cm⁴ (after reduction by corrosion and section)
EI = 200000 × 10³ × 20008 × 10⁻⁸ = 4.002 × 10⁴

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} (\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction K_h = 6910 kN/m³
calculated value β = 0.45582 m⁻¹
resultant earth force (lateral) P_0 = 45.45 kN/m
height of acting position of load h_0 = 1.28 m
moment M_0 = 58.07 kN·m/m

in consideration of ψ_m = 1.328,
maximum moment M_{max} = 77.12 kN·m/m
depth of generated position of M_{max} l_m = 0.949 m
depth of 1st fixed point l_i = 2.672 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction K_h = 6910 kN/m³
calculated value β = 0.45582 m⁻¹
resultant earth force (lateral) P_0 = 49.70 kN/m
height of acting position of load h_0 = 1.54 m
moment M_0 = 76.34 kN·m/m

in consideration of ψ_m = 1.251,
maximum moment M_{max} = 95.53 kN·m/m
depth of generated position of M_{max} l_m = 0.866 m
depth of 1st fixed point l_i = 2.589 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin t_1 = 1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate η = 0.82
Section efficiency μ = 1.00
Module of section Z_0 = 1610 cm³ (original condition)
Z = 1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{77.12\times10^6}{1320\times10^3} = 58\ \text{N/mm}^2 \leqq \sigma_a = 180\ \text{N/mm}^2 \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{95.53\times10^6}{1320\times10^3} = 72\ \text{N/mm}^2 \leqq \sigma_a = 270\ \text{N/mm}^2 \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.32	2.97 2.53	0.871 0.742	0.269 0.207	1.92 6.49	0.517 1.344
2	1.32～ 1.54	2.02 1.94	0.592 0.570	0.140 0.132	1.08 1.20	0.152 0.158
3	1.54～ 3.00	1.38 0.90	0.406 0.263	0.071 0.032	7.97 20.90	0.568 0.661
4	3.00～ 3.41	0.27 0.14	0.080 0.040	0.003 0.001	5.89 0.00	0.019 0.000
						ΣQ = 3.418

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

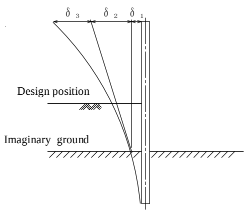
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4558\times1.28)\times45.45}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^3} = 0.00949\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4558\times1.28)\times45.45}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^2}\times(3.00+0.41) = 0.02018\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.42\times(3.00+0.41)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00339\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00949 + 0.02018 + 0.00339$
$= 0.03306$ m
$= 33.06 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.32	3.23 2.79	0.880 0.760	0.274 0.216	1.44 8.29	0.394 1.788
2	1.32～ 3.00	1.79 1.23	0.488 0.335	0.100 0.050	10.55 21.02	1.051 1.050
3	3.00～ 3.67	0.45 0.22	0.122 0.061	0.007 0.002	8.40 0.00	0.060 0.000
						ΣQ = 4.345

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

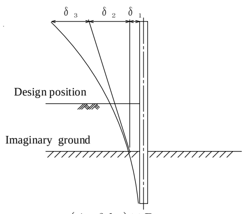
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4558\times1.54)\times49.70}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^3} = 0.01115\ \text{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4558\times1.54)\times49.70}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4558^2}\times(3.00+0.67) = 0.02634\ \text{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{4.34\times(3.00+0.67)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00537\ \text{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.01115 + 0.02634 + 0.00537$
$= 0.04286$ m
$= 42.86 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	24400 cm⁴ (after reduction by corrosion and section)
EI = 200000 × 10³ × 24400 × 10⁻⁸		=	4.880 × 10⁴

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction $K_h = 6190 kN/m^3$

Calculated value $\beta = 0.43376 m^{-1}$

Penetration length of SSP $D = 0.41 + \dfrac{3}{0.434} = 7.33m$

Whole length of SSP $L = 3.00 - 0.40 + 7.33 = 9.93m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction $K_h = 6190 kN/m^3$

Calculated value $\beta = 0.43376 m^{-1}$

Penetration length of SSP $D = 0.67 + \dfrac{3}{0.434} = 7.59m$

Whole length of SSP $L = 3.00 - 0.40 + 7.59 = 10.19m$

Therefore, whole length of SSP is set as 10.20 m in consideration of round unit of SSP length.

7 Calculation Result

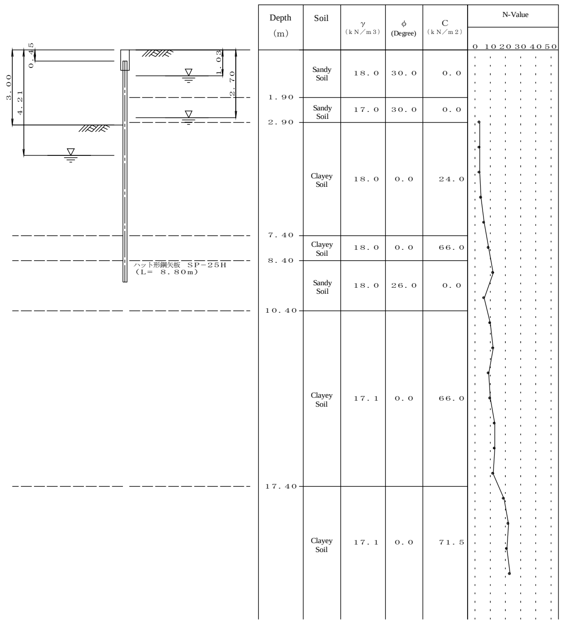
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		77.12	95.53
Stress intensity	σ (N/mm^2)		58 (180)	72 (270)
Lateral displacement	δ (mm)		33.06 (50.0)	42.86 (75.0)
Penetration depth	D (m)		7.33	7.59
Whole length of SSP	L (m)	10.20		

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 4 + 400 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	1.03 m (Normal Condition)
	L_{wa}'	=	2.70 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.21 m (Normal Condition)
	L_{wp}'	=	4.21 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \frac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake
considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C}} \cdot \tan\theta$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value	No	Depth (m)	N-Value
1	2.90	3	11	12.90	9
2	3.90	3	12	13.90	10
3	4.90	3	13	14.90	13
4	5.90	4	14	15.90	13
5	6.90	6	15	16.90	12
6	7.90	9	16	17.90	19
7	8.90	12	17	18.90	22
8	9.90	6	18	19.90	21
9	10.90	10	19	20.90	23
10	11.90	12	20	21.90	28

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	φ	C kN/m^2	a	k'	ζ (degree)		kh(kN/m^3)	
										normal	seismic	normal	seismic
1	1.90	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	——	——
2	2.90	S	3.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	——	——
3	7.40	C	4.0	18.00	9.00	0.0	24.0	0.0	0.200	auto	auto	——	——
4	8.40	C	11.0	18.00	9.00	0.0	66.0	0.0	0.200	auto	auto	——	——
5	10.40	S	9.0	18.00	9.00	26.0	0.0	0.0	0.200	auto	auto	——	——
6	17.40	C	11.0	17.10	8.10	0.0	66.0	0.0	0.200	auto	auto	——	——
7	21.90	C	22.0	17.10	8.10	0.0	71.5	0.0	0.200	auto	auto	——	——

Note) depth : from top of coping to bottom of the layer
soil : sandy(S), clayey(C), mixed (M)
N-value : average N-value in the layer
γ : wet unit weight of soil
γ' : saturated unit weight of soil
φ : internal friction angle of soil
C_o : soil adhesion
a : slope of soil adhesion
k' : design seismic coefficient (underwater)
ζ : angle of active rupture
kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active	15.00°	15.00°
passive	-15.00°	0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm（Normal）
σ_a' = 270 N/mm（Seismic）

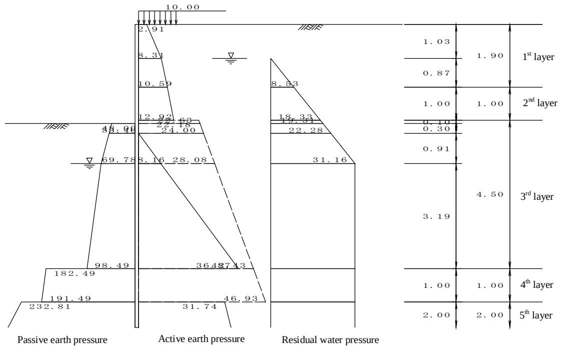
Allowable displacement　　δ_a = 50.0 mm（Normal）
δ_a' = 75.0 mm（Seismic）

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qa (kN/m²)	Ka	Ka ×cosδ
1	0.00～ 1.03	Sandy soil	18.0	30.0	— —	10.000 28.540	0.30142 0.30142	0.29115 0.29115
2	1.03～ 1.90	Sandy soil	9.0	30.0	— —	28.540 36.370	0.30142 0.30142	0.29115 0.29115
3	1.90～ 2.90	Sandy soil	8.0	30.0	— —	36.370 44.370	0.30142 0.30142	0.29115 0.29115
4	2.90～ 3.00	Clayey soil	9.0	—	24.0 24.0	44.370 45.270	— —	— —
5	3.00～ 3.30	Clayey soil	9.0	—	24.0 24.0	45.270 48.000	— —	— —
6	3.30～ 4.21	Clayey soil	9.0	—	24.0 24.0	48.000 56.160	— —	— —
7	4.21～ 7.40	Clayey soil	9.0	—	24.0 24.0	56.160 84.870	— —	— —
8	7.40～ 8.40	Clayey soil	9.0	—	66.0 66.0	84.870 93.870	— —	— —
9	8.40～ 10.40	Sandy soil	9.0	26.0	— —	93.870 111.870	0.35007 0.35007	0.33814 0.33814
10	10.40～ 12.89	Clayey soil	8.1	—	66.0 66.0	111.870 132.000	— —	— —
11	12.89～ 17.40	Clayey soil	8.1	—	66.0 66.0	132.000 168.570	— —	— —
12	17.40～ 21.90	Clayey soil	8.1	—	71.5 71.5	168.570 205.020	— —	— —

Case: Lower Marikina Section 4 + 400 J

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\frac{\sin(\phi+\delta) \cdot \sin(\phi-\beta-\theta)}{\cos(\delta+\theta) \cdot \cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	Kp	Kp ×cosδ
5	3.00～ 3.30	Clayey soil	18.0	0.0	24.0 24.0	0.000 5.460	— —	— —
6	3.30～ 4.21	Clayey soil	18.0	0.0	24.0 24.0	5.460 21.780	— —	— —
7	4.21～ 7.40	Clayey soil	9.0	0.0	24.0 24.0	21.780 50.490	— —	— —
8	7.40～ 8.40	Clayey soil	9.0	0.0	66.0 66.0	50.490 59.490	— —	— —
9	8.40～ 10.40	Sandy soil	9.0	26.0	— —	59.490 77.490	4.05145 4.05145	3.91340 3.91340
10	10.40～ 12.89	Clayey soil	8.1	0.0	66.0 66.0	77.490 97.620	— —	— —
11	12.89～ 17.40	Clayey soil	8.1	0.0	66.0 66.0	97.620 134.190	— —	— —
12	17.40～ 21.90	Clayey soil	8.1	0.0	71.5 71.5	134.190 170.640	— —	— —

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\frac{\sin(\phi+\delta) \cdot \sin(\phi-\beta-\theta)}{\cos(\delta+\theta) \cdot \cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
1	0.00～ 1.03	2.91 8.31	— —	2.91 8.31	— —	— —
2	1.03～ 1.90	8.31 10.59	— —	8.31 10.59	0.00 8.53	— —
3	1.90～ 2.90	10.59 12.92	— —	10.59 12.92	8.53 18.33	— —
4	2.90～ 3.00	-3.63 -2.73	22.18 22.63	22.18 22.63	18.33 19.31	— —
5	3.00～ 3.30	-2.73 0.00	22.63 24.00	22.63 24.00	19.31 22.28	48.00 53.46
6	3.30～ 4.21	0.00 8.16	24.00 28.08	24.00 28.08	22.28 31.16	53.46 69.78
7	4.21～ 7.40	8.16 36.87	28.08 42.43	28.08 42.43	31.16 31.16	69.78 98.49
8	7.40～ 8.40	-47.13 -38.13	42.43 46.93	42.43 46.93	31.16 31.16	182.49 191.49
9	8.40～ 10.40	31.74 37.83	— —	31.74 37.83	31.16 31.16	232.81 303.25

Case: Lower Marikina Section 4 + 400 J

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²	Pw kN/m²	Pp kN/m²
10	10.40～ 12.89	-20.13 0.00	55.93 66.00	55.93 66.00	31.16 31.16	209.49 229.62
11	12.89～ 17.40	0.00 36.57	66.00 84.28	66.00 84.28	31.16 31.16	229.62 266.19
12	17.40～ 21.90	25.57 62.02	84.28 102.51	84.28 102.51	31.16 31.16	277.19 313.64

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma\gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [\ K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}\ ] \cdot \cos\delta$

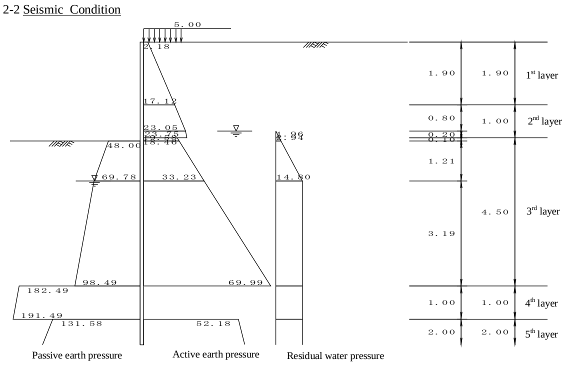
- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma\gamma h + \frac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = [\ K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}\ ] \cdot \cos\delta$

2-2 Seismic Condition



Case: Lower Marikina Section 4 + 400 J

2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Q (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 1.90	Sandy Soil	18.0	30.0	— —	5.00 39.20	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	— —
2	1.90～ 2.70	Sandy Soil	17.0	30.0	— —	39.20 52.80	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	— —
3	2.70～ 2.90	Sandy Soil	8.0	30.0	— —	52.80 54.40	0.00 1.96	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	— —
4	2.90～ 3.00	Clayey Soil	9.0	—	24.0 24.0	54.40 55.30	1.96 2.94	0.200 0.200	11.31 11.31	— —	— —	40.94 40.87
5	3.00～ 4.21	Clayey Soil	9.0	—	24.0 24.0	55.30 66.19	2.94 14.80	0.200 0.200	11.31 11.31	— —	— —	40.87 39.99
6	4.21～ 7.40	Clayey Soil	9.0	—	24.0 24.0	66.19 94.90	14.80 46.06	0.200 0.200	11.31 11.31	— —	— —	39.99 37.38
7	7.40～ 8.40	Clayey Soil	9.0	—	66.0 66.0	94.90 103.90	46.06 55.86	0.200 0.200	11.31 11.31	— —	— —	42.65 42.42
8	8.40～ 10.40	Sandy Soil	9.0	26.0	— —	103.90 121.90	55.86 75.46	0.200 0.200	11.31 11.31	0.51997 0.51997	0.50225 0.50225	— —
9	10.40～ 17.40	Clayey Soil	8.1	—	66.0 66.0	121.90 178.60	75.46 144.06	0.200 0.200	11.31 11.31	— —	— —	41.95 40.35
10	17.40～ 21.90	Clayey Soil	8.1	—	71.5 71.5	178.60 215.05	144.06 188.16	0.200 0.200	11.31 11.31	— —	— —	40.76 39.76

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ = 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\frac{\sin(\phi+\delta) \cdot \sin(\phi-\beta-\theta)}{\cos(\delta+\theta) \cdot \cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}} \cdot \tan\theta$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m³)	φ (degree)	C kN/m²	Σγh+Qp (kN/m²)	γwhw (kN/m²)	k (k')	θ (degree)	Kp	Kp ×cosδ
5	3.00～ 4.21	Clayey Soil	18.00	0.0	24.0 24.0	0.000 21.780	0.00 0.00	0.200 0.200	11.31 11.31	— —	— —
6	4.21～ 7.40	Clayey Soil	9.00	0.0	24.0 24.0	21.780 50.490	0.00 31.26	0.200 0.200	11.31 11.31	— —	— —
7	7.40～ 8.40	Clayey Soil	9.00	0.0	66.0 66.0	50.490 59.490	31.26 41.06	0.200 0.200	11.31 11.31	— —	— —
8	8.40～ 10.40	Sandy soil	9.00	26.0	— —	59.490 77.490	41.06 60.66	0.200 0.200	11.31 11.31	2.21182 2.21182	2.21182 2.21182
9	10.40～ 17.40	Clayey Soil	8.10	0.0	66.0 66.0	77.490 134.190	60.66 129.26	0.200 0.200	11.31 11.31	— —	— —
10	17.40～ 21.90	Clayey Soil	8.10	0.0	71.5 71.5	134.190 170.640	129.26 173.36	0.200 0.200	11.31 11.31	— —	— —

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
$\delta =$ -0.00、$\beta =$ 0.00、$\theta = \tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta-\theta) \cdot \left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 1.90	2.18 17.12	—— ——	—— ——
2	1.90～ 2.70	17.12 23.05	—— ——	—— ——
3	2.70～ 2.90	23.05 23.75	0.00 1.96	—— ——
4	2.90～ 3.00	18.46 19.58	1.96 2.94	—— ——
5	3.00～ 4.21	19.58 33.23	2.94 14.80	48.00 69.78
6	4.21～ 7.40	33.23 69.99	14.80 14.80	69.78 98.49
7	7.40～ 8.40	0.00 0.00	14.80 14.80	182.49 191.49
8	8.40～ 10.40	52.18 61.22	[illegible]	[illegible]
9	10.40～ 17.40	16.27 86.89	14.80 14.80	209.49 266.19
10	17.40～ 21.90	75.46 121.32	14.80 14.80	277.19 313.64

Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma \gamma h + Q - 2C$
$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$
K_c : Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = [K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}] \cdot \cos\delta$

- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

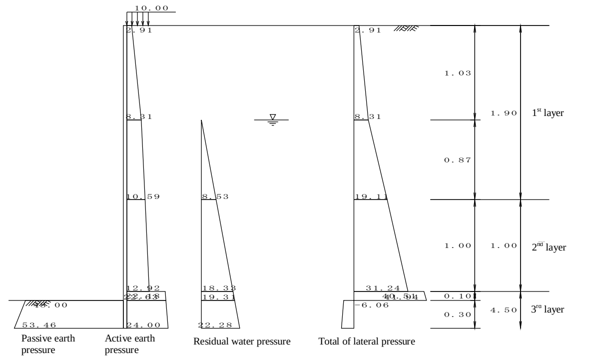
Clayey soil $P_p = \Sigma \gamma h + Q + 2C$

Mixed soil $P_p = [K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}] \cdot \cos\delta$

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition

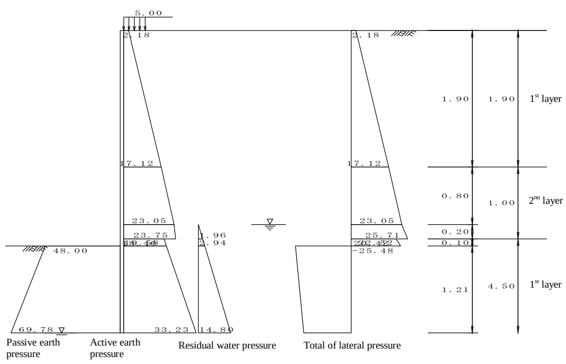


	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.03	2.91 8.31	—— ——	—— ——	2.91 8.31
2	1.03～ 1.90	8.31 10.59	0.00 8.53	—— ——	8.31 19.11
3	1.90～ 2.90	10.59 12.92	8.53 18.33	—— ——	19.11 31.24
4	2.90～ 3.00	22.18 22.63	18.33 19.31	—— ——	40.51 41.94
5	3.00～ 3.30	22.63 24.00	19.31 22.28	48.00 53.46	-6.06 -7.18

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.00 m (GL -3.00 m)

3-2 Seismic Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.90	2.18 17.12	—— ——	—— ——	2.18 17.12
2	1.90～ 2.70	17.12 23.05	—— ——	—— ——	17.12 23.05
3	2.70～ 2.90	23.05 23.75	0.00 1.96	—— ——	23.05 25.71
4	2.90～ 3.00	18.46 19.58	1.96 2.94	—— ——	20.42 22.52
5	3.00～ 4.21	19.58 33.23	2.94 14.80	48.00 69.78	-25.48 -21.75

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.00 m (GL -3.00 m)

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to $1/\beta$ depth. The modules are calculated by the formula below;

$$K_h = 6910 \times N'^{0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side)　t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)
	I	=	20008 cm⁴(after reduction by corrosion and section)
Inertia sectional moment	EI	=	$200000 \times 10^3 \times 20008 \times 10^{-8} = 4.002 \times 10^4$

	Depth (m)	N-value		Depth (m)	N-value
1	2.90	3	11	12.90	9
2	3.90	3	12	13.90	10
3	4.90	3	13	14.90	13
4	5.90	4	14	15.90	13
5	6.90	6	15	16.90	12
6	7.90	9	16	17.90	19
7	8.90	12	17	18.90	22
8	9.90	6	18	19.90	21
9	10.90	10	19	20.90	23
10	11.90	12	20	21.90	28

4-2 Normal Condition

K_h = 10817 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$$= 0.510 \quad m^{-1}$$

$$L = \frac{1}{\beta} = 1.96 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.00 m) to 1.96 m depth (GL -4.96 m).

	Depth (m)	N-value
1	3.00	3.00
2	3.90	3.00
3	4.90	3.00
4	4.96	3.06
	Σh = 12.06	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{12.06}{4}$

$= 3.02$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 3.02^{0.406} = 10817\ kN/m^3$

Kh (normal condition) = 10817 kN/m³

4-3 Seismic Condition

K_h = 10817 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.510\ m^{-1}$

$$L = \frac{1}{\beta} = 1.96\ m$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.00 m) and 1.96 m depth (GL -4.96 m).

	Depth (m)	N-value
1	3.00	3.00
2	3.90	3.00
3	4.90	3.00
4	4.96	3.06
	Σh = 12.06	

Average N-value $N' = \frac{\Sigma A}{L}$

$= \frac{12.06}{4}$

$= 3.02$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 3.02^{0.406} = 10817\ kN/m^3$

Kh (seismic condition) = 10817 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.03	1.03	2.91 8.31	1.50 4.28	2.66 2.31	3.98 9.90
2	1.03～ 1.90	0.87	8.31 19.11	3.61 8.32	1.68 1.39	6.07 11.56
3	1.90～ 2.90	1.00	19.11 31.24	9.56 15.62	0.77 0.43	7.33 6.77
4	2.90～ 3.00	0.10	40.51 41.94	2.03 2.10	0.07 0.03	0.14 0.07
				ΣP = 47.01		ΣM = 45.82

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.00 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{45.82}{47.01} = 0.97\ m$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～ 1.90	1.90	2.18 17.12	2.07 16.26	2.37 1.73	4.91 28.18
2	1.90～ 2.70	0.80	17.12 23.05	6.85 9.22	0.83 0.57	5.71 5.23
3	2.70～ 2.90	0.20	23.05 25.71	2.31 2.57	0.23 0.17	0.54 0.43
4	2.90～ 3.00	0.10	20.42 22.52	1.02 1.13	0.07 0.03	0.07 0.04
				ΣP = 41.43		ΣM = 45.10

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.00 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{45.10}{41.43} = 1.09\ m$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x}\ (\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	10817 kN/m³
calculated value	β	=	0.50986 m⁻¹
resultant earth force (lateral)	P_0	=	47.01 kN/m
height of acting position of load	h_0	=	0.97 m
moment	M_0	=	45.82 kN·m/m

in consideration of ψ_m = 1.410,

maximum moment	M_{max}	=	64.60 kN·m/m
depth of generated position of M_{max}	l_m	=	0.912 m
depth of 1st fixed point	l_i	=	2.452 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	10817 kN/m³
calculated value	β	=	0.50986 m⁻¹
resultant earth force (lateral)	P_0	=	41.43 kN/m
height of acting position of load	h_0	=	1.09 m
moment	M_0	=	45.10 kN·m/m

in consideration of ψ_m = 1.351,

maximum moment	M_{max}	=	60.94 kN·m/m
depth of generated position of M_{max}	l_m	=	0.868 m
depth of 1st fixed point	l_i	=	2.408 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{64.60\times10^6}{1320\times10^3} = 49\ \mathrm{N/mm^2} \leqq \sigma_a = 180\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{60.94\times10^6}{1320\times10^3} = 46\ \mathrm{N/mm^2} \leqq \sigma_a = 270\ \mathrm{N/mm^2} \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.03	2.66 2.31	0.886 0.771	0.276 0.221	1.50 4.28	0.414 0.945
2	1.03～ 1.90	1.68 1.39	0.560 0.463	0.128 0.091	3.61 8.32	0.461 0.755
3	1.90～ 2.90	0.77 0.43	0.256 0.144	0.030 0.010	9.56 15.62	0.286 0.155
4	2.90～ 3.00	0.07 0.03	0.022 0.011	0.000 0.000	2.03 2.10	0.001 0.000
						ΣQ = 3.017

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

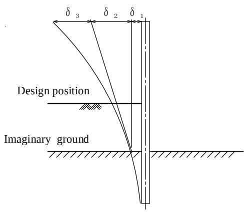
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5099\times0.97)\times47.01}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5099^3} = 0.00663\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5099\times0.97)\times47.01}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5099^2}\times(3.00+0.00) = 0.01352\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.02\times(3.00+0.00)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00204\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
= 0.00663＋0.01352＋0.00204
= 0.02218 m
= 22.18 ≦ δa = 50.00 mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.90	2.37 1.73	0.789 0.578	0.229 0.135	2.07 16.26	0.476 2.191
2	1.90～ 2.70	0.83 0.57	0.278 0.189	0.035 0.017	6.85 9.22	0.240 0.154
3	2.70～ 2.90	0.23 0.17	0.078 0.056	0.003 0.002	2.31 2.57	0.007 0.004
4	2.90～ 3.00	0.07 0.03	0.022 0.011	0.000 0.000	1.02 1.13	0.000 0.000
						ΣQ = 3.072

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

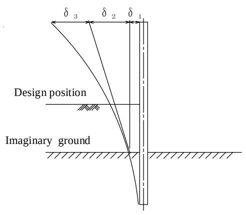
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.5099\times1.09)\times41.43}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5099^3} = 0.00607\ \mathrm{m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.5099\times1.09)\times41.43}{2\times2.00\times10^8\times20008\times10^{-8}\times0.5099^2}\times(3.00+0.00) = 0.01260\ \mathrm{m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.07\times(3.00+0.00)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00207\ \mathrm{m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
= 0.00607＋0.01260＋0.00207
= 0.02075 m
= 20.75 ≦ δ_a = 75.00 mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP（Chang）

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction	$K_h = 10817 kN/m^3$
Calculated value	$\beta = 0.48518 m^{-1}$
Penetration length of SSP	$D = 0.00 + \dfrac{3}{0.485} = 6.18m$
Whole length of SSP	$L = 3.00 - 0.40 + 6.18 = 8.78m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction	$K_h = 10817 kN/m^3$
Calculated value	$\beta = 0.48518 m^{-1}$
Penetration length of SSP	$D = 0.00 + \dfrac{3}{0.485} = 6.18m$
Whole length of SSP	$L = 3.00 - 0.40 + 6.18 = 8.78m$

Therefore, whole length of SSP is set as 8.80 m in consideration of round unit of SSP length.

7 Calculation Result

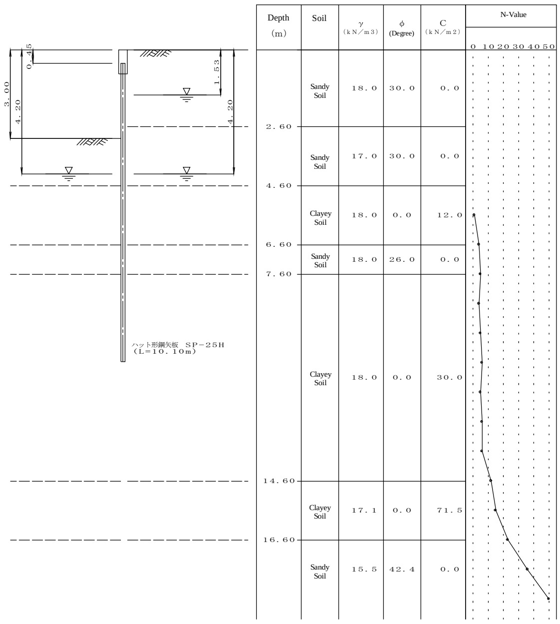
			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		64.60	60.94
Stress intensity	σ (N/mm^2)		49 (180)	46 (270)
Lateral displacement	δ (mm)		22.18 (50.0)	20.75 (75.0)
Penetration depth	D (m)		6.18	6.18
Whole length of SSP	L (m)	8.80		

－Steel Sheet Pile　Design Calculation－

Lower Marikina Section 4 + 500 J

1 Design Conditions

1-1 Longitudinal Section of SSP & Considered Geological Survey Log



1-2 Dimensions of Structure

Depth from coping top to riverbed	H	=	3.00 m
Depth from coping top to rear side ground	H_0	=	0.00 m
Depth from coping top to SSP top	H_k	=	0.40 m
Landside WL	L_{wa}	=	1.53 m (Normal Condition)
	L_{wa}'	=	4.20 m (Seismic Condition)
Riverside WL	L_{wp}	=	4.20 m (Normal Condition)
	L_{wp}'	=	4.20 m (Seismic Condition)

Imaginary riverbed　　calculated in consideration of geotechnical conditions

1-3 Applied Formula

Formula for generated stress　　Chang's formula

Penetration depths　　$L = \frac{3}{\beta}$

1-4 Constant Numbers for Design

Unit weight of water　　γ_w = 9.8 kN/m^3

Type of water pressure　　trapezoidal water pressure

Lateral pressure　　calculated in consideration of site conditions

Study case　　- Normal Condition
- Seismic Condition

Design earthquake intensity　　k = 0.100

Dynamic water pressure due to earthquake considered as distributed load

Wind load, Impact load　　not considered

Minimum angle of rupture　　ζ_0 = 10 degrees

Rear side angle of slope　　not considered

Angle of rupture (clayey soil)　　$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$

Equilibrium factor of compression　　K_c = 0.50 (considered in Seismic Condition)

1-5 Lateral Foundation Modulus

Applied formula　　$K_h = 6910 \times N'^{0.406}$

Average N-value　　calculated from average N-value between imaginary riverbed and depth as 1/β

N-value distribution

No	Depth (m)	N-Value
1	5.60	1
2	6.60	4
3	7.60	5
4	8.60	4
5	9.60	5
6	10.60	6
7	11.60	5
8	12.60	6
9	13.60	6
10	14.60	12

No	Depth (m)	N-Value
11	15.60	15
12	16.60	23
13	17.60	36
14	18.60	50
15	19.60	50
16	20.60	50
17	21.60	50

1-6 Vertical Load

Vertical load on landside　　calculated in consideration of embankment shape on landside

Vertical load on riverside　　not considered

1-7 Soil Modulus

No	Depth (m)	Soil	N-value	γ kN/m^3	γ' kN/m^3	ϕ	C kN/m^2	a	k'	ζ (degree)		kh(kN/m^3)	
										normal	seismic	normal	seismic
1	2.60	S	15.0	18.00	9.00	30.0	0.0	0.0	0.200	auto	auto	――	――
2	4.60	S	4.0	17.00	8.00	30.0	0.0	0.0	0.200	auto	auto	――	――
3	6.60	C	2.0	18.00	9.00	0.0	12.0	0.0	0.200	auto	auto	――	――
4	7.60	S	5.0	18.00	9.00	26.0	0.0	0.0	0.200	auto	auto	――	――
5	14.60	C	5.0	18.00	9.00	0.0	30.0	0.0	0.200	auto	auto	――	――
6	16.60	C	16.0	17.10	8.10	0.0	71.5	0.0	0.200	auto	auto	――	――
7	18.60	S	30.0	15.50	6.50	42.4	0.0	0.0	0.200	auto	auto	――	――

Note)
- depth : from top of coping to bottom of the layer
- soil : sandy(S), clayey(C), mixed (M)
- N-value : average N-value in the layer
- γ : wet unit weight of soil
- γ' : saturated unit weight of soil
- ϕ : internal friction angle of soil
- C_o : soil adhesion
- a : slope of soil adhesion
- k' : design seismic coefficient (underwater)
- ζ : angle of active rupture
- kh : modulus of subgrade reaction

Angle of wall friction

Angle of wall friction	Normal	Seismic
active passive	15.00° -15.00°	15.00° 0.00°

1-8 Steel Sheet Pile (SSP)

Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4
Sectional factor	Z_0	=	1610 cm^3

Corrosion margin　　t_1 = 1.00 mm (riverside)　　t_2 = 1.00 mm (landside)

Corrosion rate (to I_0)	η	=	0.82
Corrosion rate (to Z_0)	η	=	0.82
Section efficiency (to I_0)	μ	=	1.00
Section efficiency (to Z_0)	μ	=	1.00

Round unit of SSP length　　0.10 m

Allowable stress　　σ_a = 180 N/mm（Normal）
σ_a' = 270 N/mm（Seismic）

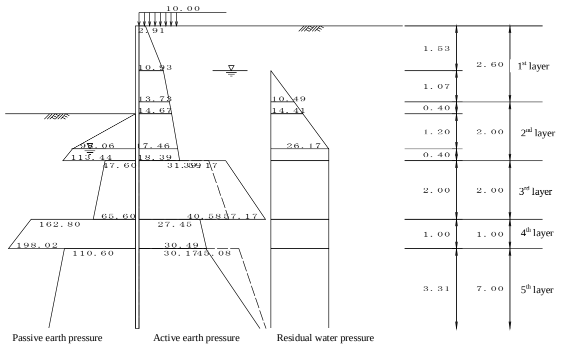
Allowable displacement　　δ_a = 50.0 mm（Normal）
δ_a' = 75.0 mm（Seismic）

Bending of cantilever beam　　calculated as distributed load of each layer

Reduction of material modulus　　Reduced: I_0 applied to calculation of lateral coefficient of subgrade reaction
Not reduced: I_0 applied to calculation of penetration depth
Reduced: I_0 applied to calculation of section forces and displacement
Reduced: Z_0 applied to calculation of stresses

2 Lateral Pressure

2-1 Normal Condition



2-1-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m^3)	ϕ (degree)	C kN/m^2	Σγh+Qa (kN/m^2)	Ka	Ka ×cosδ
1	0.00～ 1.53	Sandy soil	18.0	30.0	—— ——	10.000 37.540	0.30142 0.30142	0.29115 0.29115
2	1.53～ 2.60	Sandy soil	9.0	30.0	—— ——	37.540 47.170	0.30142 0.30142	0.29115 0.29115
3	2.60～ 3.00	Sandy soil	8.0	30.0	—— ——	47.170 50.370	0.30142 0.30142	0.29115 0.29115
4	3.00～ 4.20	Sandy soil	8.0	30.0	—— ——	50.370 59.970	0.30142 0.30142	0.29115 0.29115
5	4.20～ 4.60	Sandy soil	8.0	30.0	—— ——	59.970 63.170	0.30142 0.30142	0.29115 0.29115
6	4.60～ 6.60	Clayey soil	9.0	—	12.0 12.0	63.170 81.170	—— ——	—— ——
7	6.60～ 7.60	Sandy soil	9.0	26.0	—— ——	81.170 90.170	0.35007 0.35007	0.33814 0.33814
8	7.60～ 10.91	Clayey soil	9.0	—	30.0 30.0	90.170 120.000	—— ——	—— ——
9	10.91～ 14.60	Clayey soil	9.0	—	30.0 30.0	120.000 153.170	—— ——	—— ——
10	14.60～ 16.60	Clayey soil	8.1	—	71.5 71.5	153.170 169.370	—— ——	—— ——
11	16.60～ 18.60	Sandy soil	6.5	42.4	—— ——	169.370 182.370	0.18084 0.18084	0.17468 0.17468

Case: Lower Marikina Section 4 + 500 J

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ= 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m^3)	ϕ (degree)	C kN/m^2	Σγh+Qp (kN/m^2)	Kp	Kp ×cosδ
4	3.00～ 4.20	Sandy soil	17.0	30.0	—— ——	0.000 20.400	4.97650 4.97650	4.80693 4.80693
5	4.20～ 4.60	Sandy soil	8.0	30.0	—— ——	20.400 23.600	4.97650 4.97650	4.80693 4.80693
6	4.60～ 6.60	Clayey soil	9.0	0.0	12.0 12.0	23.600 41.600	—— ——	—— ——
7	6.60～ 7.60	Sandy soil	9.0	26.0	—— ——	41.600 50.600	4.05145 4.05145	3.91340 3.91340
8	7.60～ 10.91	Clayey soil	9.0	0.0	30.0 30.0	50.600 80.430	—— ——	—— ——
9	10.91～ 14.60	Clayey soil	9.0	0.0	30.0 30.0	80.430 113.600	—— ——	—— ——
10	14.60～ 16.60	Clayey soil	8.1	0.0	71.5 71.5	113.600 129.800	—— ——	—— ——
11	16.60～ 18.60	Sandy soil	6.5	42.4	—— ——	129.800 142.800	10.38845 10.38845	10.03447 10.03447

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ= -15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral Pressure

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m^2	Pa2 kN/m^2	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2
1	0.00～ 1.53	2.91 10.93	—— ——	2.91 10.93	—— ——	—— ——
2	1.53～ 2.60	10.93 13.73	—— ——	10.93 13.73	0.00 10.49	—— ——
3	2.60～ 3.00	13.73 14.67	—— ——	13.73 14.67	10.49 14.41	—— ——
4	3.00～ 4.20	14.67 17.46	—— ——	14.67 17.46	14.41 26.17	0.00 98.06
5	4.20～ 4.60	17.46 18.39	—— ——	17.46 18.39	26.17 26.17	98.06 113.44
6	4.60～ 6.60	39.17 57.17	31.59 40.58	39.17 57.17	26.17 26.17	47.60 65.60
7	6.60～ 7.60	27.45 30.49	—— ——	27.45 30.49	26.17 26.17	162.80 198.02
8	7.60～ 10.91	30.17 60.00	45.08 60.00	45.08 60.00	26.17 26.17	110.60 140.43
9	10.91～ 14.60	60.00 93.17	60.00 76.58	60.00 93.17	26.17 26.17	140.43 173.60

Case: Lower Marikina Section 4 + 500 J

	Depth (m)	Active side			Residual water pressure	Passive side
		Pa1 kN/m^2	Pa2 kN/m^2	Pa kN/m^2	Pw kN/m^2	Pp kN/m^2
10	14.60～ 16.60	10.17 26.37	76.58 84.68	76.58 84.68	26.17 26.17	256.60 272.80
11	16.60～ 18.60	29.58 31.86	—— ——	29.58 31.86	26.17 26.17	1302.47 1432.92

- Formula for active earth pressure

Sandy soil $P_{a1} = K_a \cdot \left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right] \cdot \cos\delta$

Clayey soil $P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c \cdot (\Sigma\gamma h + Q)$

K_c : Equilibrium coefficient of compression: 0.5

Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil $P_{a1} = \left[K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}\right] \cdot \cos\delta$

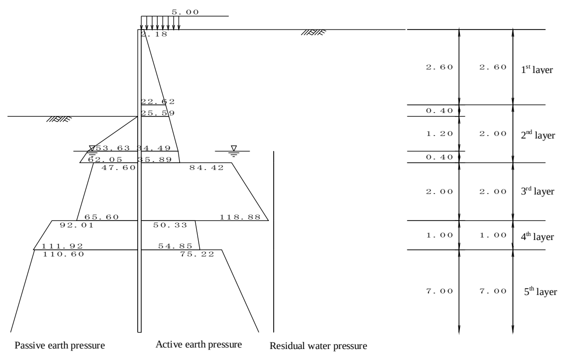
- Formula for passive earth pressure

Sandy soil $P_p = K_p \cdot \left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right] \cdot \cos\delta$

Clayey soil $P_p = \Sigma\gamma h + Q + 2C$

Mixed soil $P_p = \left[K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}\right] \cdot \cos\delta$

2-2 Seismic Condition



Case: Lower Marikina Section 4 + 500 J

2-2-1 Soil Modulus of Active Side

	Depth (m)	Soil	γ (kN/m^3)	ϕ (degree)	C kN/m^2	Σγh+Q (kN/m^2)	γwhw (kN/m^2)	k (k')	θ (degree)	Ka	Ka ×cosδ	θ (degree)
1	0.00～ 2.60	Sandy Soil	18.0	30.0	—— ——	5.00 51.80	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
2	2.60～ 3.00	Sandy Soil	17.0	30.0	—— ——	51.80 58.60	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
3	3.00～ 4.20	Sandy Soil	17.0	30.0	—— ——	58.60 79.00	0.00 0.00	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
4	4.20～ 4.60	Sandy Soil	8.0	30.0	—— ——	79.00 82.20	0.00 3.92	0.200 0.200	11.31 11.31	0.45203 0.45203	0.43663 0.43663	—— ——
5	4.60～ 6.60	Clayey Soil	9.0	—	12.0 12.0	82.20 100.20	3.92 23.52	0.200 0.200	11.31 11.31	—— ——	—— ——	27.60 19.35
6	6.60～ 7.60	Sandy Soil	9.0	26.0	—— ——	100.20 109.20	23.52 33.32	0.200 0.200	11.31 11.31	0.51997 0.51997	0.50225 0.50225	—— ——
7	7.60～ 14.60	Clayey Soil	9.0	—	30.0 30.0	109.20 172.20	33.32 101.92	0.200 0.200	11.31 11.31	—— ——	—— ——	38.20 32.61
8	14.60～ 16.60	Clayey Soil	8.1	—	71.5 71.5	172.20 188.40	101.92 121.52	0.200 0.200	11.31 11.31	—— ——	—— ——	40.93 40.50
9	16.60～ 18.60	Sandy Soil	6.5	42.4	—— ——	188.40 201.40	121.52 141.12	0.200 0.200	11.31 11.31	0.28983 0.28983	0.27995 0.27995	—— ——

Coefficient of active earth pressure of sandy soil Ka is calculated by the formula below;
δ= 15.00、β = 0.00、θ = 0.00

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos\theta \cdot \cos(\delta+\theta) \cdot \left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\delta+\theta)\cdot\cos(-\beta)}}\right]^2}$$

Angle between surface of collapse and level surface of clayey soil ζ is calculated by the formula below;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Soil Modulus of Passive Side

	Depth (m)	Soil	γ (kN/m^3)	ϕ (degree)	C kN/m^2	Σγh+Qp (kN/m^2)	γwhw (kN/m^2)	k (k')	θ (degree)	Kp	Kp ×cosδ
3	3.00～ 4.20	Sandy soil	17.00	30.0	—— ——	0.000 20.400	0.00 0.00	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
4	4.20～ 4.60	Sandy soil	8.00	30.0	—— ——	20.400 23.600	0.00 3.92	0.200 0.200	11.31 11.31	2.62913 2.62913	2.62913 2.62913
5	4.60～ 6.60	Clayey Soil	9.00	0.0	12.0 12.0	23.600 41.600	3.92 23.52	0.200 0.200	11.31 11.31	—— ——	—— ——
6	6.60～ 7.60	Sandy soil	9.00	26.0	—— ——	41.600 50.600	23.52 33.32	0.200 0.200	11.31 11.31	2.21182 2.21182	2.21182 2.21182
7	7.60～ 14.60	Clayey Soil	9.00	0.0	30.0 30.0	50.600 113.600	33.32 101.92	0.200 0.200	11.31 11.31	—— ——	—— ——
8	14.60～ 16.60	Clayey Soil	8.10	0.0	71.5 71.5	113.600 129.800	101.92 121.52	0.200 0.200	11.31 11.31	—— ——	—— ——
9	16.60～ 18.60	Sandy soil	6.50	42.4	—— ——	129.800 142.800	121.52 141.12	0.200 0.200	11.31 11.31	4.67049 4.67049	4.67049 4.67049

Coefficient of passive earth pressure of sandy soil Kp is calculated by the formula below;
$\delta = -0.00$、$\beta = 0.00$、$\theta = \tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral Pressure

No	Depth (m)	Active side		
		Pa1 kN/m²	Pa2 kN/m²	Pa kN/m²
1	0.00～ 2.60	2.18 22.62	—— ——	—— ——
2	2.60～ 3.00	22.62 25.59	—— ——	—— ——
3	3.00～ 4.20	25.59 34.49	—— ——	0.00 53.63
4	4.20～ 4.60	34.49 35.89	0.00 0.00	53.63 62.05
5	4.60～ 6.60	84.42 118.88	0.00 0.00	47.60 65.60
6	6.60～ 7.60	50.33 54.85	0.00 0.00	92.01 111.92
7	7.60～ 14.60	75.22 159.95	0.00 0.00	110.60 173.60
8	14.60～ 16.60	67.46 87.73	[illegible]	[illegible]
9	16.60～ 18.60	52.74 56.38	0.00 0.00	606.23 666.95

Passive earth pressure　Active earth pressure　Residual water pressure　Total of lateral pressure

- Formula for active earth pressure

Sandy soil　$P_{a1} = K_a\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Clayey soil　$P_{a1} = \Sigma\gamma h + Q - 2C$
$P_{a2} = K_c\cdot(\Sigma\gamma h + Q)$
K_c : Equilibrium coefficient of compression: 0.5
Larger of Pa1 or Pa2 is applied as active earth pressure (Pa)

Mixed soil　$P_{a1} = [K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}]\cdot\cos\delta$

- Formula for passive earth pressure

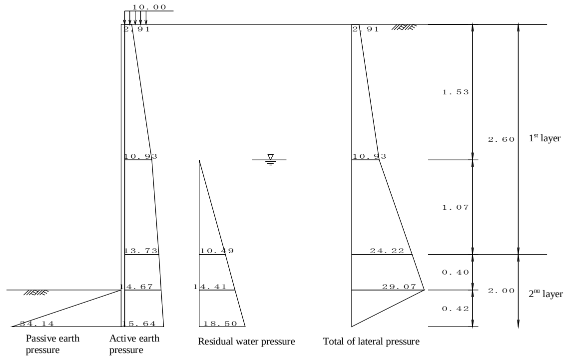
Sandy soil　$P_p = K_p\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$
Clayey soil　$P_p = \Sigma\gamma h + Q + 2C$
Mixed soil　$P_p = [K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}]\cdot\cos\delta$

Case: Lower Marikina Section 4 + 500 J

3 Imaginary Riverbed

Imaginary ground level L_k is calculated as the elevation level that the sum of active earth pressure and residual water pressure are balanced with passive earth pressure.

3-1 Normal Condition



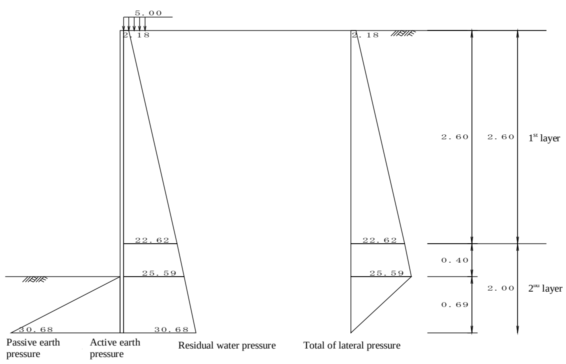
	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 1.53	2.91 10.93	—— ——	—— ——	2.91 10.93
2	1.53～ 2.60	10.93 13.73	0.00 10.49	—— ——	10.93 24.22
3	2.60～ 3.00	13.73 14.67	10.49 14.41	—— ——	24.22 29.07
4	3.00～ 3.42	14.67 15.64	14.41 18.50	0.00 34.14	29.07 0.00
5	3.42～ 4.20	15.64 17.46	18.50 26.17	34.14 98.06	0.00 -54.44

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.42 m (GL -3.42 m)

Case: Lower Marikina Section 4 + 500 J

3-2 Seismic Condition



	Depth (m)	Pa kN/m²	Pw kN/m²	Pp kN/m²	Ps kN/m²
1	0.00～ 2.60	2.18 22.62	—— ——	—— ——	2.18 22.62
2	2.60～ 3.00	22.62 25.59	—— ——	—— ——	22.62 25.59
3	3.00～ 3.69	25.59 30.68	—— ——	0.00 30.68	25.59 0.00
4	3.69～ 4.20	30.68 34.49	—— ——	30.68 53.63	0.00 -19.14

P_a : Active earth pressure
P_w : Residual water pressure
P_p : Passive earth pressure
P_s : Lateral pressure $P_s = P_a + P_w - P_p$

Imaginary riverbed L_k: 0.69 m (GL -3.69 m)

Case: Lower Marikina Section 4 + 500 J

4 Modulus of Lateral Subgrade Reaction

4-1 Formula for Modulus of Lateral Subgrade Reaction

Modulus of lateral subgrade reaction is calculated on the average N-value from imaginary riverbed to 1/β depth. The modules are calculated by the formula below;

$$K_h = 6910\times N'^{\,0.406}$$

where,

$$\beta = \sqrt[4]{\frac{Kh\cdot B}{4EI}}$$

Unit width	B	=	1.0000 m		
Corrosion margin	t_1	=	1.00 mm (active side)	t_2 =	1.00 mm (passive side)
Corrosion rate	η	=	0.82		
Section efficiency	μ	=	1.00		
Young's modulus	E	=	200000 N/mm²		
Inertia sectional moment	I_0	=	24400 cm⁴(original condition)		
	I	=	20008 cm⁴(after reduction by corrosion and section)		
Inertia sectional moment	EI	=	$200000\times10^3\times20008\times10^{-8} = 4.002\times10^4$		

	Depth (m)	N-value
1	5.60	1
2	6.60	4
3	7.60	5
4	8.60	4
5	9.60	5
6	10.60	6
7	11.60	5
8	12.60	6
9	13.60	6
10	14.60	12

	Depth (m)	N-value
11	15.60	15
12	16.60	23
13	17.60	36
14	18.60	50
15	19.60	50
16	20.60	50
17	21.60	50

4-2 Normal Condition

K_h = 6937 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh\cdot B}{4EI}}$$

$$= 0.456\ \ m^{-1}$$

$$L = \frac{1}{\beta} = 2.19\ m$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.42 m) to 2.19 m depth (GL -5.61 m).

	Depth (m)	N-value
1	3.42	1.00
2	5.60	1.00
3	5.61	1.03
	Σh = 3.03	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{3.03}{3}$

$= 1.01$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 1.01^{0.406} = 6937$ kN/m³

Kh (normal condition) = 6937 kN/m³

4-3 Seismic Condition

K_h = 7525 kN/m³ is set tentatively.

$$\beta = \sqrt[4]{\frac{Kh \cdot B}{4EI}}$$

$= 0.466$ m⁻¹

$$L = \frac{1}{\beta} = 2.15 \text{ m}$$

Therefore, average N-value is calculated on the actual N-value from imaginary riverbed (GL -3.69 m) and 2.15 m depth (GL -5.83 m).

	Depth (m)	N-value
1	3.69	1.00
2	5.60	1.00
3	5.83	1.70
	Σh = 3.70	

Average N-value $N' = \dfrac{\Sigma A}{L}$

$= \dfrac{3.70}{3}$

$= 1.23$

Calculated Kh is equal to tentative one, so modulus of lateral subgrade reaction (normal condition) is set definitely as following:

$K_h = 6910 \times N'^{0.406} = 6910 \times 1.23^{0.406} = 7525$ kN/m³

Kh (seismic condition) = 7525 kN/m³

5 Sectional Forces and Displacement

Chang's formula is applied to calculate stress, displacement and penetration depth of SSP.

5-1 Calculation of Resultant Lateral Force P_0 & Acting Elevation h_0

5-1-1 Normal Condition

	Depth Z (m)	Thickness h (m)	Total of lateral force Ps (kN/m²)	Load P (kN)	Arm length Y (m)	Moment M (kN·m)
1	0.00～1.53	1.53	2.91 10.93	2.23 8.36	2.91 2.40	6.48 20.05
2	1.53～2.60	1.07	10.93 24.22	5.85 12.96	1.53 1.17	8.95 15.22
3	2.60～3.00	0.40	24.22 29.07	4.84 5.81	0.68 0.55	3.32 3.20
4	3.00～3.42	0.42	29.07 0.00	6.07 0.00	0.28 0.14	1.69 0.00
				ΣP = 46.12		ΣM = 58.91

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.42 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{58.91}{46.12} = 1.28 \text{ m}$$

5-1-2 Seismic Condition

	Depth Z (m)	Thickness h (m)	Lateral load Ps (kN/m²)	Load P kN	Arm length Y (m)	Moment M (kN·m)
1	0.00～2.60	2.60	2.18 22.62	2.84 29.40	2.82 1.95	8.00 57.43
2	2.60～3.00	0.40	22.62 25.59	4.52 5.12	0.95 0.82	4.31 4.20
3	3.00～3.69	0.69	25.59 0.00	8.78 0.00	0.46 0.23	4.02 0.00
				ΣP = 50.66		ΣM = 77.96

P_s : active earth pressure + residual water pressure - passive earth pressure
P : load P_s x h/2 x B
B : unit width = 1.000 m
Y : height of acting position from imaginary riverbed
M : moment by load P x Y

Arbitrary load lateral load	P_t	=	0.0 kN/m
depth to acting position	H_t	=	0.00 m
moment	M_m	=	0.0 kN·m/m
depth to acting position	H_m	=	0.00 m
Height from riverbed to top of coping	H	=	3.00 m
Depth of Imaginary riverbed from riverbed	L_k	=	0.69 m

Moment M_t by arbitrary load is as below

$M_t = P_t \cdot (H + L_k - H_t) + M_m = 00.00$ kN·m

h_0, Height of acting position of P_0 from imaginary riverbed

$$h_0 = \frac{M_0}{P_0} = \frac{\Sigma M + M_t}{\Sigma P + P_t}$$

$$= \frac{77.96}{50.66} = 1.54 \text{ m}$$

5-2 Sectional Force

Corrosion rate and section efficiency for calculation of sectional forces and displacements are set as followings:

Unit width	B	=	1.0000 m
Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm²
Inertia sectional moment	I_0	=	24400 cm⁴ (original condition)
	I	=	20008 cm⁴ (after reduction by corrosion and section)
$EI = 200000 \times 10^3 \times 20008 \times 10^{-8}$		=	4.002×10^4

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$\phi_m = \frac{\sqrt{(1+2\beta h_0)^2+1}}{2\beta h_0} \times \exp\left(-\tan^{-1}\frac{1}{1+2\beta h_0}\right)$$

$$M_{max} = M_0 \cdot \phi_m$$

$$l_m = \frac{1}{\beta} \times \tan^{-1}\frac{1}{1+2\beta h_0}$$

$$l_i = \frac{1}{\beta} \times \tan^{-1}\frac{1+\beta h_0}{\beta h_0}$$

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x}\ (\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x)$$

5-2-1 Normal Condition

modulus of lateral subgrade reaction	K_h	=	6937 kN/m³
calculated value	β	=	0.45626 m⁻¹
resultant earth force (lateral)	P_0	=	46.12 kN/m
height of acting position of load	h_0	=	1.28 m
moment	M_0	=	58.91 kN·m/m

in consideration of ψ_m = 1.328

maximum moment	M_{max}	=	78.22 kN·m/m
depth of generated position of M_{max}	l_m	=	0.948 m
depth of 1st fixed point	l_i	=	2.670 m

5-2-2 Seismic Condition

modulus of lateral subgrade reaction	K_h	=	7525 kN/m³
calculated value	β	=	0.46565 m⁻¹
resultant earth force (lateral)	P_0	=	50.66 kN/m
height of acting position of load	h_0	=	1.54 m
moment	M_0	=	77.96 kN·m/m

in consideration of ψ_m = 1.243,

maximum moment	M_{max}	=	96.89 kN·m/m
depth of generated position of M_{max}	l_m	=	0.837 m
depth of 1st fixed point	l_i	=	2.524 m

5-3 Stress Intensity

Corrosion rate and section efficiency for check of stresses intensity are set as followings:

Corrosion margin	t_1	=	1.00 mm (active side) t_2 = 1.00 mm (passive side)
Corrosion rate	η	=	0.82
Section efficiency	μ	=	1.00
Module of section	Z_0	=	1610 cm³ (original condition)
	Z	=	1320 cm³ (after reduction by corrosion and section)

5-3-1 Normal Condition

$$\sigma = \frac{M_{max}}{Z} = \frac{78.22\times10^{6}}{1320\times10^{3}} = 59 \text{ N/mm}^2 \leqq \sigma_a = 180 \text{ N/mm}^2 \quad \text{(ok)}$$

5-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{96.89\times10^{6}}{1320\times10^{3}} = 73 \text{ N/mm}^2 \leqq \sigma_a = 270 \text{ N/mm}^2 \quad \text{(ok)}$$

5-4 Displacement

5-4-1 Normal Condition

Modules of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 1.53	2.91 2.40	0.851 0.702	0.259 0.189	2.23 8.36	0.577 1.576
2	1.53～ 2.60	1.53 1.17	0.448 0.344	0.085 0.052	5.85 12.96	0.499 0.677
3	2.60～ 3.00	0.68 0.55	0.200 0.161	0.019 0.012	4.84 5.81	0.091 0.072
4	3.00～ 3.42	0.28 0.14	0.081 0.041	0.003 0.001	6.07 0.00	0.020 0.000
						ΣQ = 3.512

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

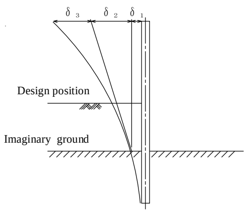
Q : $\zeta \times P$

P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Displacement



Case: Lower Marikina Section 4 + 500 J

$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4563\times1.28)\times46.12}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4563^3} = 0.00960 \text{ m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4563\times1.28)\times46.12}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4563^2}\times(3.00+0.42) = 0.02049 \text{ m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{3.51\times(3.00+0.42)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00350 \text{ m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.00960 + 0.02049 + 0.00350$
$= 0.03360$ m
$= 33.60 \leqq \delta a = 50.00$ mm (ok)

Where,
Displacement at imaginary ground
Displacement by angle of inclination slope at imaginary ground
Displacement at higher part of imaginary ground as cantilever
Displacement at top of SSP
Allowable displacement

5-4-2 Seismic Condition

Modulus of deformation

	Depth (m)	Y (m)	α	ζ	P (kN)	Q (kN)
1	0.00～ 2.60	2.82 1.95	0.765 0.530	0.218 0.116	2.84 29.40	0.619 3.398
2	2.60～ 3.00	0.95 0.82	0.259 0.222	0.031 0.023	4.52 5.12	0.138 0.117
3	3.00～ 3.69	0.46 0.23	0.124 0.062	0.007 0.002	8.78 0.00	0.065 0.000
						ΣQ = 4.337

Y : Height from imaginary riverbed to acting position

α : $\alpha = \dfrac{Y}{H+L_k}$

ζ : $\zeta = \dfrac{(3-\alpha)\times\alpha^2}{6}$

Q : $\zeta \times P$

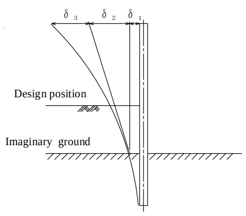
P : Lateral force

H : Depth to design position

L_k : Depth from design position to imaginary ground

Case: Lower Marikina Section 4 + 500 J

Displacement



$$\delta_1 = \frac{(1+\beta h_0)\times P_0}{2EI\beta^3}$$

$$= \frac{(1+0.4657\times1.54)\times50.66}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4657^3} = 0.01076 \text{ m}$$

$$\delta_2 = \frac{(1+2\beta h_0)\times P_0}{2EI\beta^2}\times(H+L_k)$$

$$= \frac{(1+2\times0.4657\times1.54)\times50.66}{2\times2.00\times10^8\times20008\times10^{-8}\times0.4657^2}\times(3.00+0.69) = 0.02619 \text{ m}$$

$$\delta_3 = \frac{Q\times(H+L_k)^3}{EI}$$

$$= \frac{4.34\times(3.00+0.69)^3}{2.00\times10^8\times20008\times10^{-8}} = 0.00543 \text{ m}$$

Additional displacement δ_3' generated by horizontal load (P) and moment (M) acting at top of SSP is considered.

$\delta = \delta_1 + \delta_2 + \delta_3$
$= 0.01076 + 0.02619 + 0.00543$
$= 0.04238$ m
$= 42.38 \leqq \delta_a = 75.00$ mm (ok)

Where,
δ_1 : Displacement at imaginary ground
δ_2 : Displacement by angle of inclination slope at imaginary ground
δ_3 : Displacement at higher part of imaginary ground as cantilever
δ : Displacement at top of SSP
δ_a : Allowable displacement

Case: Lower Marikina Section 4 + 500 J

6 Penetration Depth

Corrosion rate and section efficiency for calculation of penetration depth of SSP are as below:

Unit width	B	=	1.0000 m
Corrosion rate	η	=	1.00
Section efficiency	μ	=	1.00
Young's modulus	E	=	200000 N/mm^2
Inertia sectional moment	I_0	=	24400 cm^4 (original condition)
	I	=	24400 cm^4 (after reduction by corrosion and section)
EI = 200000 × 10^3 × 24400 × 10^{-8}		=	4.880 × 10^4

6-1 Penetration Depth and Whole Length of SSP (Chang)

Based on the depth of imaginary riverbed as L_k, penetration depth of SSP (D) and whole length of SSP (L) are calculated as followings:

$$D = L_k + \frac{3}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

6-1-1 Normal Condition

Modules of lateral subgrade reaction: $K_h = 6937 kN/m^3$
Calculated value: $\beta = 0.43418 m^{-1}$

Penetration length of SSP: $D = 0.42 + \dfrac{3}{0.434} = 7.33m$

Whole length of SSP: $L = 3.00 - 0.40 + 7.33 = 9.93m$

6-1-2 Seismic Condition

Modules of lateral subgrade reaction: $K_h = 7525 kN/m^3$
Calculated value: $\beta = 0.44311 m^{-1}$

Penetration length of SSP: $D = 0.69 + \dfrac{3}{0.443} = 7.46m$

Whole length of SSP: $L = 3.00 - 0.40 + 7.46 = 10.06m$

Therefore, whole length of SSP is set as 10.10 m in consideration of round unit of SSP length.

7 Calculation Result

			Normal condition	Seismic condition
Inertia sectional moment	I (cm^4)	24400		
Section modulus	Z (cm^3)	1610		
Maximum bending moment	M_{max} (kN·m/m)		78.22	96.89
Stress intensity	σ (N/mm^2)		59 (180)	73 (270)
Lateral displacement	δ (mm)		33.60 (50.0)	42.38 (75.0)
Penetration depth	D (m)		7.33	7.46
Whole length of SSP	L (m)	10.10		

CHAPTER 5 CALCULATION OF CONSOLIDATION SETTLEMENT AT BACKFILL SITE

The detailed calculation of consolidation settlement at backfill site is indicated from the following page.

Calculation of Immediate of Settlement in Backfill Site

1. Design Condition

1-1 Material of Embankment

Reclamation of the ground shall be done with improved dredging soil with r=18.0 kN/m^3.

1-2 Design Condition

(1) Embankment Height

Project area: East side 30.16 ha
West side 16.46 ha
Total 46.62 ha

Reclamation area can be obtained by multiplying 0.95 by the total area

A = 46.62 ha * 0.95 = 44.3 ha

Backfill soil

V = 880,000 m^3

Embankment Height

H = V / A = 880,000 / 443.000 = 1.99 m = 2.00 m

Average height at Project site

$H_{average}$ = 11.75m, H_{lower} = 11.50m, Difference = 11.75 – 11.50 = 0.25m

Embankment height in calculation = H + 0.25m = 2.00 + 0.25 = 2.25 m

(2) Ground Condition

Average load on ground: 18.0 * 2.25 = 40.50 kN

Soil: silty clay with fine sand (CL)

Average N-value: 3

Liquid Limit LL: 70 (Clay: 50 ~ 130: from the book " Soil Test, Basic and Guide, The Japanese Geotechnical Society")

Complex Index Cc = 0.009 (LL – 10) = 0.009 (70 – 10) = 0.54

Void Ratio: 1.40

Plastic Limit PL: 50

Layers of ground: Correct layer's condition is unknown. (Estimate 12m to 8m)

Silty Clay: (Consolidation settlement depth) 12.0m, 10m, 8m

Ground Water: at bottom of foundation

2. Calculation of Settlement

2.1 Immediate Settlement

Deformation of soil: $E = 700 * N = 700 * 3 = 2{,}100\ kN/m^3$

$Se = Is * (1 - \nu 2)*qB / E$

Where,

Se: immediate settlement (m)

Is : factor depending on shape and rigidity of foundation (=1.12, wide area)

ν : Poison's ratio (=0.30)

qB: surcharge (kN/m^2)

$Se = 1.12 * (1 - 0.30^2) * 40.50 / 2{,}100$

$= 0.020$ m

2.3 Consolidation Settlement

p_0: natural ground pressure

$= (17.0 - 9.8) *6.00 = 43.2\ kN/m^2$ (12m)

$= (17.0 - 9.8) *5.00 = 36.0\ kN/m^2$ (10m)

$= (17.0 - 9.8) *4.00 = 28.8\ kN/m^2$ (8m)

Δp: increased load = $40.50\ kN/m^2$

Therefore, consolidation settlement is obtained as follows.

$s = Cc / (1+e) * \log ((po+\Delta p)/po) * H$

$= 0.54 / (1+1.40)*\log ((43.2+40.5)/43.2)*12.0 = 0.776$ m (12m)

$= 0.54 / (1+1.40)*\log ((36.0+40.5)/36.0)*10.0 = 0.737$ m (10m)

$= 0.54 / (1+1.40)*\log ((28.8+40.5)/28.8)* 8.0 = 0.686$ m (8m)

2.4 Residual Settlement

(Residual Settlement) = (Immediate Settlement) + (Consolidation Settlement)

= 0.020 + 0.776 = 0.796 m (12m)

= 0.020 + 0.737 = 0.757 m (10m)

= 0.020 + 0.686 = 0.706 m (8m)

2.5 Settlement Time

$t = T_V * d^2 / Cv$

Where, t : settlement time (day)

Tv: degree of consolidation, In case of U = 90%, T v = 0.848

d : discharge distance = H/2 = 12.0 / 2 = 6.00 m (12m)

= 10.0 / 2 = 5.00 m (10m)

= 8.0 / 2 = 4.00 m (8m)

Cv: coefficient of consolidation, Cv = 0.0150 m^2/day

(In case of Lower Marikina River, Cv = 0.0150 m^2/day at STA 3 +450)

$t = 0.848 * 6.00^2 / 0.0150 = 2035$ days (5.6 years: 12m)

$t = 0.848 * 5.00^2 / 0.0150 = 1413$ days (3.9 years: 10m)

$t = 0.848 * 4.00^2 / 0.0150 = 905$ days (2.5 years: 8m)

CHAPTER 6 CALCULATION OF DRAINAGE FACILITIES AT BACKFILL SITE

The detailed calculation of drainage facilities at backfill site is indicated from the following page.

Calculation of Laguna RC Pipe

1. Design Condition

(1) Catchment Area (one sand basin takes a half area of the project site)
Catchment area in project site is assumed to be 95% of total project area.

Case 1 East side (Permanent)

Catchment area A1= 30.16 * 0.95 = 28.65 ha

Developed area back to grass field

Case 2 West side (Permanent)

Catchment area A2= 16.46 * 0.95 = 15.64 ha

Developed area back to grass field

Case 3 Outer channels on East and west side (Permanent)

Catchment area A3= 100m*700m = 70,000m2 = 7.00 ha

Area of developed area and grass field

(2) Average Runoff Coefficient

Case1 and Case 2

A runoff coefficient of C=0.60 for the area after construction is to be selected because the project site changes to a cultivated area, and grass fields can be seen everywhere.

Case 3

A runoff coefficient of C=0.70 for the area outside of project site because flood water comes from both sides of urban area and grass field.

	Infiltration: small	Infiltration: medium	Infiltration: large
Forest Land	0.6~0.7	0.5~0.6	0.3~0.5
Grass Field	0.7~0.8	0.6~0.7	0.4~0.6
Cultivated Area		0.7~0.8	0.5~0.7
Bare Area	1.0	0.9~1.0	0.8~0.9

2. Design Rainfall Discharge

(1) Design Rainfall (Q_R)

Bricks experimental formula is to be selected for very flat area.

Q_R: Design Rainfall Discharge (m^3/sec)

$Q_R = R \cdot C \cdot A \cdot \sqrt[6]{(S/A)}$

t: time of concentration (60minutes is selected because of flat area)

Kerby equation

$T = (2/3 \cdot 3.28 \cdot L \cdot n / \sqrt{S})^{0.467}$

Where, L: slope Distance (=700m)

n: lag coefficient (= 0.20, rough grass field, cultivated area and town)

S: slope gradient (=1/1000 in case 1,2, 1.3 /700 in case of 3))

Case 1,2

$T = (2/3 \cdot 3.28 \cdot 700 \cdot 0.20 / \sqrt{1/1000})^{0.467}$

$= 73$ minutes

Case 3

$T = (2/3 \cdot 3.28 \cdot 700 \cdot 0.20 / \sqrt{1.3 / 700})^{0.467}$

$= 63$ minutes

Therefore, 60 minutes is adopted for all cases for obtaining safety more against unknown factors.

R : Rainfall Intensity in 10 years

$r = 1474.2 / (t^{0.65} + 4.02) = 1474.2 / (60^{0.65} + 4.02) = 80.4$ mm/hr

R= (80.4 mm/hr) = 0.223 (m^3/sec・ha)

S: Grand Gradient (S/1000=1/1000)

However, a calculation item of $\sqrt[6]{(S/A)}$ is neglected in case 3 because gradient of land is not sure and Q_R should follow the calculation as sewage system.

(2)

Case 1 East side

C: Runoff Coefficient=0.70

A: Catchment Area = 7.00 ha

$Q_R = R \cdot C \cdot A$

$= 0.223 * 0.60 * 28.66 * \sqrt[6]{(1 / 28.66)}$

$= 2.19\ m^3/S$

Case 2 West side

C: Runoff Coefficient=0.60

A: Catchment Area = 15.64 ha

Q_R=R・C・A

$=0.223*0.60*15.64*\sqrt[6]{(1 / 15.64)}$

$=1.32\ m^3/S$

Case 3 Outer channel

C: Runoff Coefficient=0.70

A: Catchment Area = 7.00 ha

Q_R=R・C・A

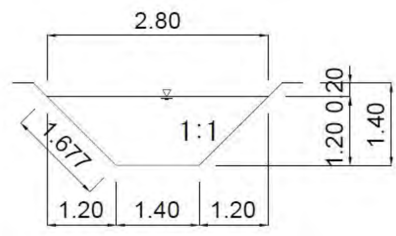
$=0.223*0.70*7.00$

$=1.09\ m^3/S$

3. Calculation of Ditches

(1) East Side

$Q_R = 2.19m^3/s$



A = (3.80 + 1.40) *1.20*1/2 = 3.12 m2

P = 1.697*2 + 1.40 = 4.794 m

R= 3.12 / 4.794 = 0.6508

n = 0.030

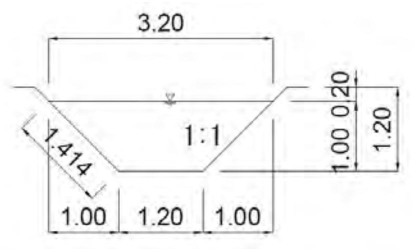
I = 1 / 1000

$V = 1 / 0.030 * 0.6508^{2/3} * (1 / 1000)1/2 = 0.79\ m^3/s$

$Q = 3.12 * 0.79 = 2.46\ m^3/s > Q_R = 2.19m^3/s$

(2) West Side

$Q_R = 1.32m^3/s$



$A = (3.20 + 1.20) *1.00*1/2 = 2.20\ m^2$

$P = 1.414*2 + 1.20 = 4.028\ m$

$R= 2,20 / 4.028 = 0.5462$

$n = 0.030$

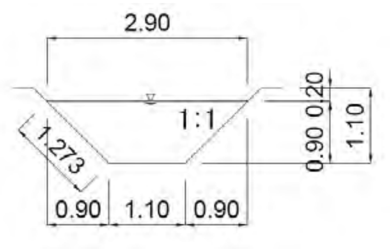
$I = 1 / 1000$

$V = 1 / 0.030 * 0.5462^{2/3} * (1 / 1000)1/2 = 0.70\ m^3/s$

$Q = 2.20 * 0.70 = 1.54\ m^3/s > Q_R = 1.32m^3/s$

(3) Outer Channel

Q_R =1.09m3/s



$A = (2.90 + 1.10) *0.90*1/2 = 1.80\ m^2$

$P = 1.273*2 + 1.10 = 3.646\ m$

$R= 1.80 / 3.646 = 0.4937$

$n = 0.030$

$I = 1 / 1000$

$V = 1 / 0.030 * 0.4937^{2/3} * (1 / 1000)^{1/2} = 0.66\ m^3/s$

$Q = 1.80 * 0.66 = 1.19\ m^3/s > Q_R = 1.09m3/s$

4. Discharge Capacity of Pipes

4.1 Discharge Capacity per One Pipe

D: diameter (= 0.91m)

H: depth of water (0.80 * D = 0.80 * 0.91 = 0.728m)

$A = 0.6736 * D^2 = 0.6736 * 0.91^2 = 0.5578\ m^2$

$R = 0.3042 * D = 0.3042 * 0.91 = 0.2768\ m$

n = 0.013

I = 1/150 (setting gradient)

Manning Equation

$V = 1/n * R^{2/3} * I^{1/2}$

$= 1/0.013 * 0.2768^{2/3} * (1/150)^{1/2}$

$= 2.67\ m^3/s$

$Q = AV = 0.5578 * 2.67 = 1.49\ m^3/s$

4.2 Number of Pipes

Case	Design Discharge	Diameter of Pipe (m)	Discharge per One Pipe	Number	Adopted Number
Case 1	2.19 m^3/s	0.91	1.49 m^3/s	1.47	2
Case 2	1.32 m^3/s	0.91	1.49 m^3/s	0.89	1
Case 3	1.09 m^3/s	0.91	1.49 m^3/s	0.73	1

5. Design of Sand Basin against Sediment of Sand

5.1 Generating Sand Volume

In reclamation area, 1.50m^3/ha/year of sand is thought to be deposited. Therefore management of the project site have to plow out at least at an interval of 6 months.

Case 1 East Side

$V = 1.50 * 28.65 * 6/12 = 21.49\ m^3$

Case 2 West Side

$V = 1.50 * 15.64 * 6/12 = 11.73\ m^3$

5.2 Size of Sand Basin (Length)

Case 1 East Side

L = 21.49 / (5.00*0.80) = 5.37 m….. 6.00m is adopted.

Case 2 West Side

L = 11.73 / (5.00*1.00) = 2.35 m….3.00m is adopted.

Calculation of temporary installation of Laguna RC Pipe

1. Design Condition

(1) Catchment Area (one sand basin takes a half area of the project site)

Catchment area is assumed to be 95% of total project area.

Case 1 East side (Temporary)

Catchment area A= 30.16 * 0.95 = 28.65 ha

Development area A1= 28.65 * 1/2 = 14.33 ha

Area Back to grass field or natural A2= 14.33 ha

Case 2 East side + West side (Temporary)

Catchment area A = (30.16+16.46)*0.95 = 44.29 ha

Development area A1= 16.46 * 1/2 = 8.23 ha

Area Back to grass field or natural A2= 44.29 – 8.23 = 36.06 ha

(2) Average Runoff Coefficient

Development Area C1 = 0.90 (bare area)

Area Back to Grass field C2 = 0.60 (grass area)

	Infiltration: small	Infiltration: medium	Infiltration: large
Forest Land	0.6~0.7	0.5~0.6	0.3~0.5
Grass Field	0.7~0.8	0.6~0.7	0.4~0.6
Cultivated Area		0.7~0.8	0.5~0.7
Bare Area	1.0	0.9~1.0	0.8~0.9

Average runoff coefficient

Case 1 C1 = (14.33*0.90 + 14.33*0.60) / 28.66 = 0.75

Case 2 C2 = (8.23*0.90 + 36.06*0.60) / 44.29 = 0.66

2. Design Rainfall Discharge

(1) Design Rainfall (Q_R)

Q_R: Design Rainfall Discharge (m3/sec)

$Q_R = R \cdot C \cdot A \cdot \sqrt[6]{(S/A)}$

R : Rainfall Intensity (54.0 mm/hr) = 0.1500 (m^3/sec・ha)

$r = 935.4 / (t^{0.64} + 3.57) = 935.4 / (60^{0.64} + 3.57) = 54.0$ mm/hr

t=60mm is adopted due to very flat area for the safety.

S: Grand Gradient (S/1000=1/1000)

Case 1

C: Runoff Coefficient=0.75

A: Catchment Area = 28.66 ha

Q_R=R・C・A

=0.150*0.75*28.66*$\sqrt[6]{(1 / 28.66)}$

=1.84 m^3/S

Case 2

C: Runoff Coefficient=0.66

A: Catchment Area = 44.29 ha

Q_R=R・C・A

=0.150*0.66*44.29*$\sqrt[6]{(1 / 44.29)}$

=2.33 m^3/S

4. Discharge Capacity

4.1 Discharge Capacity per One Pipe

D: diameter (= 0.91m)

H: depth of water (0.91 * D = 0.80 * 0.91 = 0.728m)

$A = 0.6736 * D^2 = 0.6736 * 0.91^2 = 0.5578\ m^2$

$R = 0.3042 * D = 0.3042 * 0.91 = 0.2768\ m$

n = 0.013

I = 1/150 (setting gradient)

Manning Equation

$V = 1/n * R^{2/3} * I^{1/2}$

$= 1/0.013 * 0.2768^{2/3} * (1/150)^{1/2}$

$= 2.67\ m^3/s$

$Q = AV = 0.5578 * 2.67 = 1.49\ m^3/s$

4.2 Number of Pipes

Case	Design Discharge	Discharge per One Pipe	Number	Adopted Number
Case 1	1.84 m^3/s	1.49 m^3/s	1.23	2
Case 2	2.33 m^3/s	1.49 m^3/s	1.56	2

5. Calculation of Sand Basin

(1) Shape and Section of Sand Basin

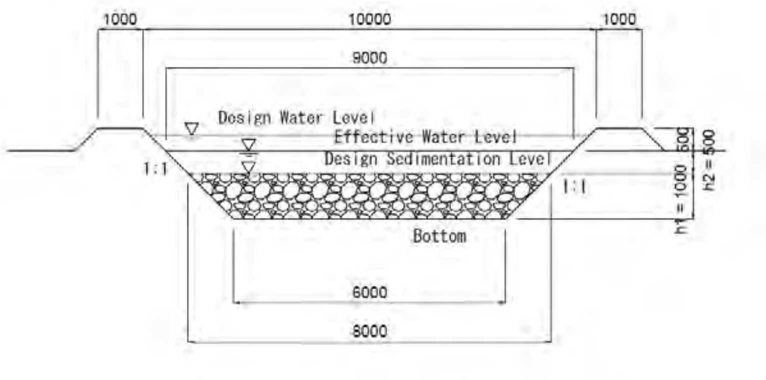
Width of Sand Basin B=9.00m

Length of Sand Basin L = 34.00m

Sediment Depth h1=1.00m (Average)

Effective Water Depth h2= 0.50m

L / B = 34.00 / 9.00 = 3.78 > 3.0 …OK



(3) Design Sediment Height

Period from development till growing of grass: Average 1.5 months

Generating Sand Volume:

$200m^3/(ha \cdot year)*1.5\ months/\ 12\ months = 25\ m^3/ha$

Therefore a design sediment volume is supposed to be $25m^3/ha$.

Sediment Volume = $8.23ha*25m^3/ha=206\ m^3$

Design Sediment Volume = $7.00*32.50*1.00 = 227.5\ m^3 > 206m^3$…OK

(4) Overflow Depth

Overflow Width: w=9.00m

Overflow depth: $h3= (Q_R / (1.8*w))^{2/3} =(2.33/ (1.8*9.00))^{2/3} =0.27m$

(5) Average Water Velocity in Sand Basin : V

Flow Area WA = B*h3

= 9.00*0.27

= $2.43m^2$

Average Velocity V = QR / WA = 2.33/ 2.43 = 0.96 m/sec

(6) Storage Time in Sand Basin : T

Storage Time T = L / V = 34.00/ 0.96 = 35.4 sec > 30 sec…OK

Structural Calculation of Laguna Drainage Facility

Structural calculation of side wall is done as a cantilever beam and as a two-side fix slab.

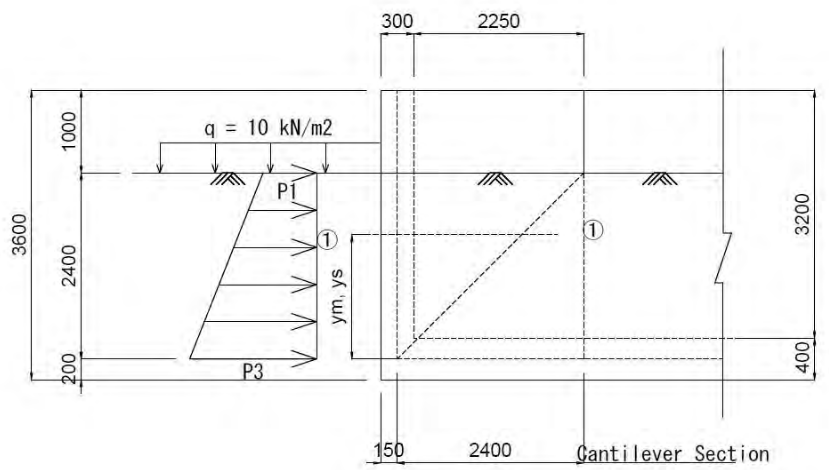
(1) Stress Calculation as a cantilever beam

P1 = 0.5*18.0*0 + 10.0*0.5 = 5.00 kN/m

P3P1 = 0.5*18.0*2.40 + 10.0*0.5 = 26.60 kN/m

M = 1/6 * 2.40^2*(2*5.00 + 26.60) = 35.14 kNm/m

S = 1/2 *2.40 * (5.00 + 26.60) = 37.92 kN/m



(2) Stress Calculation as a Two-side Fix Slab (Horizontal Direction)

P2 = 26.60 – 5.00 = 21.60 kN/m

Fix end moment

Height where a moment is maximum

ym = 2/3*h*(P1+P2)/P2

= 2/3*2.40*(5.00+21.60) / 21.60

= 1.97m

Force at ym

Py = 5.00 + 21.60 – 21.60*1.97/2.40

= 8.87 kN

Moment at ym

My = 1/2*Py*ym^2

= 1/2 * 8.87 * 1.97^2

= 17.21 kNm

Shearing force

Height where a shearing force is maximum

ys = 1/2*h*(P1+P2)/P2

= 1/2*2.40*(5.00+21.60) / 21.60

= 1.47m

Force at ys

Py = P1 +P2-P2*ys/h

= 5.00 + 21.60 – 21.60*1.47/2.40

= 13.37 kN

Shearing force at ys

Smax = Py * ys

= 13.37 * 1.47

= 19.65 kN

(3) Section Calculation

Calculation Item		Cantilever Beam	Two-side Fixed Slab	Remark
M (kN・m / m)		35.14	17.21	
S (kN / m)		37.92	19.65	
t (mm)		300	300	
d’(mm)		75	75	
d(mm)		225	225	
fsy (N/mm2)		275	275	
f’c (N/mm2)		20.7	20.7	
Ast1	D16@125	D16@125	D12@125	
	1609	1609	905	
p		0.0072	0.0040	
k		0.369	0.292	
j		0.877	0.903	
σ c = 2・M/(k・j・b・d^2)		4.3	2.6	
f’ck = 0.4 f’c		8.3	8.3	
σ s = M / (As・j・d)		111	93	
fs		140	140	
τ = S / (b・j・d)		0.19	0.09	
τ a		0.36	0.36	
Decision		OK	OK	

Structural Calculation of RC Pipe ϕ 910

1. Design Condition

   Class of reinforced concrete pipe : Class Ⅱ, Wall B

   Diameter : ϕ 910 (t = 100mm)

   D-load to produce a 0.30-mm crack : 50 KN / m / 0.91m

   Earth covering: 0.60m

2. Foundation

   Foundation : sand

   Thickness of foundation : 0.20 * (outer diameter)

   = 0.20 * 1110mm

   = 222 mm…….250mm is adopted.

3. Safety Factor of Pipe

   Strength of pipe shall satisfy the following equation.

   $f \leq Mr / M$

   Where,

   f : safety factor, 1.25

   Mr: resistant bending moment (kNm)

   M : maximum moment occurred in pipe (kNm)

   Japan Sewage Works Association recommends the following equations.

   Mr = 0.318 * Q * R + 0.239 * W * R

   M = k * q * R

   Where,

   Q : strength against outer pressure depending on cracking load (kN/m)

   = 50 kN/ 0.91m = 54.9 kN/m

   R : center radius of pipe thickness (m)

   = (0.91 + 0.10 * 2) * 1/2 = 0.550 m

   W : pipe weight (kN/m)

   $= (1.11^2 – 0.91^2) * \pi /4 * 23.5 = 7.46$ kN/m

   k : factor due to supporting condition (=0.275 : sand foundation)

   q : distribution load (kN/m2)

   = 18.0 * 0.60 + 10.0 = 20.80 kN/m^2

   Mr = 0.318 * 54.9 * 0.550 + 0.239 * 7.46 * 0.550

   = 10.58 kNm

   M = 0.275 * 20.80 * 0.550

   = 3.15 kNm

   f = Mr / M = 3.36 > 1.25…..OK

CHAPTER 7 CALCULATION OF TEMPORARY BRIDGE/JETTY AT BACKFILL SITE

The detailed calculation of temporary bridge/jetty at backfill site is indicated from the following page.

Calculation of Laguna Temporary Bridge (Jetty)

1. Design Condition

1-1 Form Size

Calculation is performed as a temporary pier (jetty) which road deck panels (2.0 m x 1.0 m) are carried on main girders arranged for every 2-m span, and H beams support them.

Calculation Width: 10.0m

Span of beam : 6.0m

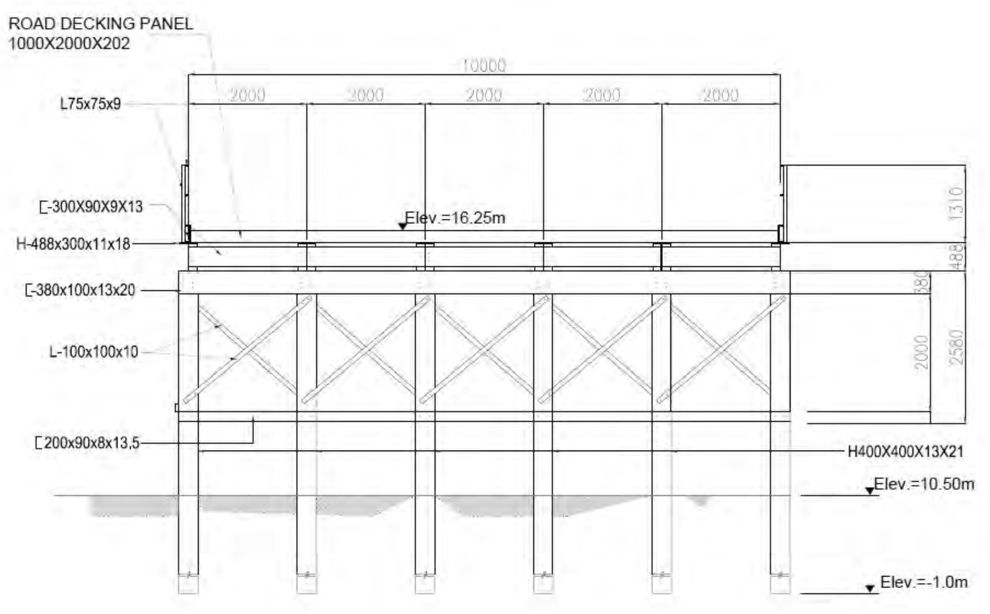


Fig.1 Cross Section of Temporary Bridge

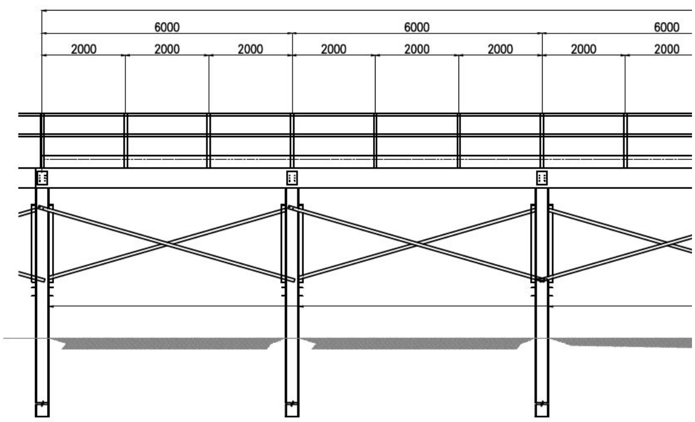


Fig.2 Profile of Temporary Bridge

1-2 Loads

(1) Dead Load

Road decking panel: $2 kN/m^2$

Main beam : 1.67 kN/m (H-594*302*14*24)

(2) Live Load

A dump truck T-25 and crawler crane loads give consideration to the design of a temporary bridge.

Crawler crane load is as follows

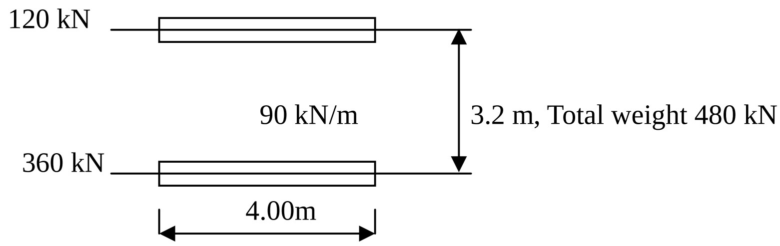


Fig.3

(3) Impact Load

i = 0.3

(4) Horizontal Load

10% of live load

1-3 Allowable Stress

* Allowable Axial Tensile Stress: $205 N/mm^2$

* Allowable Axial Compressive Stress:

$l / r \leq 20$: $205 N/mm^2$

$20 < l / r < 93$: $(140 - 0.84(l/r-20))*1.5 N/mm^2$

$l / r \geq 93$: 〔$12000000 / (6700+(l/r)^2)$〕$*1.5 N/mm^2$

Where, l : length of a member

r: radius of gyration of area (cm)

* Allowable Bending Tensile Stress: $205 N/mm^2$

* Allowable Bending Compressive Stress:

$l / b \leq 4.5$: $205 N/mm^2$

$4.5 < l / b \leq$ 〔$(140 - 2.4(l/b-4.5))$〕$* 1.5 N/mm^2$

Where, l : distance of fixed points of flange (cm)

b : compressive flange width (cm)

* Allowable shearing stress : 120 N/mm^2

* Allowable shearing stress of bolt : 125 N/mm^2

* Allowable bearing stress : 285 N/mm^2

1-4 Allowable Deflection of Main Beam

1/400 of span, or 25mm or less

1-5 Bearing Capacity of Pile

$Qa = Qu / 2$

$Qu = 200NA + 10(NcAc+1/5NsAs)$

Where, Qa : allowable bearing capacity of a pile (kN)

Qu : ultimate bearing capacity of a pile (kN)

2. Design of Main Beam

2-1 Stress due to Dead Load

$M_{max}= M_{C1}=1/8*(2*2.0 +1.27)*6.0^2 = 23.7$ kNm

$S_{max}= S_{A1} =1/2*(2*2.0 +1.27)*6.0 =15.8$ kN

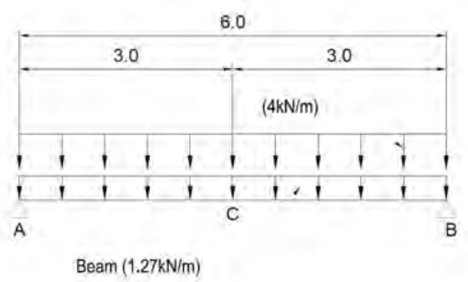


Fig.4

2-2 Stress due to Truck Load

$P_{rB} = (1+i)\ P_r\ (0.125+1.0+0.5)$

$=(1+0.3)*100*(0.125+1.0+0.5)$

$= 211.3$ kN

$P_{fB}=(1+i)P_f(0.125+1.0+0.5)$

=(1+0.3)*25*(0.125+1.0+0.5)

= 52.8kN

Where, I : impact factor (=0.3)

P_r: load of rear wheel (=100kN)

P_f: load of front wheel(=25kN)

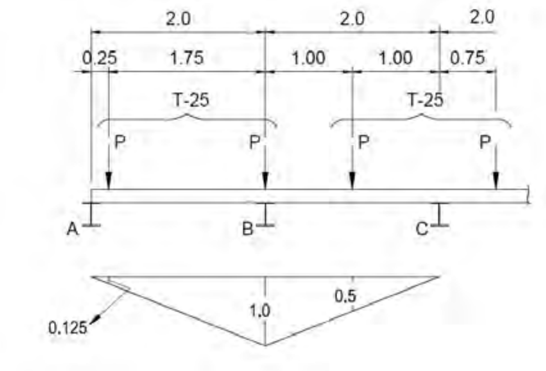


Fig.5

(1) Bending Moment

Case 1

R_A = 1/6.0*(P_{fB}*6.00+P_{rB}*2.00)

= 1/6.00*(52.8*6.00+211.3*2.00)

=123.2 kN

R_B = P_{fB} +P_{rB} – R_A

= 52.8 + 211.3 – 123.2

= 140.9 kN

M_{c2} = R_B*2.00

= 140.9*2.00

= 281.80 kN

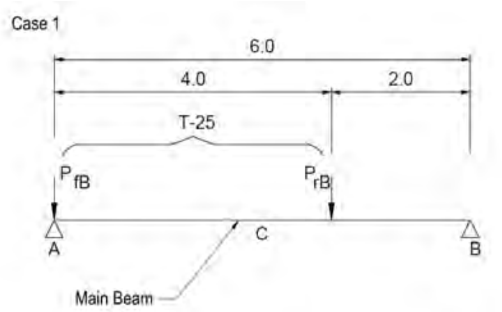


Fig.6

Case 2

R_A = 211.3*1/2 = 105.7 kN

Mc2 = 105.7*3.00

= 317.1 kN (= M_{max})

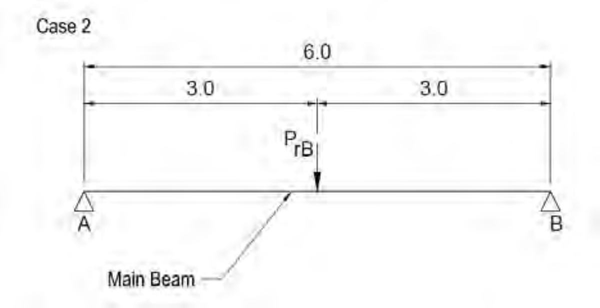


Fig.7

(2) Shearing

Case 3

R_A = 211.3 +52.8 *3.00 / 6.00 = 237.7 kN (= S_{max})

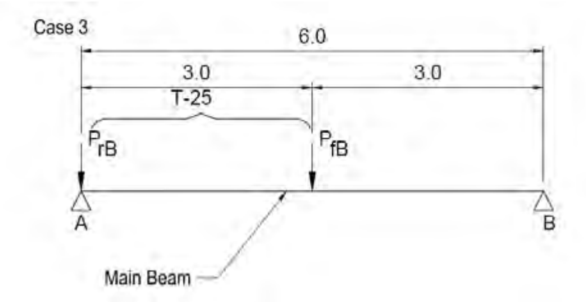


Fig.8

2-3 Stress due to Crawler Crane Load

q = 1/4.0*(1+i)*480:0.75

= 1/4.0*(1+0.3)*480*0.75

= 117.0 kN/m

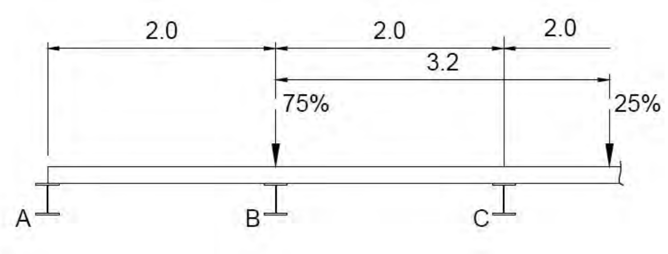


Fig. 9

(1) Bending Moment

M_{C3} = 1/2*q*4.0*3.00-1/2*q(4.0-2.0)^2

=1/2*117.0*4.0*3.00 – 1/2*117.0*(4.00-2.00)^2

= 468.0 kNm

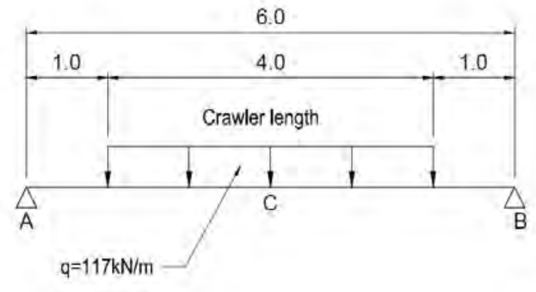


Fig.10

(2) Shearing

SA3 = RA=q*4.00*(6.00 – 1/2*4.00)*1/6.00

= 117.0*4.00*4.00 / 6.00

=312.0 kN

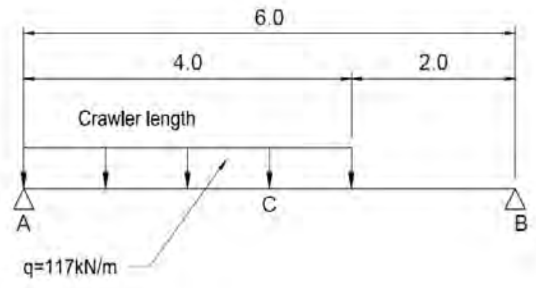


Fig.11

2-4 Stress of Main Beam

M_{max} and S_{max} are as follows.

M_{max} = 23.7 + 468.0 = 491.7 kNm

S_{max} = 15.8 + 312.2 = 328.0 kN

H-shaped beam used for main beams is as follows.

H-shaped beam : H-488*300*11*18

Weight of beam : 1.23 kN/m

Section modulus: Z_x= 2,820 cm^3

Moment of inertia: Ix = 68,900cm^4

Width of flange : b=30cm

Area of web : A_w = 1.1*48.8 =53.7 cm^2

l_y = 600 cm

σ ba = 210 – 3.6*(l/b – 20) =210 – 3.6(600/30 – 20) = 210 N/mm^2

σ b = M_{max} / Z_x = 491.7 *10^3/ 2,820 = 175.4 N/mm^2 < 210 N/mm^2

τ = S_{max} / A_w = 328.0*10^3 / ((488 – 18*2)*11) =66.0 N/mm^2 <120 N/mm^2

2-5 Deflection of Main Beam

(1) Deflection due to Truck Load

δ c = P_{rB}*l^3 / (48EI)

= 211.3 * 6.00^3 / (48*2.0*10^8*68,900*10^{-8})

= 0.0069 m = 0.69cm < l / 400 = 600 / 400 = 1.5 cm

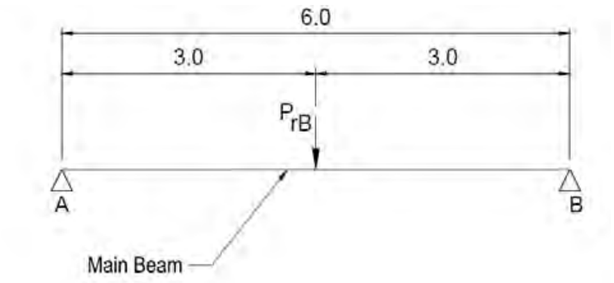


Fig.12

(2) Deflection due to Crawler Crane

q = 117.0 kN/m

δ c = q(l - 2a)^4/(384EI) +qbl(1.5l^2 – b^2 – 2bc – 2c^2) / (48EI)

= 117.0* (6.0 – 2*1.0)^4 / (384*2.0*10^8*68,900*10^{-8})

+117.0*4.0*6.0*(1.5*6.0^2 – 4.00^2 – 2*4.0*1.0 – 2*1.0^2) / (48*2.0*10^8*68,900*10^{-8})

= 0.00057 + 0.01189 = 0.01246 m = 1.25 cm < l / 400 = 600 / 400 = 1.5 cm

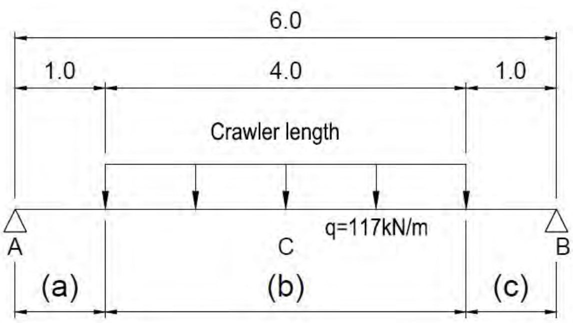


Fig.13

3. Design of Bearing Pile

(1) Reaction to Pile

Weight of Steel Member

Road decking panel	: 2 kN / m2 *2.0m*6.0m	= 24.0 kN
Main Beam	: 1.23kN / m *6.00m	= 7.4 kN
Seat Beam	: 0.66kN/m*2*2.00m	= 2.6 kN
Cross Beam	: 0.37 kN/m*3*2.00m	= 2.2 kN
Vertical Brace	: 0.15 kN/m*2*2.80m	= 0.8 kN
Pile	: 1.96 kN/m*14.20m	= 27.8 kN
Total		64.8 kN $= R_{B1}$

(2) Vertical Force due to Crawler Crane

R_{B2} = 117.0*2.00*(6.00-2.00 /2) / 6.00*2 = 390 kN / pile

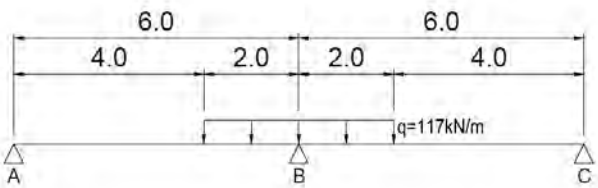


Fig.14

(3) Maximum Vertical Force on Pile

$N = R_{B1}+R_{B2}$ = 64.8 + 390.0 = 454.8 kN

3-2 Horizontal Force

Ten (10) % of live load would act in the lateral direction.

(1) Horizontal Force due to Truck Loads

P_{rB} = 2 * 3 *Pr = 2 * 3 *100 = 600 kN

P_{fB} = 2 * 3 * Pf = 2 * 3 * 25 = 150 kN

Case 1

H = 0.1 * R_B

= 0.1 * (600*1+150*(2.0/6.0+3.0/6.0))

= 72.5 kN

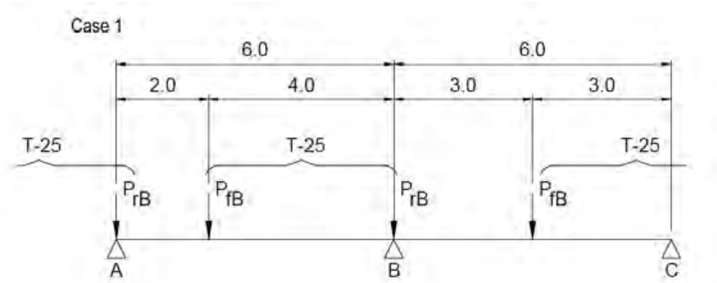


Fig.15

Case 2

H = 0.1 * (600* (1.0/6.0+4.0/6.0) + 150*(4.0/6.0+1.0/6.0))

= 62.5 kN

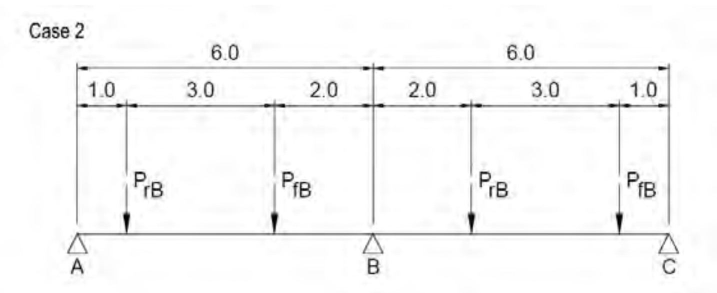


Fig.16

Therefore, P_{H1} = 72.5 / 6.0 =12.1 kN/pile

(2) Horizontal Force due to Crawler Crane

P_{H2}= 480 * 0.1 /6 = 8.0 kN/pile

(3) Maximum Horizontal Force

H_0 = 12.1 kN/pile

3-3 Bending Moment of Bearing Pile

N Value= 5 (Average)

$K_h = 1/3 * \alpha * E_0 * (B_H/0.3)^{-3/4}$

$\beta = (K_h D / (4EIy))^{1/4}$

Where, Kh : coefficient of lateral reaction of soil (kN/m^3)

α : coefficient (=1.0 from N value)

E_0: modulus of deformation of soil, 2800N (kN/m^2)

B_H: converted loading width of foundation in load direction (m)

D : size of H-shaped beam, 0.40m

E: elastic (Young's) modulus, $2.0*10^8$ kN/m^2

Iy: moment of inertia, 22,400 cm^4

From a repeated calculation, the values as shown below are obtained.

β = 0.525 m^{-1}

K_h = 33,923 kN/m^3

B_H = 0.459 m

Restriction moment at the top (Height of 7.00 m shows an average height of river bed)

$M0 = H0 (1+\beta h) / 2\beta$

= 12.1 * (1 + 0.525*4.70) / (2*0.525)

= 40.0 kNm

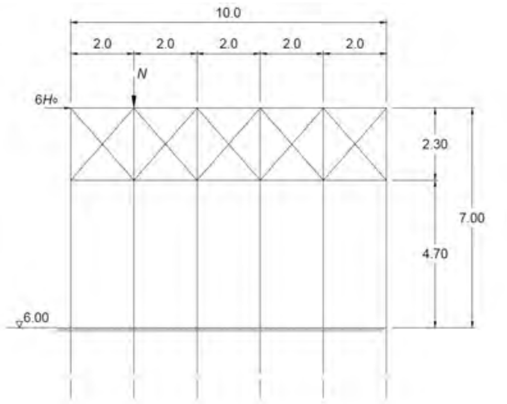


Fig.17

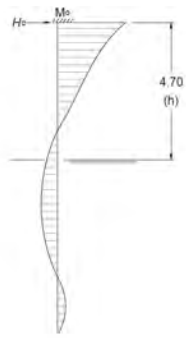


Fig.18

3-4 Horizontal Deformation of Bearing Pile

Chang's method is applied to a calculation of deformation.

$\delta = H_0 h^3 / (12\ EI) * ((1+\beta h)^3 + 2) / (\beta h)^3$

$= 12.1*4.70^3 / (12*2.0*10^8*22{,}400*10^{-8})*((1+0.525*4.70)^3 + 2) / (0.525*4.70)^3$

$= 0.0068$ m = 0.68 cm

3-5 Stress of Bearing Pile

$\sigma c / \sigma ca + \sigma bc / \sigma ba < 1.0$

Bearing Pile: H-400*400*13*21

A = 218.7 cm^2, ix = 17.5 cm, iy= 10.1 cm, Zy = 1,120 cm^3

Longitudinal buckling length lx = 700 cm, lx /ix = 700 / 17.5 = 40.0

σ ca = 1.5*(140 – 0.84 (lx/ix – 20)) = 184.8 N/mm^2

Lateral buckling length ly = 230 cm, ly/iy = 22.8

σ ca = 1.5*(140 – 0.84 (ly/iy – 20)) = 206.5 N/mm^2

σ ba = 210.0 N/mm^2

Stress of pile

σ c = N/A = 454.8*10^3 / (218.7*10^2) = 20.8 N/mm^2

σ bc = M / Zy = 40.0 * 10^6 / (1,120*10^3) = 35.7 N/mm^2

σ c/ σ ca + σ bc/ σ ba = 20.8 / 184.8 + 35.7 / 210.0

= 0.11 + 0.17 = 0.28 < 1.0

3-6 Bearing Capacity of Pile

Qu = 200NA + 10NcAc

Where, N: N value at a tip of pile (=20)

A : Area of pile, (=0.40(a)*0.40(b) = 0.160 m^2)

Nc: Average N value of cohesive soil (= 5)

Ac: perimeter of pile (=1.20*10.00m (average)= 12.00m^2)

Qu = 200 * 20 * 0.160 + 10 * 5 * 12.00

= 1,240 kN

Qa = 1/2 * 1,240 = 620 kN > 454.8 kN….OK

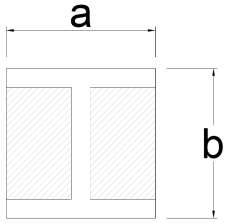


Fig. 19

4 Design of Bracing

4-1 Diagonal Member

T = 1/4 * H/cos θ = 1/4 * 72.5 / (2.0/$\sqrt{}$(2.00^2+2.30^2)) = 27.6 kN/piece

Steel member: L-100*100*10

A = 19.0 cm^2

σ t = T/A = 27.6 *10^3 / 19*10^2 = 14.5 N/mm^2 < 210 N/mm^2

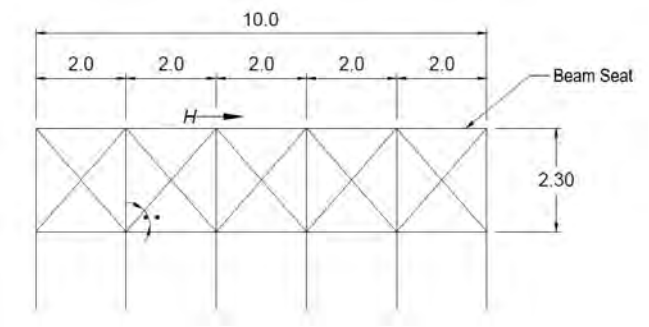


Fig.20

4-2 Tie Beam

[－200*90*8*13.5

$A = 38.65\ cm^2$

$iy = 2.68\ cm$

Length of member l = 2.0m

l / iy = 200/ 2.68 = 74.6

σ ca = 1.5*(140 – 0.84 (ly/iy – 20))

=1.5*(140 – 0.84 (74.6 – 20))

$= 141.2\ N/mm^2$

σ c = H / 5 *1/A*1/2

$= 72.5*10^3 / (5*38.65*10^2 *2)$

$= 1.9\ N/mm^2 < 141.2\ N/mm^2$….OK

Calculation of Road Deck Panel

1. Design Condition

Span : L = 2.00m (1.996m)

Height: H = 0.20m(0.202m)

Load : T-25

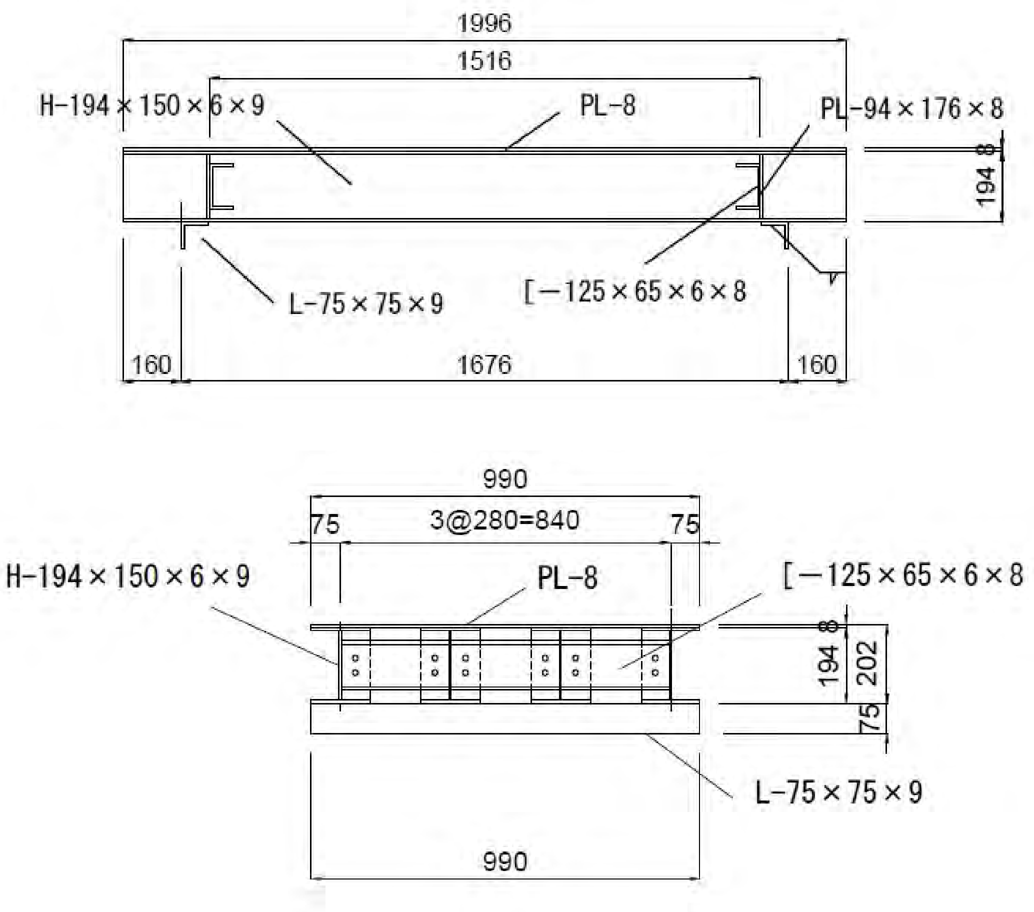


Fig.1 Road Deck Panel

2. Stress Calculation

(1) Live load

$P = 0.4W(1+i)$

$=100kN \times (1+0.385)$

$= 138.5$ kN

$i = 20 / (50+2.00) = 0.385$

(2) Dead Load

H-194×150×6×9 W = 29.9 kg/m (0.293 kN/m)* 4 nos

Section Modulus Zx = 271 cm^3

PL – 8 W = 0.008×1.00×77kN/m^3 = 0.62 kN/m

(3) Bending Moment

$M = 1/4 \times 138.5 \times 2.00 + 1/8 \times (0.293 \times 4 + 0.62) \times 2.00^2$

$= 69.9$ kNm

(4) Bending Stress

$\sigma = M / Zx$

$= 69.9 \times 10^6 / (271 \times 10^3 \times 4)$

= 64.5 N/mm2 ＜140×1.30 = 182 N/mm^2

3. Unit Weight

L-75×75×9 W = 9.96 kg/m (0.098 kN/m)*2nos

[-125×65×6×8 W = 13.4 kg/m (0.131 kN/m)*2nos

Total Weight = 0.62 ×2.00 + 0.293 ×4×2.00 + 0.098×1.00×2 + 0.131×0.78×2

= 3.98 kN

Unit Weight = 3.98 / 2.00 = 2.00 kN/m^2

CHAPTER 8 STABILITY ANALYSIS AT BACKFILL SITE

The detailed calculation of stability analysis at back fill site is indicated from the following page.

8.2

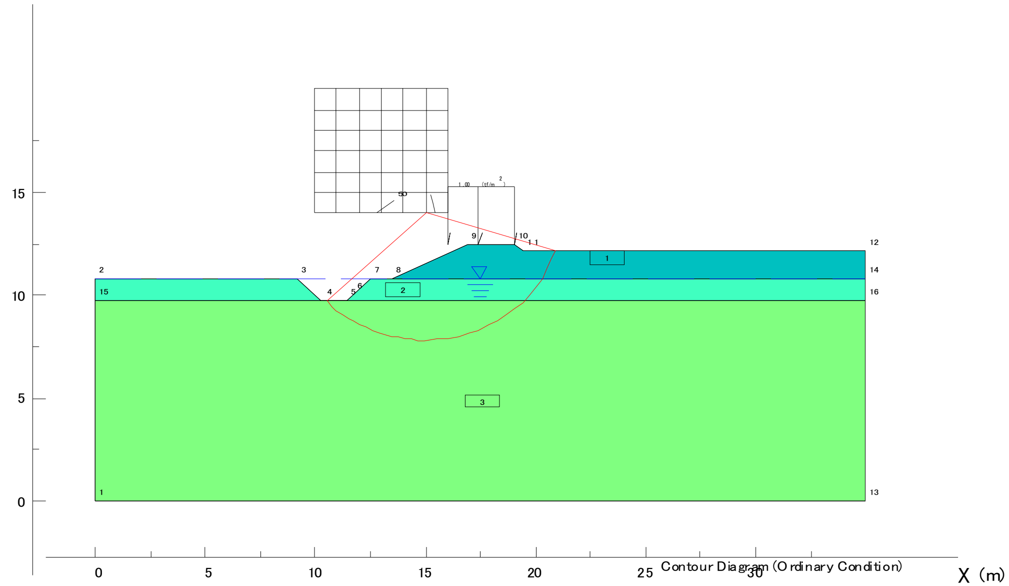
Laguna Embankment Case 1 i=1.5 H=0.7

Scale ; 1/ 277

Min. safety factor	F S MIN	=	4.388	
Center of arc	X	=	15.00	(m)
	Y	=	14.00	(m)
Radius	R	=	6.19	(m)
Resisting moment	M R	=	154.18	(tfm)
Sliding moment	M D	=	35.13	(tfm)

Layer Number	Saturated Unit Weight (tf/m^3)	Wet Unit Weight (tf/m^3)	Friction Angle (Degree)	Cohesion (tf/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	1.800	0900	30.00	01 0	0.00	0.000	0.000
2	1.700	0800	24.00	07 0	0.00	0.000	0.000
3	1.700	0800	24.00	18 0	0.00	0.000	0.000

Water unit weight = 1.000 (tf/m^3)



Contour Diagram (Ordinary Condition)

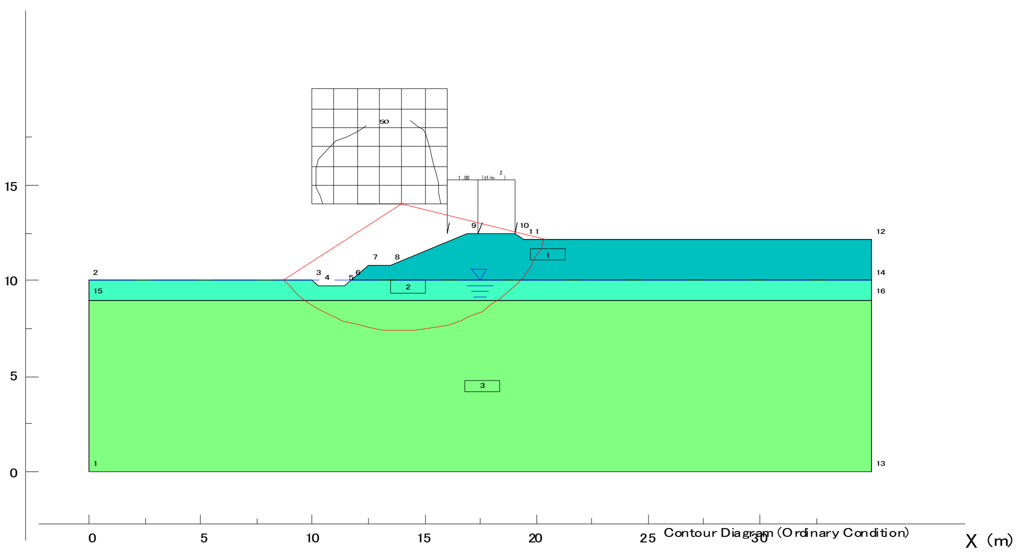
Laguna Embankment Case 1 i=1.5 H=0

Scale ; 1/ 277

Min. safety factor	$F_{S\ MIN}$	=	3.985	
Center of arc	X	=	14.00	(m)
	Y	=	14.00	(m)
Radius	R	=	6.64	(m)
Resisting moment	M_R	=	187.59	(tfm)
Sliding moment	M_D	=	47.07	(tfm)

Layer Number	Saturated Unit Weight (tf/m^3)	Wet Unit Weight (tf/m^3)	Friction Angle (Degree)	Cohesion (tf/m^2)	Rate of Increase of Cohesion	Horizontal Seismic Coefficient	Vertical Seismic Coefficient
1	1.800	0900	30.00	010	0.00	0.000	0.000
2	1.700	0800	24.00	070	0.00	0.000	0.000
3	1.700	0800	24.00	180	0.00	0.000	0.000

Water unit weight = 1.000 (tf/m^3)



Contour Diagram (Ordinary Condition)

CHAPTER 9 STRUCTURAL CALCULATION OF DRAINAGE FACILITIES

9.1 MANHOLE

9.1.1 Design Conditions

Manhole and junction box are regard structure as fixed beam at three or four sides. And structural state is under ground structure, consequently, state of each pressure is regard as earth pressure at rest.

A passenger vehicle's passing is considered as the load effecting on cover and top slab of manhole.

(1) Concrete

Materials	f'_{ck} (N/mm^2)	E_c 10^4 (N/mm^2)
20.7	20.7	2.380

Unit weight γ_c = 24.00(kN/m^3)

(2) Reinforcing Steel Bar

Grade	F_{yk} (N/mm^2)	$E_s \times 10^5$ (N/mm^2)
275	275	2.000

(3) Groundwater Level

Depth (H) : 0.900(m)

Unit weight : 9.8(kN/m^3)

(4) Soil Conditions

Depth Z (m)	Unit weight (Wet) γ (kN/m^3)	Unit weight (saturated) γ_{sat} (kN/m^3)	Coefficient for earth pressure at rest K	Coefficient for vertical pressure α
10.000	18.000	20.000	0.5000	1.0000

(5) Live Load

(a) On Top Slab and Cover

Vehicle : T-2, 8.0 (kN)

Impact Coefficient : 0.300

(b) On the Ground

Live Load: 10.0 (kN/m^2)

(6) Calculation Case

(a) Manhole

Regarding case1 to 4, in Pasig River there are same cases, hence in this Vol.5, these cases are omitted.

Table R 9.1.1 Calculation Case of Manhole

Case		Inside Width b_1 (mm)	Inside Width b_2 (mm)	H (mm)	Remarks
1 -	1	1500	700	0 - 2000	Calculation Result is omitted.
	2			2000 - 2500	“
	3			2500 - 3000	“
	4			3000 - 3500	“
	5			3500 - 4000	“
	6			4000 - 4500	Refer to Vol.III-1 Chapter4
2 -	1	2000	700	0 - 2000	Calculation Result is omitted.
	2			2000 - 2500	“
	3			2500 - 3000	Refer to Vol.III-1 Chapter4
	4			3000 - 3500	“
	5			3500 - 4000	“
3 -	1	1500	1500	0 - 2000	Calculation Result is omitted
	2			2000 - 2500	“
	3			2500 - 3000	“
	4			3000 - 3500	“
	5			3500 - 4000	Refer to Vol.III-1 Chapter4
4 -	1	1500	2300	0 - 2000	Calculation Result is omitted
	2			2000 - 2500	Refer to Vol.III-1 Chapter4
	3			2500 - 3000	“
	4			3000 - 3500	“
	5			3500 - 4000	“
5 -	1	2000	2500	2000 - 2500	MSL-5,MSR-4
	2			2500 - 3000	MSL-1,MSR-2
6		2000	3000	3000 - 3500	MSL-6
7		2000	3600	2000 - 2500	MSL-3
8		2500	2600	3000 - 3500	MSL-2,MSL-4
9		1500	6100	3000 - 3200	MSR-3

(b) Manhole Cover

Regarding manhole cover, calculation is same as Pasig River, hence the detail of calculation is omitted.

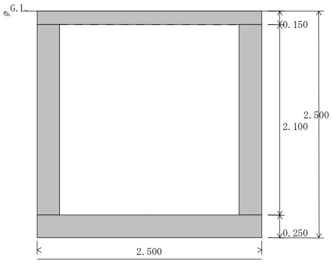
Table R 9.1.2 Calculation Case of Manhole Cover

Case	Width a (mm)	Length b (mm)	H (mm)	Remarks
1	600	1200	150	Refer to Vol.III-1 Chapter4

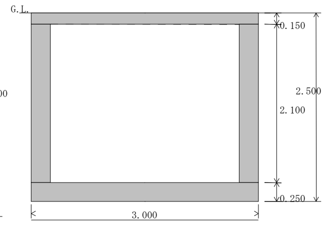
9.1.2 Structural Calculation of Manhole

The detail of structural calculation of manhole is indicated from the following page.

1 CASE1-5-1 (2,000 X 2,500 X H2,500 M)



Front



Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	2.500	0.000	3.000	0.000	0.150
Side Wall	2.500	2.000	3.000	2.500	2.100
Bottom Slab	2.500	0.000	3.000	0.000	0.250

1.1 Vertical Load

(1) Weight of Body

Top Slab	2.500×3.000×0.150×24.000	27.000 (kN)
Pedestrian Load	2.500×3.000×5.000	37.500 (kN)
Side Wall	(2.500×3.000 - 2.000×2.500)×2.100×24.000	126.000 (kN)
Bottom Slab	2.500×3.000×0.250×24.000	45.000 (kN)

(1) Water Pressure for Bottom Slab

$Ww = \gamma w \cdot (h - hw)$
$= 9.800\times(2.500 - 0.900)$
$= 15.680$ (kN/m^2)

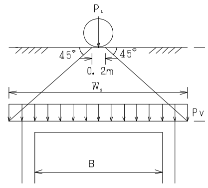
(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i)$$
$$= \frac{2\times 8.000}{2.75}\times(1+0.300)$$
$= 7.564$ (kN/m)

(b) Vertical Load of Live Load

$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h+0.2}$$
$$= \frac{7.564\times 0.9}{2\times 0.000+0.2}$$
$= 34.036$ (kN/m^2)



$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0 - W_l) \text{ より}$$
$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0 - W_l)$$
$$q_{vl} = \frac{34.036\times 0.200}{2.500^2} \cdot (2\times 2.500-0.200)$$
$= 5.228$ (kN/m^2)

1.2 Horizontal Load

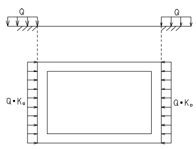
(1) Earth Pressure

Ps = $\Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$
Ps : Horizontal Earth Pressure (kN/m^2)
K_0 : Coefficient of earth pressure at rest
γt : Wet unit weight (kN/m^3)
γ' : Submerged unit weight (kN/m^3)
h : Thickness (m)
hw : Thickness inside water (m)

(2) Water Pressure

Pw = $\gamma w \cdot hw$
Pw : Water Pressure (kN/m^2)
γw : Unit weight of water = 9.800 (kN/m^3)
hw : Depth (m)

(3) Horizontal load by live load
$P_l = Q \cdot K_0$
Q : = 10.000 (kN/m^2)



(3) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.075	Top Slab	Center	18.000	0.675	0.000	5.000	5.675
0.150	Sidewall	Top	18.000	1.350	0.000	5.000	6.350
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
2.250	Sidewall	Bottom	10.000	14.850	13.230	5.000	33.080
2.375	Bottom Slab	Center	10.000	15.475	14.455	5.000	34.930

1.3 Calculation of Bottom Slab

(1) Load

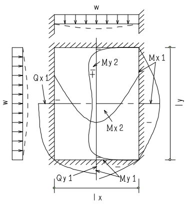
$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{64.500 + 0.000}{7.500} + 5.228$$
$= 13.828$ (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 13.828 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.750 (m)
α : ly/lx = 1.222



[1]Bending Moment

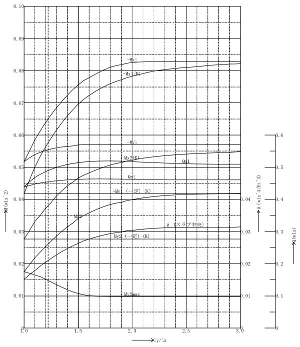
short	α	M(kN.m)
Mx1	−0.0650	−4.551
Mx2	0.0381	2.666

long	α	M(kN.m)
My1	−0.0554	−3.880
My2	0.0277	1.939
My2max	0.0148	1.039

[2]Shear Force

short	α	Q (kN)
Qx1	0.4901	15.248

long	α	Q (kN)
Qy1	0.4548	14.149



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-3.8798	1.9391	-3.8798
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.0908
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	80.0	150.0	80.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	28.8574	26.5869	28.8574
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.8200	2.3855	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	60.9293	35.0569	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1718
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

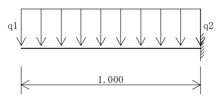
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-4.5510	2.6664	-4.5510
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.5376
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	150.0	82.0	150.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	26.1108	29.2969	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	5.8761	2.5198	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	84.8427	40.7972	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0887
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

1.4 Calculation of Open of Top Slab

(1) Design Condition

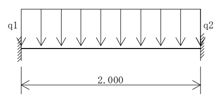
(a) Cantilever Beam

Span Length L (m)	1.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	12.253
Load q_2 (kN/m^2)	12.253



(a) Fixed Ended Beam

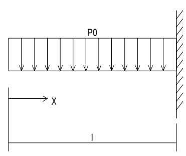
Span Length L (m)	2.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	5.228
Load q_2 (kN/m^2)	5.228



(2) Moment and Shear Force

(a) Cantilever Beam

$$Mx = -\frac{p_0 x^2}{2}$$
$$Qx = -p_0 x$$



p_0 : Effective Load = 12.253 (kN/m^2)
l : Span Length = 1.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
End	1.000	-6.127
Center	0.500	-1.532

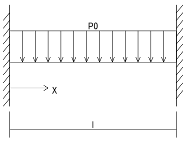
[2]Shear Force

	x (m)	Qx (kN)
End	1.000	-12.253

(b) Fixed Ended Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



p_0 : Effective Load = 5.228 (kN/m^2)
l : Span Length = 2.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
Edge	0.000	-1.743
Center	1.000	0.871

[2]Shear Force

	x (m)	Qx (kN)
Edge	0.000	5.228

(1) Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-6.1265	-1.5316
Axial Force	N	kN	——	——
Shear Force	V	kN	-12.2530	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	150.0	150.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	26.1108	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	7.9103	1.9776
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	114.2129	28.5532
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0867	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

(b) Fixed Ended Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-1.7427	0.8713
Axial Force	N	kN	——	——
Shear Force	V	kN	5.2280	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	80.0	150.0
Applied area of Reinforcement				
(Tension)	As'	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	28.8574	26.5869
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.7158	1.0719
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	27.3673	15.7527
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0743	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

1.5 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

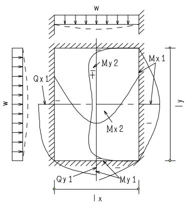
W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{190.500 + 0.000}{7.500} + 5.228$$

= 30.628 (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 30.628 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.750 (m)
α : ly/lx = 1.222



[1]Bending Moment

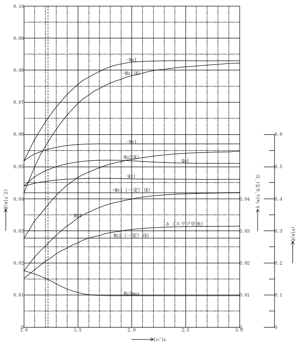
short	α	M(kN.m)
Mx1	-0.0650	-10.080
Mx2	0.0381	5.906

long	α	M(kN.m)
My1	-0.0554	-8.593
My2	0.0277	4.295
My2max	0.0148	2.302

[2]Shear Force

short	α	Q (kN)
Qx1	0.4901	33.774

long	α	Q (kN)
Qy1	0.4548	31.339



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.5934	4.2950	-8.5934
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	25.6406
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	148.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3066	30.8838	39.3066
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.2904	2.0200	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	80.3837	68.9418	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1929
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

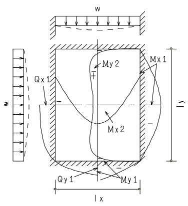
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-10.0802	5.9058	-10.0802
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	26.2689
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.3291	2.4540	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	85.7139	87.4651	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1797
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

1.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.300 (m)
α : ly/lx = 1.022



[1]Bending Moment

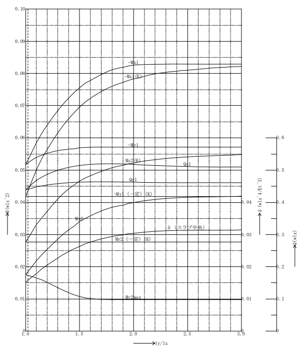
short	α	M(kN.m)
Mx1	-0.0533	-1.532
Mx2	0.0289	0.830

long	α	M(kN.m)
My1	-0.0523	-1.504
My2	0.0277	0.796
My2max	0.0174	0.499

[2]Shear Force

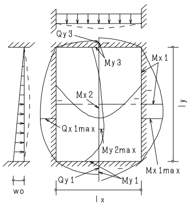
short	α	Q (kN)
Qx1	0.4460	5.695

long	α	Q (kN)
Qy1	0.4410	5.631



(b) Moment and Shear Force by Uniformly Varying Load

M = α・w・lx^2
Q = α・w・lx
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 21.855 (kN/m^2)
lx : Length (short) =2.250 (m)
ly : Length (long) = 2.300 (m)
α : ly/lx = 1.022



[1]Bending Moment

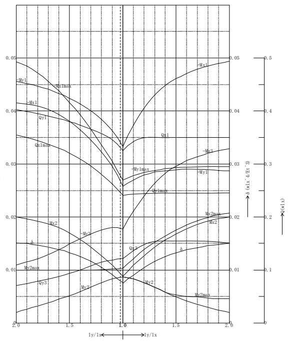
short	α	M(kN.m)
Mx1	-0.0267	-2.958
Mx1max	-0.0278	-3.071
Mx2	0.0093	1.028

long	α	M(kN.m)
My1	-0.0341	-3.768
My2	0.0086	0.953
My2max	0.0103	1.135
My3	-0.0178	-1.973

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2457	12.081

long	α	Q (kN)
Qy1	0.3307	16.260
Qy3	0.1210	5.951



Total Moment and Shear Force

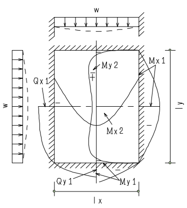
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.532	-3.071	-4.603
	Center	0.830	1.028	1.858
Vertical	Top	-1.504	-1.973	-3.477
	Center	0.796	1.135	1.930
	Bottom	-1.504	-3.768	-5.271

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	5.695	12.081	17.776
	Center	4.429	9.397	13.826
Vertical	Top	5.631	5.951	11.582
	Center	4.407	13.846	18.253
	Bottom	5.631	16.260	21.891

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

M = α・w・lx^2
Q = α・w・lx
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.300 (m)
ly : Length (long) = 2.750 (m)
α : ly/lx = 1.196



[1]Bending Moment

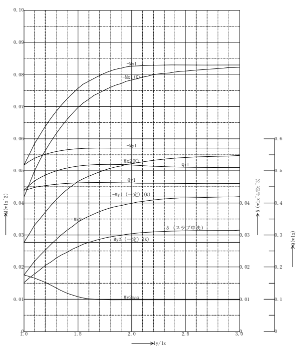
short	α	M(kN.m)
Mx1	-0.0637	-1.912
Mx2	0.0370	1.112

long	α	M(kN.m)
My1	-0.0551	-1.656
My2	0.0277	0.832
My2max	0.0153	0.458

[2]Shear Force

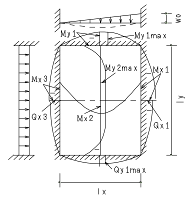
short	α	Q (kN)
Qx1	0.4862	6.346

long	α	Q (kN)
Qy1	0.4534	5.918



(b) Moment and Shear Force by Uniformly Varying Load

M = α・w・lx^2
Q = α・w・lx
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 21.855 (kN/m^2)
lx : Length (short) = 2.300 (m)
ly : Length (long) = 2.750 (m)
α : ly/lx = 1.196



[1]Bending Moment

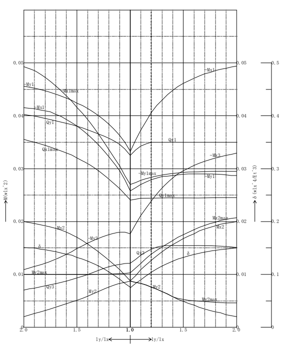
short	α	M(kN.m)
Mx1	-0.0405	-4.678
Mx2	0.0126	1.459
Mx2max	0.0137	1.587
Mx3	-0.0238	-2.749

long	α	M(kN.m)
My1	-0.0279	-3.222
My1max	-0.0284	-3.280
My2	0.0076	0.883
My2max	0.0076	0.883

[2]Shear Force

short	α	Q (kN)
Qx1	0.3495	17.571
Qx3	0.1473	7.406

long	α	Q (kN)
Qy1max	0.2442	12.274



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.656	-3.280	-4.936
	Center	0.832	0.883	1.714
Vertical	Top	-1.912	-2.749	-4.661
	Center	1.112	1.587	2.699
	Bottom	-1.912	-4.678	-6.590

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	5.918	12.274	18.191
	Center	4.842	10.042	14.884
Vertical	Top	6.346	7.406	13.752
	Center	4.966	14.856	19.822
	Bottom	6.346	17.571	23.917

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.2715	1.9305	-5.2715
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	18.2527
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	148.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3066	30.8838	39.3066
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.0184	0.9079	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	49.3096	30.9876	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1373
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-4.6025	1.8580	-4.6025
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	13.8258
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D13×4.00 452.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	32.2571	32.2571	32.2571
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.9125	0.7721	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	68.1629	27.5171	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.0926
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-6.5896	2.6990	-6.5896
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	19.8221
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	146.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3066	30.6396	39.3066
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.5232	1.2967	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	61.6395	43.9400	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1492
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-4.9360	1.7143	-4.9360
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	14.8838
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D13×4.00 452.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	32.2571	32.2571	32.2571
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.0510	0.7123	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	73.1014	25.3880	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.0997
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

1.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_W \cdot V_h$

$= 9.800 \times 12.000$

$= 117.600$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	2.500×3.000×1.350	10.125
3	2.500×3.000×0.250	1.875
Total	——	12.000

(b) Vertical Load

W = Wc + Wu

= 235.500 + 0.000

= 235.500 (kN)

(c) safety factor

Safety Factor F = W / U

= 235.500 / 117.600

= 2.003 ≧ Allowable Safety Factor F_a = 1.200

(2) Stability against Bearing

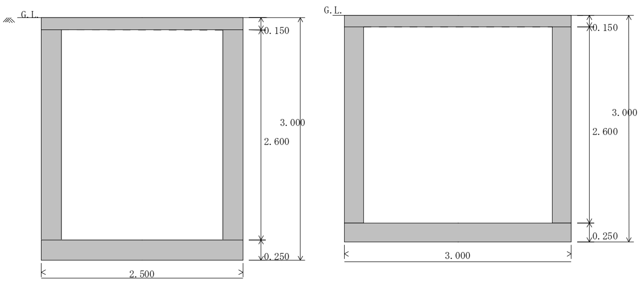
Weight of Soil Same Volume as Body

Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	2.500×3.000×0.150×18.000	20.250
2	Side Wall	2.500×3.000×2.100×18.000	283.500
3	Bottom Slab	2.500×3.000×0.250×18.000	33.750
Total Ws			337.500

Ws / Wc = 337.500 /235.500

= 1.433 ≧ 1.0

2 CASE 5-2 (2,000 X 2,500 X H3,000 M)



Front Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	2.500	0.000	3.000	0.000	0.150
Side Wall	2.500	2.000	3.000	2.500	2.600
Bottom Slab	2.500	0.000	3.000	0.000	0.250

2.1 Vertical Load

(1) Weight of Body

Top Slab	2.500×3.000×0.150×24.000	27.000 (kN)
Pedestrian Load	2.500×3.000×5.000	37.500 (kN)
Side Wall	(2.500×3.000 - 2.000×2.500)×2.600×24.000	156.000 (kN)
Bottom Slab	2.500×3.000×0.250×24.000	45.000 (kN)

(1) Water Pressure for Bottom Slab

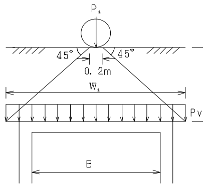
$Ww = \gamma w \cdot (h - hw)$
$= 9.800 \times (2.500 - 0.900)$
$= 20.580$ (kN/m^2)

(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i)$$
$$= \frac{2 \times 8.000}{2.75} \times (1+0.300)$$
$= 7.564$ (kN/m)

(b) Vertical Load of Live Load



$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h + 0.2}$$
$$= \frac{7.564 \times 0.9}{2 \times 0.000 + 0.2}$$
$= 34.036$ (kN/m^2)

$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0 - W_l) \text{ より}$$
$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0 - W_l)$$
$$q_{vl} = \frac{34.036 \times 0.200}{2.500^2} \cdot (2 \times 2.500 - 0.200)$$
$= 5.228$ (kN/m^2)

2.2 Horizontal Load

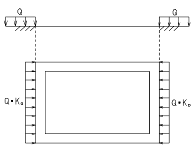
(1) Earth Pressure

Ps = $\Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$
Ps : Horizontal Earth Pressure (kN/m^2)
K_0 : Coefficient of earth pressure at rest
γt : Wet unit weight (kN/m^3)
γ' : Submerged unit weight (kN/m^3)
h : Thickness (m)
hw : Thickness inside water (m)

(2) Water Pressure

Pw = $\gamma w \cdot hw$
Pw : Water Pressure (kN/m^2)
γw : Unit weight of water = 9.800 (kN/m^3)
hw : Depth (m)

(3) Horizontal load by live load
$P_l = Q \cdot K_0$
Q : = 10.000 (kN/m^2)



(3) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.075	Top Slab	Center	18.000	0.675	0.000	5.000	5.675
0.150	Sidewall	Top	18.000	1.350	0.000	5.000	6.350
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
2.750	Sidewall	Bottom	10.000	17.350	18.130	5.000	40.480
2.875	Bottom Slab	Center	10.000	17.975	19.355	5.000	42.330

2.3 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{64.500 + 0.000}{7.500} + 5.228$$
$= 13.828$ (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$

$Q = \alpha \cdot w \cdot lx$

ここに、

M : Bending Moment (kN.m)

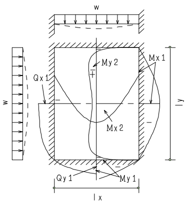
Q : Shear Force (kN)

w : Distributed Load = 13.828 (kN/m^2)

lx : Length (short) = 2.250 (m)

ly : Length (long) = 2.750 (m)

α : ly/lx = 1.222



[1]Bending Moment

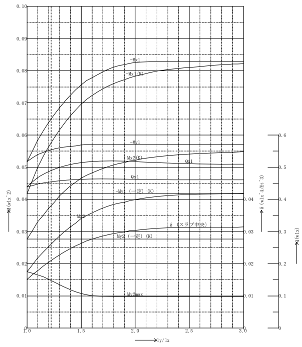
short	α	M(kN.m)
Mx1	−0.0650	−4.551
Mx2	0.0381	2.666

long	α	M(kN.m)
My1	−0.0554	−3.880
My2	0.0277	1.939
My2max	0.0148	1.039

[2]Shear Force

short	α	Q (kN)
Qx1	0.4901	15.248

long	α	Q (kN)
Qy1	0.4548	14.149



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-3.8798	1.9391	-3.8798
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.0908
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	80.0	150.0	80.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	28.8574	26.5869	28.8574
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.8200	2.3855	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	60.9293	35.0569	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1718
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

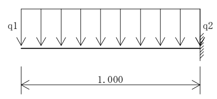
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-4.5510	2.6664	-4.5510
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.5376
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	150.0	82.0	150.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	26.1108	29.2969	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	5.8761	2.5198	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	84.8427	40.7972	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0887
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

2.4 Calculation of Open of Top Slab

(1) Design Condition

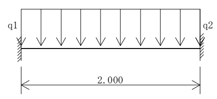
(a) Cantilever Beam

Span Length L (m)	1.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	12.253
Load q_2 (kN/m^2)	12.253



(a) Fixed Ended Beam

Span Length L (m)	2.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	5.228
Load q_2 (kN/m^2)	5.228

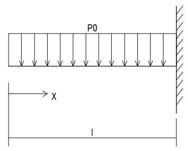


(2) Moment and Shear Force

(a) Cantilever Beam

$$Mx = -\frac{p_0 x^2}{2}$$

$$Qx = -p_0 x$$



p_0 : Effective Load = 12.253 (kN/m^2)

l : Span Length = 1.000 (m)

Mx : Bending Moment at position x (kN.m)

Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
End	1.000	-6.127
Center	0.500	-1.532

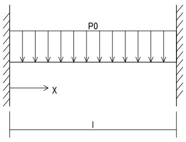
[2]Shear Force

	x (m)	Qx (kN)
End	1.000	-12.253

(b) Fixed Ended Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



p_0 : Effective Load = 5.228 (kN/m^2)
l : Span Length = 2.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
Edge	0.000	-1.743
Center	1.000	0.871

[2]Shear Force

	x (m)	Qx (kN)
Edge	0.000	5.228

(1) Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-6.1265	-1.5316
Axial Force	N	kN	——	——
Shear Force	V	kN	-12.2530	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	150.0	150.0
Applied area of Reinforcement			D13×8.00 904.80	D13×8.00 904.80
(Tension)	As	mm^2	0.00	0.00
(Compression)	As	mm^2	9	9
Young's modulus	n		26.1108	26.1108
Neutral Axis	X	mm	20.7	20.7
Concrete	f'_{ck}	N/ mm^2	415.0	415.0
Reinforcement Bar	f_{yk}	N/ mm^2	7.9103	1.9776
Stress Intensity (Concrete)	σ_c	N/mm^2	8.2000	8.2000
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	○	○
Evaluation of Compression			114.2129	28.5532
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	140.0000	140.0000
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	○	○
Evaluation of Compression			-6.1265	-1.5316
Stress Intensity by Shear Force	τ	N/mm2	0.0867	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

(b) Fixed Ended Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-1.7427	0.8713
Axial Force	N	kN	——	——
Shear Force	V	kN	5.2280	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	80.0	150.0
Applied area of Reinforcement				
(Tension)	As'	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	28.8574	26.5869
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.7158	1.0719
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	27.3673	15.7527
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0743	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

2.5 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

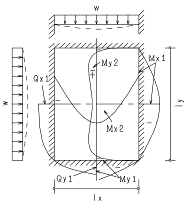
W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{220.500 + 0.000}{7.500} + 5.228$$

= 34.628 (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 34.628 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.750 (m)
α : ly/lx = 1.222



[1]Bending Moment

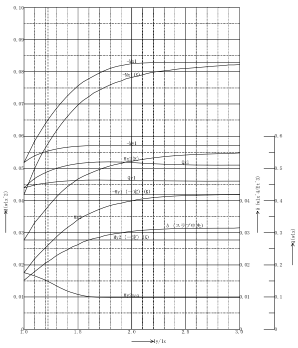
short	α	M(kN.m)
Mx1	-0.0650	-11.397
Mx2	0.0381	6.677

long	α	M(kN.m)
My1	-0.0554	-9.716
My2	0.0277	4.856
My2max	0.0148	2.602

[2]Shear Force

short	α	Q (kN)
Qx1	0.4901	38.185

long	α	Q (kN)
Qy1	0.4548	35.431



(b) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-11.3967	6.6771	-11.3967
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	29.6996
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.7639	2.7745	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	96.9081	98.8880	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2032
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(3) Calculation Result of Bottom Slab

(a) Back and Forth

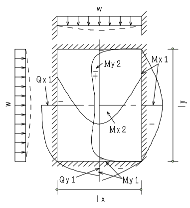
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-9.7157	4.8559	-9.7157
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	28.9893
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	148.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3066	30.8838	39.3066
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.7202	2.2838	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	90.8817	77.9455	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2181
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

2.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.244



[1]Bending Moment

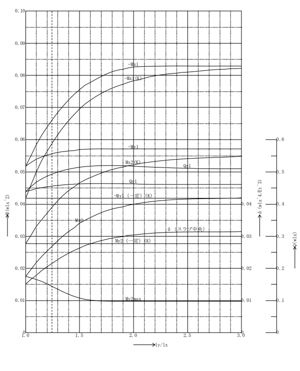
short	α	M(kN.m)
Mx1	-0.0661	-1.900
Mx2	0.0390	1.120

long	α	M(kN.m)
My1	-0.0556	-1.599
My2	0.0277	0.796
My2max	0.0145	0.416

[2]Shear Force

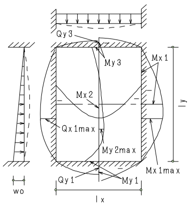
short	α	Q (kN)
Qx1	0.4933	6.299

long	α	Q (kN)
Qy1	0.4559	5.821



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 36.655 (kN/m^2)
lx : Length (short) =2.250 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.244



[1]Bending Moment

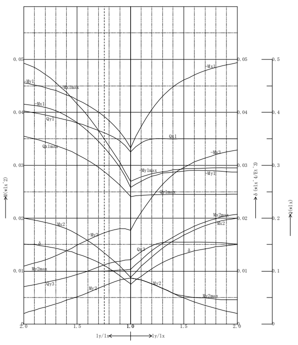
short	α	M(kN.m)
Mx1	-0.0333	-6.173
Mx1max	-0.0349	-6.482
Mx2	0.0134	2.489

long	α	M(kN.m)
My1	-0.0392	-7.274
My2	0.0072	1.344
My2max	0.0100	1.858
My3	-0.0172	-3.198

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2873	23.697

long	α	Q (kN)
Qy1	0.3613	29.801
Qy3	0.1118	9.223



Total Moment and Shear Force

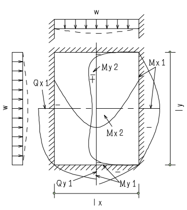
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.900	-6.482	-8.382
	Center	1.120	2.489	3.608
Vertical	Top	-1.599	-3.198	-4.797
	Center	0.796	1.858	2.654
	Bottom	-1.599	-7.274	-8.873

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	6.299	23.697	29.995
	Center	4.899	18.431	23.330
Vertical	Top	5.821	9.223	15.045
	Center	4.782	26.316	31.098
	Bottom	5.821	29.801	35.622

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.750 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.018



[1]Bending Moment

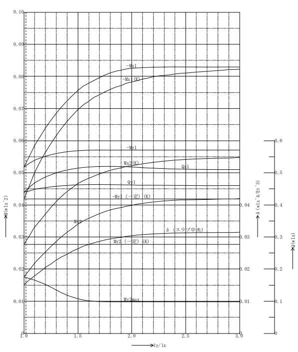
short	α	M(kN.m)
Mx1	-0.0530	-2.276
Mx2	0.0287	1.231

long	α	M(kN.m)
My1	-0.0522	-2.242
My2	0.0277	1.189
My2max	0.0174	0.748

[2]Shear Force

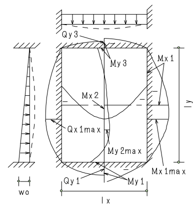
short	α	Q (kN)
Qx1	0.4447	6.940

long	α	Q (kN)
Qy1	0.4406	6.877



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 36.655 (kN/m^2)
lx : Length (short) = 2.750 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.018



[1]Bending Moment

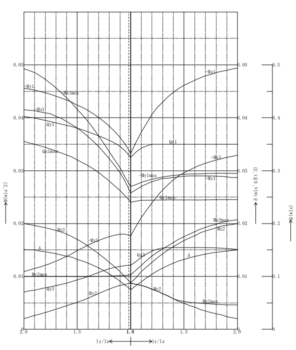
short	α	M(kN.m)
Mx1	-0.0266	-7.364
Mx1max	-0.0276	-7.656
Mx2	0.0092	2.550

long	α	M(kN.m)
My1	-0.0339	-9.402
My2	0.0086	2.392
My2max	0.0103	2.845
My3	-0.0178	-4.937

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2448	24.676

long	α	Q (kN)
Qy1	0.3297	33.231
Qy3	0.1211	12.202



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-2.276	-7.656	-9.932
	Center	1.231	2.550	3.781
Vertical	Top	-2.242	-4.937	-7.179
	Center	1.189	2.845	4.034
	Bottom	-2.242	-9.402	-11.644

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	6.940	24.676	31.616
	Center	5.678	20.190	25.868
Vertical	Top	6.877	12.202	19.079
	Center	5.649	29.175	34.824
	Bottom	6.877	33.231	40.108

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.3821	3.6085	-8.3821
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	23.3297
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.7683	1.4994	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	71.2743	53.4411	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1596
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.8728	2.6535	-8.8728
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	31.0981
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	144.0	148.0	144.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.0015	30.8838	39.0015
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.4748	1.2480	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	84.1926	42.5935	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2374
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-11.6441	4.0339	-11.6441
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	34.8235
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	144.0	146.0	144.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.0015	30.6396	39.0015
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	4.5601	1.9381	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	110.4884	65.6735	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2658
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-9.9320	3.7812	-9.9320
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	25.8680
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.2802	1.5712	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	84.4535	55.9995	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1770
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

2.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_w \cdot V_h$

$= 9.800 \times 15.750$

$= 154.350$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	2.500×3.000×1.850	13.875
3	2.500×3.000×0.250	1.875
Total	——	15.750

(b) Vertical Load

W = Wc + Wu

= 265.500 + 0.000

= 265.500 (kN)

(c) safety factor

Safety Factor F = W / U

= 265.500 / 154.350

= 1.720 ≧ Allowable Safety Factor $F_a = 1.200$

(2) Stability against Bearing

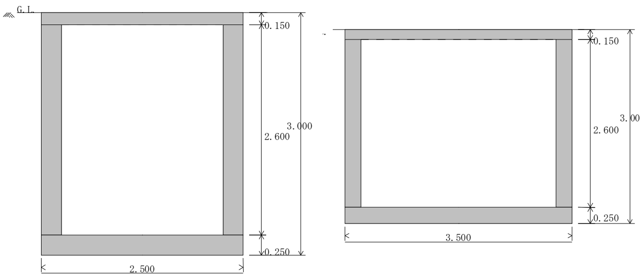
Weight of Soil Same Volume as Body

Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	2.500×3.000×0.150×18.000	20.250
2	Side Wall	2.500×3.000×2.600×18.000	351.000
3	Bottom Slab	2.500×3.000×0.250×18.000	33.750
Total Ws			405.000

Ws / Wc = 405.000 /265.500

= 1.525 ≧ 1.0

3 CASE6 (2,000 X 3,000 X H3,000 M)



Front

Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	2.500	0.000	3.500	0.000	0.150
Side Wall	2.500	2.000	3.500	3.000	2.600
Bottom Slab	2.500	0.000	3.500	0.000	0.250

3.1 Vertical Load

(1) Weight of Body

Top Slab	2.500×3.500×0.150×24.000	31.500 (kN)
Pedestrian Load	2.500×3.500×5.000	43.750 (kN)
Side Wall	(2.500×3.500 – 2.000×3.000)×2.600×24.000	171.600 (kN)
Bottom Slab	2.500×3.500×0.250×24.000	52.500 (kN)

(1) Water Pressure for Bottom Slab

$Ww = \gamma w \cdot (h - hw)$
$= 9.800 \times (3.000 - 0.900)$
$= 20.580$ (kN/m^2)

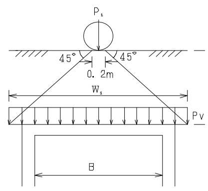
(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i)$$
$$= \frac{2 \times 8.000}{2.75} \times (1+0.300)$$
$= 7.564$ (kN/m)

(b) Vertical Load of Live Load

$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h + 0.2}$$
$$= \frac{7.564 \times 0.9}{2 \times 0.000 + 0.2}$$
$= 34.036$ (kN/m^2)



$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0 - W_l) \text{ より}$$
$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0 - W_l)$$
$$q_{vl} = \frac{34.036 \times 0.200}{2.500^2} \cdot (2 \times 2.500 - 0.200)$$
$= 5.228$ (kN/m^2)

3.2 Horizontal Load

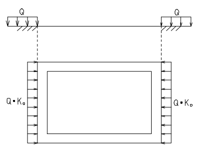
(1) Earth Pressure

Ps = $\Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$
Ps : Horizontal Earth Pressure (kN/m^2)
K_0 : Coefficient of earth pressure at rest
γt : Wet unit weight (kN/m^3)
γ' : Submerged unit weight (kN/m^3)
h : Thickness (m)
hw : Thickness inside water (m)

(2) Water Pressure

Pw = γw ・hw
Pw : Water Pressure (kN/m^2)
γw : Unit weight of water = 9.800 (kN/m^3)
hw : Depth (m)

(3) Horizontal load by live load



$P_l = Q \cdot K_0$
Q : = 10.000 (kN/m^2)

(4) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.075	Top Slab	Center	18.000	0.675	0.000	5.000	5.675
0.150	Sidewall	Top	18.000	1.350	0.000	5.000	6.350
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
2.750	Sidewall	Bottom	10.000	17.350	18.130	5.000	40.480
2.875	Bottom Slab	Center	10.000	17.975	19.355	5.000	42.330

3.3 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

W3 : Subgrade Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{75.250 + 0.000}{8.750} + 5.228$$
$= 13.828$ (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$

$Q = \alpha \cdot w \cdot lx$

ここに、

M : Bending Moment (kN.m)

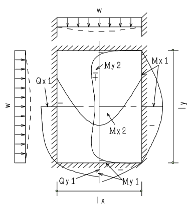
Q : Shear Force (kN)

w : Distributed Load = 13.828 (kN/m^2)

lx : Length (short) = 2.250 (m)

ly : Length (long) = 3.250 (m)

α : ly/lx = 1.444



[1]Bending Moment

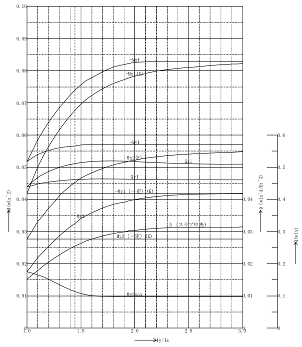
short	α	M(kN.m)
Mx1	-0.0739	-5.174
Mx2	0.0453	3.169

long	α	M(kN.m)
My1	-0.0567	-3.968
My2	0.0277	1.939
My2max	0.0114	0.796

[2]Shear Force

short	α	Q (kN)
Qx1	0.5117	15.922

long	α	Q (kN)
Qy1	0.4621	14.376



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-3.9677	1.9391	-3.9677
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.6069
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	80.0	150.0	80.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	28.8574	26.5869	28.8574
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.9065	2.3855	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	62.3096	35.0569	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1791
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

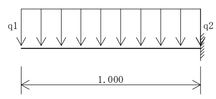
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.1741	3.1689	-5.1741
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	13.0910
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	150.0	82.0	150.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	26.1108	29.2969	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	6.6806	2.9947	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	96.4576	48.4854	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0926
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

3.4 Calculation of Open of Top Slab

(1) Design Condition

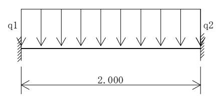
(a) Cantilever Beam

Span Length L (m)	1.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	12.253
Load q_2 (kN/m^2)	12.253



(a) Fixed Ended Beam

Span Length L (m)	2.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	5.035
Load q_2 (kN/m^2)	5.035

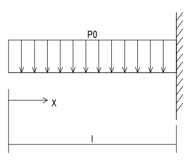


(2) Moment and Shear Force

(a) Cantilever Beam

$$Mx = -\frac{p_0 x^2}{2}$$

$$Qx = -p_0 x$$



p_0 : Effective Load = 12.253 (kN/m^2)

l : Span Length = 1.000 (m)

Mx : Bending Moment at position x (kN.m)

Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
End	1.000	-6.127
Center	0.500	-1.532

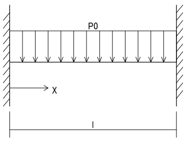
[2]Shear Force

	x (m)	Qx (kN)
End	1.000	-12.253

(b) Fixed Ended Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



p_0 : Effective Load = 5.035 (kN/m^2)
l : Span Length = 2.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
Edge	0.000	-1.678
Center	1.000	0.839

[2]Shear Force

	x (m)	Qx (kN)
Edge	0.000	5.035

(1) Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-6.1265	-1.5316
Axial Force	N	kN	——	——
Shear Force	V	kN	-12.2530	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	150.0	150.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	26.1108	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	7.9103	1.9776
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	114.2129	28.5532
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0867	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

(b) Fixed Ended Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-1.6783	0.8392
Axial Force	N	kN	——	——
Shear Force	V	kN	5.0350	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	80.0	150.0
Applied area of Reinforcement				
(Tension)	As'	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	28.8574	26.5869
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.6525	1.0323
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	26.3570	15.1711
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0715	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

3.5 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

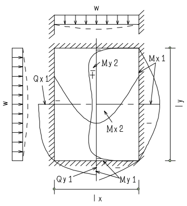
W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{246.850 + 0.000}{8.750} + 5.228$$

= 33.439 (kN/m^2)

(2) Moment and Shear Force

M = α・w・lx²
Q = α・w・lx
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 33.439 (kN/m²)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.444



[1]Bending Moment

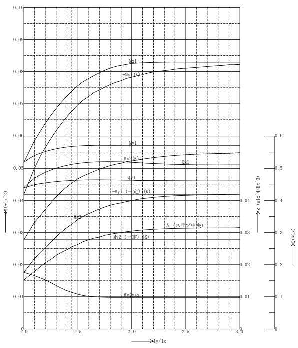
short	α	M(kN.m)
Mx1	-0.0739	-12.512
Mx2	0.0453	7.663

long	α	M(kN.m)
My1	-0.0567	-9.595
My2	0.0277	4.689
My2max	0.0114	1.924

[2]Shear Force

short	α	Q (kN)
Qx1	0.5117	38.502

long	α	Q (kN)
Qy1	0.4621	34.765



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-9.5948	4.6893	-9.5948
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	29.4168
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	130.0	148.0	130.0
Applied area of Reinforcement					
(Tension)	As	mm²	D19×4.00 1256.80	D13×4.00 452.40	——
(Compression)	As	mm²	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	44.0979	30.8838	44.0979
Concrete	f'_{ck}	N/ mm²	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm²	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm²	3.7760	2.2054	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm²	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm²	66.2009	75.2701	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm²	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2551
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

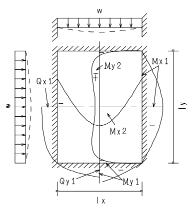
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-12.5122	7.6631	-12.5122
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	29.9461
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm²	D19×4.00 1256.80	D13×4.00 452.40	——
(Compression)	As	mm²	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	49.8962	32.2571	49.8962
Concrete	f'_{ck}	N/ mm²	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm²	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm²	3.4969	3.1842	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm²	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm²	69.4485	113.4894	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm²	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2089
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

3.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load

M = α・w・lx²
Q = α・w・lx
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m²)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.244



[1]Bending Moment

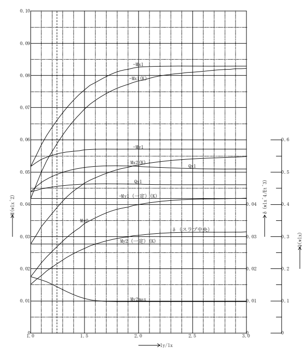
short	α	M(kN.m)
Mx1	-0.0661	-1.900
Mx2	0.0390	1.120

long	α	M(kN.m)
My1	-0.0556	-1.599
My2	0.0277	0.796
My2max	0.0145	0.416

[2]Shear Force

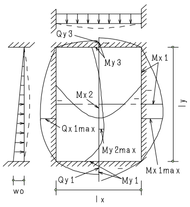
short	α	Q (kN)
Qx1	0.4933	6.299

long	α	Q (kN)
Qy1	0.4559	5.821



(b) Moment and Shear Force by Uniformly Varying Load

$M = α・w・lx^2$
$Q = α・w・lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 36.655 (kN/m^2)
lx : Length (short) =2.250 (m)
ly : Length (long) = 2.800 (m)
α : ly/lx = 1.244



[1]Bending Moment

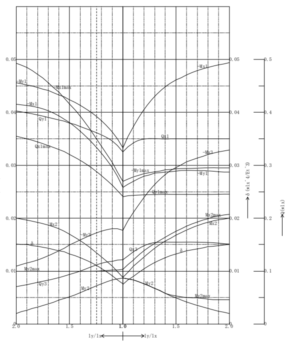
short	α	M(kN.m)
Mx1	-0.0333	-6.173
Mx1max	-0.0349	-6.482
Mx2	0.0134	2.489

long	α	M(kN.m)
My1	-0.0392	-7.274
My2	0.0072	1.344
My2max	0.0100	1.858
My3	-0.0172	-3.198

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2873	23.697

long	α	Q (kN)
Qy1	0.3613	29.801
Qy3	0.1118	9.223



Total Moment and Shear Force

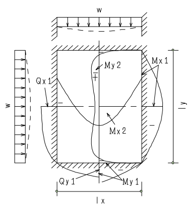
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.900	-6.482	-8.382
	Center	1.120	2.489	3.608
Vertical	Top	-1.599	-3.198	-4.797
	Center	0.796	1.858	2.654
	Bottom	-1.599	-7.274	-8.873

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	6.299	23.697	29.995
	Center	4.899	18.431	23.330
Vertical	Top	5.821	9.223	15.045
	Center	4.782	26.316	31.098
	Bottom	5.821	29.801	35.622

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

$M = α・w・lx^2$
$Q = α・w・lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.161



[1]Bending Moment

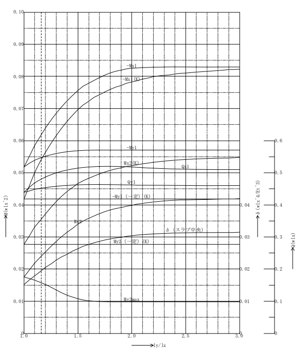
short	α	M(kN.m)
Mx1	-0.0619	-2.756
Mx2	0.0356	1.585

long	α	M(kN.m)
My1	-0.0547	-2.435
My2	0.0277	1.232
My2max	0.0158	0.701

[2]Shear Force

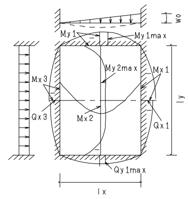
short	α	Q (kN)
Qx1	0.4805	7.634

long	α	Q (kN)
Qy1	0.4515	7.174



(b) Moment and Shear Force by Uniformly Varying Load

$M = α・w・lx^2$
$Q = α・w・lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 36.655 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.161



[1]Bending Moment

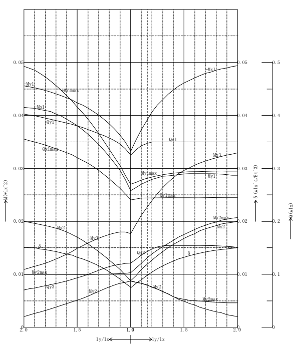
short	α	M(kN.m)
Mx1	-0.0393	-11.306
Mx2	0.0120	3.446
Mx2max	0.0132	3.785
Mx3	-0.0228	-6.552

long	α	M(kN.m)
My1	-0.0276	-7.927
My1max	-0.0282	-8.094
My2	0.0079	2.274
My2max	0.0079	2.274

[2]Shear Force

short	α	Q (kN)
Qx1	0.3475	35.668
Qx3	0.1436	14.734

long	α	Q (kN)
Qy1max	0.2440	25.039



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-2.435	-8.094	-10.529
	Center	1.232	2.274	3.507
Vertical	Top	-2.756	-6.552	-9.308
	Center	1.585	3.785	5.370
	Bottom	-2.756	-11.306	-14.062

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	7.174	25.039	32.213
	Center	6.070	21.187	27.257
Vertical	Top	7.634	14.734	22.368
	Center	6.271	31.167	37.439
	Bottom	7.634	35.668	43.302

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.8728	2.6535	-8.8728
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	31.0981
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	132.0	148.0	132.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1256.80	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	44.4946	30.8838	44.4946
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.4041	1.2480	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	60.2525	42.5935	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2654
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.3821	3.6085	-8.3821
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	23.3297
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.7683	1.4994	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	71.2743	53.4411	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1596
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-14.0618	5.3703	-14.0618
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	37.4386
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	132.0	148.0	132.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1256.80	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	44.4946	30.8838	44.4946
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	5.3949	2.5258	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	95.4891	86.2025	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.3195
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-10.5287	3.5068	-10.5287
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	27.2572
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.4772	1.4572	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	89.5272	51.9354	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1865
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

3.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_w \cdot V_h$

$= 9.800 \times 18.375$

$= 180.075$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	2.500×3.500×1.850	16.188
3	2.500×3.500×0.250	2.188
Total	——	18.375

(b) Vertical Load

W = Wc + Wu

= 299.350 + 0.000

= 299.350 (kN)

(c) safety factor

Safety Factor F = W / U

= 299.350 / 180.075

= 1.662 ≧ Allowable Safety Factor F_a = 1.200

(2) Stability against Bearing

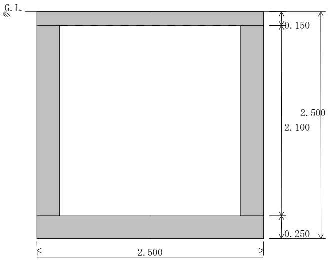
Weight of Soil Same Volume as Body

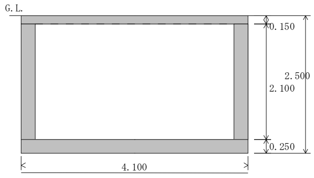
Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	2.500×3.500×0.150×18.000	23.625
2	Side Wall	2.500×3.500×2.600×18.000	409.500
3	Bottom Slab	2.500×3.500×0.250×18.000	39.375
Total Ws			472.500

Ws / Wc = 472.500 /299.350

= 1.578 ≧ 1.0

4 CASE 7 (2000 X 3,600 X H2,500 M)





Front

Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	2.500	0.000	4.100	0.000	0.150
Side Wall	2.500	2.000	4.100	3.600	2.100
Bottom Slab	2.500	0.000	4.100	0.000	0.250

4.1 Vertical Load

(1) Weight of Body

Top Slab	2.500×4.100×0.150×24.000	36.900 (kN)
Pedestrian Load	2.500×4.100×5.000	51.250 (kN)
Side Wall	(2.500×4.100 – 2.000×3.600)×2.100×24.000	153.720 (kN)
Bottom Slab	2.500×4.100×0.250×24.000	61.500 (kN)

(1) Water Pressure for Bottom Slab

$Ww = \gamma w \cdot (h - hw)$
$= 9.800\times(2.500 - 0.900)$
$= 15.680$ (kN/m^2)

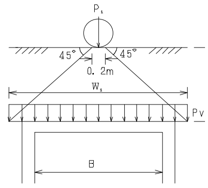
(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i)$$
$$= \frac{2\times 8.000}{2.75}\times(1+0.300)$$
$= 7.564$ (kN/m)

(b) Vertical Load of Live Load

$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h+0.2}$$
$$= \frac{7.564\times 0.9}{2\times 0.000+0.2}$$
$= 34.036$ (kN/m^2)



$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0-W_l) \text{ より}$$
$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0-W_l)$$
$$q_{vl} = \frac{34.036\times 0.200}{2.500^2} \cdot (2\times 2.500-0.200)$$
$= 5.228$ (kN/m^2)

4.2 Horizontal Load

(1) Earth Pressure

$Ps = \Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$

Ps	:	Horizontal Earth Pressure (kN/m^2)
K_0	:	Coefficient of earth pressure at rest
γt	:	Wet unit weight (kN/m^3)
γ'	:	Submerged unit weight (kN/m^3)
h	:	Thickness (m)
hw	:	Thickness inside water (m)

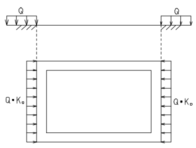
(2) Water Pressure

$Pw = \gamma w \cdot hw$

Pw	:	Water Pressure (kN/m^2)
γw	:	Unit weight of water = 9.800 (kN/m^3)
hw	:	Depth (m)

(3) Horizontal load by live load

$P_l = Q \cdot K_0$
Q : = 10.000 (kN/m^2)



(4) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.075	Top Slab	Center	18.000	0.675	0.000	5.000	5.675
0.150	Sidewall	Top	18.000	1.350	0.000	5.000	6.350
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
2.250	Sidewall	Bottom	10.000	14.850	13.230	5.000	33.080
2.375	Bottom Slab	Center	10.000	15.475	14.455	5.000	34.930

4.3 Calculation of Bottom Slab

(1) Load

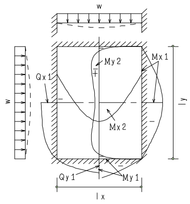
$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

W3	:	Subgrade Reaction for Bottom Slab (kN/m^2)
Wc	:	Weight of Body (kN)
Wu	:	Weight of Soil (kN)
A	:	Area (m^2)
P_{vl}	:	Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{88.150 + 0.000}{10.250} + 5.228$$
$= 13.828$ (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 13.828 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 3.850 (m)
α : ly/lx = 1.711



[1]Bending Moment

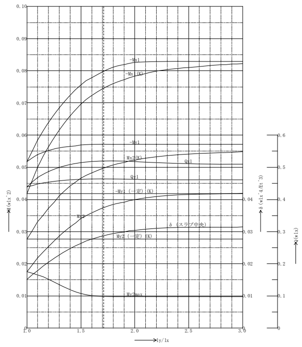
short	α	M(kN.m)
Mx1	-0.0799	-5.590
Mx2	0.0497	3.482

long	α	M(kN.m)
My1	-0.0571	-3.997
My2	0.0277	1.939
My2max	0.0099	0.691

[2]Shear Force

short	α	Q (kN)
Qx1	0.5195	16.164

long	α	Q (kN)
Qy1	0.4635	14.422



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-3.9972	1.9391	-3.9972
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	12.9238
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	80.0	150.0	80.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	28.8574	26.5869	28.8574
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.9356	2.3855	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	62.7738	35.0569	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1836
Allowable Stress Intensity by Shear Force	$\tau_{\_a1}$	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

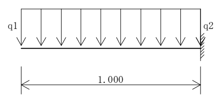
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.5902	3.4815	-5.5902
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	13.2900
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	150.0	82.0	150.0
Applied area of Reinforcement (Tension)	As	mm^2	D13×8.00 1013.60	D13×8.00 1013.60	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	27.2644	30.6152	27.2644
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	6.9617	3.1674	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	93.6128	47.8453	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0943
Allowable Stress Intensity by Shear Force	$\tau_{\_a1}$	N/mm^2	——	——	0.3600
Evaluation			——	——	○

4.4 Calculation of Open of Top Slab

(1) Design Condition

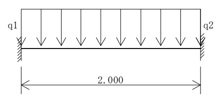
(a) Cantilever Beam

Span Length L (m)	1.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	12.253
Load q_2 (kN/m^2)	12.253



(a) Fixed Ended Beam

Span Length L (m)	2.000
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	5.228
Load q_2 (kN/m^2)	5.228

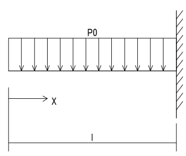


(2) Moment and Shear Force

(a) Cantilever Beam

$$Mx = -\frac{p_0 x^2}{2}$$
$$Qx = -p_0 x$$

p_0 : Effective Load = 12.253 (kN/m^2)
l : Span Length = 1.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)



[1]Bending Moment

	x (m)	Mx (kN.m)
End	1.000	-6.127
Center	0.500	-1.532

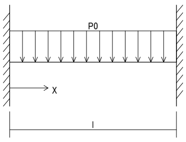
[2]Shear Force

	x (m)	Qx (kN)
End	1.000	-12.253

(b) Fixed Ended Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



p_0 : Effective Load = 5.228 (kN/m^2)
l : Span Length = 2.000 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
Edge	0.000	-1.743
Center	1.000	0.871

[2]Shear Force

	x (m)	Qx (kN)
Edge	0.000	5.228

（1） Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-6.1265	-1.5316
Axial Force	N	kN	——	——
Shear Force	V	kN	-12.2530	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	150.0	150.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	26.1108	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	7.9103	1.9776
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	114.2129	28.5532
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0867	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

(b) Fixed Ended Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-1.7427	0.8713
Axial Force	N	kN	——	——
Shear Force	V	kN	5.2280	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	80.0	150.0
Applied area of Reinforcement				
(Tension)	As'	mm^2	D13×8.00 904.80	D13×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	28.8574	26.5869
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.7158	1.0719
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	27.3673	15.7527
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0743	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

4.5 Calculation of Bottom Slab

（1） Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

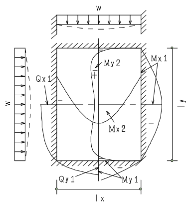
W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{241.870 + 0.000}{10.250} + 5.228$$

= 28.825 (kN/m^2)

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 28.825 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 3.850 (m)
α : ly/lx = 1.711



[1]Bending Moment

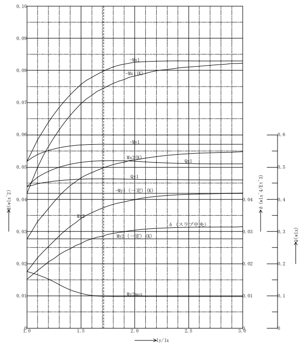
short	α	M(kN.m)
Mx1	-0.0799	-11.653
Mx2	0.0497	7.257

long	α	M(kN.m)
My1	-0.0571	-8.332
My2	0.0277	4.042
My2max	0.0099	1.441

[2]Shear Force

short	α	Q (kN)
Qx1	0.5195	33.694

long	α	Q (kN)
Qy1	0.4635	30.064



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.3324	4.0422	-8.3324
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	26.1594
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	144.0	148.0	144.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.0015	30.8838	39.0015
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.2632	1.9011	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	79.0648	64.8835	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1997
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

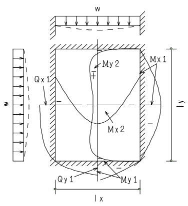
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-11.6531	7.2574	-11.6531
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	26.2061
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.8486	3.0157	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	99.0878	107.4820	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1793
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

4.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.250 (m)
ly : Length (long) = 2.300 (m)
α : ly/lx = 1.022



[1]Bending Moment

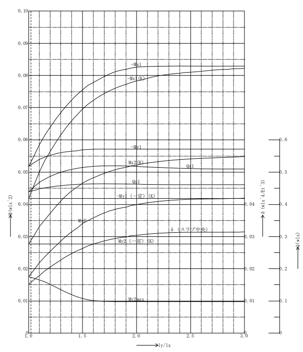
short	α	M(kN.m)
Mx1	-0.0533	-1.532
Mx2	0.0289	0.830

long	α	M(kN.m)
My1	-0.0523	-1.504
My2	0.0277	0.796
My2max	0.0174	0.499

[2]Shear Force

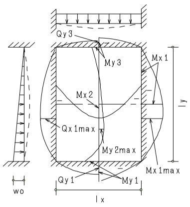
short	α	Q (kN)
Qx1	0.4460	5.695

long	α	Q (kN)
Qy1	0.4410	5.631



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 29.255 (kN/m^2)
lx : Length (short) =2.250 (m)
ly : Length (long) = 2.300 (m)
α : ly/lx = 1.022



[1]Bending Moment

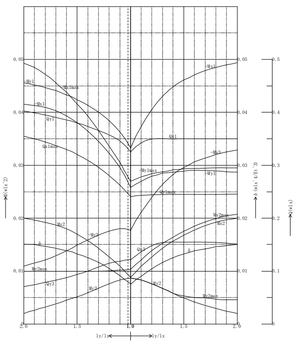
short	α	M(kN.m)
Mx1	-0.0267	-3.959
Mx1max	-0.0278	-4.111
Mx2	0.0093	1.376

long	α	M(kN.m)
My1	-0.0341	-5.044
My2	0.0086	1.275
My2max	0.0103	1.519
My3	-0.0178	-2.641

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2457	16.172

long	α	Q (kN)
Qy1	0.3307	21.766
Qy3	0.1210	7.966



Total Moment and Shear Force

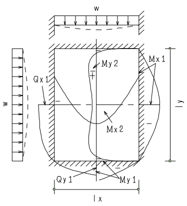
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.532	-4.111	-5.642
	Center	0.830	1.376	2.206
Vertical	Top	-1.504	-2.641	-4.145
	Center	0.796	1.519	2.315
	Bottom	-1.504	-5.044	-6.547

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	5.695	16.172	21.867
	Center	4.429	12.578	17.007
Vertical	Top	5.631	7.966	13.597
	Center	4.407	18.534	22.941
	Bottom	5.631	21.766	27.397

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.300 (m)
ly : Length (long) = 3.850 (m)
α : ly/lx = 1.674



[1]Bending Moment

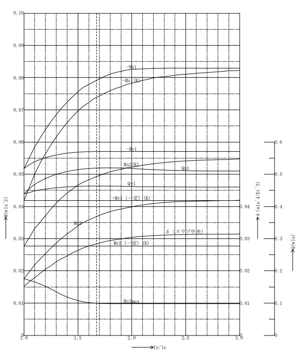
short	α	M(kN.m)
Mx1	-0.0792	-2.379
Mx2	0.0493	1.480

long	α	M(kN.m)
My1	-0.0571	-1.714
My2	0.0277	0.832
My2max	0.0100	0.299

[2]Shear Force

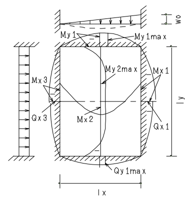
short	α	Q (kN)
Qx1	0.5192	6.777

long	α	Q (kN)
Qy1	0.4635	6.050



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 29.255 (kN/m^2)
lx : Length (short) = 2.300 (m)
ly : Length (long) = 3.850 (m)
α : ly/lx = 1.674



[1]Bending Moment

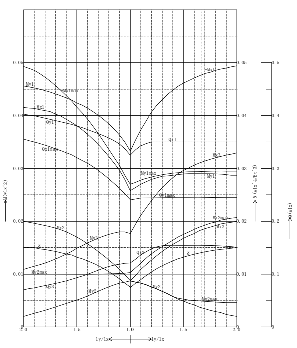
short	α	M(kN.m)
Mx1	-0.0477	-7.381
Mx2	0.0184	2.846
Mx2max	0.0191	2.955
Mx3	-0.0312	-4.827

long	α	M(kN.m)
My1	-0.0290	-4.488
My1max	-0.0294	-4.557
My2	0.0037	0.566
My2max	0.0049	0.758

[2]Shear Force

short	α	Q (kN)
Qx1	0.3500	23.550
Qx3	0.1545	10.396

long	α	Q (kN)
Qy1max	0.2449	16.478



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.714	-4.557	-6.271
	Center	0.832	0.758	1.590
Vertical	Top	-2.379	-4.827	-7.206
	Center	1.480	2.955	4.434
	Bottom	-2.379	-7.381	-9.759

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	6.050	16.478	22.528
	Center	5.264	14.338	19.603
Vertical	Top	6.777	10.396	17.173
	Center	5.304	19.860	25.165
	Bottom	6.777	23.550	30.328

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-6.5473	2.3147	-6.5473
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	22.9409
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	148.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D13×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3066	30.8838	39.3066
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.5070	1.0886	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	61.2435	37.1546	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.1726
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.3600
Evaluation			—	—	○

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.6423	2.2060	-5.6423
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	17.0075
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D13×4.00 452.40	D13×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	32.2571	32.2571	32.2571
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.3445	0.9167	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	83.5621	32.6708	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.1140
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.3600
Evaluation			—	—	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-9.7592	4.4342	-9.7592
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	25.1646
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	146.0	148.0	146.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 805.60	D13×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	39.3372	30.8838	39.3372
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.7352	2.0855	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	91.1530	71.1758	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.1894
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.3600
Evaluation			—	—	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-6.2715	1.5899	-6.2715
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	19.6025
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D13×4.00 452.40	D13×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	32.2571	32.2571	32.2571
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.6060	0.6606	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	92.8804	23.5462	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1313
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

4.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_w \cdot V_h$

$= 9.800 \times 16.400$

$= 160.720$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	2.500×4.100×1.350	13.838
3	2.500×4.100×0.250	2.563
Total	——	16.400

(b) Vertical Load

W = Wc + Wu

= 303.370 + 0.000

= 303.370 (kN)

(c) safety factor

Safety Factor F = W / U

= 303.370 / 160.720

= 1.888 ≧ Allowable Safety Factor $F_a = 1.200$

(2) Stability against Bearing

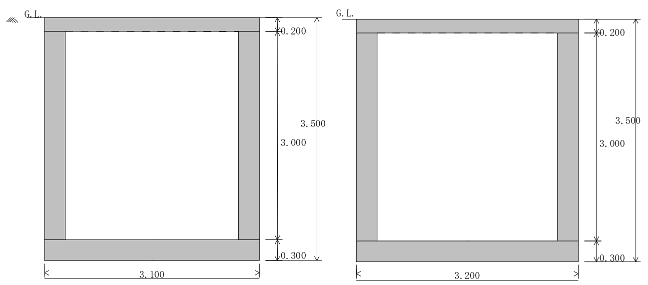
Weight of Soil Same Volume as Body

Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	2.500×4.100×0.150×18.000	27.675
2	Side Wall	2.500×4.100×2.100×18.000	387.450
3	Bottom Slab	2.500×4.100×0.250×18.000	46.125
Total Ws			461.250

Ws / Wc = 461.250 /303.370

= 1.520 ≧ 1.0

5 CASE 8 (2,500 X 2,600 X H3,500 M)



Front Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	3.100	0.000	3.200	0.000	0.200
Side Wall	3.100	2.500	3.200	2.600	3.000
Bottom Slab	3.100	0.000	3.200	0.000	0.300

5.1 Vertical Load

(1) Weight of Body

Top Slab	3.100×3.200×0.200×24.000	47.616 (kN)
Pedestrian Load	3.100×3.200×5.000	49.600 (kN)
Side Wall	(3.100×3.200 - 2.500×2.600)×3.000×24.000	246.240 (kN)
Bottom Slab	3.100×3.200×0.300×24.000	71.424 (kN)

(1) Water Pressure for Bottom Slab

$$Ww = \gamma w \cdot (h - hw)$$
$$= 9.800 \times (3.500 - 0.900)$$
$$= 25.480 \text{ (kN/m}^2)$$

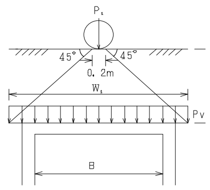
(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i)$$
$$= \frac{2 \times 8.000}{2.75} \times (1+0.300)$$
$$= 7.564 \text{ (kN/m)}$$

(b) Vertical Load of Live Load

$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h + 0.2}$$
$$= \frac{7.564 \times 0.9}{2 \times 0.000 + 0.2}$$
$$= 34.036 \text{ (kN/m}^2)$$



$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0 - W_l) \text{ より}$$

$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0 - W_l)$$

$$q_{vl} = \frac{34.036 \times 0.200}{3.100^2} \cdot (2 \times 3.100 - 0.200)$$

$$= 4.250 \text{ (kN/m}^2)$$

5.2 Horizontal Load

(1) Earth Pressure

$Ps = \Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$

- Ps : Horizontal Earth Pressure (kN/m^2)
- K_0 : Coefficient of earth pressure at rest
- γt : Wet unit weight (kN/m^3)
- γ' : Submerged unit weight (kN/m^3)
- h : Thickness (m)
- hw : Thickness inside water (m)

(2) Water Pressure

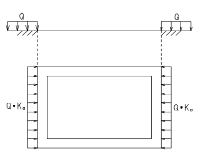
$Pw = \gamma w \cdot hw$

- Pw : Water Pressure (kN/m^2)
- γw : Unit weight of water = 9.800 (kN/m^3)
- hw : Depth (m)

(3) Horizontal load by live load

$P_l = Q \cdot K_0$

Q : = 10.000 (kN/m^2)



(4) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.100	Top Slab	Center	18.000	0.900	0.000	5.000	5.900
0.200	Sidewall	Top	18.000	1.800	0.000	5.000	6.800
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
3.200	Sidewall	Bottom	10.000	19.600	22.540	5.000	47.140
3.350	Bottom Slab	Center	10.000	20.350	24.010	5.000	49.360

5.3 Calculation of Bottom Slab

(1) Load

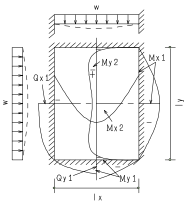
$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

- W3 : Subgrade Reaction for Bottom Slab (kN/m^2)
- Wc : Weight of Body (kN)
- Wu : Weight of Soil (kN)
- A : Area (m^2)
- P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{97.216 + 0.000}{9.920} + 4.250$$
$$= 14.050 \text{ (kN/m}^2)$$

(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 14.050 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 2.900 (m)
α : ly/lx = 1.036



[1]Bending Moment

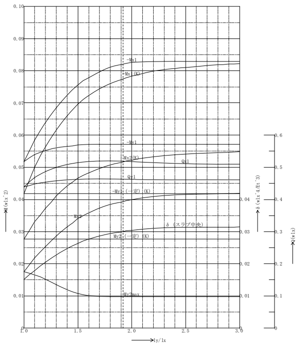
short	α	M(kN.m)
Mx1	-0.0542	-5.973
Mx2	0.0296	3.264

long	α	M(kN.m)
My1	-0.0527	-5.800
My2	0.0277	3.051
My2max	0.0172	1.899

[2]Shear Force

short	α	Q (kN)
Qx1	0.4502	17.712

long	α	Q (kN)
Qy1	0.4422	17.397



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.8003	3.0512	-5.8003
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	14.3974
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	200.0	200.0	200.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	100.0	100.0	100.0
Applied area of Reinforcement (Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	33.0322	33.0322	33.0322
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.9477	2.0767	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	72.0307	37.8913	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1618
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

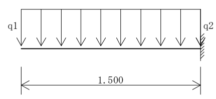
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.9734	3.2637	-5.9734
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	14.5488
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	200.0	200.0	200.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	200.0	112.0	200.0
Applied area of Reinforcement (Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	30.5908	35.3271	30.5908
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	5.0228	1.8427	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	84.8353	35.9948	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0767
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

5.4 Calculation of Open of Top Slab

(1) Design Condition

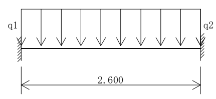
(a) Cantilever Beam

Span Length L (m)	1.500
Member Thickness t (m)	0.200
Load q_1 (kN/m^2)	8.471
Load q_2 (kN/m^2)	8.471



(b) Fixed Ended Beam

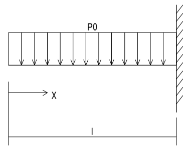
Span Length L (m)	2.600
Member Thickness t (m)	0.200
Load q_1 (kN/m^2)	4.250
Load q_2 (kN/m^2)	4.250



(2) Moment and Shear Force

(a) Cantilever Beam

$$M_x = -\frac{p_0 x^2}{2}$$
$$Q_x = -p_0 x$$



p_0 : Effective Load = 8.471 (kN/m^2)
l : Span Length = 1.500 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
End	1.500	-9.530
Center	0.750	-2.382

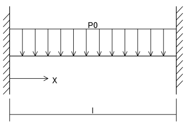
[2]Shear Force

	x (m)	Qx (kN)
End	1.500	-12.707

(b) Fixed Ended Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



p_0 : Effective Load = 4.250 (kN/m^2)
l : Span Length = 2.600 (m)
Mx : Bending Moment at position x (kN.m)
Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
Edge	0.000	-2.394
Center	1.300	1.197

[2]Shear Force

	x (m)	Qx (kN)
Edge	0.000	5.525

（1） Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-9.5299	-2.3825
Axial Force	N	kN	——	——
Shear Force	V	kN	-12.7065	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	200.0	200.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	100.0	100.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	33.0322	33.0322
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	6.4861	1.6215
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	118.3454	29.5863
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.1428	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

(a) Fixed Ended Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-2.3942	1.1971
Axial Force	N	kN	——	——
Shear Force	V	kN	5.5250	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	200.0	200.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	100.0	100.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	33.0322	33.0322
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.6295	0.8147
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	29.7316	14.8658
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0621	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

5.5 Calculation of Bottom Slab

（1） Load

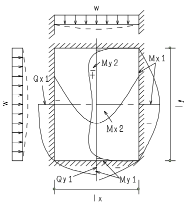
$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{343.456 + 0.000}{9.920} + 4.250$$

= 38.873 (kN/m^2)

(2) Moment and Shear Force



$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 38.873 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 2.900 (m)
α : ly/lx = 1.036

[1]Bending Moment

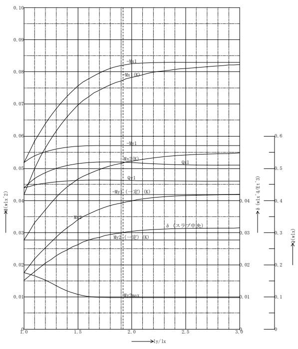
short	α	M(kN.m)
Mx1	-0.0542	-16.527
Mx2	0.0296	9.030

long	α	M(kN.m)
My1	-0.0527	-16.048
My2	0.0277	8.442
My2max	0.0172	5.255

[2]Shear Force

short	α	Q (kN)
Qx1	0.4502	49.003

long	α	Q (kN)
Qy1	0.4422	48.132



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-16.0479	8.4419	-16.0479
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	38.1738
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	194.0	198.0	194.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	46.2524	36.2915	46.2524
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.8858	2.5029	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	111.7137	100.3740	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2138
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

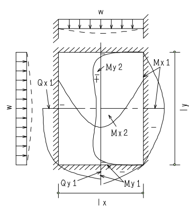
(b) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-16.5268	9.0297	-16.5268
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	38.5023
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	210.0	210.0	210.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	48.3765	37.4817	48.3765
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.5244	2.4395	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	105.9724	101.0572	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1986
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

5.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load



$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.900 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.161

[1]Bending Moment

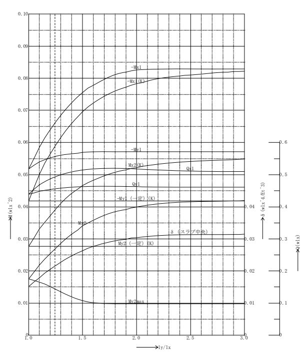
short	α	M(kN.m)
Mx1	-0.0619	-2.865
Mx2	0.0356	1.648

long	α	M(kN.m)
My1	-0.0547	-2.532
My2	0.0277	1.281
My2max	0.0157	0.729

[2]Shear Force

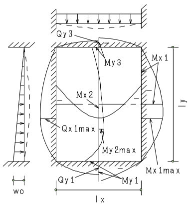
short	α	Q (kN)
Qx1	0.4805	7.937

long	α	Q (kN)
Qy1	0.4515	7.458



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 43.460 (kN/m^2)
lx : Length (short) =2.800 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.161



[1]Bending Moment

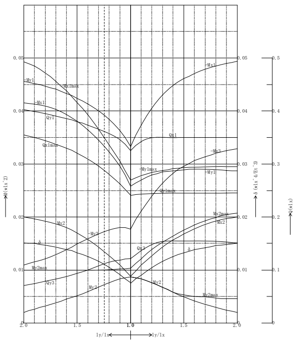
short	α	M(kN.m)
Mx1	-0.0313	-10.650
Mx1max	-0.0323	-11.020
Mx2	0.0120	4.086

long	α	M(kN.m)
My1	-0.0376	-12.816
My2	0.0079	2.697
My2max	0.0101	3.441
My3	-0.0177	-6.043

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2732	33.239

long	α	Q (kN)
Qy1	0.3536	43.028
Qy3	0.1162	14.144



Total Moment and Shear Force

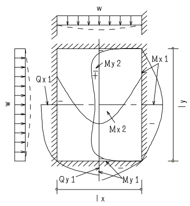
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-2.865	-11.020	-13.885
	Center	1.648	4.086	5.734
Vertical	Top	-2.532	-6.043	-8.575
	Center	1.281	3.441	4.723
	Bottom	-2.532	-12.816	-15.348

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	7.937	33.239	41.176
	Center	6.236	26.116	32.353
Vertical	Top	7.458	14.144	21.602
	Center	6.081	37.751	43.832
	Bottom	7.458	43.028	50.486

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.900 (kN/m^2)
lx : Length (short) = 2.900 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.121



[1]Bending Moment

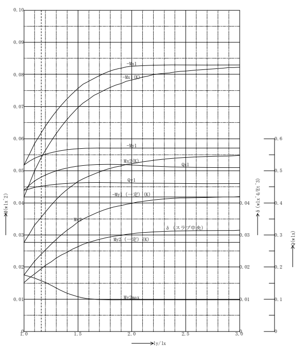
short	α	M(kN.m)
Mx1	-0.0596	-2.959
Mx2	0.0340	1.686

long	α	M(kN.m)
My1	-0.0542	-2.689
My2	0.0277	1.374
My2max	0.0163	0.806

[2]Shear Force

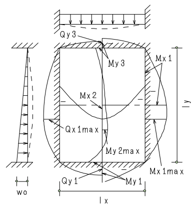
short	α	Q (kN)
Qx1	0.4725	8.084

long	α	Q (kN)
Qy1	0.4493	7.688



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 43.460 (kN/m^2)
lx : Length (short) = 2.900 (m)
ly : Length (long) = 3.250 (m)
α : ly/lx = 1.121



[1]Bending Moment

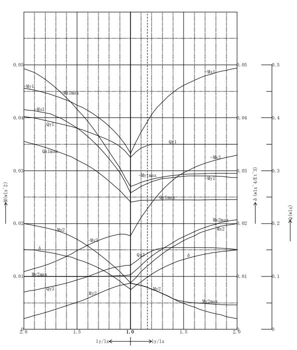
short	α	M(kN.m)
Mx1	-0.0302	-11.030
Mx1max	-0.0311	-11.375
Mx2	0.0113	4.120

long	α	M(kN.m)
My1	-0.0368	-13.434
My2	0.0082	2.988
My2max	0.0102	3.713
My3	-0.0179	-6.549

[2]Shear Force

short	α	Q (kN)
Qx1max	0.2659	33.508

long	α	Q (kN)
Qy1	0.3486	43.932
Qy3	0.1178	14.849



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-2.959	-11.375	-14.334
	Center	1.686	4.120	5.806
Vertical	Top	-2.689	-6.549	-9.238
	Center	1.374	3.713	5.087
	Bottom	-2.689	-13.434	-16.123

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	8.084	33.508	41.592
	Center	6.412	26.575	32.987
Vertical	Top	7.688	14.849	22.536
	Center	6.269	38.506	44.775
	Bottom	7.688	43.932	51.620

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-15.3477	4.7226	-15.3477
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	43.8321
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	194.0	198.0	194.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	46.2524	36.2915	46.2524
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.7162	1.4002	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	106.8395	56.1519	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.2454
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.4200
Evaluation			—	—	○

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-13.8850	5.7343	-13.8850
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	32.3528
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	210.0	210.0	210.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	48.3765	37.4817	48.3765
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.9610	1.5492	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	89.0325	64.1767	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.1669
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.4200
Evaluation			—	—	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-16.1228	5.0874	-16.1228
Axial Force	N	kN	—	—	—
Shear Force	V	kN	—	—	44.7746
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	194.0	198.0	194.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	—
(Compression)	As	mm^2	0.00	0.00	—
Young's modulus	n		9	9	9
Neutral Axis	X	mm	46.2524	36.2915	46.2524
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.9039	1.5084	—
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	—
Evaluation of Compression			○	○	—
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	112.2351	60.4891	—
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	—
Evaluation of Compression			○	○	—
Stress Intensity by Shear Force	τ	N/mm2	—	—	0.2507
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	—	—	0.4200
Evaluation			—	—	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-14.3339	5.8056	-14.3339
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	32.9870
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	210.0	210.0	210.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	48.3765	37.4817	48.3765
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	275.0	275.0	275.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.0567	1.5685	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	91.9113	64.9740	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1701
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.4200
Evaluation			——	——	○

5.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_W \cdot V_h$

$= 9.800 \times 25.792$

$= 252.762$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	3.100×3.200×2.300	22.816
3	3.100×3.200×0.300	2.976
Total	——	25.792

(b) Vertical Load

W = Wc + Wu

= 414.880 + 0.000

= 414.880 (kN)

(c) safety factor

Safety Factor F = W / U

= 414.880 / 252.762

= 1.641 ≧ Allowable Safety Factor F_a = 1.200

(2) Stability against Bearing

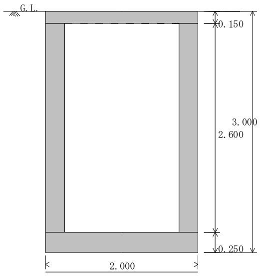
Weight of Soil Same Volume as Body

Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	3.100×3.200×0.200×18.000	35.712
2	Side Wall	3.100×3.200×3.000×18.000	535.680
3	Bottom Slab	3.100×3.200×0.300×18.000	53.568
Total Ws			624.960

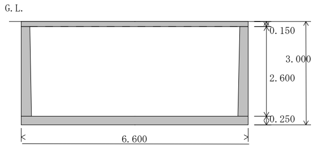
Ws / Wc = 624.960 /414.880

= 1.506 ≧ 1.0

6 CASE 9 (1,500 X 6,100 X H3,000 M)



Front



Side

Member	Front		Side		Height (m)
	Outside (m)	Inside (m)	Outside (m)	Inside (m)	
Top Slab	2.000	0.000	6.600	0.000	0.150
Side Wall	2.000	1.500	6.600	6.100	2.600
Bottom Slab	2.000	0.000	6.600	0.000	0.250

6.1 Vertical Load

(1) Weight of Body

Top Slab	2.000×6.600×0.150×24.000	47.520 (kN)
Pedestrian Load	2.000×6.600×5.000	66.000 (kN)
Side Wall	10.725×24.000	257.400 (kN)
Bottom Slab	2.000×6.600×0.250×24.000	79.200 (kN)

(1) Water Pressure for Bottom Slab

$$Ww = \gamma w \cdot (h - hw) = 9.800\times(3.000 - 0.900) = 20.580\ (kN/m^2)$$

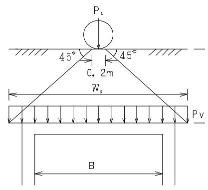
(2) Live Load

(a) Vehicle

$$P_l = \frac{2 \cdot P}{2.75} \cdot (1+i) = \frac{2\times 8.000}{2.75}\times(1+0.300) = 7.564\ (kN/m)$$

(b) Vertical Load of Live Load

$$P_{vl} = \frac{P_l \cdot \beta}{W_l} = \frac{P_l \cdot \beta}{2 \cdot h+0.2} = \frac{7.564\times 0.9}{2\times 0.000+0.2} = 34.036\ (kN/m^2)$$



$$\frac{1}{8} \cdot q_{vl} \cdot B_0^2 = \frac{1}{8} \cdot P_{vl} \cdot W_l \cdot (2B_0 - W_l) \text{ より}$$

$$q_{vl} = \frac{P_{vl} \cdot W_l}{B_0^2} \cdot (2B_0 - W_l)$$

$$q_{vl} = \frac{34.036\times 0.200}{2.000^2} \cdot (2\times 2.000 - 0.200) = 6.467\ (kN/m^2)$$

6.2 Horizontal Load

(1) Earth Pressure

$Ps = \Sigma K_0 \cdot \gamma t \cdot (h - hw) + \Sigma K_0 \cdot \gamma' \cdot hw$

- Ps : Horizontal Earth Pressure (kN/m^2)
- K_0 : Coefficient of earth pressure at rest
- γt : Wet unit weight (kN/m^3)
- γ' : Submerged unit weight (kN/m^3)
- h : Thickness (m)
- hw : Thickness inside water (m)

(2) Water Pressure

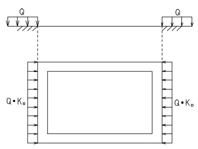
$Pw = \gamma w \cdot hw$

- Pw : Water Pressure (kN/m^2)
- γw : Unit weight of water = 9.800 (kN/m^3)
- hw : Depth (m)

(3) Horizontal load by live load

$P_l = Q \cdot K_0$

Q : = 10.000 (kN/m^2)



(4) Total of Horizontal Load

Depth (m)	Member	Location	Unit weight of soil	Ps (kN/m^2)	Pw (kN/m^2)	P_l (kN/m^2)	Total (kN/m^2)
0.075	Top Slab	Center	18.000	0.675	0.000	5.000	5.675
0.150	Sidewall	Top	18.000	1.350	0.000	5.000	6.350
0.900	Sidewall	Water surface	18.000	8.100	0.000	5.000	13.100
2.750	Sidewall	Bottom	10.000	17.350	18.130	5.000	40.480
2.875	Bottom Slab	Center	10.000	17.975	19.355	5.000	42.330

6.3 Calculation of Bottom Slab

(1) Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

- W3 : Subgrade Reaction for Bottom Slab (kN/m^2)
- Wc : Weight of Body (kN)
- Wu : Weight of Soil (kN)
- A : Area (m^2)
- P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W1 = \frac{113.520 + 0.000}{13.200} + 6.467 = 15.067\ (kN/m^2)$$

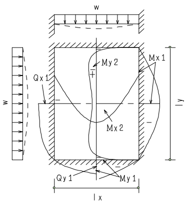
(2) Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$

$Q = \alpha \cdot w \cdot lx$

ここに、

- M : Bending Moment (kN.m)
- Q : Shear Force (kN)
- w : Distributed Load = 15.067 (kN/m^2)
- lx : Length (short) = 1.750 (m)
- ly : Length (long) = 6.350 (m)
- α : ly/lx = 3.629

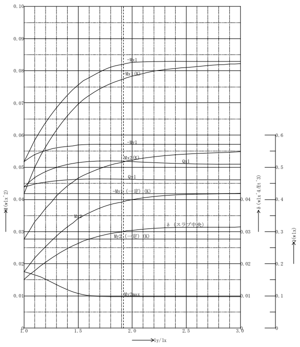


[1]Bending Moment

short	α	M(kN.m)
Mx1	-0.0829	-3.825
Mx2	0.0561	2.587
long	**α**	**M(kN.m)**
My1	-0.0571	-2.635
My2	0.0277	1.278
My2max	0.0098	0.452

[2]Shear Force

short	α	Q (kN)
Qx1	0.5097	13.439
long	**α**	**Q (kN)**
Qy1	0.4610	12.155



(3) Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-2.6347	1.2781	-2.6347
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	11.3895
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	80.0	150.0	80.0
Applied area of Reinforcement (Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	28.8574	26.5869	28.8574
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.5941	1.5724	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	41.3766	23.1074	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.1618
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

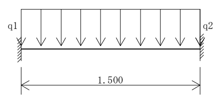
Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-3.8252	2.5866	-3.8252
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	10.3675
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	150.0	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	150.0	82.0	150.0
Applied area of Reinforcement (Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	26.1108	29.2969	26.1108
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	4.9389	2.4445	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	71.3112	39.5767	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm^2	——	——	0.0734
Allowable Stress Intensity by Shear Force	τ_{a1}	N/mm^2	——	——	0.3600
Evaluation			——	——	○

6.4 Calculation of Open of Top Slab

(1) Design Condition

(a) Cantilever Beam

Span Length L (m)	1.500
Member Thickness t (m)	0.150
Load q_1 (kN/m^2)	6.467
Load q_2 (kN/m^2)	6.467

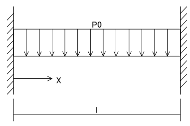


(2) Moment and Shear Force

(a) Cantilever Beam

$$Mx = \frac{p_0 l^2}{2}\left\{-\frac{1}{6} + \frac{x}{l} - \left(\frac{x}{l}\right)^2\right\}$$

$$Qx = \frac{p_0 l}{2} - p_0 x$$



- p_0 : Effective Load = 6.467 (kN/m^2)
- l : Span Length = 1.500 (m)
- Mx : Bending Moment at position x (kN.m)
- Qx : Shear Force at position x (kN)

[1]Bending Moment

	x (m)	Mx (kN.m)
End	0.000	-1.213
Center	0.750	0.606

[2]Shear Force

	x (m)	Qx (kN)
End	0.000	4.850

（1） Calculation Results

(a) Cantilever Beam

Item		Unit	Edge	Center
Bending Moment	M	kN.m	-1.2126	0.6063
Axial Force	N	kN	——	——
Shear Force	V	kN	4.8502	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	150.0	150.0
Effective Width	b_w	mm	1000.0	1000.0
Effective Height	d	mm	80.0	150.0
Applied area of Reinforcement				
(Tension)	As	mm^2	D12×8.00 904.80	D12×8.00 904.80
(Compression)	As	mm^2	0.00	0.00
Young's modulus	n		9	9
Neutral Axis	X	mm	28.8574	26.5869
Concrete	f'_{ck}	N/ mm^2	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.1939	0.7458
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	19.0424	10.9609
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm2	0.0689	——
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	0.3600	——
Evaluation			○	——

6.5 Calculation of Bottom Slab

（1） Load

$$W3 = \frac{Wc + Wu}{A} + P_{vl}$$

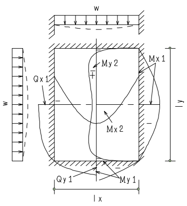
W3 : Subgrage Reaction for Bottom Slab (kN/m^2)
Wc : Weight of Body (kN)
Wu : Weight of Soil (kN)
A : Area (m^2)
P_{vl} : Vertical Load by by Live Load (kN/m^2)

$$W3 = \frac{370.920 + 0.000}{13.200} + 6.467$$

$= 34.567$ (kN/m^2)

（2） Moment and Shear Force

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 34.567 (kN/m^2)
lx : Length (short) = 1.750 (m)
ly : Length (long) = 6.350 (m)
α : ly/lx = 3.629



[1]Bending Moment

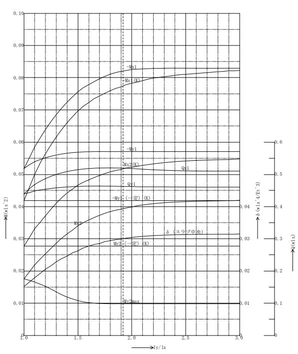
short	α	M(kN.m)
Mx1	-0.0829	-8.776
Mx2	0.0561	5.934

long	α	M(kN.m)
My1	-0.0571	-6.045
My2	0.0277	2.932
My2max	0.0098	1.037

[2]Shear Force

short	α	Q (kN)
Qx1	0.5097	30.833

long	α	Q (kN)
Qy1	0.4610	27.887



（3） Calculation Result of Bottom Slab

(a) Back and Forth

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-6.0447	2.9324	-6.0447
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	25.6910
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	130.0	148.0	130.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1256.80	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	44.0979	30.8838	44.0979
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.3789	1.3791	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	41.7061	47.0691	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2228
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-8.7759	5.9343	-8.7759
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	22.0234
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1256.80	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	49.8962	32.2571	49.8962
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	2.4527	2.4658	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	48.7103	87.8862	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1536
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

6.6 Calculation of Side Wall

(1) Back and Forth

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$

$Q = \alpha \cdot w \cdot lx$

ここに、

M : Bending Moment (kN.m)

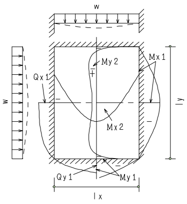
Q : Shear Force (kN)

w : Distributed Load = 5.675 (kN/m^2)

lx : Length (short) = 1.750 (m)

ly : Length (long) = 2.800 (m)

α : ly/lx = 1.600



[1]Bending Moment

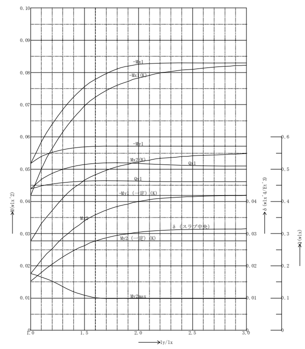
short	α	M(kN.m)
Mx1	-0.0779	-1.354
Mx2	0.0483	0.839

long	α	M(kN.m)
My1	-0.0571	-0.992
My2	0.0277	0.481
My2max	0.0101	0.176

[2]Shear Force

short	α	Q (kN)
Qx1	0.5178	5.142

long	α	Q (kN)
Qy1	0.4632	4.600



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$

$Q = \alpha \cdot w \cdot lx$

ここに、

M : Bending Moment (kN.m)

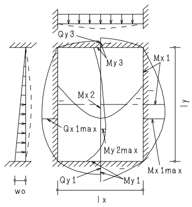
Q : Shear Force (kN)

w : Distributed Load = 36.655 (kN/m^2)

lx : Length (short) =1.750 (m)

ly : Length (long) = 2.800 (m)

α : ly/lx = 1.600



[1]Bending Moment

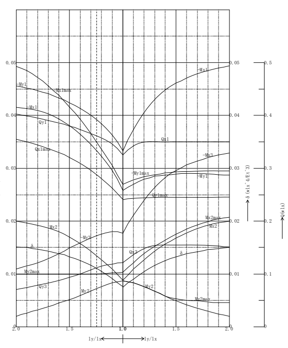
short	α	M(kN.m)
Mx1	-0.0393	-4.412
Mx1max	-0.0438	-4.917
Mx2	0.0180	2.021

long	α	M(kN.m)
My1	-0.0433	-4.861
My2	0.0045	0.505
My2max	0.0100	1.123
My3	-0.0138	-1.549

[2]Shear Force

short	α	Q (kN)
Qx1max	0.3294	21.130

long	α	Q (kN)
Qy1	0.3854	24.722
Qy3	0.0876	5.619



Total Moment and Shear Force

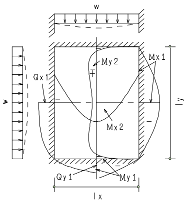
		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-1.354	-4.917	-6.271
	Center	0.839	2.021	2.860
Vertical	Top	-0.992	-1.549	-2.542
	Center	0.481	1.123	1.604
	Bottom	-0.992	-4.861	-5.853

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	5.142	21.130	26.272
	Center	3.673	15.093	18.766
Vertical	Top	4.600	5.619	10.219
	Center	3.779	22.013	25.792
	Bottom	4.600	24.722	29.322

(2) Left and Right

(a) Moment and Shear Force by Uniformly Distributed Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 5.675 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 6.350 (m)
α : ly/lx = 2.268



[1]Bending Moment

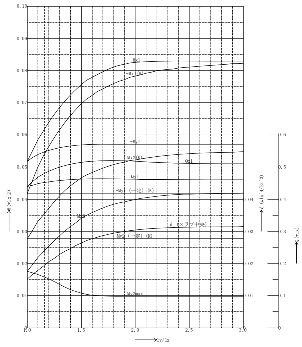
short	α	M(kN.m)
Mx1	-0.0829	-3.688
Mx2	0.0535	2.382

long	α	M(kN.m)
My1	-0.0571	-2.540
My2	0.0277	1.232
My2max	0.0098	0.436

[2]Shear Force

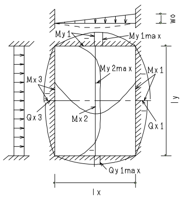
short	α	Q (kN)
Qx1	0.5135	8.159

long	α	Q (kN)
Qy1	0.4615	7.334



(b) Moment and Shear Force by Uniformly Varying Load

$M = \alpha \cdot w \cdot lx^2$
$Q = \alpha \cdot w \cdot lx$
ここに、
M : Bending Moment (kN.m)
Q : Shear Force (kN)
w : Distributed Load = 36.655 (kN/m^2)
lx : Length (short) = 2.800 (m)
ly : Length (long) = 6.350 (m)
α : ly/lx = 2.268



[1]Bending Moment

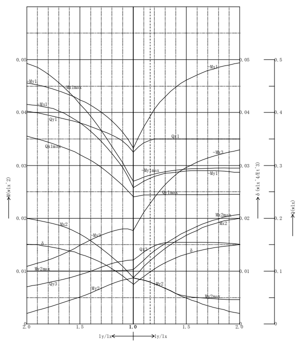
short	α	M(kN.m)
Mx1	-0.0505	-14.504
Mx2	0.0211	6.055
Mx2max	0.0212	6.103
Mx3	-0.0340	-9.763

long	α	M(kN.m)
My1	-0.0287	-8.248
My1max	-0.0295	-8.478
My2	0.0009	0.267
My2max	0.0046	1.322

[2]Shear Force

short	α	Q (kN)
Qx1	0.3500	35.922
Qx3	0.1461	14.993

long	α	Q (kN)
Qy1	0.2452	25.166



(c) Total Moment and Shear Force

		Bending Moment (kN.m)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	-2.540	-8.478	-11.018
	Center	1.232	1.322	2.554
Vertical	Top	-3.688	-9.763	-13.451
	Center	2.382	6.103	8.485
	Bottom	-3.688	-14.504	-18.193

		Shear Force (kN)		
		Uniformly Distributed Load	Uniformly Varying Load	Total
Horizontal	Edge	7.334	25.166	32.500
	Center	6.756	23.184	29.941
Vertical	Top	8.159	14.993	23.151
	Center	6.702	31.376	38.078
	Bottom	8.159	35.922	44.081

(3) Calculation Result of Side Slab

(a) Back and Forth (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-5.8531	1.6040	-5.8531
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	25.7916
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	182.0	198.0	182.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1256.80	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	53.8330	36.2915	53.8330
Concrete	f'_{ck}	N/ mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/ mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.3250	0.4756	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	28.3905	19.0712	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.1572
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(b) Back and Forth (Horizontal)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-6.2707	2.8600	-6.2707
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	18.7658
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	300.0	300.0	300.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	210.0	210.0	210.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	48.3765	37.4817	48.3765
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	1.3372	0.7727	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	40.2086	32.0088	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.0968
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(c) Right and Left (Vertical)

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-18.1926	8.4845	-18.1926
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	38.0779
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	132.0	148.0	132.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D19×4.00 1209.19	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	43.8232	30.8838	43.8232
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	7.0761	3.9904	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	128.1412	136.1905	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.3244
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

(d) Right and Left

Item		Unit	Edge	Center	h/2
Bending Moment	M	kN.m	-11.0181	2.5544	-11.0181
Axial Force	N	kN	——	——	——
Shear Force	V	kN	——	——	29.9405
Width of Member	B	mm	1000.0	1000.0	1000.0
Height of Member	H	mm	250.0	250.0	250.0
Effective Width	b_w	mm	1000.0	1000.0	1000.0
Effective Height	d	mm	160.0	160.0	160.0
Applied area of Reinforcement					
(Tension)	As	mm^2	D16×4.00 804.40	D12×4.00 452.40	——
(Compression)	As	mm^2	0.00	0.00	——
Young's modulus	n		9	9	9
Neutral Axis	X	mm	41.4429	32.2571	41.4429
Concrete	f'_{ck}	N/mm^2	20.7	20.7	20.7
Reinforcement Bar	f_{yk}	N/mm^2	415.0	415.0	415.0
Stress Intensity (Concrete)	σ_c	N/mm^2	3.6389	1.0614	——
Allowable Stress Intensity (Concrete)	σ_{ca}	N/mm^2	8.2000	8.2000	——
Evaluation of Compression			○	○	——
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	93.6883	37.8298	——
Allowable Stress Intensity (Reinforcing Bar)	σ_{sa}	N/mm^2	140.0000	140.0000	——
Evaluation of Compression			○	○	——
Stress Intensity by Shear Force	τ	N/mm2	——	——	0.2048
Allowable Stress Intensity by Shear Force	τ_a1	N/mm2	——	——	0.3600
Evaluation			——	——	○

6.7 Stability Analysis

(1) Stability against Buoyancy Force

(a) Buoyancy Force

$U = \gamma_W \cdot V_h$

$= 9.800 \times 27.720$

$= 271.656$ (kN)

Volume of body under water

Number	Area × Height	Volume (m^3)
2	2.000×6.600×1.850	24.420
3	2.000×6.600×0.250	3.300
Total	——	27.720

(b) Vertical Load

W = Wc + Wu

= 450.120 + 0.000

= 450.120 (kN)

(c) safety factor

Safety Factor F = W / U

= 450.120 / 271.656

= 1.657 ≧ Allowable Safety Factor $F_a = 1.200$

(2) Stability against Bearing

Weight of Soil Same Volume as Body

Number	Member	Volume of body × Unit Weight	Weight (kN)
1	Top Slab	2.000×6.600×0.150×18.000	35.640
2	Side Wall	2.000×6.600×2.600×18.000	617.760
3	Bottom Slab	2.000×6.600×0.250×18.000	59.400
Total Ws			712.800

Ws / Wc = 712.800 /450.120

= 1.584 ≧ 1.0

9.2 Box Culvert (0.9 x 0.9)

(1) Design Condition

In section II and section III, box culverts which dimension are 0.9 x 0.9 are used as collector pipes

The condition of concrete and reinforcing bar is same as manhole. Only the condition which is different from other structure is indicated as follows

(a) Live Load

(i) On Top Slab and Cover

Vehicle : T-25, 250 (kN)

(ii) On the Ground

Live Load: 10.0 (kN/m^2)

(b) Thickness of soil

2.2m* (Maximum height in section III, from top of box culver to dike crown)

* In this design, the future plan should be considered. Expansion of dike width and vehicles passing on the top of dike in the future is assumed.

(c) Reinforcing

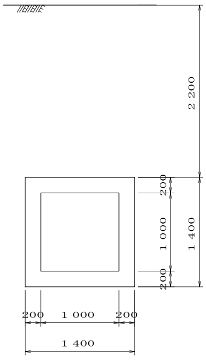
Existing pipes would be connected to the side wall of box culvert. For the connecting pipe and box culvert easily, single reinforcing is apllied.

(2) Structural Calculation

The detail of structural calculation of manhole is indicated from the following page

1 SectionIII_Box Culvert (Cross Section)

1.1 Design Condition

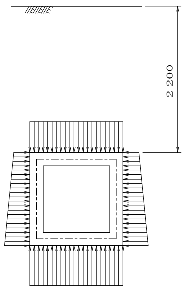


［ T-25 ］

Thickness of Embankment is less than 4.0m

Intensity of uniformly distributed load q = 10.00 (kN/m^2)

1.1.1 Dead Load (Case 1)



Weight of Body

(1) Top Slab

w = 0.200 × 24.00 = 4.80 (kN/m^2)

(2) Left Side Wall

w = 0.200 × 24.00 = 4.80 (kN/m^2)

(3) Right Side Wall

w = 0.200 × 24.00 = 4.80 (kN/m^2)

Load

(1) Pavement and Embankment

α

Pavement = 1.000 × 0.000 × 22.50 = 0.00 (kN/m^2)

Embankment = 1.000 × 2.200 × 18.00 = 39.60 (kN/m^2)

Σwd = 39.60 (kN/m^2)

(2) Load Effecting on Top Slab

Uniformly Distributed Load

w = 39.60 + 0.00 = 39.60 (kN/m^2)

Water Pressure and Earth Pressure

Intensity of Earth Pressure and Water Pressure

pi = Ko×(qd + Yo×γa + Zo×γ)

Ko : Coefficient of Earth Pressure at Rest Left = 0.500

Right = 0.500

qd : Load Effecting top of Embankment = 0.00 (kN/m^2)

Yo : Thickness of Pavement = 0.050 (m)

γa: Unit Weight of Pavement = 22.50 (kN/m^3)

γ : Unit Weight of Soil = 18.00 (kN/m^3)

Zo: Depth (m)

(1) Left Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	19.80
p_2	Axis Line of Top Slab	2.300	20.70
p_3	Axis Line of Bottom Slab	3.500	31.50
p_4	Under side of Bottom Slab	3.600	32.40

(2) Right Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	19.80
p_2	Axis Line of Top Slab	2.300	20.70
p_3	Axis Line of Bottom Slab	3.500	31.50
p_4	Under side of Bottom Slab	3.600	32.40

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Weight of Body	Top Slab	6.72		0.700		4.70
	Left Side Wall	4.80		0.100		0.48
	Right Side Wall	4.80		1.300		6.24
Load		55.44		0.700		38.81
Earth Pressure	Left Side Wall		36.54		0.644	23.52
	Right Side Wall		-36.54		0.644	-23.52
Total		71.76				50.23

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 0.700 \text{ (m)}$$

$$e = \frac{B}{2} - X = 0.000 \text{ (m)}$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

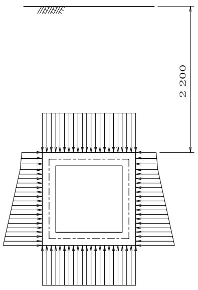
$$ql = \frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} = 51.26 \text{ (kN/m}^2\text{)}$$

$$qr = \frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} = 51.26 \text{ (kN/m}^2\text{)}$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 51.26 \text{ (kN/m}^2\text{)}$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 51.26 \text{ (kN/m}^2\text{)}$$

1.1.2 Dead Load (Case 2)



Weight of Body

(1) Top Slab

w = 0.200 × 24.00 = 4.80 (kN/m^2)

(2) Left Side Wall

w = 0.200 × 24.00 = 4.80 (kN/m^2)

(3) Right Side Wall

w = 0.200 × 24.00 = 4.80 (kN/m^2)

Load

(1) Pavement and Embankment

α

Pavement = 1.000 × 0.000 × 22.50 = 0.00 (kN/m^2)

Embankment = 1.000 × 2.200 × 18.00 = 39.60 (kN/m^2)

Σwd = 39.60 (kN/m^2)

(2) Load Effecting on Top Slab

Uniformly Distributed Load

w = 39.60 + 0.00 = 39.60 (kN/m^2)

Water Pressure and Earth Pressure

Intensity of Earth Pressure and Water Pressure

$pi = Ko \times (qd + Yo \times \gamma a + Zo \times \gamma)$

Ko : Coefficient of Earth Pressure at Rest — Left = 0.500; Right = 0.500

qd :	Load Effecting top of Embankment	=	0.00 (kN/m^2)
Yo :	Thickness of Pavement	=	0.050 (m)
γa:	Unit Weight of Pavement	=	22.50 (kN/m^3)
Z_u:	Thickness of soil in above water level	=	2.080 (m)
γ :	Unit Weight of Soil (wet)	=	18.00 (kN/m^3)
γ’:	Unit Weight of Soil (Submerged)	=	10.20 (kN/m^3)
γ_{w}:	Unit weight of water	=	9.80 (kN/m^3)
Zo:	Depth (m)		

(1) Left Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	19.80
p_2	Axis Line of Top Slab	2.300	20.70
p_3	Water Surface	2.460	22.14
p_4	Axis Line of Bottom Slab	3.500	37.64
P_5	Under side of Bottom Slab	3.600	39.13

(2) Right Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	19.80
p_2	Axis Line of Top Slab	2.300	20.70
p_3	Water Surface	2.460	22.14
p_4	Axis Line of Bottom Slab	3.500	37.64
P_5	Under side of Bottom Slab	3.600	39.13

Summary of Effecting Force

Item			V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Weight of Body	Top Slab		6.72		0.700		4.70
	Left Side Wall		4.80		0.100		0.48
	Right Side Wall		4.80		1.300		6.24
Load			55.44		0.700		38.81
Earth Pressure	Left Side Wall	Above water surface		5.45		1.268	6.91
		Under water surface		28.55		0.548	15.65
	Right Side Wall	Above water surface		-5.45		1.268	-6.91
		Under water surface		-28.55		0.548	-15.65
Uplift			-15.64		0.700		-10.95
Total			56.12				39.28

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 0.700 \ (m)$$

$$e = \frac{B}{2} - X = 0.000 \ (m)$$

(2) Intensity of Subgrade Reaction

$Me = \Sigma V \times e = 0.00$ (kN.m/m)

$$ql = \frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} = 40.09 \ (kN/m^2)$$

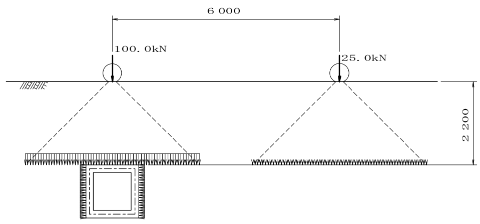
$$qr = \frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} = 40.09 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 40.09 \ (kN/m^2)$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 40.09 \ (kN/m^2)$$

1.1.3 Live Load (Case 1)

［ 2 Axis 250 (kN) ］



Intensity of Wheel Load

$$Pl+i = \frac{2 \times P \times (1+i)}{2.75}$$

$$Pvl = \frac{(Pl+i) \times \beta}{2 \times D + Do}$$

Pl+I	:	Live Load at Unit Length (kN/m)
P	:	Wheel Load (kN)
i	:	Impact Coefficient
Pvl	:	Converted Uniformly Distributed Load (kN/m^2)
D	:	Depth = 1.850 (m)
Do	:	Width of Wheel (m)
β	:	Coefficient of Reduction

Rear wheel $$Pl+i = \frac{2 \times 100.0 \times (1 + 0.300)}{2.75} = 94.55 \ (kN/m)$$

$$Pvl = \frac{94.55 \times 0.900}{2 \times 2.200 + 0.20} = 18.50 \ (kN/m^2)$$

Front wheel $$Pl+i = \frac{2 \times 25.0 \times (1 + 0.300)}{2.75} = 23.64 \ (kN/m)$$

$$Pvl = \frac{23.64 \times 1.000}{2 \times 2.200 + 0.20} = 5.14 \ (kN/m^2)$$

Load

(1) Vertical Load Effecting Top Slab

	Intensity (kN/m^2)	Position (m)	Width (m)
Rear Wheel	18.50	0.000	1.200
Front Wheel	5.14	0.000	0.000

(2) Horizontal Load Effecting on Left Side Wall
Converted Uniformly Distributed Load
wl = 18.50 (kN/m^2)
p = Ko×wl = 0.500 × 18.50 = 9.25 (kN/m^2)

(3) Horizontal Load Effecting on Right Side Wall
Converted Uniformly Distributed Load
wl = 18.50 (kN/m^2)
p = Ko×wl = 0.500 × 18.50 = 9.25 (kN/m^2)

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Top Slab	Distributed 1	25.90		0.700		18.13
	Distributed 2	0.00		0.000		0.00
Left Side Wall	Distributed		12.95		0.700	9.06
Right Side Wall	Distributed		-12.95		0.700	-9.06
Total		25.90				18.13

Subgrade Reaction
(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 0.700 \ (m)$$

$$e = \frac{B}{2} - X = 0.000 \ (m)$$

(2) Intensity of Subgrade Reaction
Me = ΣV × e = 0.00 (kN.m/m)

$$ql = \left(\frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} \right) \times 1.000 = 18.50 \ (kN/m^2)$$

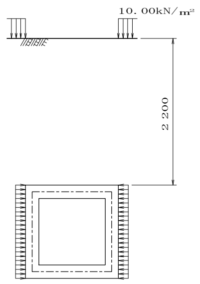
$$qr = \left(\frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} \right) \times 1.000 = 18.50 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 18.50 \ (kN/m^2)$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 18.50 \ (kN/m^2)$$

1.1.4 Live Load (Case 2)

［Lateral Pressure］



Load
(1) Horizontal Load Effecting on Left Side Wall
p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)
(2) Horizontal Load Effecting on Right Side Wall
p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

Summary of Effecting Force

Item		H (kN/m)	y (m)	M (kN.m/m)
Left Side Wall	Distribute	7.00	0.700	4.90
Right Side Wall	Distribute	-7.00	0.700	-4.90
Total				0.00

Subgrade Reaction
(1) Subgrade Reaction

$$ql = \pm \left(\frac{6 \times Me}{B^2} \right) \times 1.000 = 0.00 \ (kN/m^2)$$

$$qr = 0.00 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 0.00 \ (kN/m^2)$$

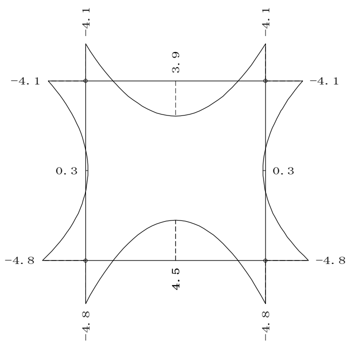
$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 0.00 \ (kN/m^2)$$

1.1.5 Calculation Case

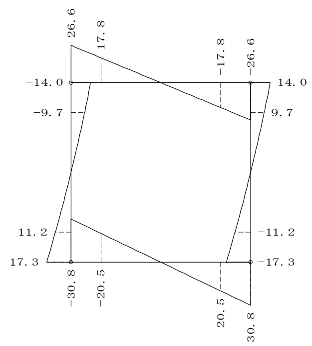
No	
1	Dead Load (Case1)
2	Dead Load (Case2)
3	Dead Load (Case1) + Live Load 1
4	Dead Load (Case1) + Live Load 2
5	Dead Load (Case2) + Live Load 1
6	Dead Load (Case2) + Live Load 2

1.2 Cross Section Force

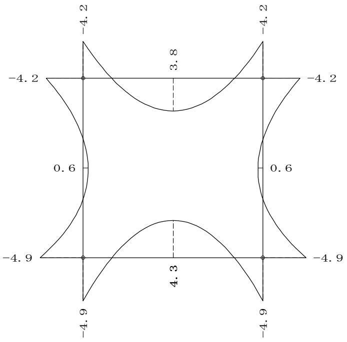
(1) Bending Moment (Case 1)



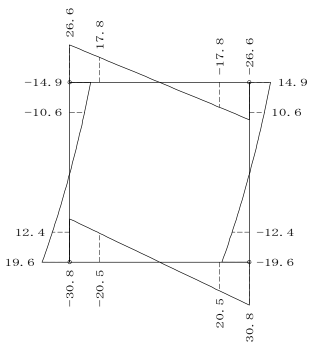
(2) Shear Force (Case 1)



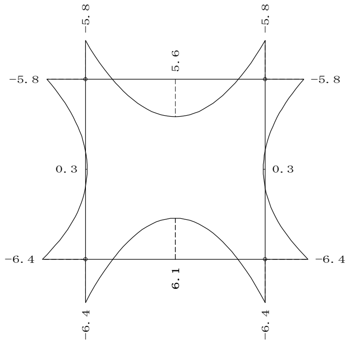
(3) Bending Moment (Case 2)



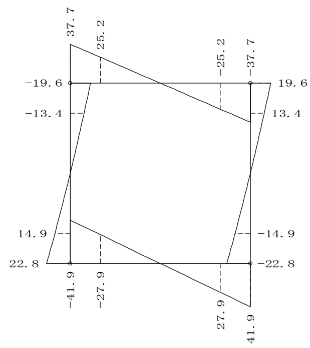
(4) Shear Force (Case2)



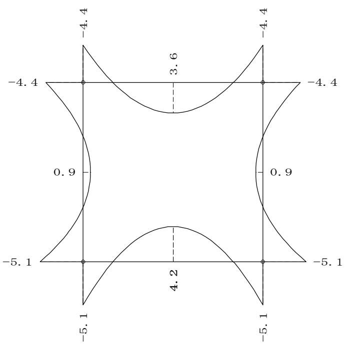
(5) Bending Moment (Case 3)



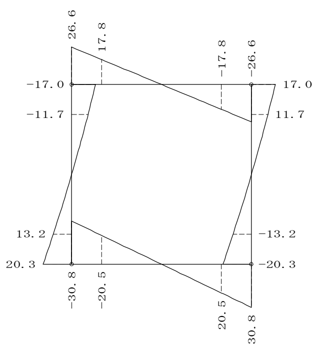
(6) Shear Force (Case3)



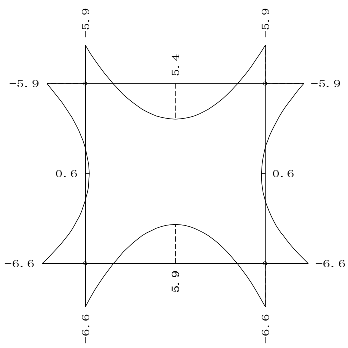
(7) Bending Moment (Case 4)



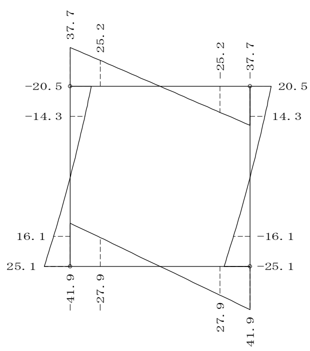
(8) Shear Force (Case 4)



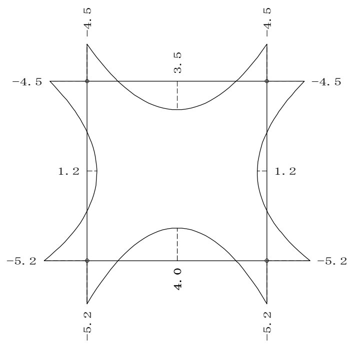
(9) Bending Moment (Case 5)



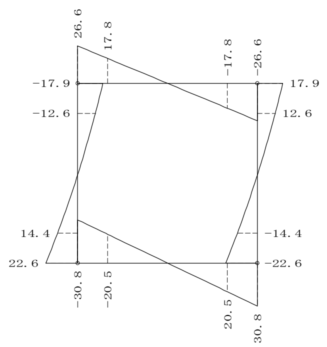
(10) Shear Force (Case 5)



(11) Bending Moment (Case 6)



(12) Shear Force (Case 6)



1.3 Stress Calculation

1.3.1 Intensity of Bending Stress

(1) Top Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-5.9	5.6	-5.9
Axial Force	N	kN	20.5	19.6	20.5
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	20.0	20.0	20.0
Effective Width	d	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Outside)	d1	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Inside)	d2	cm	10.0	10.0	10.0
Required Area of Reinforcement	Outside	cm^2	3.11	0.00	3.11
	Inside	cm^2	0.00	2.89	0.00
Reinforcement	Outside	cm^2	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525
	Inside	cm^2	D─ @── D─ @── ───	D12 @250 D─ @── 4.525	D─ @── D─ @── ───
Neutral Axis	X	cm	2.921	2.931	2.921
Stress Intensity	σc	N/mm^2	4.45	4.17	4.45
	σs	N/mm^2	98.30	91.78	98.30
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	──	───	5	3	5

(2) Left Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-5.9	1.2	-6.6
Axial Force	N	kN	37.7	29.0	42.5
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	20.0	20.0	20.0
Effective Width	d	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Outside)	d1	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Inside)	d2	cm	10.0	10.0	10.0
Required Area of Reinforcement	Outside	cm^2	1.89	0.00	2.11
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D─ @── 4.525	D─ @── D─ @── ───	D12 @250 D─ @── 4.525
	Inside	cm^2	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525
Neutral Axis	X	cm	3.518	17.615	3.531
Stress Intensity	σc	N/mm^2	3.78	0.32	4.22
	σs	N/mm^2	63.73	-1.24	70.72
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	──	───	5	6	5

(3) Right Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-5.9	1.2	-6.6
Axial Force	N	kN	37.7	29.0	42.5
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	20.0	20.0	20.0
Effective Width	d	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Outside)	d1	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Inside)	d2	cm	10.0	10.0	10.0
Required Area of Reinforcement	Outside	cm^2	1.89	0.00	2.11
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D─ @── 4.525	D─ @── D─ @── ───	D12 @250 D─ @── 4.525
	Inside	cm^2	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525
Neutral Axis	X	cm	3.518	17.615	3.531
Stress Intensity	σc	N/mm^2	3.78	0.32	4.22
	σs	N/mm^2	63.73	-1.24	70.72
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	──	───	5	6	5

(4) Bottom Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-6.4	6.1	-6.4
Axial Force	N	kN	22.8	22.8	22.8
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	20.0	20.0	20.0
Effective Width	d	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Outside)	d1	cm	10.0	10.0	10.0
Thickness of Cover Concrete (Inside)	d2	cm	10.0	10.0	10.0
Required Area of Reinforcement	Outside	cm^2	3.36	0.00	3.36
	Inside	cm^2	0.00	3.11	0.00
Reinforcement	Outside	cm^2	D12 @250 D— @—— 4.525	D12 @250 D— @—— 4.525	D12 @250 D— @—— 4.525
	Inside	cm^2	D— @—— D— @—— ——	D12 @250 D— @—— 4.525	D— @—— D— @—— ——
Neutral Axis	X	cm	2.935	2.965	2.935
Stress Intensity	σc	N/mm^2	4.82	4.55	4.82
	σs	N/mm^2	105.96	98.53	105.96
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	—	—	3	3	3

1.3.2 Intensity of Shear Stress

$$\tau m = \frac{S}{b \times d} \leqq \tau a$$

b = 100.0（cm）

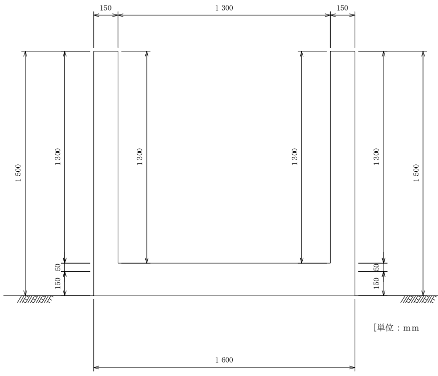
Member	Position	S (kN)	d (cm)	τm (N/mm^2)	τa (N/mm^2)	Case	L (m)
Top Slab	Left Side τ	25.2	10.0	0.249	0.360	3	0.200
	Right Side τ	-25.2	10.0	0.249	0.360	3	0.200
Left Side Wall	UpperSide	-14.3	10.0	0.141	0.360	5	0.200
	UnderSide	16.1	10.0	0.159	0.360	5	0.200
Right Side Wall	UpperSide	14.3	10.0	0.141	0.360	5	0.200
	UnderSide	-16.1	10.0	0.159	0.360	5	0.200
Bottom Slab	Left Side τ	-27.9	10.0	0.276	0.360	3	0.200
	Right Side τ	27.9	10.0	0.276	0.360	3	0.200

9.3 U-ditch (1.3 x 1.3)

In section II, the large U-ditch, which dimension is more than 1.0m x 1.0m, is used for collect the discharge from the catchment area.

The detail of structural calculation of manhole is indicated from the following page

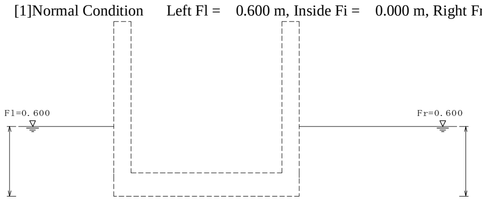
1 MSR3 Wing Wall (U-shaped)



(Unit : mm)

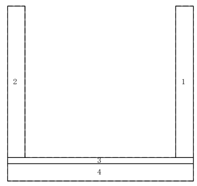
1.1 Water Level

[1]Normal Condition Left Fl = 0.600 m, Inside Fi = 0.000 m, Right Fr = 0.600 m



1.2 Stability Analysis

(1) Center of Gravity



	Width × Height × Depth	$Vi(m^3)$	Center of gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2 3	0.150× 1.300× 1.000 0.150× 1.300× 1.000 1.600× 0.050× 1.000 1.600× 0.150× 1.000	0.195 0.195 0.080 0.240	1.525 0.075 0.800 0.800	0.850 0.850 0.175 0.075	0.297 0.015 0.064 0.192	0.166 0.166 0.014 0.018	
Σ		0.710	——	——	0.568	0.363	

Center of Gravity $XG = \Sigma(Vi \cdot Xi) / \Sigma Vi = 0.568 / 0.710 = 0.800$ (m)

$YG = \Sigma(Vi \cdot Yi) / \Sigma Vi = 0.363 / 0.710 = 0.512$ (m)

(2) Weight of Body and Uplift

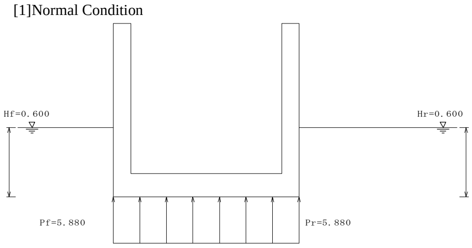
(a) Weight of Body

[1]Normal Condition

	Vertical Force $W = \gamma \cdot V$ (kN)	Position X (m)
Side Wall	24.000 × 0.390 ＝ 9.360	0.800
Bottom Slab	24.000 × 0.320 ＝ 7.680	0.800

(b) Uplift

[1]Normal Condition



Water level Hf ＝ 0.600 (m)

Strength of Water Pressure Pf ＝ 5.880 (kN/m^2)

Buoyancy Effecting on Body

$$U = \frac{Pf + Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 9.408 \text{ (kN)}$$

Position (From the Left Edge)

$$X = \frac{Pf + 2 \cdot Pr}{3 \cdot (Pf + Pr)} \cdot Bj = 0.800 \text{ (m)}$$

Where,

Bj : Width of Footing Bj ＝ 1.600 (m)

Bc : Length of Footing Bc ＝ 1.000 (m)

λ : Reduction Coefficient of buoyancy λ ＝ 1.000

(3) Stability against Buoyancy Force

$$F_s = \frac{\Sigma Vu + \alpha \cdot Pv}{U}$$

Where,

ΣVu : Total Vertical Force (kN)

α : Efficiency Rate of Earth Pressure α＝0.000

Pv : Vertical Element of Earth Pressure (kN)

U : Buoyancy (kN)

	ΣVu (kN)	Pv (kN)	U (kN)	Safty Factor f_s	Required FS f_{sa}
Normal Condition	17.040	0.000	9.408	1.811 ≧	1.200

1.3 Structural Calculation

1.3.1 Calculation of Earth Pressure

[1]Normal Condition

■Earth Pressure at Rest

Height of Imaginary Back Side H ＝ 1.400 m

Height above Water Surface H1 ＝ 0.900 m

Height under Water Surface H2 ＝ 0.500 m

Angle Between Back side Surface of Wall and Vertical Plane α ＝ 0.000 °

Unit Weight of Back fill Ma γs ＝ 18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1 p2 p3	5.000 13.100 15.650	5.000 13.100 15.650	0.000 0.000 0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1 + p2) \cdot H1 = \frac{1}{2} \times (5.000 + 13.100) \times 0.900 = 8.145 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p2 + p3) \cdot H2 = \frac{1}{2} \times (13.100 + 15.650) \times 0.500 = 7.188 \text{ kN}$$

Total Earth Pressure

$P_0 = P = P1 + P2 = 8.145 + 7.188 = 15.332$ kN

1.3.2 Side wall

(1) Water Pressure

pi ＝ hi・Gw

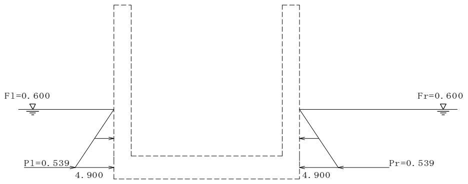
Where,

pi：Strength of Water Pressure at the Edge of Bottom Slab (kN/m^2)

hi：Water Depth (m)

Gw：Unit Weight of Water (kN/m^3), Gw ＝ 9.800

[1]Normal Condition



Water Depth hi (m)	Strength pi (kN/m^2)	Water Depth ho (m)	Strength po (kN/m^2)	concentrated load Po (kN/m)
0.500	4.900	0.600	5.880	0.539

(2) Subgrade Reaction

[1]Normal Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	1.300	3.600	3.600	4.680	0.000	0.000
Weight of Right Side wall	Right Side Wall	1.300	3.600	3.600	4.680	1.450	6.786
Weight of Bottom Slab	Bottom Slab	1.450	4.800	4.800	6.960	0.725	5.046
Weight of Bottom Slab	Bottom Slab	0.000	0.360	0.000	0.360	0.000	0.000
Weight of Bottom Slab	Bottom Slab	0.000	0.360	0.000	0.360	1.450	0.522
Buoyancy	Bottom Slab	1.450	-5.880	-5.880	-8.526	0.725	-6.181
Buoyancy	Bottom Slab	0.000	-0.441	0.000	-0.441	0.000	0.000
Buoyancy	Bottom Slab	0.000	-0.441	0.000	-0.441	1.450	-0.639
Total	——	——	——	——	7.632	——	5.533

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Left Side Wall	0.900	5.000	13.100	8.145	0.883	7.191
Earth Pressure (Left)	Left Side Wall	0.500	13.100	15.650	7.187	0.243	1.744
Earth Pressure (Left)	Left Side Wall	0.000	1.591	0.000	1.591	0.000	0.000
Earth Pressure (Right)	Right Side Wall	0.900	-5.000	-13.100	-8.145	0.883	-7.191
Earth Pressure (Right)	Right Side Wall	0.500	-13.100	-15.650	-7.187	0.243	-1.744
Earth Pressure (Right)	Right Side Wall	0.000	-1.591	0.000	-1.591	0.000	0.000
Water Pressure (Left)	Left Side Wall	0.500	0.000	4.900	1.225	0.167	0.204
Water Pressure (Left)	Left Side Wall	0.000	0.539	0.000	0.539	0.000	0.000
Water Pressure (Right)	Right Side Wall	0.500	0.000	-4.900	-1.225	0.167	-0.204
Water Pressure (Right)	Right Side Wall	0.000	-0.539	0.000	-0.539	0.000	0.000
Total	——	——	——	——	0.000	——	0.000

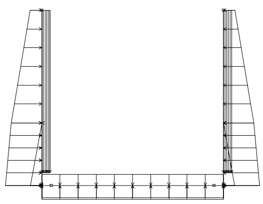
Vertical Force V ＝ ΣVi ＝ 7.632 (kN)

Horizontal Force H ＝ ΣHi ＝ 0.000 (kN)

Moment Mo ＝ ΣMxi＋ΣMyi ＝5.333 ＋0.000 ＝5.333(kN.m)

Summary of Load

[1]Normal Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	1.300	3.600	3.600
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	1.300	3.600	3.600
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	1.450	4.800	4.800
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	0.000	0.360	0.000
Weight of Bottom Slab	Bottom Slab	Vertical	1.450	0.000	0.360	0.000

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.900	5.000	13.100
Earth Pressure (Left)	Left Side Wall	Horizontal	0.900	0.500	13.100	15.650
Earth Pressure (Left)	Left Side Wall	Horizontal	1.400	0.000	1.591	0.000
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.900	-5.000	-13.100
Earth Pressure (Right)	Right Side Wall	Horizontal	0.900	0.500	-13.100	-15.650
Earth Pressure (Right)	Right Side Wall	Horizontal	1.400	0.000	-1.591	0.000
Water Pressure (Left)	Left Side Wall	Horizontal	0.900	0.500	0.000	4.900
Water Pressure (Left)	Left Side Wall	Horizontal	1.400	0.000	0.539	0.000
Water Pressure (Right)	Right Side Wall	Horizontal	0.900	0.500	0.000	-4.900
Water Pressure (Right)	Right Side Wall	Horizontal	1.400	0.000	-0.539	0.000

■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	1.450	-5.880	-5.880
Buoyancy	Bottom Slab	Vertical	0.000	0.000	-0.441	0.000
Buoyancy	Bottom Slab	Vertical	1.450	0.000	-0.441	0.000

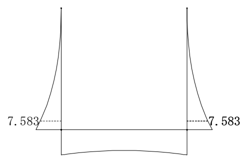
■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	1.450	-5.263	-5.263

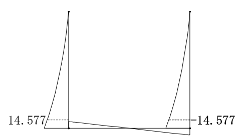
(3) Cross Section Force

[1]Normal Condition

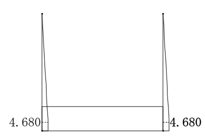
(a) Bending Moment



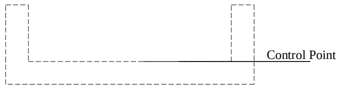
(b) Shear Force



(c) Axial Force



(4) Stress Calculation



1) Bending Stress

(a) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	–	——	——
	2'	——	–	——	——
Outside	1	7.5	D16	5.00	10.055
	2	——	–	——	——

Required Area of Reinforcement 9.439 (cm^2)

(b) Stress Calculation

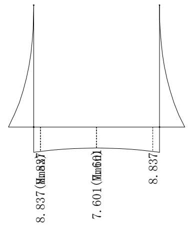
			Normal Condition
Bending Moment	M	kN.m	7.583
Axial Force	N	kN	——
Shear Force	S	kN	14.577
Width of Member	B	mm	1000.0
Height of Member	H	mm	150.0
Effective Width	d	mm	75.0
Applied area of Reinforcement (Tension)	A_s	cm^2	10.055
(Compression)	A_s'	cm^2	
Neutral Axis	X	mm	28.894
Stress Intensity (Concrete)	σ_c	N/mm^2	8.033
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200
Evaluation of Compression			○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	115.360
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	140.000
Evaluation of Compression			○
Stress Intensity by Shear Force	τ	N/mm^2	0.194
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360
Evaluation			○

1.3.3 Bottom Slab

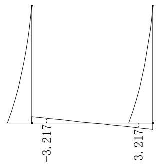
(1) Cross Section Force

[1]Normal Condition

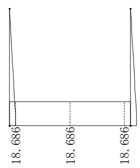
(a) Bending Moment



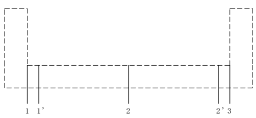
(b) Shear Force



(c) Axial Force



(2) Stress Calculation



1) Control point of Bending Stress

	1	2	3
Control point	0.075	0.725	1.375

2) Control Point of Shear Stress

	1'	2'
Control point	0.175	1.275

(3) Calculation of Bending Stress

(a) Control point 1

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	——
	2'	——	—	——	——
Outside	1	10.0	D16	5.00	10.055
	2	——	—	——	——

Required Area of Reinforcement 7.008 (cm^2)

2) Stress Calculation

			Normal Condition
Bending Moment	M	kN.m	8.837
Axial Force	N	kN	——
Shear Force	S	kN	——
Width of Member	B	mm	1000.0
Height of Member	H	mm	200.0
Effective Width	d	mm	100.0
Applied area of Reinforcement (Tension)	A_s	cm^2	10.055
(Compression)	A_s'	cm^2	
Neutral Axis	X	mm	34.448
Stress Intensity (Concrete)	σ_c	N/mm^2	5.797
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200
Evaluation of Compression			○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	99.285
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	140.000
Evaluation of Compression			○

(b) Control point 2

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	10.0	D16	5.00	10.055
	2	——	—	——	———

Required Area of Reinforcement 5.986 (cm^2)

2) Stress Calculation

			Normal Condition
Bending Moment	M	kN.m	7.601
Axial Force	N	kN	——
Shear Force	S	kN	——
Width of Member	B	mm	1000.0
Height of Member	H	mm	200.0
Effective Width	d	mm	100.0
Applied area of Reinforcement (Tension)	A_s	cm^2	10.055
(Compression)	A_s'	cm^2	
Neutral Axis	X	mm	34.448
Stress Intensity (Concrete)	σ_c	N/mm^2	4.987
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200
Evaluation of Compression			○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	85.401
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	140.000
Evaluation of Compression			○

(4) Calculation of Shear Stress

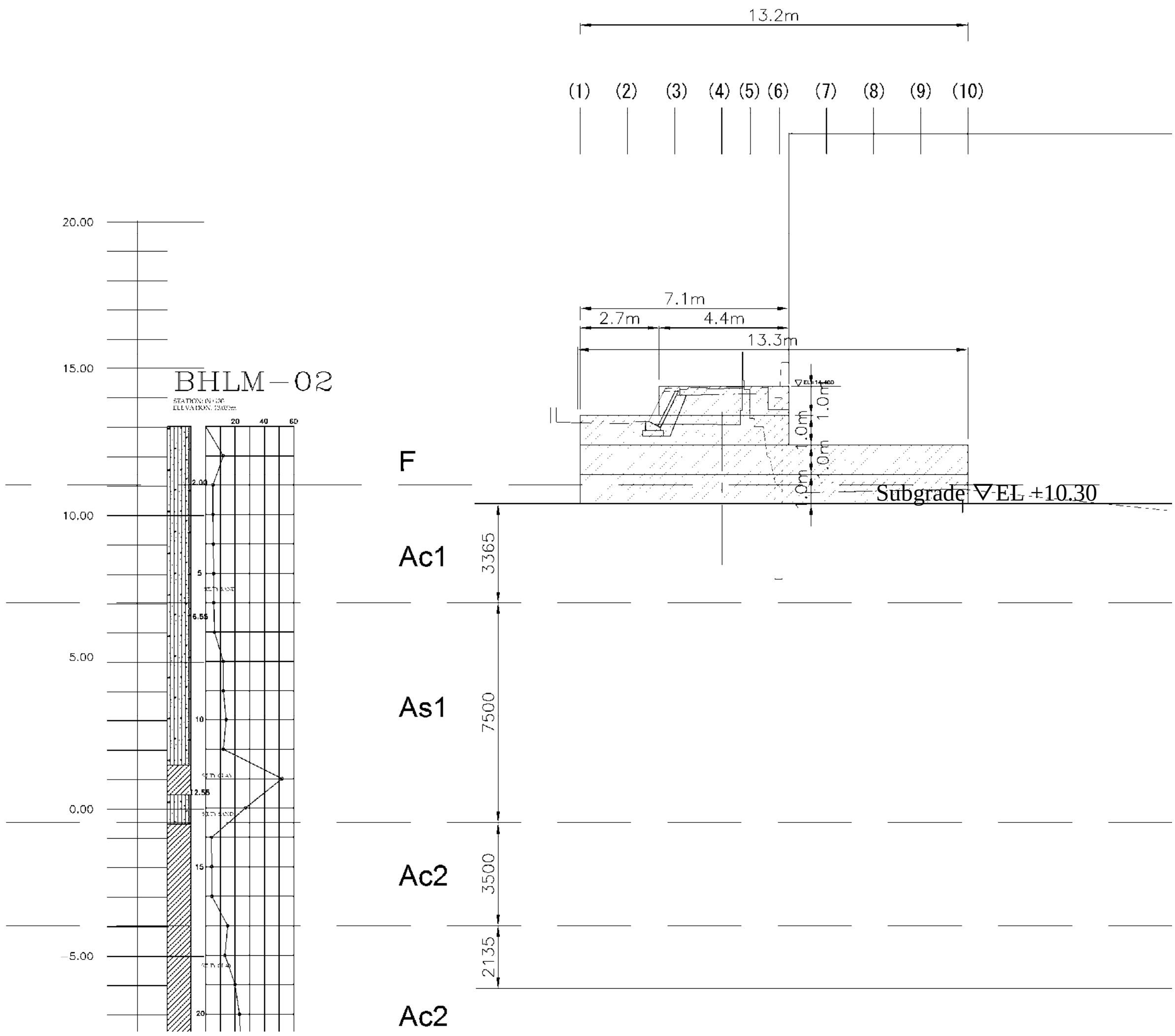
			Normal Condition
Bending Moment	M	kN.m	——
Axial Force	N	kN	——
Shear Force	S	kN	3.217
Width of Member	B	mm	1000.0
Height of Member	H	mm	200.0
Bending Moment	d	mm	100.0
Stress Intensity by Shear Force	τ	N/mm^2	0.032
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360
Evaluation			○

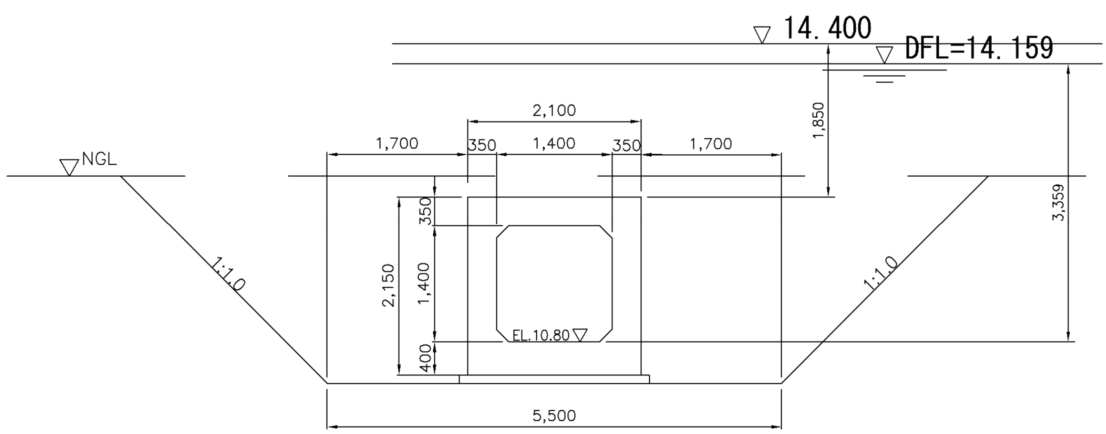
CHAPTER 10 CALCULATION OF RESIDUAL SETTLEMENT AT SLUICEWAY SITE

Calculation of residual settlement at sluiceway site is shown from the following page.

1 Calculation of MSL-1

1.1 Calculation Model





1.2 Results of Consolidation test

Sluiceway No **MSL-1**

Geological Data	
Borehole	BHML-02
station	1+100
elevation	13.035
northing	1610801.415
easting	507807.944

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	2.0	F	17.0
	6.0	AC1	18.1
	13.5	AS1	18.0
	25.0	DC	16.9
	27.0	DS	18.0
	30.0	GF	15.5

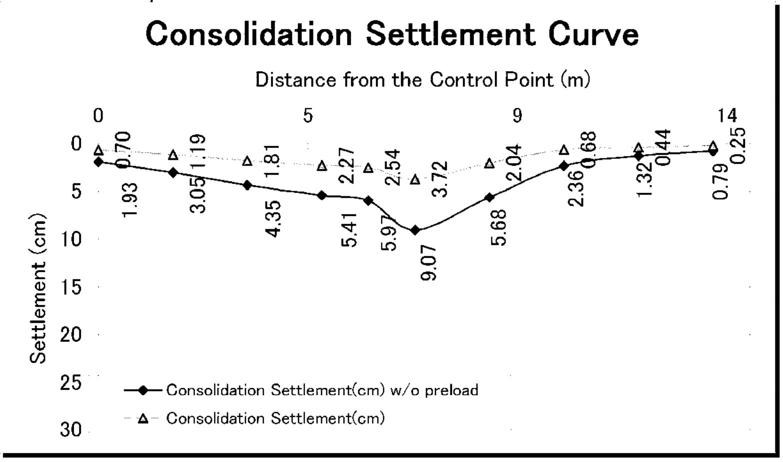
Consolidation Data		AC1	DC
PC kgf/cm^2		0.4650	1.825
	p kgf/cm2		
e-logp	0.1	1.1148	1.683
	0.2	1.0763	1.650
	0.4	1.0170	1.608
	0.8	0.9456	1.568
	1.6	0.8644	1.490
	3.2	0.7798	1.240
	6.4	0.6886	1.065
	12.8	0.5898	0.880
Cv 10^{-3} cm^2/sec	0.05	2.6900	15.450
	0.15	3.5630	9.100
	0.3	2.5350	4.400
	0.6	2.2730	6.150
	1.2	1.9260	3.750
	2.4	3.0010	5.700
	4.8	2.5760	3.900
	9.6	2.1800	4.650

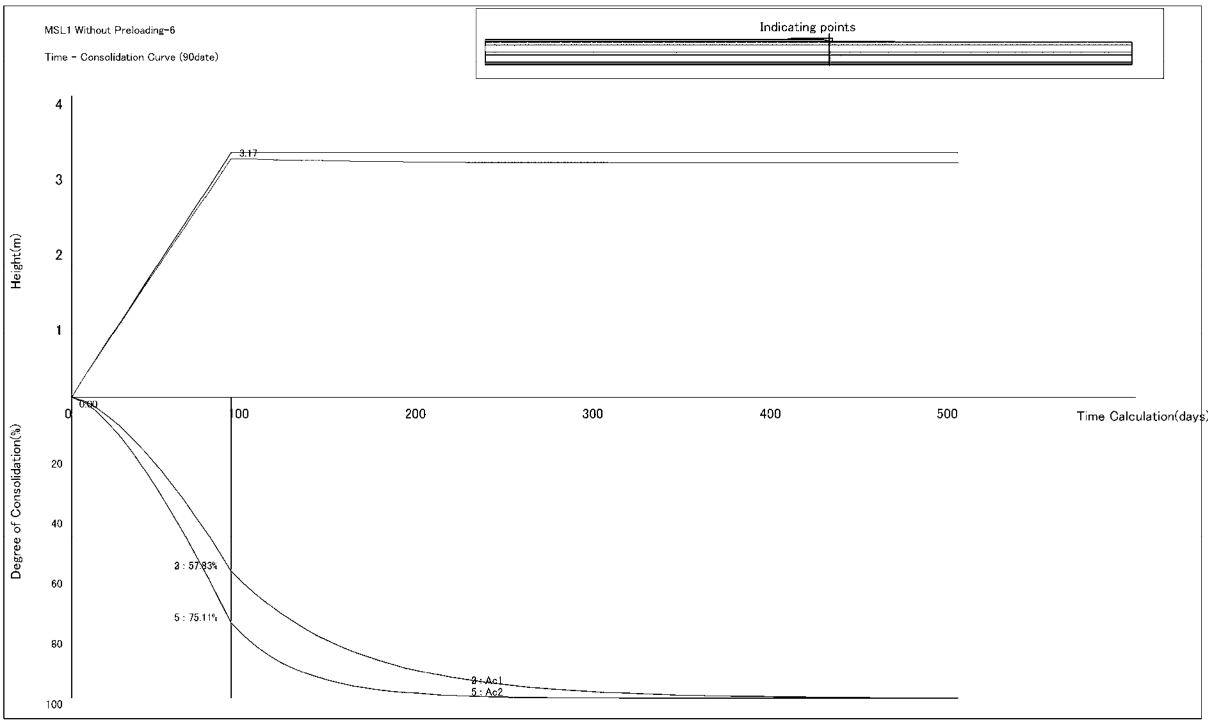
AC1 : BHLM02 2m
DC : Average of BMLL5 11m & 12m

1.3 Considering Pre-Load Effect

In this site, pre-load effect darling 90 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	5.8	6.8	8.4	10.0	11.6	13.2
Consolidation Settlement(cm) w/o preload	1.93	3.05	4.35	5.41	5.97	9.07	5.68	2.36	1.32	0.79
after 90days	1.24	1.86	2.54	3.14	3.43	5.35	3.64	1.68	0.88	0.54
Consolidation Settlement(cm)	0.70	1.19	1.81	2.27	2.54	3.72	2.04	0.68	0.44	0.25





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

E' : Modulus of Deformation of the Ground after Pre-Load (kN/m^2)

E_0 : Original Modulus of Deformation of the Ground (kN/m^2) (= 700 x N)

C : Cohesion after Pre-Load (kN/m^2)

C_0 : Original Cohesion (kN/m^2)

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

m : Strength Increase Ratio (Cohesive soil : $m = 0.25$)

ΔP : Load Increase (kN/m^2)

U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BHML-02)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
Ac1	11	0	5.88	0.25	17.2	0.5	13	1.19	7.01
As	0	28	7.70	-	-	-	-	-	7.70
Ac2	49	0	7.15	0.25	5.1	0.7	50	1.02	7.28

1.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BHLM-02)

Loading Width	B (m)	13.400
Loading Length	L (m)	5.500
Boring		BHLM-02

No.	Classification	E_i (MN/m^2)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	Ac1	7.01	3.37	3.37	-0.03995	
2	As	7.70	7.50	10.87	-0.03213	
3	Ac2	7.28	5.63	16.50	-0.01154	
4						
5						
6						
7						
8						
9						
10						
11						
	Total		16.50		-0.08362	-0.61143
E_m (MN/m^2)						7.3
(kN/m^2)						7312
(tf/m^2)						731

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment (E.L. m)	Sign	Load (kN/m3)		(m)		(kN/m2)	Loading Width (m)	1/2 Loading Width (m)	Location of the Center of the Load from the Control Point (m)			
14.40	q1	18.0	×	1.0	=	18.0	13.20	6.60	6.60		=	6.60
	q2	18.0	×	1.0	=	18.0	13.20	6.60	0.00 +	6.60	=	6.60
	q3	18.0	×	1.0	=	18.0	7.10	3.55	0.00 +	3.55	=	3.55
	q4	18.0	×	1.0	=	18.0	4.40	2.20	2.70 +	2.20	=	4.90

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	18.0	6.60	6.60
q2	Load 2	18.0	6.60	6.60
q3	Load 3	18.0	3.55	3.55
q4	Load 4	18.0	2.20	4.90

● Ground Conditon BHLM-02

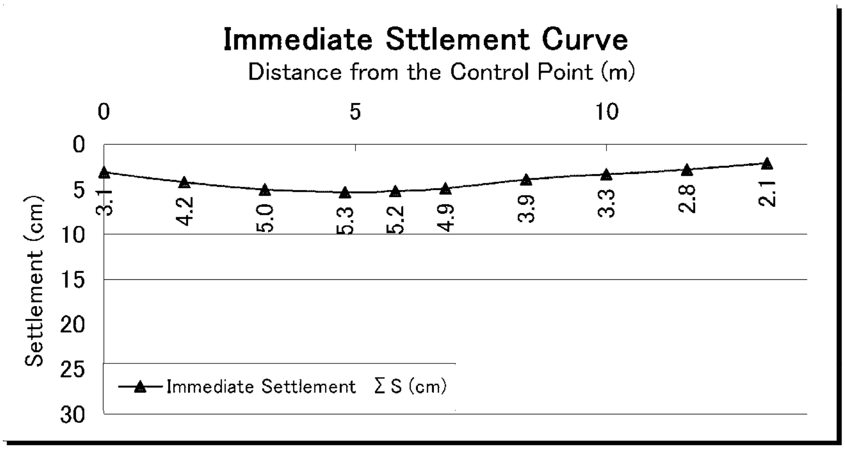
Thickness of Layer 16.5 m

Average Modulus of Elasticity 7312 kN/m2

$$Six = \sum_{i-i}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0-\frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right|+\left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

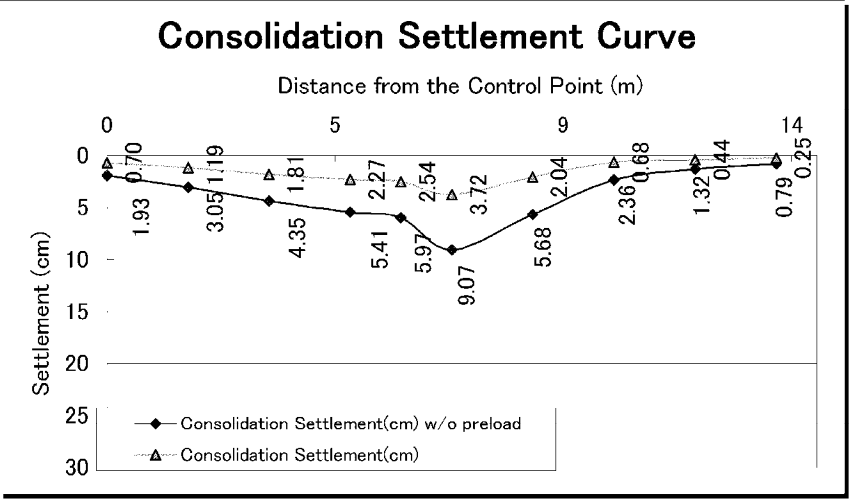
$$= \sum_{i-i}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0-0.2387\left[\alpha\log|\alpha|+\beta\log|\beta|\right]\right\}$$

	X	(m)	Location of Caluculation	Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
				0.0	1.6	3.2	4.8	5.8	6.8	8.4	10.0	11.6	13.2
q1	X1	(m)	Xo1-X	6.6	5.0	3.4	1.8	0.8	0.2	1.8	3.4	5.0	6.6
	S1	(cm)		1.03	1.30	1.43	1.51	1.53	1.54	1.51	1.43	1.30	1.03
q2	X2	(m)	Xo2-X	6.6	5.0	3.4	1.8	0.8	0.2	1.8	3.4	5.0	6.6
	S2	(cm)		1.03	1.30	1.43	1.51	1.53	1.54	1.51	1.43	1.30	1.03
q3	X3	(m)	Xo3-X	3.6	2.0	0.4	1.3	2.3	3.3	4.9	6.5	8.1	9.7
	S3	(cm)		0.87	1.20	1.30	1.26	1.17	0.98	0.55	0.34	0.19	0.07
q4	X4	(m)	Xo4-X	4.9	3.3	1.7	0.1	0.9	1.9	3.5	5.1	6.7	8.3
	S4	(cm)		0.16	0.39	0.88	1.05	1.00	0.82	0.35	0.14	0.00	0.00
Immediate Settlement			ΣS (cm)	**3.1**	**4.2**	**5.0**	**5.3**	**5.2**	**4.9**	**3.9**	**3.3**	**2.8**	**2.1**



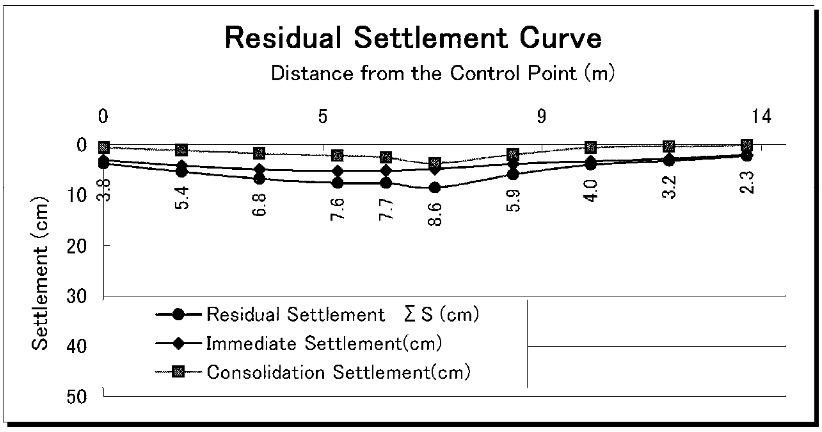
1.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	5.8	6.8	8.4	10.0	11.6	13.2
nsolidation Settlement(cm) w/o preload	1.93	3.05	4.35	5.41	5.97	9.07	5.68	2.36	1.32	0.79
after 90days	1.24	1.86	2.54	3.14	3.43	5.35	3.64	1.68	0.88	0.54
Consolidation Settlement(cm)	0.70	1.19	1.81	2.27	2.54	3.72	2.04	0.68	0.44	0.25



1.6 Residual Settlement

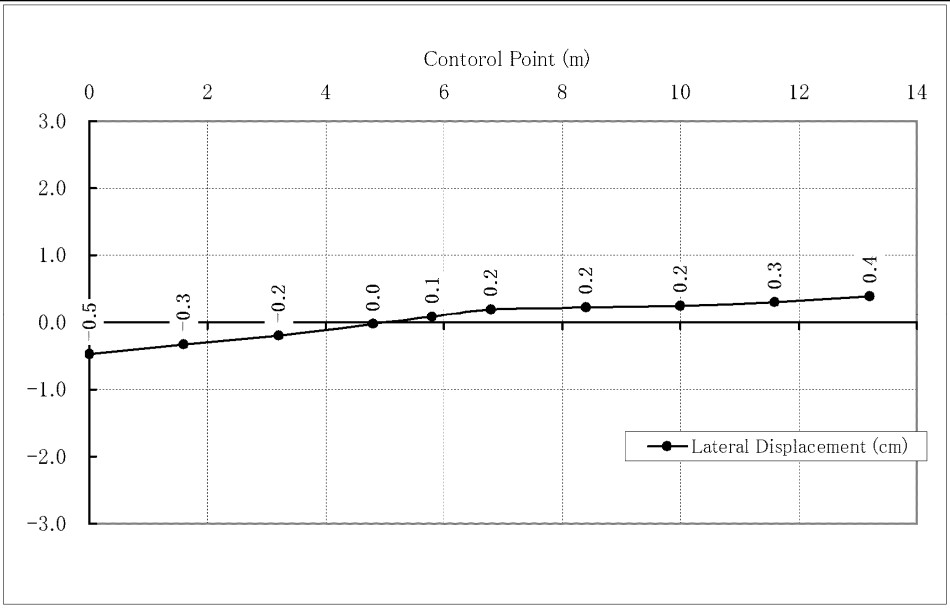
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	5.8	6.8	8.4	10.0	11.6	13.2
Immediate Settlement(cm)	3.10	4.20	5.00	5.30	5.20	4.90	3.90	3.30	2.80	2.10
Consolidation Settlement(cm)	0.70	1.19	1.81	2.27	2.54	3.72	2.04	0.68	0.44	0.25
Residual Settlement ΣS (cm)	**3.8**	**5.4**	**6.8**	**7.6**	**7.7**	**8.6**	**5.9**	**4.0**	**3.2**	**2.3**



1.7 Lateral Displacement

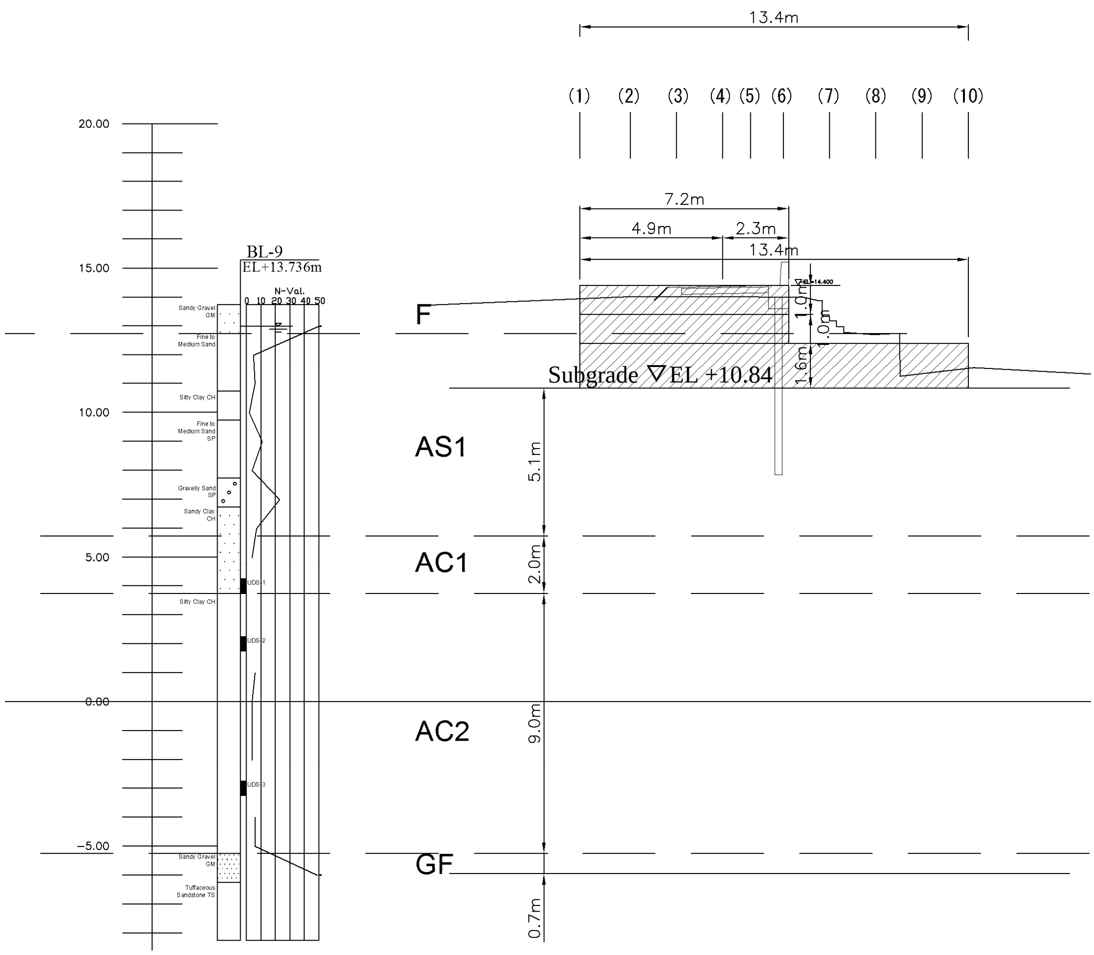
Lateral Displacement (BHLM-02)

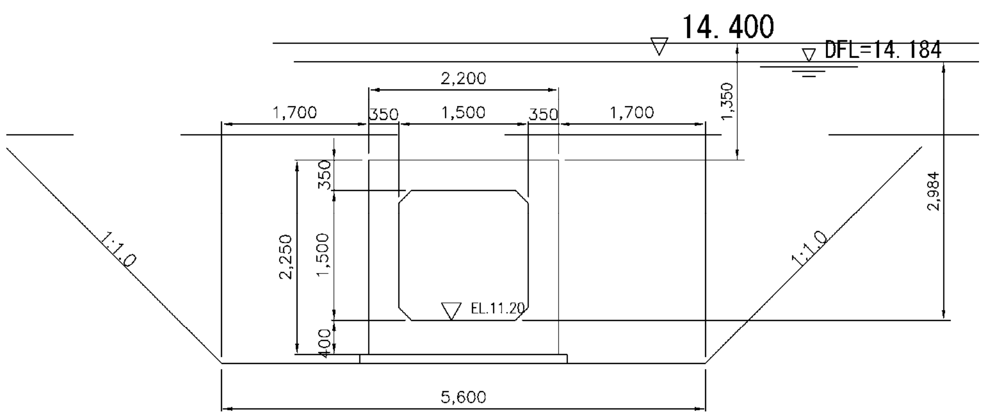
			q1		q2		q3		q4		Total
q_i (kN/m^2)				18.000		18.000		18.000		18.000	
a_i (m)				6.600		6.600		3.550		2.200	
b_i (m)				2.750		2.750		2.750		2.750	
ν				0.400		0.400		0.400		0.400	
x_i (m)				6.600		6.600		3.550		4.900	
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	7312	6.6	-0.16	6.6	-0.16	3.5	-0.12	4.9	-0.05	-0.5
(2)	1.6	7312	5.0	-0.10	5.0	-0.10	2.0	-0.06	3.3	-0.07	-0.3
(3)	3.2	7312	3.4	-0.06	3.4	-0.06	0.4	-0.01	1.7	-0.07	-0.2
(4)	4.8	7312	1.8	-0.03	1.8	-0.03	-1.3	0.04	0.1	0.00	0.0
(5)	5.8	7312	0.8	-0.01	0.8	-0.01	-2.3	0.07	-0.9	0.04	0.1
(6)	6.8	7312	-0.2	0.00	-0.2	0.00	-3.3	0.11	-1.9	0.08	0.2
(7)	8.4	7312	-1.8	0.03	-1.8	0.03	-4.9	0.09	-3.5	0.07	0.2
(8)	10.0	7312	-3.4	0.06	-3.4	0.06	-6.5	0.07	-5.1	0.05	0.2
(9)	11.6	7312	-5.0	0.10	-5.0	0.10	-8.1	0.05	-6.7	0.04	0.3
(10)	13.2	7312	-6.6	0.16	-6.6	0.16	-9.6	0.05	-8.3	0.03	0.4



2 Calculation of MSL-2

2.1 Calculation Model





2.2 Results of Consolidation test

Sluiceway No	MSL-2
Geological Data	
Borehole	BL-9
station	R 1+325
elevation	13.736
northing	1611031.716
easting	507843.905

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	1.0	F	17.0
	8.0	AS1	18.0
	10.0	AC1	18.1
	19.0	DC	16.9
		GF	15.5

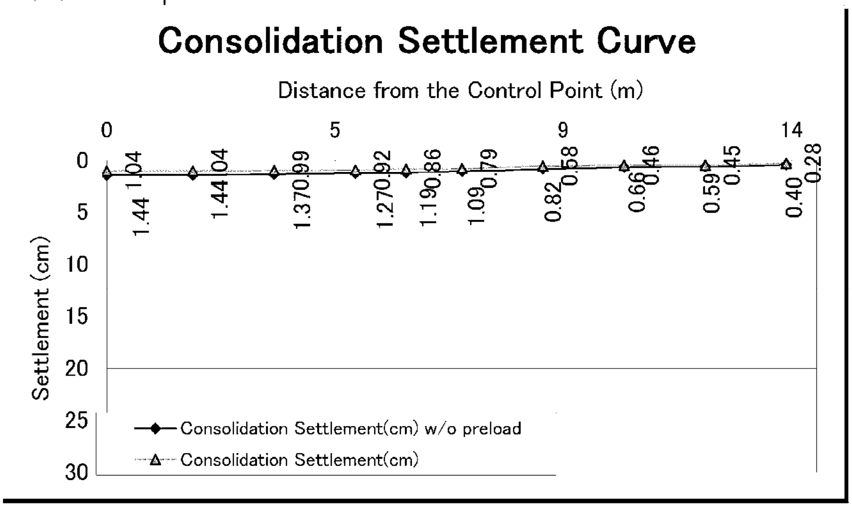
Consolidation Data		AC1	DC
PC kgf/cm^2		1.9300	1.495
	p kgf/cm2		
e-logp	0.1	1.1800	1.763
	0.2	1.1700	1.743
	0.4	1.1500	1.710
	0.8	1.1250	1.668
	1.6	1.0800	1.600
	3.2	0.9600	1.393
	6.4	0.8350	1.130
	12.8	0.7500	0.905
Cv 10^{-3} cm^2/sec	0.05	7.1000	6.275
	0.15	5.1000	4.350
	0.3	5.3000	3.800
	0.6	5.2000	3.975
	1.2	10.9000	5.000
	2.4	2.8000	1.200
	4.8	1.9000	0.700
	9.6	2.1000	0.500

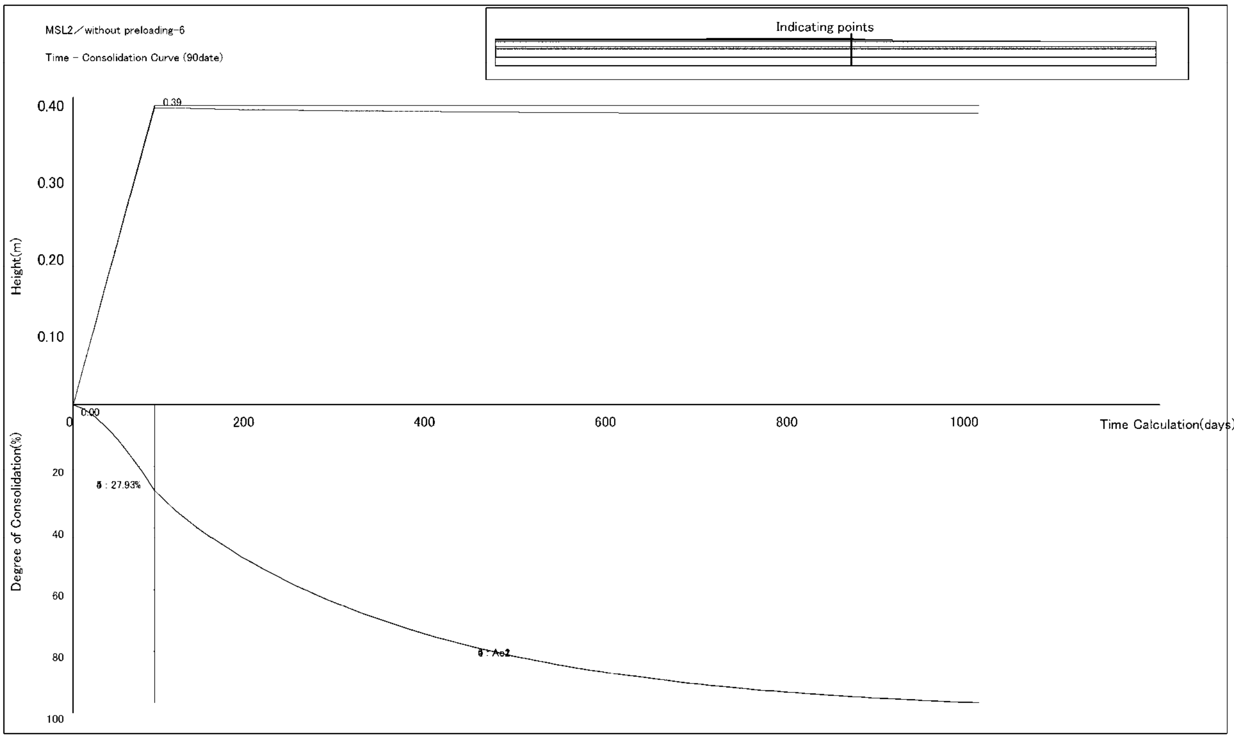
AC1: Average of BHLM17 3m & 8m
DC : BHLM05 14m

2.3 Considering Pre-Load Effect

In this site, pre-load effect darling 90 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point Location of Caluculation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)		0.0	1.7	3.3	4.9	5.9	7.0	8.6	10.2	11.8	13.4
Consolidation Settlement(cm) w/o preload		1.44	1.44	1.37	1.27	1.19	1.09	0.82	0.66	0.59	0.40
	after 90days	0.40	0.40	0.38	0.35	0.33	0.31	0.24	0.20	0.15	0.12
Consolidation Settlement(cm)		1.04	1.04	0.99	0.92	0.86	0.79	0.58	0.46	0.45	0.28





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

- E' : Modulus of Deformation of the Ground after Pre-Load（kN/m^2）
- E_0 : Original Modulus of Deformation of the Ground（kN/m^2）(= 700 x N)
- C : Cohesion after Pre-Load（kN/m^2）
- C_0 : Original Cohesion（kN/m^2）

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

- m : Strength Increase Ratio（Cohesive soil：m＝0.25）
- ΔP : Load Increase（kN/m^2）
- U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BL-9)

	c_0 (kN/m2)	φ(˚)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
F	0	0	-	-		-	-	-	-
As	0	27	4.20	-		-	-	-	4.20
Ac1	11	0	2.80	0.25	4.9	0.2	11	1.02	2.86
Ac2	49	0	2.80	0.25	3.7	0.2	49	1.00	2.81
GF	0	42	35.00	-		-	-	-	35.00

2.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BL-9)

Loading Width	B (m)	13.400
Loading Length	L (m)	5.600
Boring		BL-9

No.	Classification	E_i (MN/m²)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	As1	4.20	5.10	5.10	-0.08434	
2	Ac1	2.86	2.00	7.10	-0.02448	
3	Ac2	2.81	9.00	16.10	-0.05999	
4	GF	35.00	0.70	16.80	-0.00023	
5						
6						
7						
8						
9						
10						
11						
	Total		16.80		-0.16904	-0.60081
E_m (MN/m²)						3.6
(kN/m²)						3554
(tf/m²)						355

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment	Sign	Load					Loading Width	1/2 Loading Width	Location of the Center of the Load from the Control Point				
(E.L. m)		(kN/m3)		(m)		(kN/m2)	(m)	(m)	(m)				
14.40	q1	18.0	×	1.6	=	28.8	13.40	6.70	6.70			=	6.70
	q2	18.0	×	1.0	=	18.0	7.20	3.60	0.00	+	3.60	=	3.60
	q3	18.0	×	1.0	=	18.0	7.20	3.60	0.00	+	3.60	=	3.60
	q4	18.0	×	0.0	=	0.0	0.00	0.00	0.00	+	0.00	=	0.00

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Widtl ai (m)	Distance Xoi (m)
q1	Load 1	28.8	6.70	6.70
q2	Load 2	18.0	3.60	3.60
q3	Load 3	18.0	3.60	3.60
q4	Load 4	0.0	0.00	0.00

● Ground Conditon BL-9

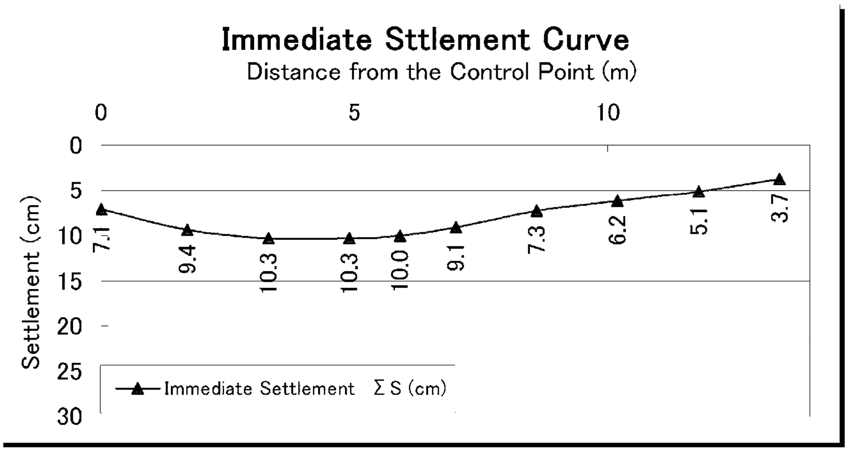
Thickness of Layer 16.8 m

Average Modulus of Elasticit 3554 kN/m2

$$Six = \sum_{i=i}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0-\frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right|+\left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

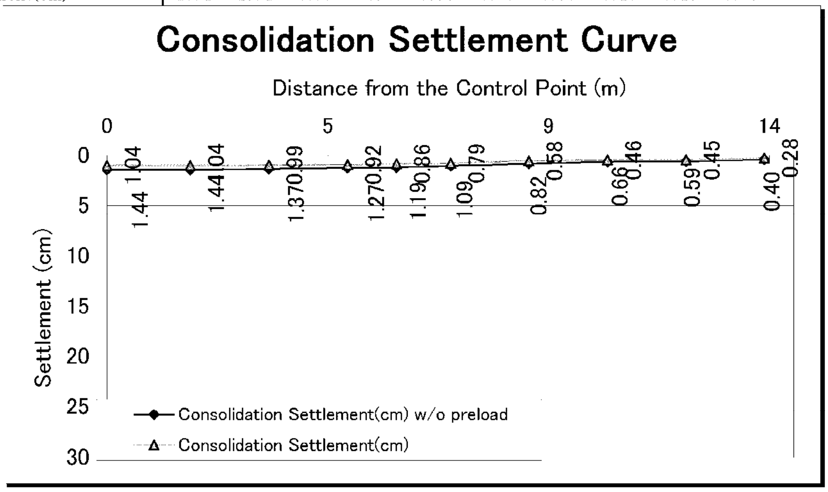
$$=\sum_{i=i}^{n}\frac{-0.9550\, a_i\cdot q_i}{E_m}\log\sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0-0.2387\left[\alpha\log|\alpha|+\beta\log|\beta|\right]\right\}$$

				Control Point									
			Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	X	(m)	Caluculation	0.0	1.7	3.3	4.9	5.9	7.0	8.6	10.2	11.8	13.4
q1	X1	(m)	Xo1-X	6.7	5.0	3.4	1.8	0.8	0.3	1.9	3.5	5.1	6.7
	S1	(cm)		3.44	4.38	4.82	5.06	5.13	5.15	5.05	4.80	4.34	3.44
q2	X2	(m)	Xo2-X	3.6	1.9	0.3	1.3	2.3	3.4	5.0	6.6	8.2	9.8
	S2	(cm)		1.82	2.53	2.72	2.63	2.43	1.99	1.13	0.71	0.40	0.15
q3	X3	(m)	Xo3-X	3.6	1.9	0.3	1.3	2.3	3.4	5.0	6.6	8.2	9.8
	S3	(cm)		1.82	2.53	2.72	2.63	2.43	1.99	1.13	0.71	0.40	0.15
Immediate Settlement			**ΣS (cm)**	**7.1**	**9.4**	**10.3**	**10.3**	**10.0**	**9.1**	**7.3**	**6.2**	**5.1**	**3.7**



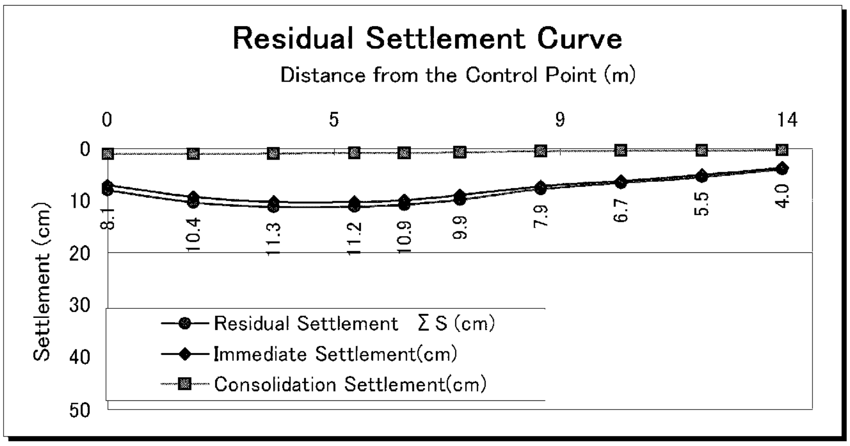
2.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.3	4.9	5.9	7.0	8.6	10.2	11.8	13.4
Consolidation Settlement(cm) w/o preload	1.44	1.44	1.37	1.27	1.19	1.09	0.82	0.66	0.59	0.40
after 90days	0.40	0.40	0.38	0.35	0.33	0.31	0.24	0.20	0.15	0.12
Consolidation Settlement(cm)	1.04	1.04	0.99	0.92	0.86	0.79	0.58	0.46	0.45	0.28



2.6 Residual Settlement

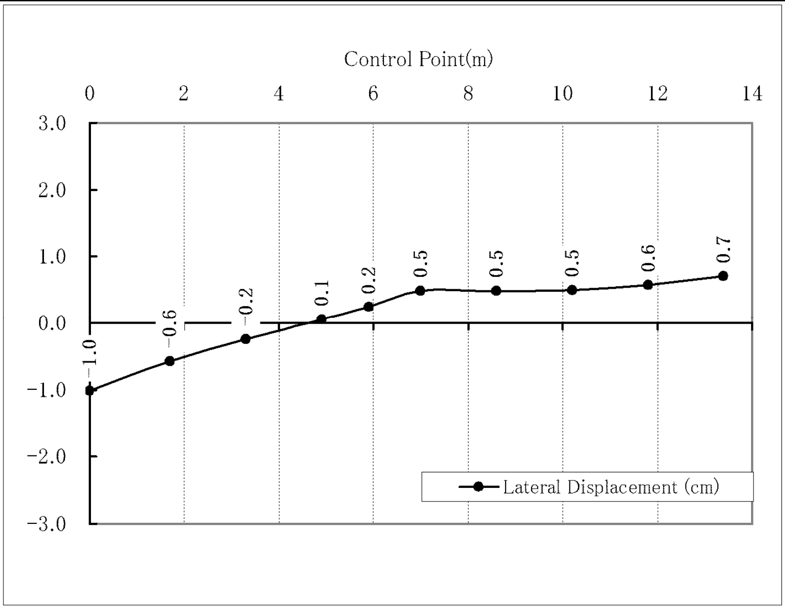
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.3	4.9	5.9	7.0	8.6	10.2	11.8	13.4
Immediate Settlement(cm)	7.10	9.40	10.30	10.30	10.00	9.10	7.30	6.20	5.10	3.70
Consolidation Settlement(cm)	1.04	1.04	0.99	0.92	0.86	0.79	0.58	0.46	0.45	0.28
Residual Settlement ΣS (cm)	**8.1**	**10.4**	**11.3**	**11.2**	**10.9**	**9.9**	**7.9**	**6.7**	**5.5**	**4.0**



2.7 Lateral Displacement

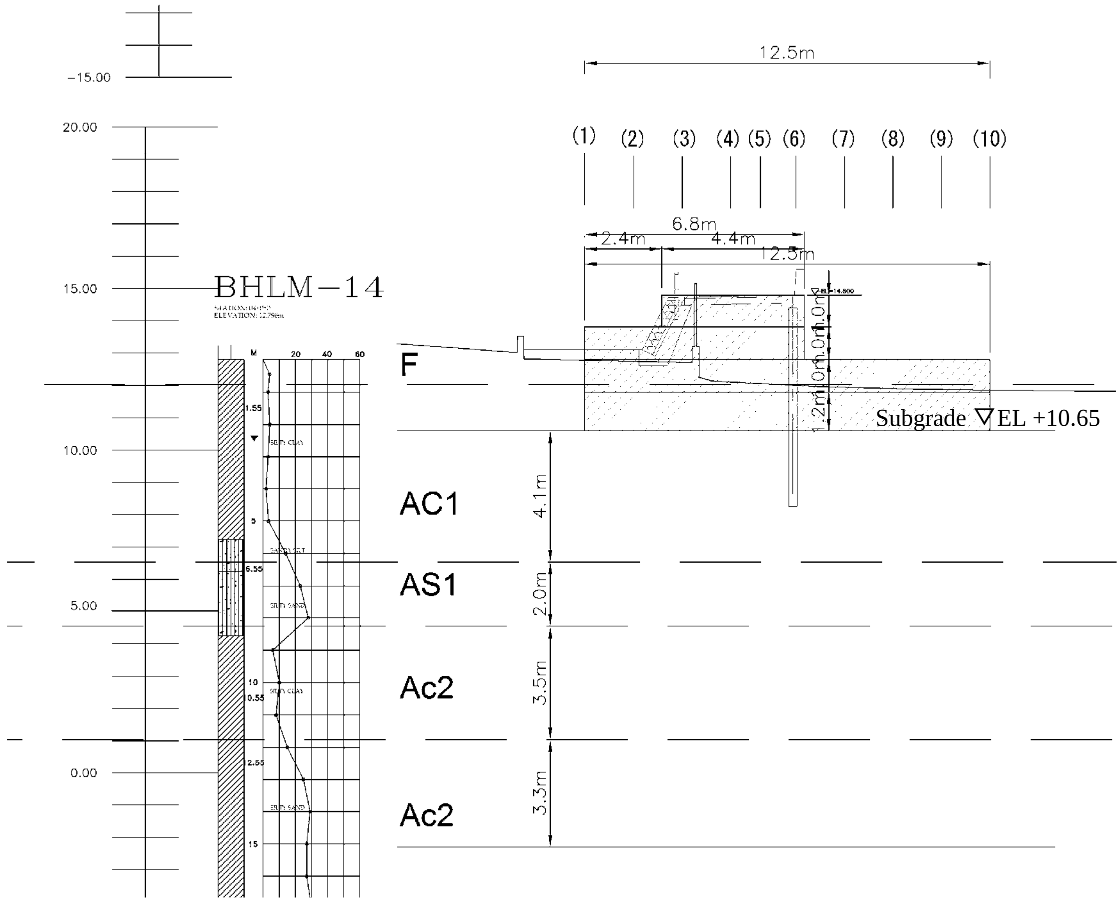
Lateral Displacement (B-9)

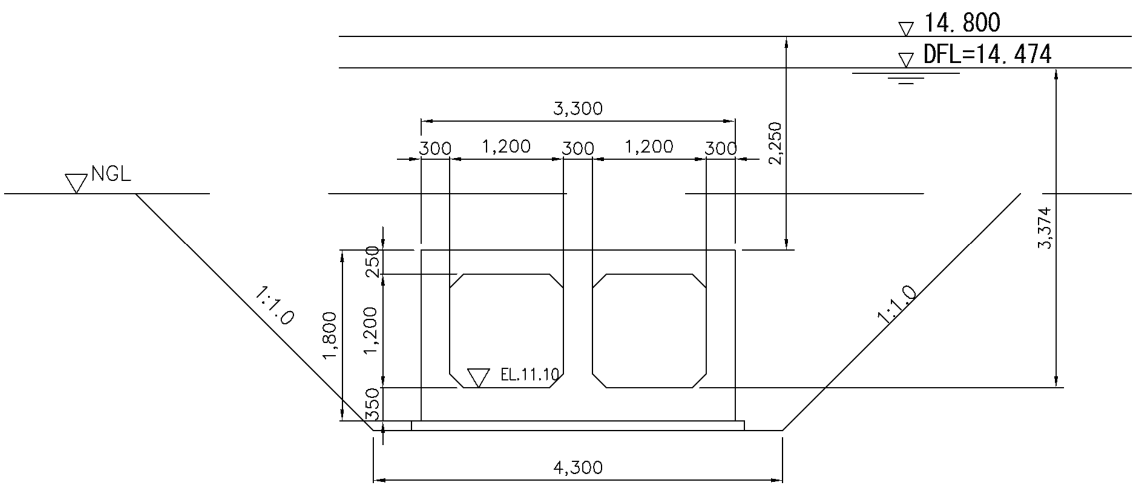
			q1		q2		q3		Total
q_i (kN/m^2)			28.800		18.000		18.000		
a_i (m)			6.700		3.600		3.600		
b_i (m)			2.800		2.800		2.800		
ν			0.400		0.400		0.400		
x_i (m)			6.700		3.600		3.600		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	R(cm)
(1)	0.0	3554	6.7	-0.52	3.6	-0.25	3.6	-0.25	-1.0
(2)	1.7	3554	5.0	-0.34	1.9	-0.12	1.9	-0.12	-0.6
(3)	3.3	3554	3.4	-0.21	0.3	-0.02	0.3	-0.02	-0.2
(4)	4.9	3554	1.8	-0.10	-1.3	0.08	-1.3	0.08	0.1
(5)	5.9	3554	0.8	-0.05	-2.3	0.15	-2.3	0.15	0.2
(6)	7.0	3554	-0.3	0.02	-3.4	0.23	-3.4	0.23	0.5
(7)	8.6	3554	-1.9	0.11	-5.0	0.19	-5.0	0.19	0.5
(8)	10.2	3554	-3.5	0.22	-6.6	0.14	-6.6	0.14	0.5
(9)	11.8	3554	-5.1	0.35	-8.2	0.11	-8.2	0.11	0.6
(10)	13.4	3554	-6.7	0.52	-9.8	0.09	-9.8	0.09	0.7



3 Calculation of MSL-3

3.1 Calculation Model





3.2 Results of Consolidation test

Sluiceway No **MSL-3**

Geological Data	
Borehole	BHLM-14
station	R 4+050
elevation	
northing	
easting	

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	1.0	F	17.0
	5.5	AC1	18.0
	6.5	AC1	18.0
	8.5	AS1	18.0
	15.5	DC	17.1
	20.0	DC	17.1

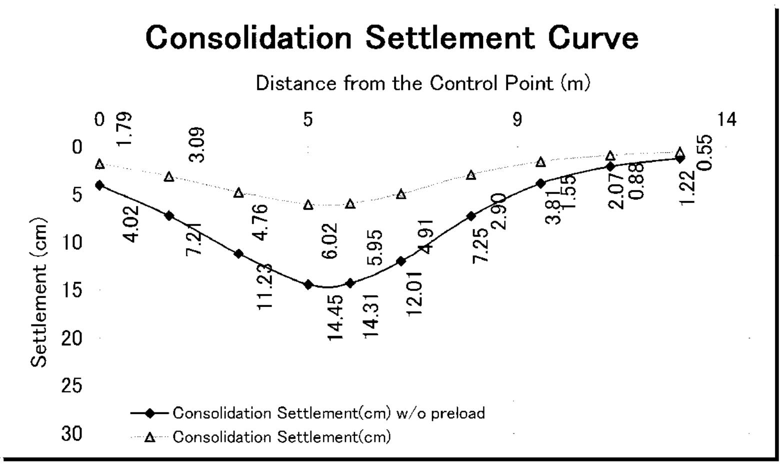
Consolidation Data		AC1	DC
PC kgf/cm^2		0.498	1.360
	p kgf/cm2		
e–logp	0.1	1.4783	1.3537
	0.2	1.4442	1.3415
	0.4	1.3791	1.3217
	0.8	1.2813	1.2874
	1.6	1.1346	1.2378
	3.2	0.9942	1.1491
	6.4	0.8539	1.0059
	12.8	0.7160	0.8404
Cv 10^{-3} cm^2/sec	0.05	12.702	14.063
	0.15	7.485	19.333
	0.3	6.673	7.545
	0.6	4.489	3.354
	1.2	9.061	2.932
	2.4	3.524	1.798
	4.8	3.061	0.943
	9.6	1.791	0.432

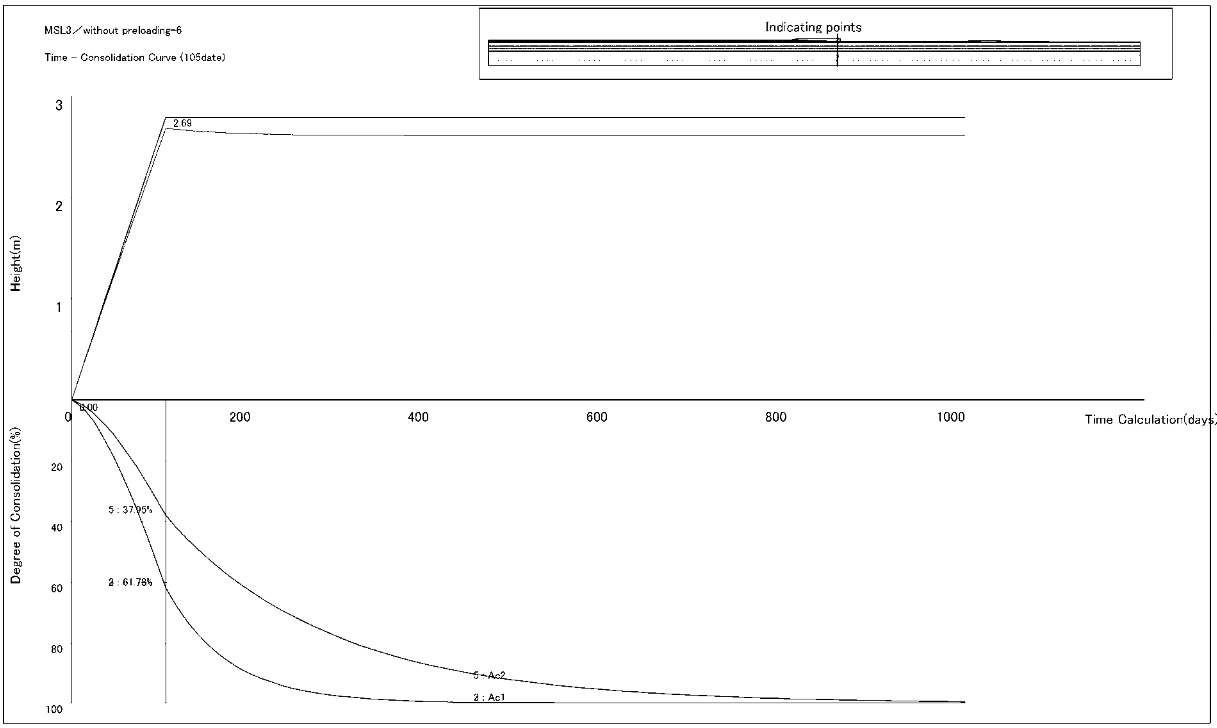
AC1 : BHLM15 4m
DC : BHLM14 15m

3.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

Control Point Location of Calculation X (m)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	0.0	1.5	3.0	4.5	5.4	6.5	8.0	9.5	11.0	12.5
Consolidation Settlement(cm) w/o preload	4.02	7.21	11.23	14.45	14.31	12.01	7.25	3.81	2.07	1.22
after 105days Consolidation Settlement(cm)	2.24	4.12	6.47	8.43	8.36	7.10	4.34	2.26	1.19	0.67
	1.79	3.09	4.76	6.02	5.95	4.91	2.90	1.55	0.88	0.55





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

- E' : Modulus of Deformation of the Ground after Pre-Load (kN/m^2)
- E_0 : Original Modulus of Deformation of the Ground (kN/m^2) (= 700 x N)
- C : Cohesion after Pre-Load (kN/m^2)
- C_0 : Original Cohesion (kN/m^2)

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

- m : Strength Increase Ratio (Cohesive soil : m＝0.25)
- ΔP : Load Increase (kN/m^2)
- U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BHML-14)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
F	0	23		-	-	-	-	-	-
Ac1	13	0	2.10	0.25	29.5	0.6	17	1.35	2.83
As	0	26	14.70	-	-	-	-	-	14.70
Ac2	72	0	2.80	0.25	13.6	0.3	73	1.01	2.84
Ac2	72	0	16.80	0.25	-	-	-	-	16.80

(1) Immediate Settlement

(2) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BHLM-14)

Loading Width	B (m)	12.500
Loading Length	L (m)	4.300
Boring		BHLM-14

No.	Classification	E_i (MN/m^2)	Thickness of Layer (m)	h_i (m)	Denominator	Numerator
1	Ac1	2.83	4.07	4.07	-0.14806	
2	As1	14.70	2.00	6.07	-0.00697	
3	Ac2	2.84	3.50	9.57	-0.04130	
4	Ac2	16.80	3.53	13.10	-0.00452	
5						
6						
7						
8						
9						
10						
11						
	Total		13.10		-0.20084	-0.71484
E_m (MN/m^2)						3.6
(kN/m^2)						3559
(tf/m^2)						356

(3) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment	Sign	Load						Loading Width	1/2 Loading Width	Location of the Center of the Load from the Control Point				
(E.L. m)		(kN/m3)		(m)		(kN/m2)		(m)	(m)	(m)				
14.80	q1	18.0	×	1.2	=	21.6		12.50	6.25	6.25			=	6.25
	q2	18.0	×	1.0	=	18.0		6.80	3.40	0.00	+	3.40	=	3.40
	q3	18.0	×	1.0	=	18.0		6.80	3.40	0.00	+	3.40	=	3.40
	q4	18.0	×	1.0	=	18.0		4.40	2.20	2.40	+	2.20	=	4.60

(4) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	21.6	6.25	6.25
q2	Load 2	18.0	3.40	3.40
q3	Load 3	18.0	3.40	3.40
q4	Load 4	18.0	2.20	4.60

● Ground Conditon BHLM-14

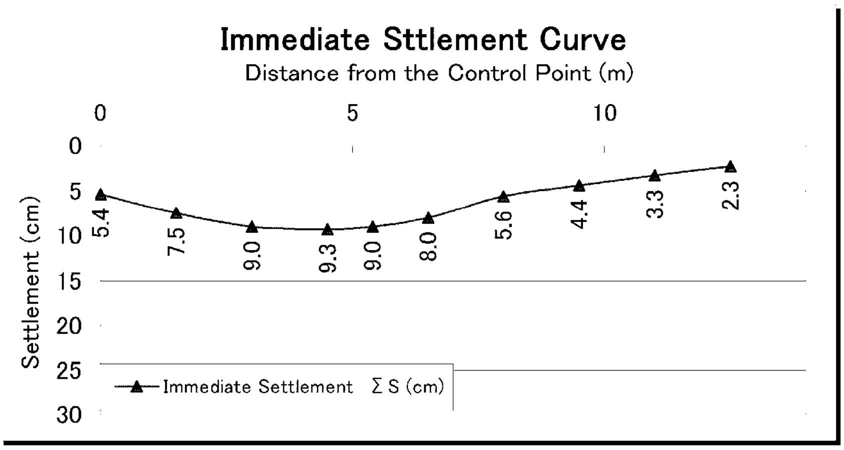
Thickness of Layer 13.1 m

Average Modulus of Elasticity 3559 kN/m2

$$Six = \sum_{i=1}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1} \frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

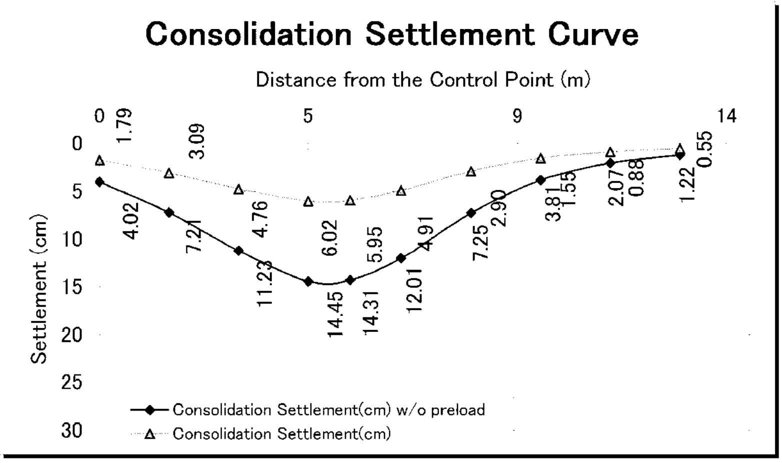
$$= \sum_{i=1}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1} \frac{a_i}{H}\right) \cdot \left\{1.0 - 0.2387\left[\alpha \log|\alpha| + \beta \log|\beta|\right]\right\}$$

				Control Point									
			Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	X	(m)	Caluculation	0.0	1.5	3.0	4.5	5.4	6.5	8.0	9.5	11.0	12.5
q1	X1	(m)	Xo1-X	6.3	4.8	3.3	1.8	0.9	0.3	1.8	3.3	4.8	6.3
	S1	(cm)		2.04	2.58	2.85	2.99	3.04	3.05	2.99	2.85	2.58	2.04
q2	X2	(m)	Xo2-X	3.4	1.9	0.4	1.1	2.0	3.1	4.6	6.1	7.6	9.1
	S2	(cm)		1.52	2.09	2.26	2.21	2.07	1.71	0.98	0.61	0.35	0.15
q3	X3	(m)	Xo3-X	3.4	1.9	0.4	1.1	2.0	3.1	4.6	6.1	7.6	9.1
	S3	(cm)		1.52	2.09	2.26	2.21	2.07	1.71	0.98	0.61	0.35	0.15
q4	X4	(m)	Xo4-X	4.6	3.1	1.6	0.1	0.8	1.9	3.4	4.9	6.4	7.9
	S4	(cm)		0.36	0.78	1.64	1.91	1.85	1.51	0.68	0.30	0.04	0.00
Immediate Settlement		ΣS (cm)		5.4	7.5	9.0	9.3	9.0	8.0	5.6	4.4	3.3	2.3



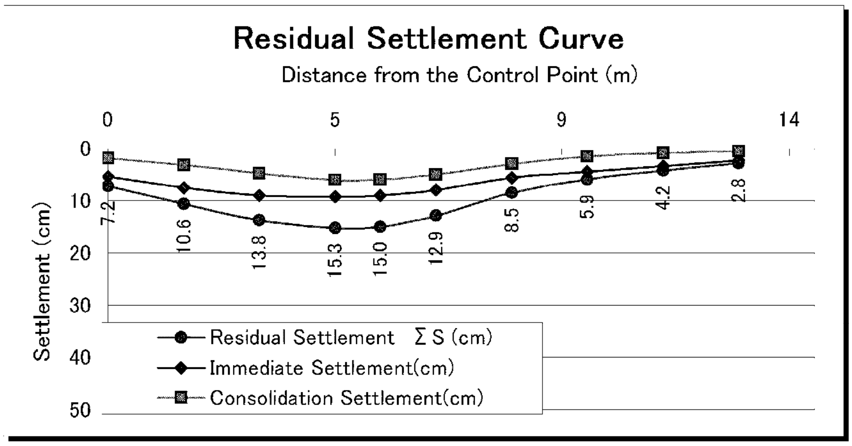
3.4 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.5	3.0	4.5	5.4	6.5	8.0	9.5	11.0	12.5
Consolidation Settlement(cm) w/o preload	4.02	7.21	11.23	14.45	14.31	12.01	7.25	3.81	2.07	1.22
after 105days	2.24	4.12	6.47	8.43	8.36	7.10	4.34	2.26	1.19	0.67
Consolidation Settlement(cm)	1.79	3.09	4.76	6.02	5.95	4.91	2.90	1.55	0.88	0.55



3.5 Residual Settlement

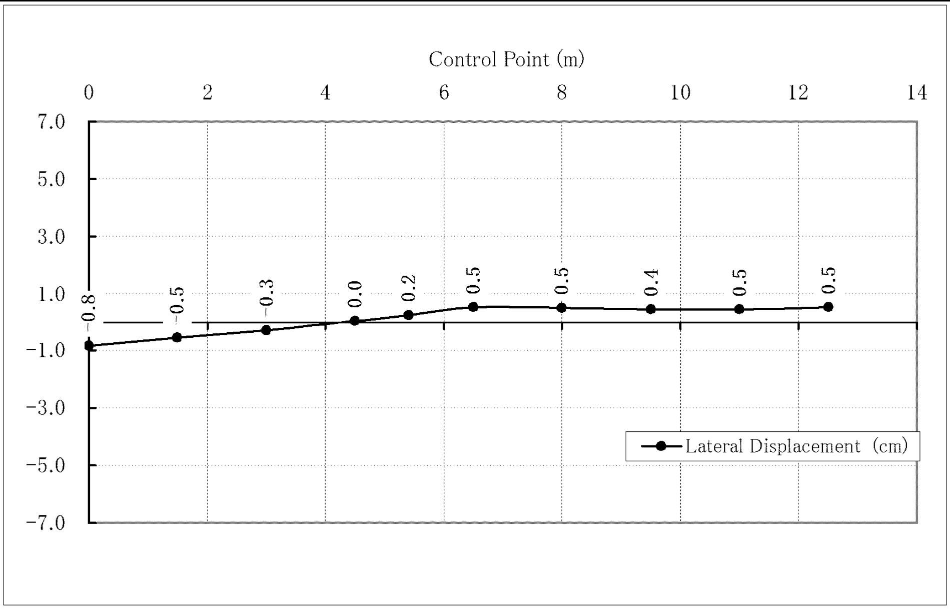
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.5	3.0	4.5	5.4	6.5	8.0	9.5	11.0	12.5
Immediate Settlement(cm)	5.40	7.50	9.00	9.30	9.00	8.00	5.60	4.40	3.30	2.30
Consolidation Settlement(cm)	1.79	3.09	4.76	6.02	5.95	4.91	2.90	1.55	0.88	0.55
Residual Settlement ΣS (cm)	**7.2**	**10.6**	**13.8**	**15.3**	**15.0**	**12.9**	**8.5**	**5.9**	**4.2**	**2.8**



3.6 Lateral Displacement

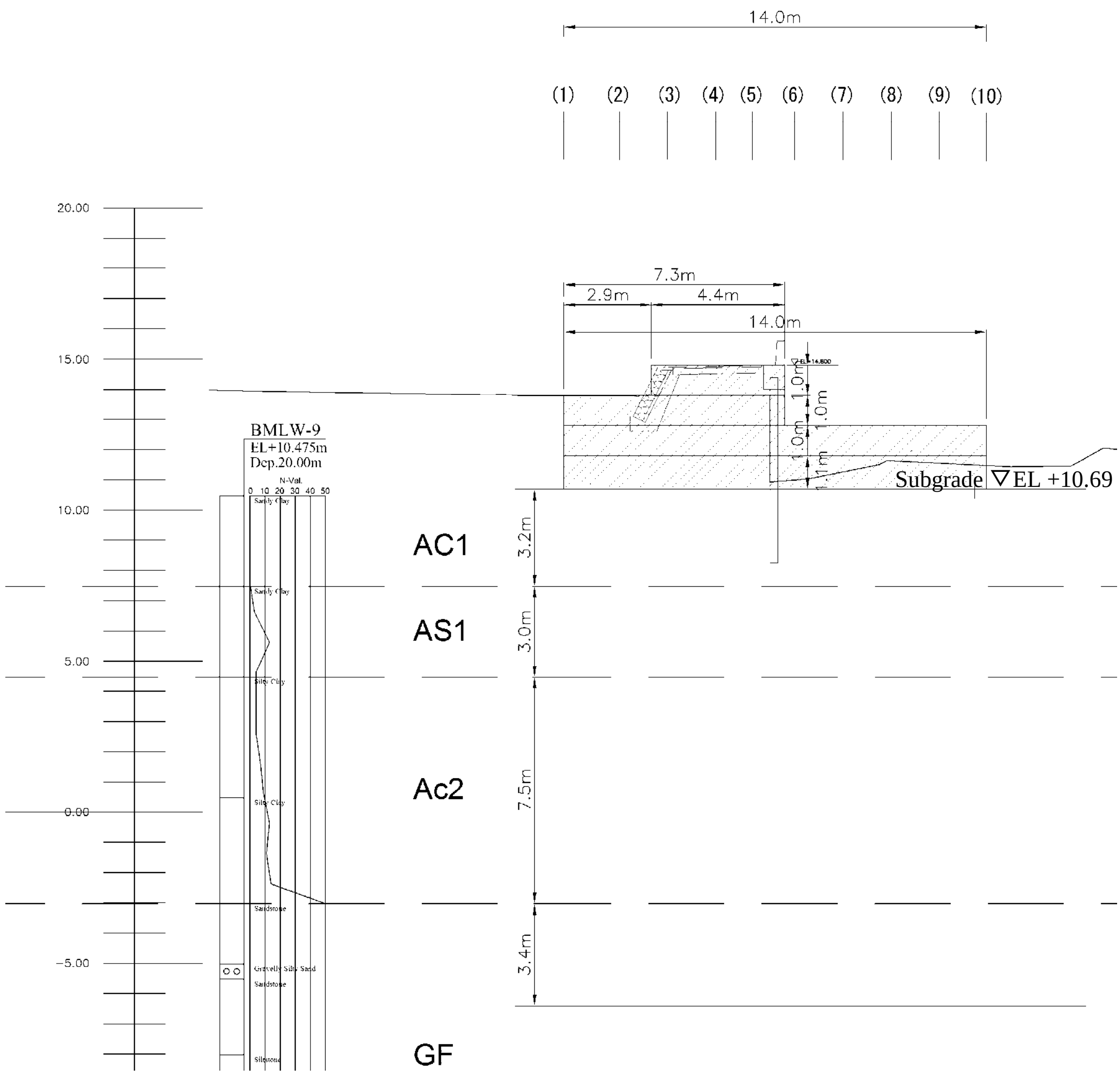
Lateral Displacement (BHLM-14)

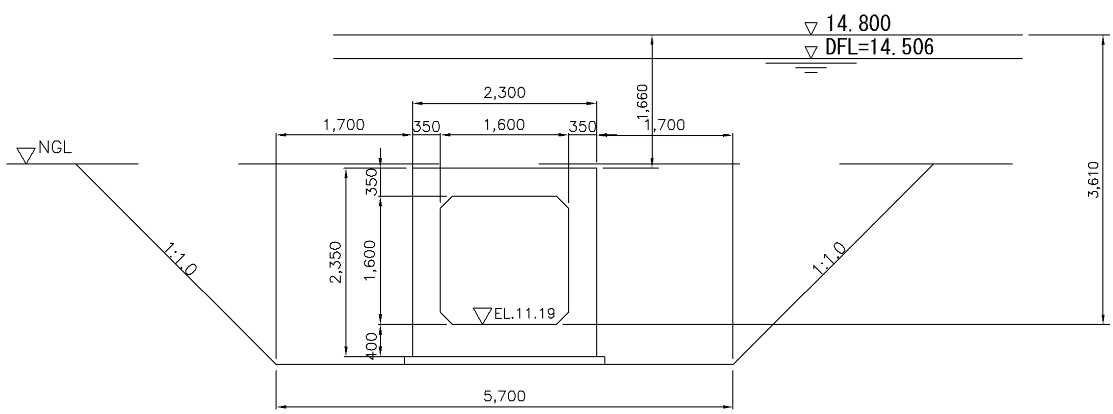
			q1		q2		q3		q4		Total
q_i (kN/m^2)			21.600		18.000		18.000		18.000		
a_i (m)			6.250		3.400		3.400		2.200		
b_i (m)			2.150		2.150		2.150		2.150		
ν			0.400		0.400		0.400		0.400		
x_i (m)			6.250		3.400		3.400		4.600		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3559	6.2	−0.32	3.4	−0.21	3.4	−0.21	4.6	−0.09	−0.8
(2)	1.5	3559	4.8	−0.21	1.9	−0.10	1.9	−0.10	3.1	−0.13	−0.5
(3)	3.0	3559	3.3	−0.13	0.4	−0.02	0.4	−0.02	1.6	−0.12	−0.3
(4)	4.5	3559	1.8	−0.06	−1.1	0.06	−1.1	0.06	0.1	−0.01	0.0
(5)	5.4	3559	0.9	−0.03	−2.0	0.11	−2.0	0.11	−0.8	0.06	0.2
(6)	6.5	3559	−0.3	0.01	−3.1	0.19	−3.1	0.19	−1.9	0.14	0.5
(7)	8.0	3559	−1.8	0.06	−4.6	0.15	−4.6	0.15	−3.4	0.12	0.5
(8)	9.5	3559	−3.3	0.13	−6.1	0.11	−6.1	0.11	−4.9	0.09	0.4
(9)	11.0	3559	−4.8	0.21	−7.6	0.09	−7.6	0.09	−6.4	0.07	0.5
(10)	12.5	3559	−6.2	0.32	−9.1	0.07	−9.1	0.07	−7.9	0.05	0.5



4 Calculation of MSL-4

4.1 Calculation Model





4.2 Results of Consolidation test

Sluiceway No	**MSL-4**
Geological Data	
Borehole	BMLW-9
station	R 4+250
elevation	10.475
northing	1613659.9
easting	508831.1

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	0.0	F	17.0
	3.0	AC1	18.0
	6.0	AS1	18.0
	13.5	DC	17.1
		GF	15.5

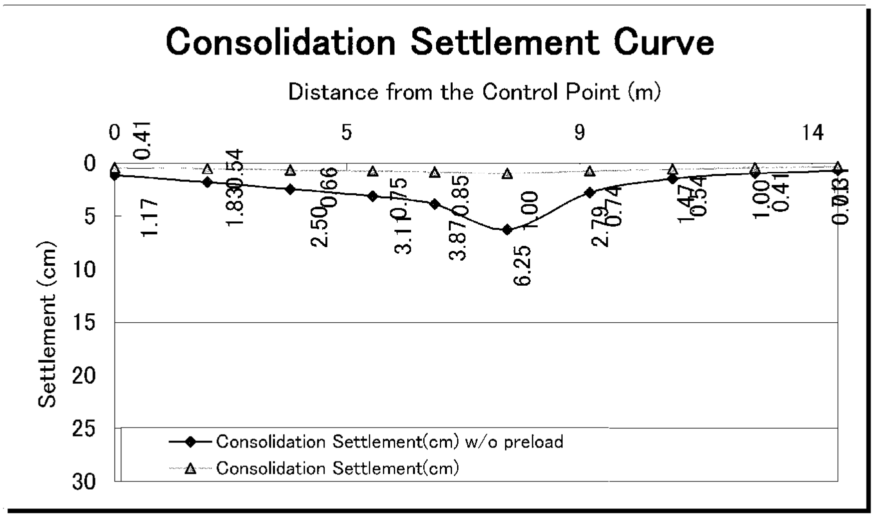
Consolidation Data		AC1	DC
PC kgf/cm^2		0.6200	1.360
	p kgf/cm2		
e-logp	0.1	1.1156	1.780
	0.2	1.0896	1.770
	0.4	1.0475	1.740
	0.8	0.9925	1.705
	1.6	0.9277	1.650
	3.2	0.8513	1.520
	6.4	0.7678	1.180
	12.8	0.6782	1.300
Cv 10^{-3} cm^2/sec	0.05	1.972	8.300
	0.15	9.302	5.800
	0.3	4.486	6.600
	0.6	5.494	6.000
	1.2	2.519	5.500
	2.4	2.341	3.500
	4.8	3.647	0.800
	9.6	3.307	0.7

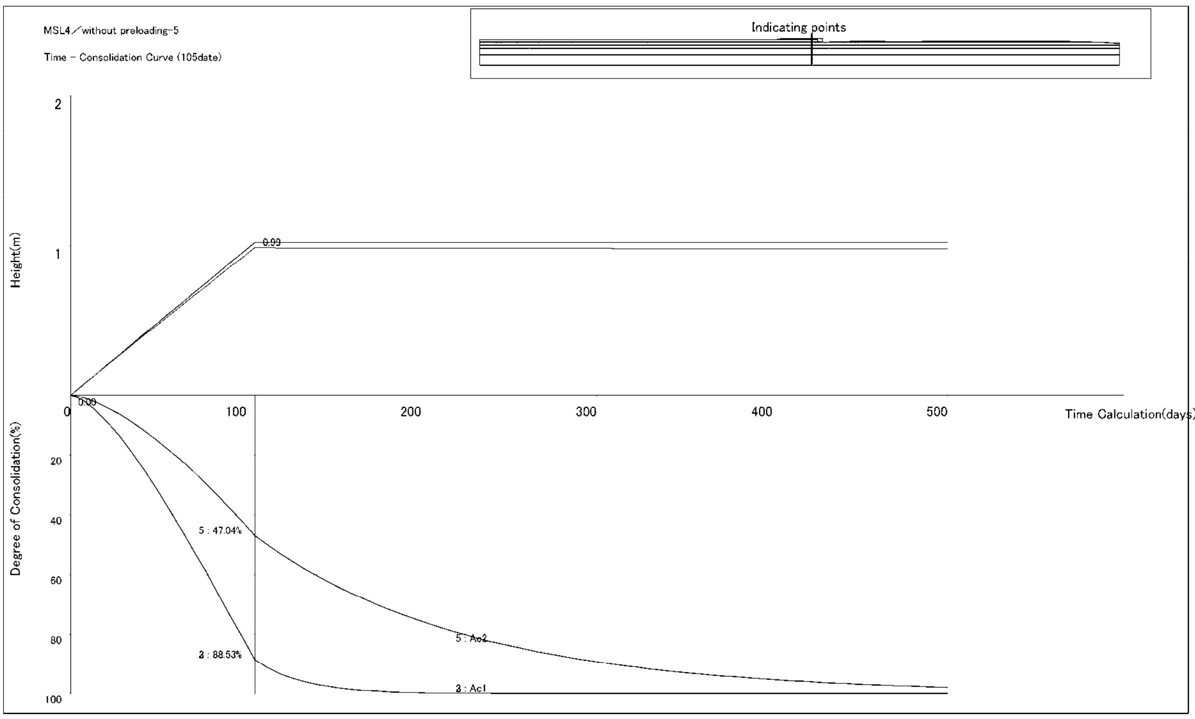
AC1 : BHLM-05 2m
DC : BMLW-9 7m

4.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.8	3.4	5.0	6.2	7.6	9.2	10.8	12.4	14.0
Consolidation Settlement(cm) w/o preload	1.17	1.83	2.50	3.11	3.87	6.25	2.79	1.47	1.00	0.71
After 105days	0.77	1.29	1.84	2.36	3.02	5.25	2.05	0.93	0.59	0.40
Consolidation Settlement(cm)	0.41	0.54	0.66	0.75	0.85	1.00	0.74	0.54	0.41	0.31





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

- E' : Modulus of Deformation of the Ground after Pre-Load (kN/m^2)
- E_0 : Original Modulus of Deformation of the Ground (kN/m^2)
- C : Cohesion after Pre-Load (kN/m^2)
- C_0 : Original Cohesion (kN/m^2)

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

- m : Strength Increase Ratio (Cohesive soil : m＝0.25)
- ΔP : Load Increase (kN/m^2)
- U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BMLW-9)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
Ac1	13	0	1.49	0.25	14.8	0.9	16	1.26	1.88
As	0	26	4.20	-	-	-	-	-	4.20
Ac2	72	0	7.00	0.25	5.3	0.4	72	1.01	7.05
GF	0	42	35.00		-	-	-	-	35.00

4.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BMLM-09)

Loading Width	B (m)	18.000
Loading Length	L (m)	5.700
Boring		BMLM-09

No.	Classification	E_i (MN/m^2)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	Ac1	1.88	3.21	3.21	-0.16666	
2	As	4.20	3.00	6.21	-0.03939	
3	Ac2	7.05	7.50	13.71	-0.03103	
4	GF	35.00	3.39	17.10	-0.00164	
5						
6						
7						
8						
9						
10						
11						
	Total		17.10		-0.23873	-0.75542
E_m (MN/m^2)						3.2
(kN/m^2)						3164
(tf/m^2)						316

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment	Sign	Load						Loading Width	1/2 Loading Width	Location of the Center of the Load from the Control Point				
(E.L. m)		(kN/m3)		(m)		(kN/m2)		(m)	(m)	(m)				
15.00	q1	18.0	×	1.1	=	19.8		14.00	7.00	7.00			=	7.00
	q2	18.0	×	1.0	=	18.0		7.30	3.65	0.00	+	3.65	=	3.65
	q3	18.0	×	1.0	=	18.0		7.30	3.65	0.00	+	3.65	=	3.65
	q4	18.0	×	1.0	=	18.0		4.40	2.20	2.90	+	2.20	=	5.10

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	19.8	7.00	7.00
q2	Load 2	18.0	3.65	3.65
q3	Load 3	18.0	3.65	3.65
q4	Load 4	18.0	2.20	5.10

● Ground Conditon BMLM-09

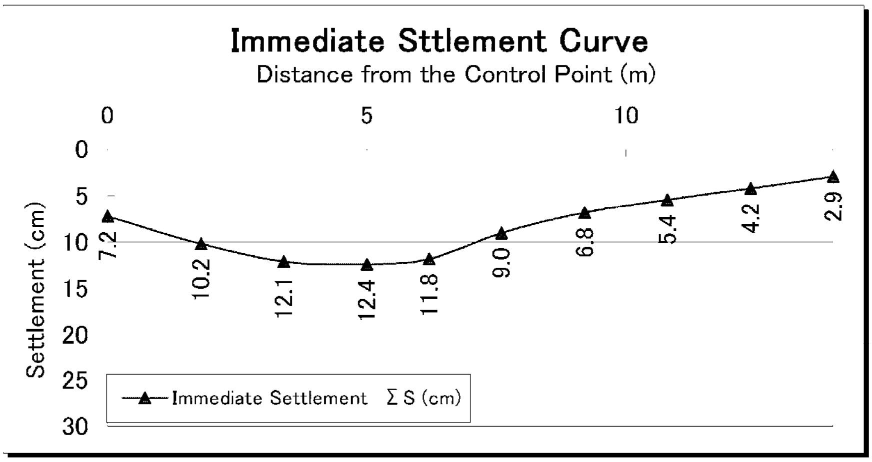
Thickness of Layer 17.1 m

Average Modulus of Elasticity 3164 kN/m2

$$Six = \sum_{i=c}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

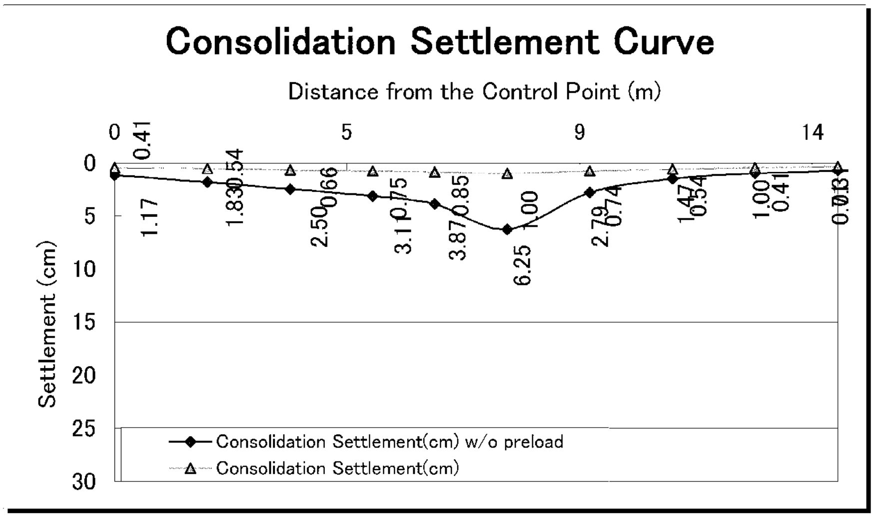
$$= \sum_{i=c}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - 0.2387\left[\alpha \log|\alpha| + \beta \log|\beta|\right]\right\}$$

			Location of Caluculation	Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	X	(m)		0.0	1.8	3.4	5.0	6.2	7.6	9.2	10.8	12.4	14.0
q1	X1	(m)	Xo1-X	7.0	5.2	3.6	2.0	0.8	0.6	2.2	3.8	5.4	7.0
	S1	(cm)		2.72	3.46	3.79	3.98	4.05	4.05	3.96	3.76	3.41	2.72
q2	X2	(m)	Xo2-X	3.7	1.9	0.3	1.4	2.6	4.0	5.6	7.2	8.8	10.4
	S2	(cm)		2.08	2.91	3.10	3.00	2.71	1.82	1.13	0.70	0.37	0.11
q3	X3	(m)	Xo3-X	3.7	1.9	0.3	1.4	2.6	4.0	5.6	7.2	8.8	10.4
	S3	(cm)		2.08	2.91	3.10	3.00	2.71	1.82	1.13	0.70	0.37	0.11
q4	X4	(m)	Xo4-X	5.1	3.3	1.7	0.1	1.1	2.5	4.1	5.7	7.3	8.9
	S4	(cm)		0.34	0.91	2.06	2.46	2.31	1.35	0.62	0.20	0.00	0.00
Immediate Settlement			ΣS (cm)	7.2	10.2	12.1	12.4	11.8	9.0	6.8	5.4	4.2	2.9



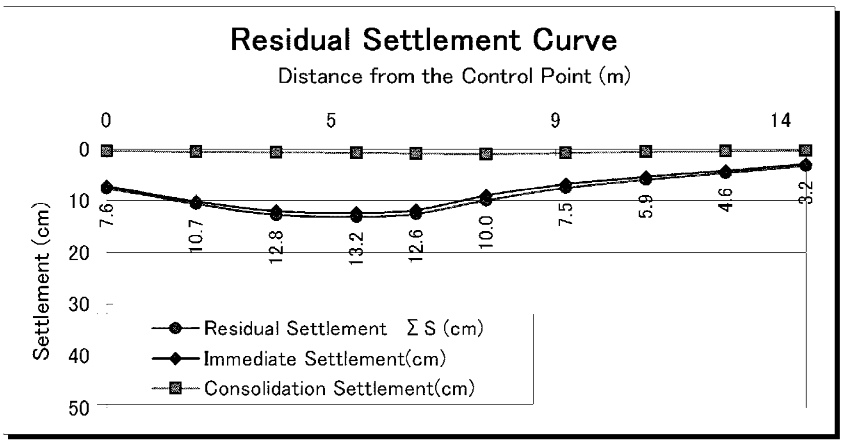
4.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.8	3.4	5.0	6.2	7.6	9.2	10.8	12.4	14.0
Consolidation Settlement(cm) w/o preload	1.17	1.83	2.50	3.11	3.87	6.25	2.79	1.47	1.00	0.71
After 105days	0.77	1.29	1.84	2.36	3.02	5.25	2.05	0.93	0.59	0.40
Consolidation Settlement(cm)	0.41	0.54	0.66	0.75	0.85	1.00	0.74	0.54	0.41	0.31



4.6 Residual Settlement

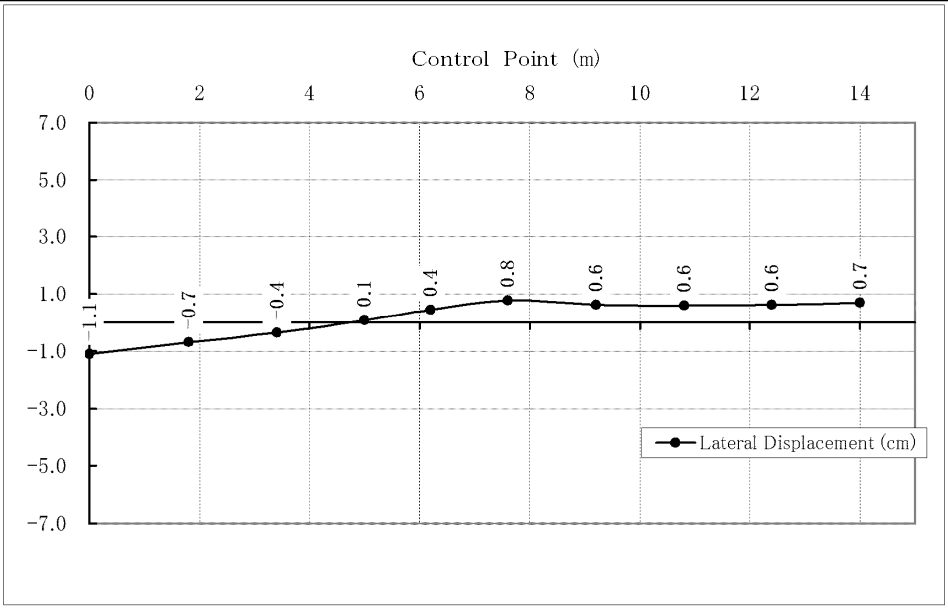
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.8	3.4	5.0	6.2	7.6	9.2	10.8	12.4	14.0
Immediate Settlement(cm)	7.20	10.20	12.10	12.40	11.80	9.00	6.80	5.40	4.20	2.90
Consolidation Settlement(cm)	0.41	0.54	0.66	0.75	0.85	1.00	0.74	0.54	0.41	0.31
Residual Settlement ΣS (cm)	**7.6**	**10.7**	**12.8**	**13.2**	**12.6**	**10.0**	**7.5**	**5.9**	**4.6**	**3.2**



4.7 Lateral Displacement

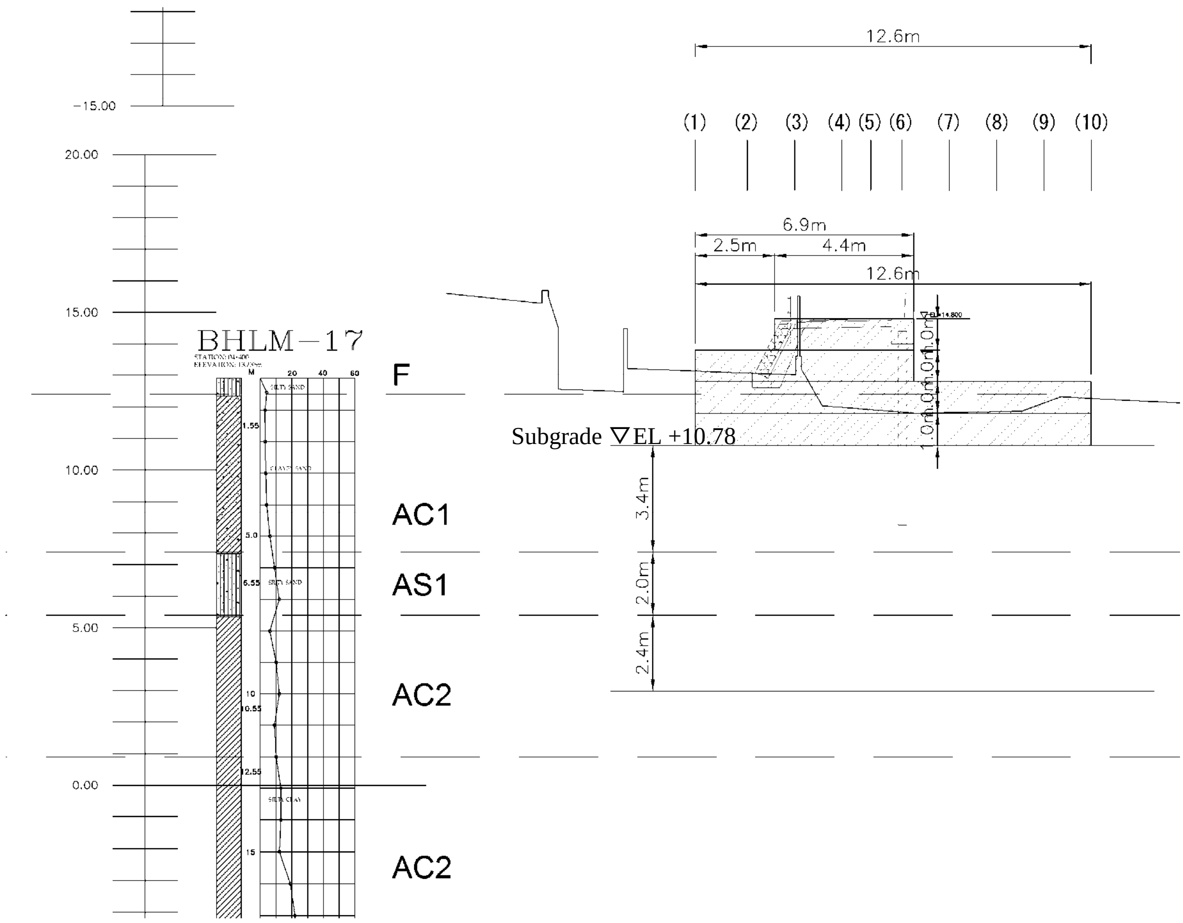
Lateral Displacement (BMLM-09)

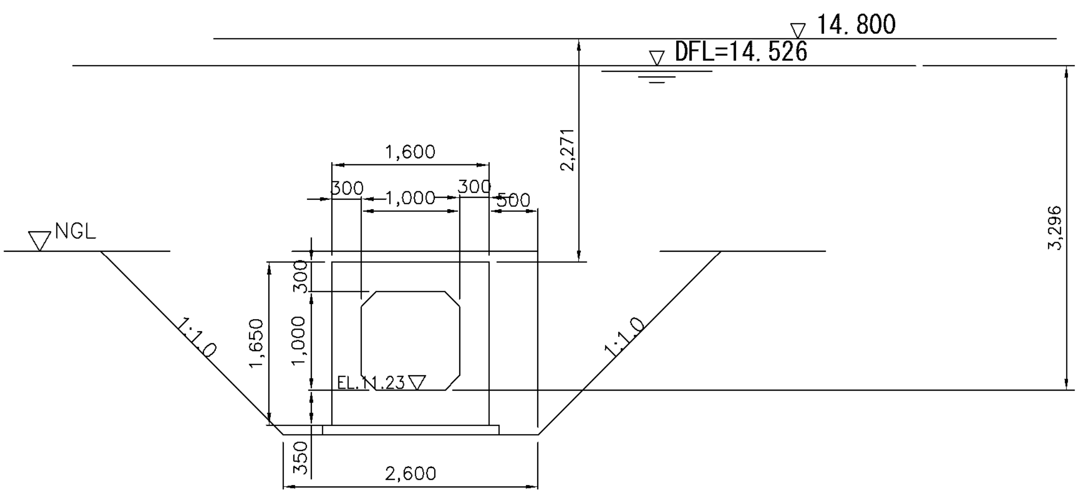
			q1		q2		q3		q4		Total
q_i (kN/m^2)			19.800		18.000		18.000		18.000		
a_i (m)			7.000		3.650		3.650		2.200		
b_i (m)			2.850		2.850		2.850		2.850		
ν			0.400		0.400		0.400		0.400		
x_i (m)			7.000		3.650		3.650		5.100		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3164	7.0	-0.41	3.6	-0.28	3.6	-0.28	5.1	-0.12	-1.1
(2)	1.8	3164	5.2	-0.26	1.9	-0.13	1.9	-0.13	3.3	-0.17	-0.7
(3)	3.4	3164	3.6	-0.17	0.3	-0.02	0.3	-0.02	1.7	-0.16	-0.4
(4)	5.0	3164	2.0	-0.09	-1.4	0.09	-1.4	0.09	0.1	-0.01	0.1
(5)	6.2	3164	0.8	-0.03	-2.6	0.18	-2.6	0.18	-1.1	0.10	0.4
(6)	7.6	3164	-0.6	0.03	-4.0	0.27	-4.0	0.27	-2.5	0.20	0.8
(7)	9.2	3164	-2.2	0.10	-5.6	0.19	-5.6	0.19	-4.1	0.14	0.6
(8)	10.8	3164	-3.8	0.18	-7.2	0.15	-7.2	0.15	-5.7	0.11	0.6
(9)	12.4	3164	-5.4	0.28	-8.8	0.12	-8.8	0.12	-7.3	0.09	0.6
(10)	14.0	3164	-7.0	0.41	-10.3	0.10	-10.3	0.10	-8.9	0.07	0.7



5 Calculation of MSL-5

5.1 Calculation Model





5.2 Results of Consolidation test

Sluiceway No	MSL-5
Geological Data	
Borehole	BHLM-17
station	R 4+400
elevation	13.235
northing	1613674.3
easting	508843.5

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	0.5	F	17.0
	5.5	AC1	18.0
	7.5	AS1	18.0
	9.5	AC1	18.0
	18.5	DC	17.1
	20.0	GF	15.5

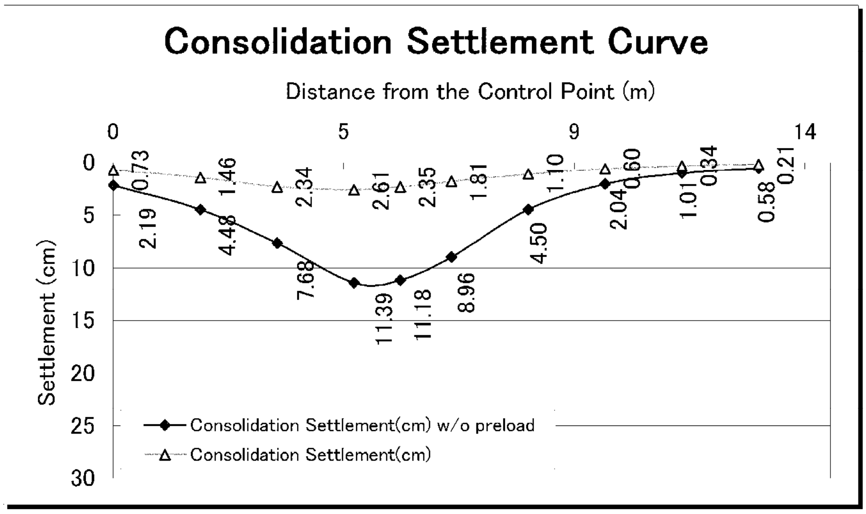
Consolidation Data		AC1	DC
PC kgf/cm^2		0.8025	1.360
	p kgf/cm2		
e-logp	0.1	1.412	0.4512
	0.2	1.376	0.4446
	0.4	1.316	0.4327
	0.8	1.232	0.4176
	1.6	1.108	0.4008
	3.2	0.977	0.3804
	6.4	0.834	0.3563
	12.8	0.687	0.332
Cv 10^{-3} cm^2/sec	0.05	6.418	7.168
	0.15	5.388	7.638
	0.3	3.602	6.100
	0.6	7.453	4.410
	1.2	3.360	10.883
	2.4	3.725	5.699
	4.8	3.203	8.622
	9.6	2.581	6.067

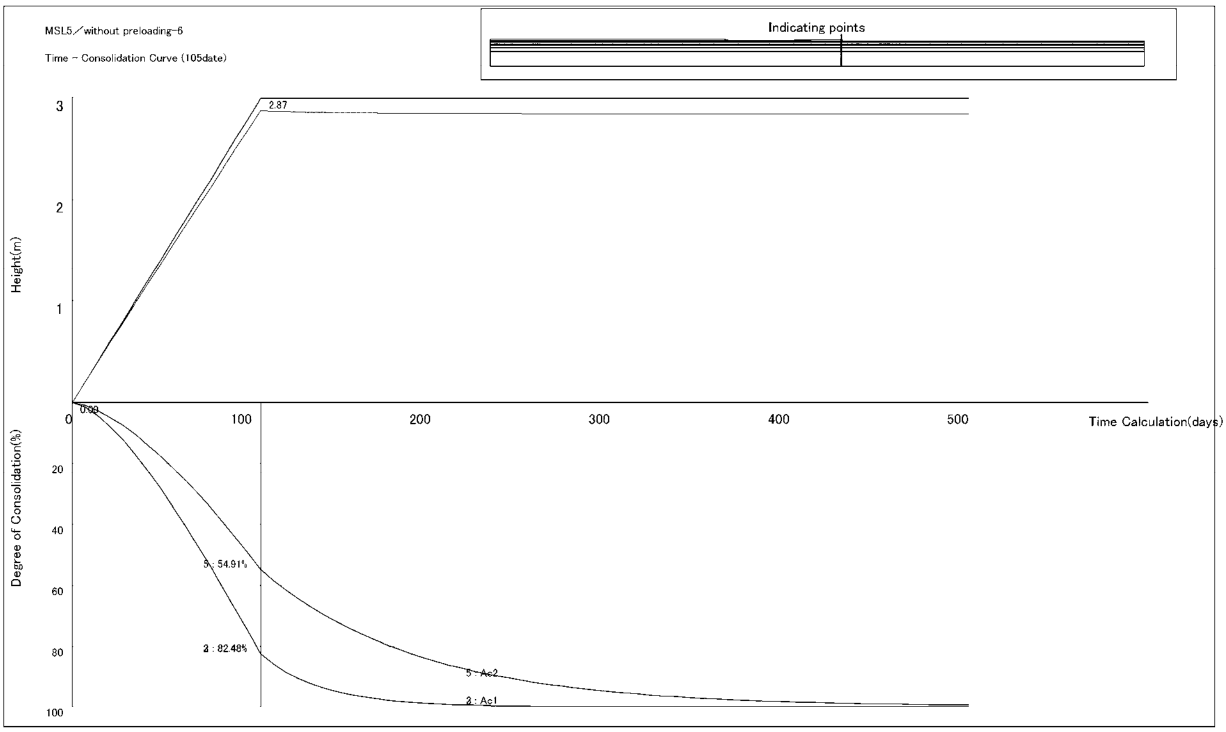
AC1 : Average of BHLM17 3m & 8m
DC : BHLM05 14m

5.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

Control Point										
Location of Caluculation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	0.0	1.7	3.2	4.7	5.6	6.6	8.1	9.6	11.1	12.6
Consolidation Settlement(cm) w/o preload	2.19	4.48	7.68	11.39	11.18	8.96	4.50	2.04	1.01	0.58
after 105days	1.46	3.01	5.35	8.78	8.83	7.15	3.40	1.44	0.67	0.38
Consolidation Settlement(cm)	0.73	1.46	2.34	2.61	2.35	1.81	1.10	0.60	0.34	0.21





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

E' : Modulus of Deformation of the Ground after Pre-Load（kN/m^2）

E_0 : Original Modulus of Deformation of the Ground（kN/m^2）(= 700 x N)

C : Cohesion after Pre-Load（kN/m^2）

C_0 : Original Cohesion（kN/m^2）

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

m : Strength Increase Ratio（Cohesive soil：m＝0.25）

ΔP : Load Increase（kN/m^2）

U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BMLM-17)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
Ac1	13	0	2.10	0.25	32.5	0.8	19	1.51	3.17
As	0	26	7.00	-	-	-	-	-	7.00
Ac2	72	0	6.30	0.25	13.5	0.5	73	1.02	6.45

5.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BHLM-17)

Loading Width	B (m)	18.500
Loading Length	L (m)	2.600
Boring		BHLM-17

No.	Classification	E_i (MN/m^2)	Thickness of Layer (m)	h_i (m)	Denominator	Numerator
1	Ac1	3.17	3.38	3.38	-0.22850	
2	As	7.00	2.00	5.38	-0.02940	
3	Ac2	6.45	2.42	7.80	-0.02610	
4						
5						
6						
7						
8						
9						
10						
11						
	Total		7.80		-0.28399	-1.09914
E_m (MN/m^2)						3.9
(kN/m^2)						3870
(tf/m^2)						387

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment (E.L. m)	Sign	Load (kN/m3)		(m)		(kN/m2)	Loading Width (m)	1/2 Loading Width (m)	Location of the Center of the Load from the Control Point (m)			
14.80	q1	18.0	×	1.0	=	18.0	12.60	6.30	6.30		=	6.30
	q2	18.0	×	1.0	=	18.0	6.90	3.45	0.00 +	3.45	=	3.45
	q3	18.0	×	1.0	=	18.0	6.90	3.45	0.00 +	3.45	=	3.45
	q4	18.0	×	1.0	=	18.0	4.40	2.20	2.50 +	2.20	=	4.70

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	18.0	6.30	6.30
q2	Load 2	18.0	3.45	3.45
q3	Load 3	18.0	3.45	3.45
q4	Load 4	18.0	2.20	4.70

● Ground Conditon BHLM-17

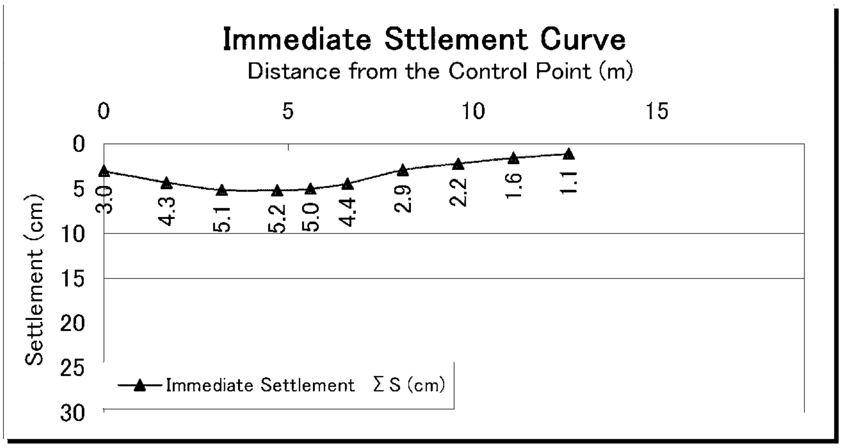
Thickness of Layer 7.8 m

Average Modulus of Elasticity 3870 kN/m2

$$Six = \sum_{i-c}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

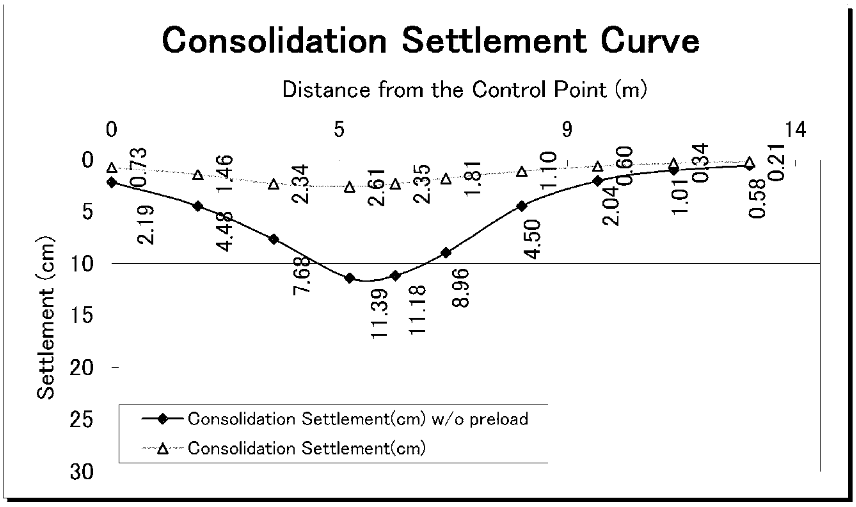
$$= \sum_{i-c}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \{1.0 - 0.2387\,[\alpha \log|\alpha| + \beta \log|\beta|]\}$$

	X	(m)	Location of Caluculation	Control Point (1) 0.0	(2) 1.7	(3) 3.2	(4) 4.7	(5) 5.6	(6) 6.6	(7) 8.1	(8) 9.6	(9) 11.1	(10) 12.6
q1	X1	(m)	Xo1-X	6.3	4.6	3.1	1.6	0.7	0.3	1.8	3.3	4.8	6.3
	S1	(cm)		0.87	1.12	1.22	1.28	1.30	1.30	1.27	1.21	1.10	0.87
q2	X2	(m)	Xo2-X	3.5	1.8	0.3	1.3	2.2	3.2	4.7	6.2	7.7	9.2
	S2	(cm)		0.93	1.30	1.39	1.34	1.25	1.05	0.60	0.38	0.22	0.10
q3	X3	(m)	Xo3-X	3.5	1.8	0.3	1.3	2.2	3.2	4.7	6.2	7.7	9.2
	S3	(cm)		0.93	1.30	1.39	1.34	1.25	1.05	0.60	0.38	0.22	0.10
q4	X4	(m)	Xo4-X	4.7	3.0	1.5	0.0	0.9	1.9	3.4	4.9	6.4	7.9
	S4	(cm)		0.23	0.54	1.12	1.27	1.22	1.00	0.45	0.20	0.03	0.00
Immediate Settlement			ΣS (cm)	**3.0**	**4.3**	**5.1**	**5.2**	**5.0**	**4.4**	**2.9**	**2.2**	**1.6**	**1.1**



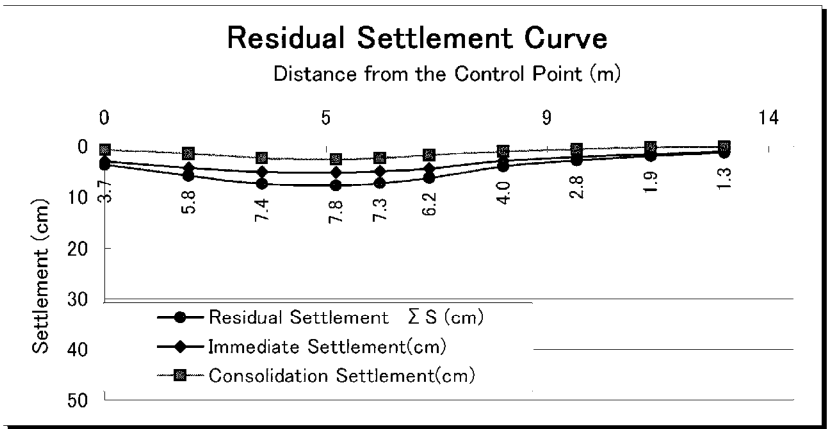
5.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.2	4.7	5.6	6.6	8.1	9.6	11.1	12.6
Consolidation Settlement(cm) w/o preload	2.19	4.48	7.68	11.39	11.18	8.96	4.50	2.04	1.01	0.58
after 105days	1.46	3.01	5.35	8.78	8.83	7.15	3.40	1.44	0.67	0.38
Consolidation Settlement(cm)	0.73	1.46	2.34	2.61	2.35	1.81	1.10	0.60	0.34	0.21



5.6 Residual Settlement

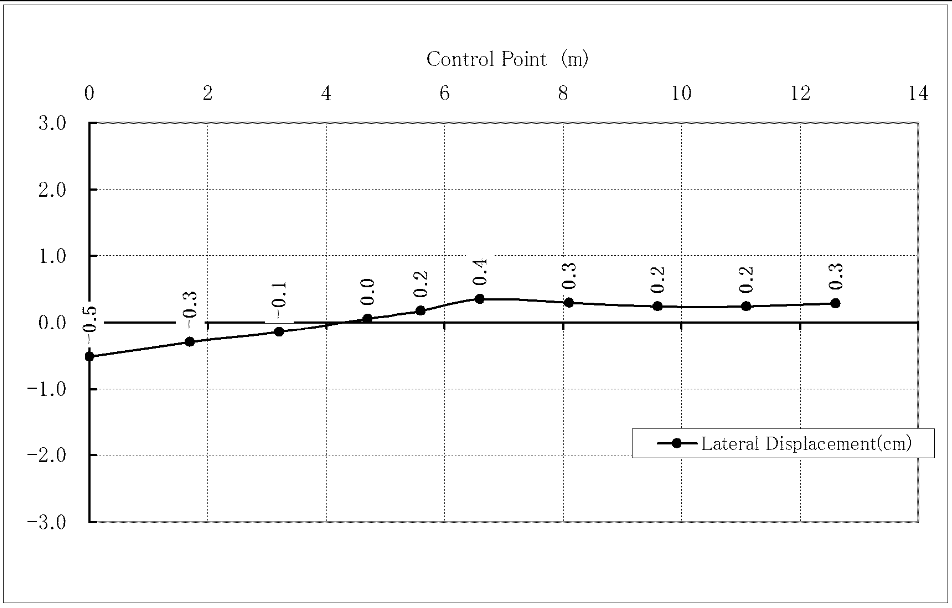
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.2	4.7	5.6	6.6	8.1	9.6	11.1	12.6
Immediate Settlement(cm)	3.00	4.30	5.10	5.20	5.00	4.40	2.90	2.20	1.60	1.10
Consolidation Settlement(cm)	0.73	1.46	2.34	2.61	2.35	1.81	1.10	0.60	0.34	0.21
Residual Settlement ΣS (cm)	**3.7**	**5.8**	**7.4**	**7.8**	**7.3**	**6.2**	**4.0**	**2.8**	**1.9**	**1.3**



5.7 Lateral Displacement

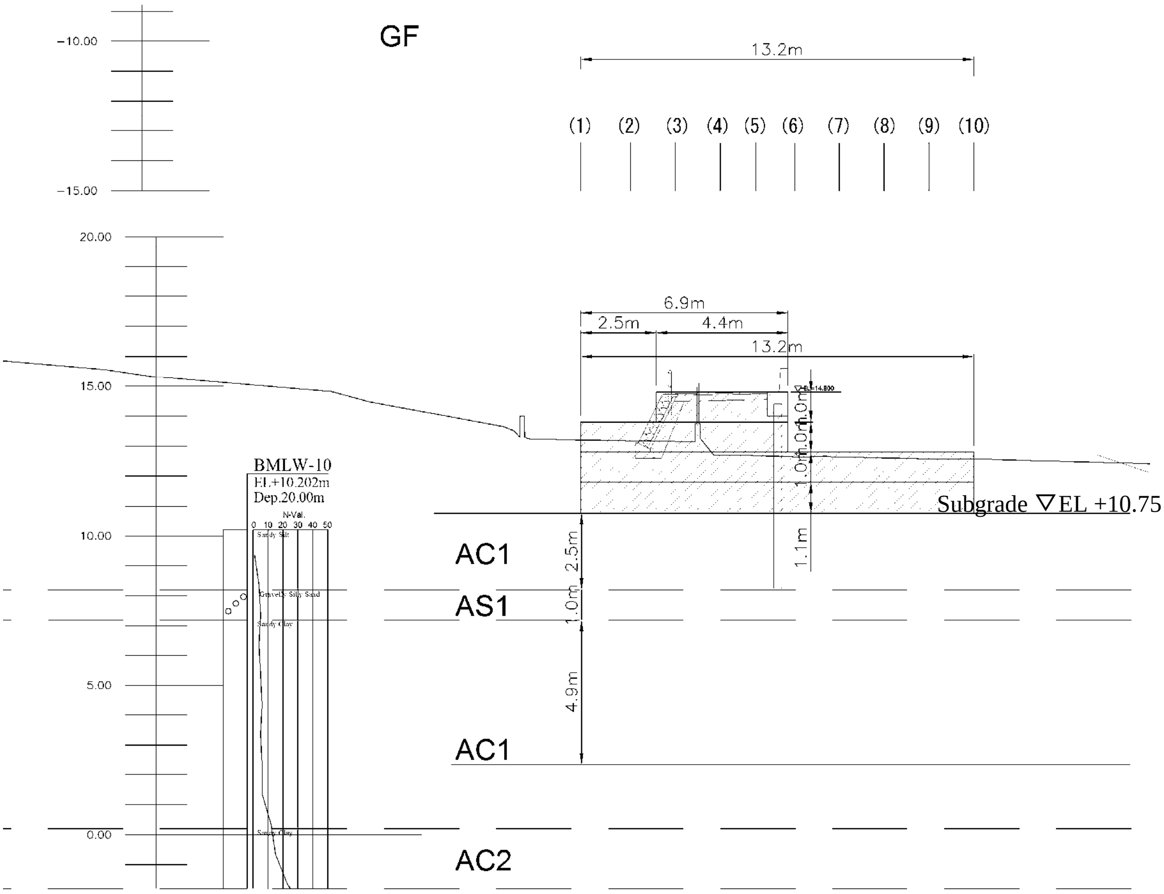
Lateral Displacement（BHLM-17）

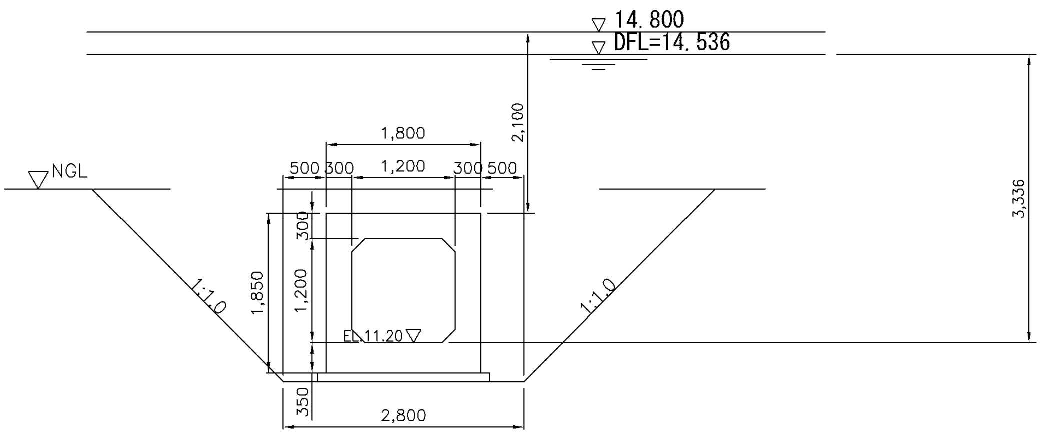
			q1		q2		q3		q4		Total
q_i (kN/m^2)			18.000		18.000		18.000		18.000		
a_i (m)			6.300		3.450		3.450		2.200		
b_i (m)			1.300		1.300		1.300		1.300		
ν			0.400		0.400		0.400		0.400		
x_i (m)			6.300		3.450		3.450		4.700		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3870	6.3	-0.18	3.4	-0.14	3.4	-0.14	4.7	-0.05	-0.5
(2)	1.7	3870	4.6	-0.10	1.8	-0.06	1.8	-0.06	3.0	-0.09	-0.3
(3)	3.2	3870	3.1	-0.06	0.3	-0.01	0.3	-0.01	1.5	-0.07	-0.1
(4)	4.7	3870	1.6	-0.03	-1.3	0.04	-1.3	0.04	0.0	0.00	0.0
(5)	5.6	3870	0.7	-0.01	-2.2	0.07	-2.2	0.07	-0.9	0.04	0.2
(6)	6.6	3870	-0.3	0.01	-3.2	0.12	-3.2	0.12	-1.9	0.10	0.4
(7)	8.1	3870	-1.8	0.03	-4.7	0.10	-4.7	0.10	-3.4	0.08	0.3
(8)	9.6	3870	-3.3	0.06	-6.2	0.07	-6.2	0.07	-4.9	0.05	0.2
(9)	11.1	3870	-4.8	0.10	-7.7	0.05	-7.7	0.05	-6.4	0.04	0.2
(10)	12.6	3870	-6.3	0.18	-9.1	0.04	-9.1	0.04	-7.9	0.03	0.3



6 Calculation of MSL-6

6.1 Calculation Model





6.2 Results of Consolidation test

Sluiceway No	MSL-6
Geological Data	
Borehole	BMLW-10
station	R 4+500
elevation	10.202
northing	1613908.5
easting	508788.75

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation		F	17.0
	2.0	AC1	18.0
	3.0	AS1	18.0
	10.0	AC1	18.0
	12.0	DC	17.1
	20.0	GF	15.5

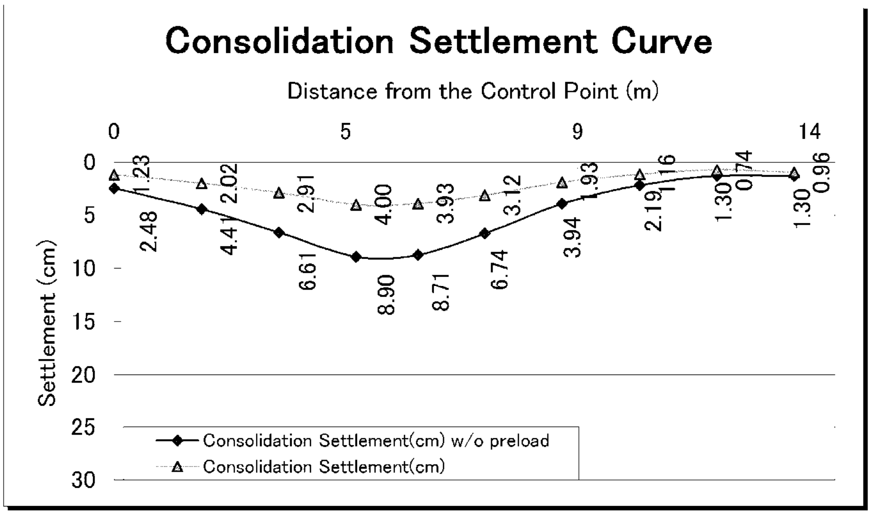
Consolidation Data		AC1	DC
PC kgf/cm^2		0.5818	0.800
	p kgf/cm2		
e-logp	0.1	1.4131	0.7352
	0.2	1.3513	0.7237
	0.4	1.2730	0.7072
	0.8	1.1659	0.6867
	1.6	1.0467	0.6599
	3.2	0.9258	0.6262
	6.4	0.8019	0.5880
	12.8	0.6696	0.5382
Cv 10^{-3} cm^2/sec	0.05	2.1975	2.9080
	0.15	1.8865	7.1060
	0.3	1.7410	11.3250
	0.6	1.6560	4.1770
	1.2	2.8880	13.0370
	2.4	1.3823	12.5700
	4.8	1.4393	7.7090
	9.6	1.6745	14.0460

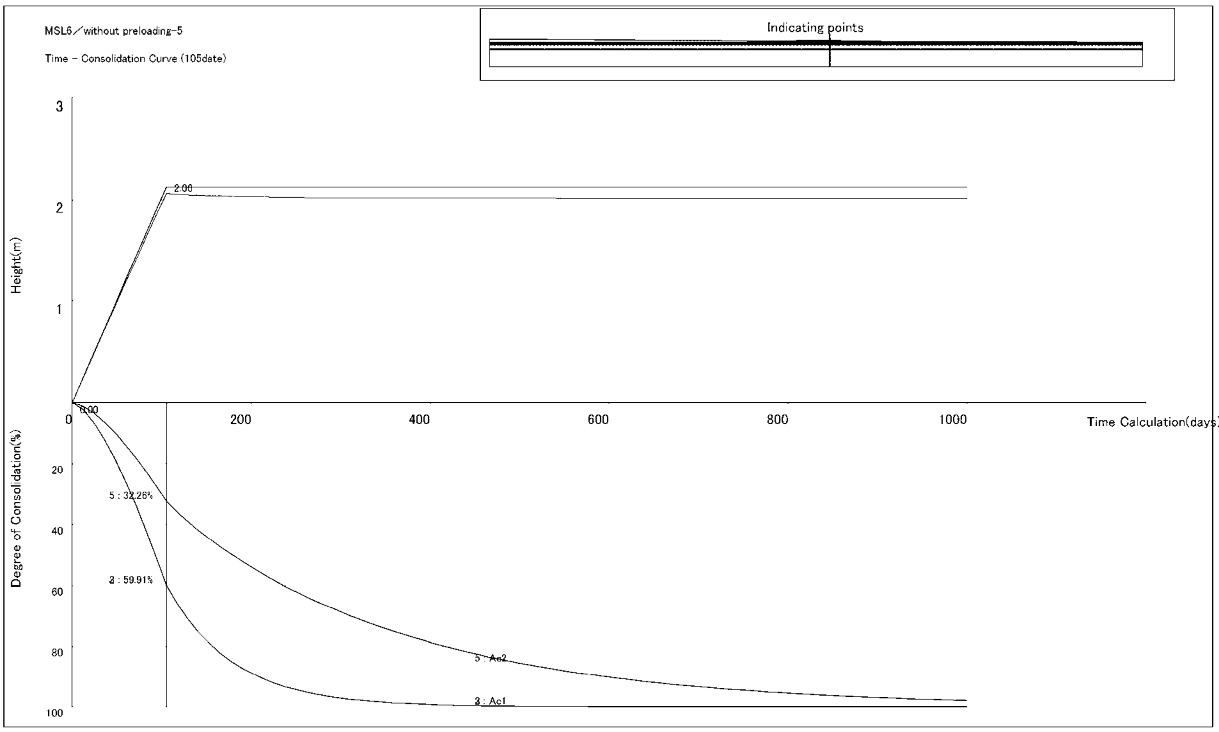
AC1 : Average of BHLM18 3m, 4m BHLM19 1m & 2m
DC : BHLM06 11m

6.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.2	4.7	5.9	7.2	8.7	10.2	11.7	13.2
Consolidation Settlement(cm) w/o preload	2.48	4.41	6.61	8.90	8.71	6.74	3.94	2.19	1.30	1.30
After105days	1.25	2.40	3.70	4.90	4.78	3.62	2.01	1.03	0.57	0.34
Consolidation Settlement(cm)	1.23	2.02	2.91	4.00	3.93	3.12	1.93	1.16	0.74	0.96





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

- E' : Modulus of Deformation of the Ground after Pre-Load（kN/m^2）
- E_0 : Original Modulus of Deformation of the Ground（kN/m^2）(= 700 x N)
- C : Cohesion after Pre-Load（kN/m^2）
- C_0 : Original Cohesion（kN/m^2）

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

- m : Strength Increase Ratio（Cohesive soil：m＝0.25）
- ΔP : Load Increase（kN/m^2）
- U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BMLW-10)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
Ac1	13	0	1.40	0.25	22.3	0.5	15	1.22	1.71
As	0	26	3.50	-	-	-	-	-	3.50
Ac1	72	0	3.50	0.25	10.1	0.3	72	1.01	3.54

6.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BHLM-10)

Loading Width	B (m)	19.500
Loading Length	L (m)	2.800
Boring		BHLM-10

No.	Classification	E_i (MN/m^2)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	Ac1	1.71	2.55	2.55	-0.33838	
2	As	3.50	1.00	3.55	-0.03799	
3	Ac2	3.54	4.85	8.40	-0.10781	
4						
5						
6						
7						
8						
9						
10						
11						
	Total		8.40		-0.48418	-1.09206
E_m (MN/m^2)						2.3
(kN/m^2)						2255
(tf/m^2)						226

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment (E.L. m)	Sign	Load (kN/m3)		(m)		(kN/m2)	Loading Width (m)	1/2 Loading Width (m)	Location of the Center of the Load from the Control Point (m)				
14.80	q1	18.0	×	1.1	=	19.8	13.20	6.60	6.60			=	6.60
	q2	18.0	×	1.0	=	18.0	6.90	3.45	0.00	+	3.45	=	3.45
	q3	18.0	×	1.0	=	18.0	6.90	3.45	0.00	+	3.45	=	3.45
	q4	18.0	×	1.0	=	18.0	4.40	2.20	2.50	+	2.20	=	4.70

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	19.8	6.60	6.60
q2	Load 2	18.0	3.45	3.45
q3	Load 3	18.0	3.45	3.45
q4	Load 4	18.0	2.20	4.70

● Ground Conditon BHLM-10

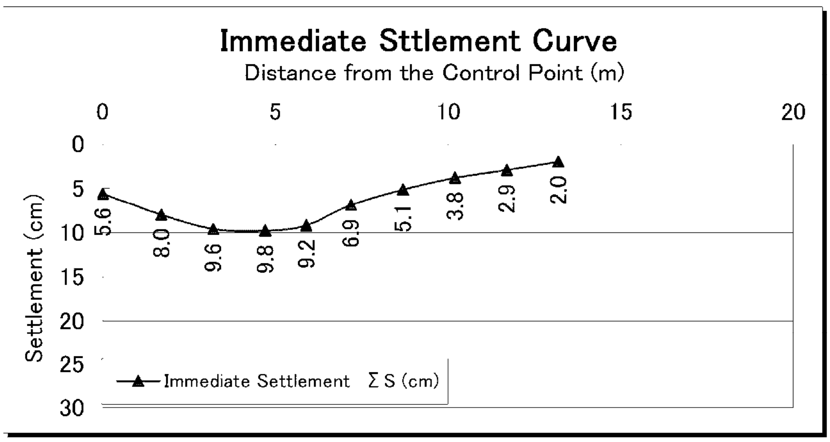
Thickness of Layer 8.4 m

Average Modulus of Elasticity 2255 kN/m2

$$Six = \sum_{i=\ell}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

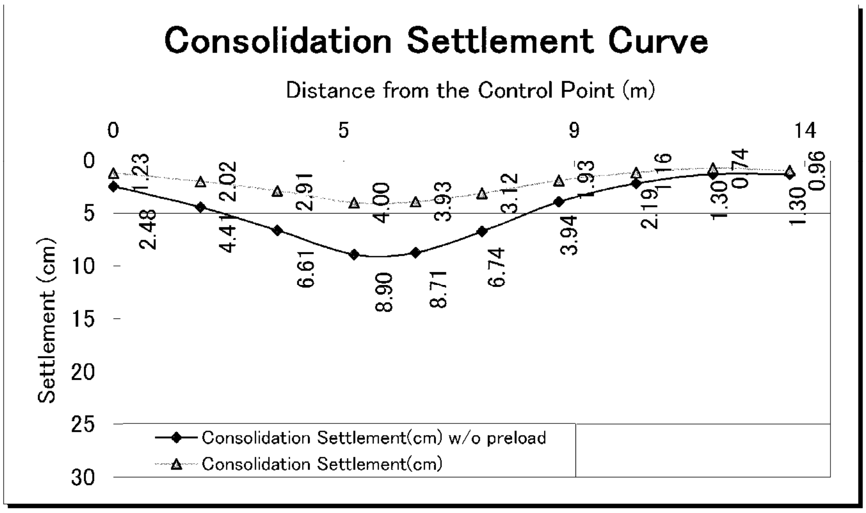
$$= \sum_{i=\ell}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - 0.2387\left[\alpha \log|\alpha| + \beta \log|\beta|\right]\right\}$$

			Location of	Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	X	(m)	Caluculation	0.0	1.7	3.2	4.7	5.9	7.2	8.7	10.2	11.7	13.2
q1	X1	(m)	Xo1-X	6.6	4.9	3.4	1.9	0.7	0.6	2.1	3.6	5.1	6.6
	S1	(cm)		1.78	2.27	2.49	2.61	2.66	2.66	2.60	2.46	2.23	1.78
q2	X2	(m)	Xo2-X	3.5	1.8	0.3	1.3	2.5	3.8	5.3	6.8	8.3	9.8
	S2	(cm)		1.70	2.38	2.54	2.46	2.21	1.48	0.92	0.57	0.31	0.09
q3	X3	(m)	Xo3-X	3.5	1.8	0.3	1.3	2.5	3.8	5.3	6.8	8.3	9.8
	S3	(cm)		1.70	2.38	2.54	2.46	2.21	1.48	0.92	0.57	0.31	0.09
q4	X4	(m)	Xo4-X	4.7	3.0	1.5	0.0	1.2	2.5	4.0	5.5	7.0	8.5
	S4	(cm)		0.41	0.98	2.02	2.30	2.13	1.26	0.61	0.23	0.00	0.00
Immediate Settlement		ΣS (cm)		**5.6**	**8.0**	**9.6**	**9.8**	**9.2**	**6.9**	**5.1**	**3.8**	**2.9**	**2.0**



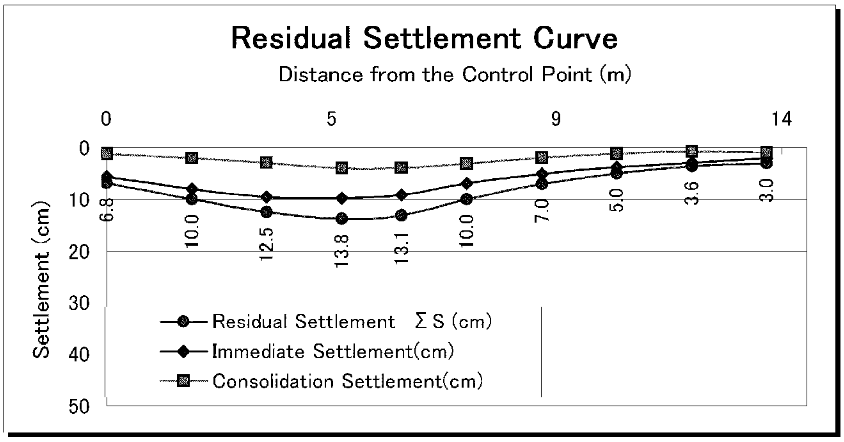
6.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.2	4.7	5.9	7.2	8.7	10.2	11.7	13.2
Consolidation Settlement(cm) w/o preload	2.48	4.41	6.61	8.90	8.71	6.74	3.94	2.19	1.30	1.30
After105days	1.25	2.40	3.70	4.90	4.78	3.62	2.01	1.03	0.57	0.34
Consolidation Settlement(cm)	1.23	2.02	2.91	4.00	3.93	3.12	1.93	1.16	0.74	0.96



6.6 Residual Settlement

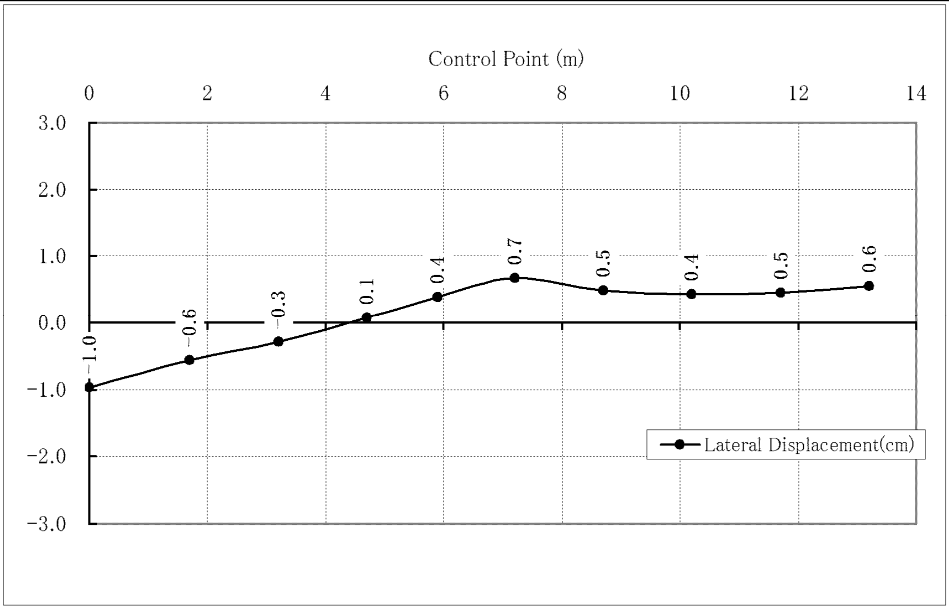
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.7	3.2	4.7	5.9	7.2	8.7	10.2	11.7	13.2
Immediate Settlement(cm)	5.60	8.00	9.60	9.80	9.20	6.90	5.10	3.80	2.90	2.00
Consolidation Settlement(cm)	1.23	2.02	2.91	4.00	3.93	3.12	1.93	1.16	0.74	0.96
Residual Settlement ΣS (cm)	**6.8**	**10.0**	**12.5**	**13.8**	**13.1**	**10.0**	**7.0**	**5.0**	**3.6**	**3.0**



6.7 Lateral Displacement

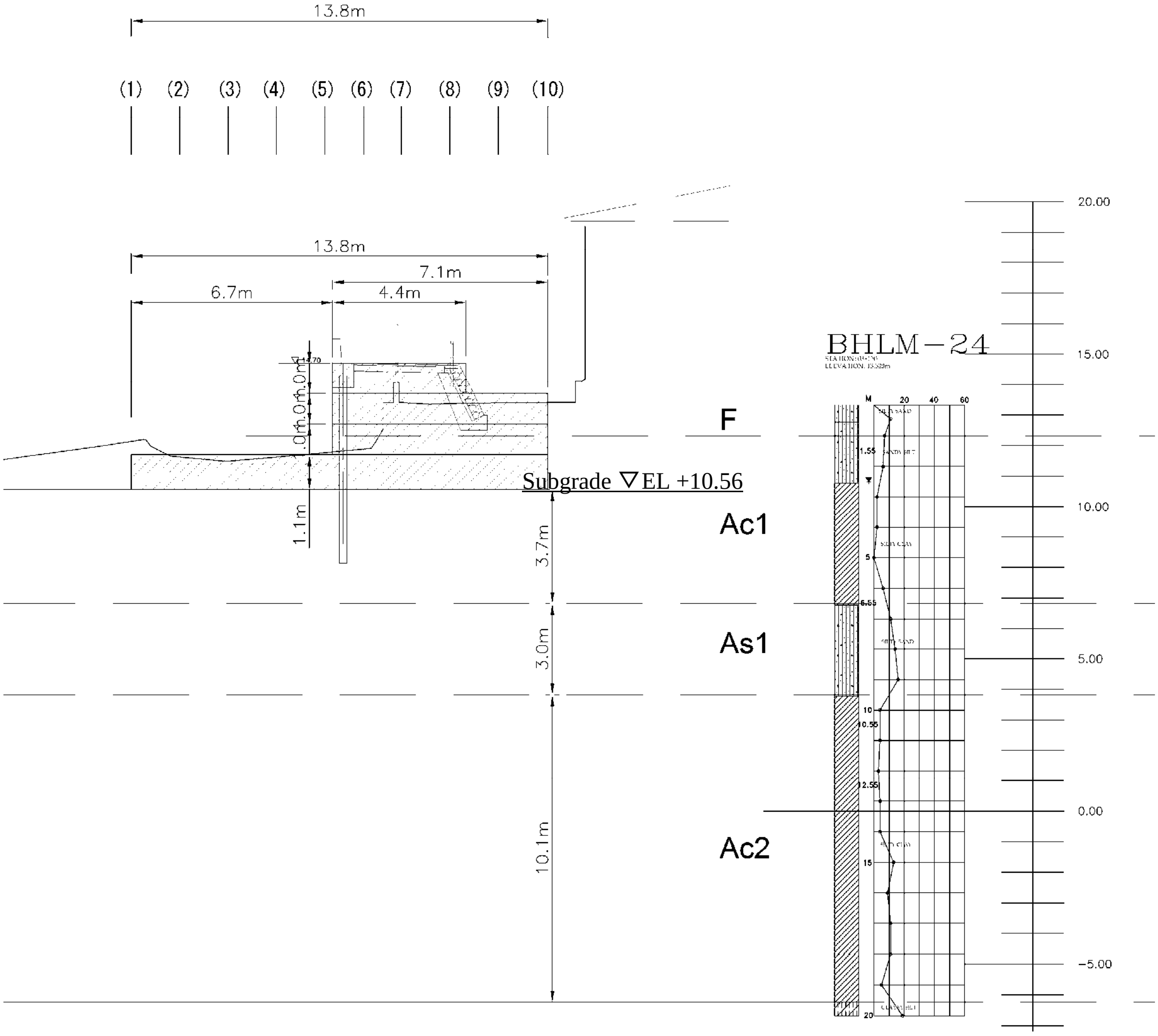
Lateral Displacement (BHLM-10)

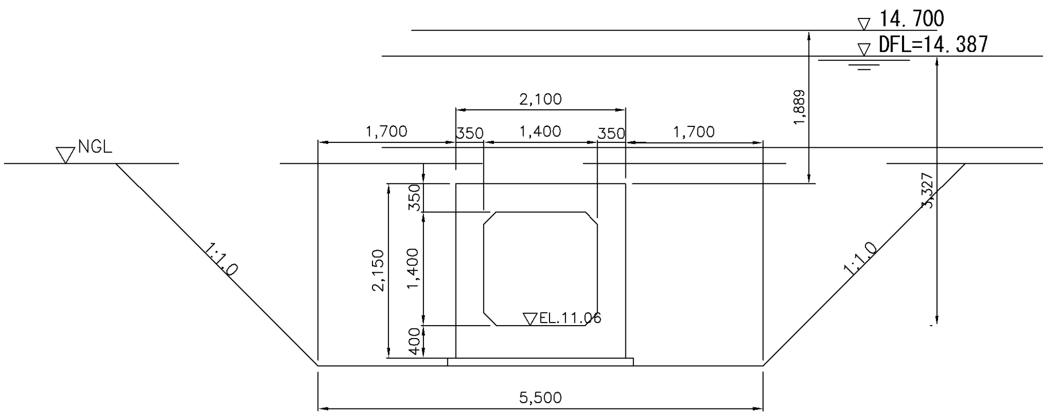
			q1		q2		q3		q4		Total
q_i (kN/m^2)			19.800		18.000		18.000		18.000		
a_i (m)			6.600		3.450		3.450		2.200		
b_i (m)			1.400		1.400		1.400		1.400		
ν			0.400		0.400		0.400		0.400		
x_i (m)			6.600		3.450		3.450		4.700		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	2255	6.6	−0.36	3.4	−0.26	3.4	−0.26	4.7	−0.10	−1.0
(2)	1.7	2255	4.9	−0.20	1.8	−0.10	1.8	−0.10	3.0	−0.16	−0.6
(3)	3.2	2255	3.4	−0.12	0.3	−0.01	0.3	−0.01	1.5	−0.13	−0.3
(4)	4.7	2255	1.9	−0.06	−1.3	0.07	−1.3	0.07	0.0	0.00	0.1
(5)	5.9	2255	0.7	−0.02	−2.5	0.16	−2.5	0.16	−1.2	0.10	0.4
(6)	7.2	2255	−0.6	0.02	−3.8	0.23	−3.8	0.23	−2.5	0.19	0.7
(7)	8.7	2255	−2.1	0.07	−5.3	0.15	−5.3	0.15	−4.0	0.12	0.5
(8)	10.2	2255	−3.6	0.13	−6.8	0.11	−6.8	0.11	−5.5	0.08	0.4
(9)	11.7	2255	−5.1	0.21	−8.3	0.09	−8.3	0.09	−7.0	0.06	0.5
(10)	13.2	2255	−6.6	0.36	−9.7	0.07	−9.7	0.07	−8.5	0.05	0.6



7 Calculation of MSR-2

7.1 Calculation Model





7.2 Results of Consolidation test

Sluiceway No	MSR-2
Geological Data	
Borehole	BHLM-24
station	R 3+170
elevation	13.32
northing	
easting	

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	1.0	F	17.0
	2.5	AC1	17.3
	6.5	AC1	17.3
	9.5	AS1	18.0
	19.5	DC	18.0
	20.0	DC	18.0

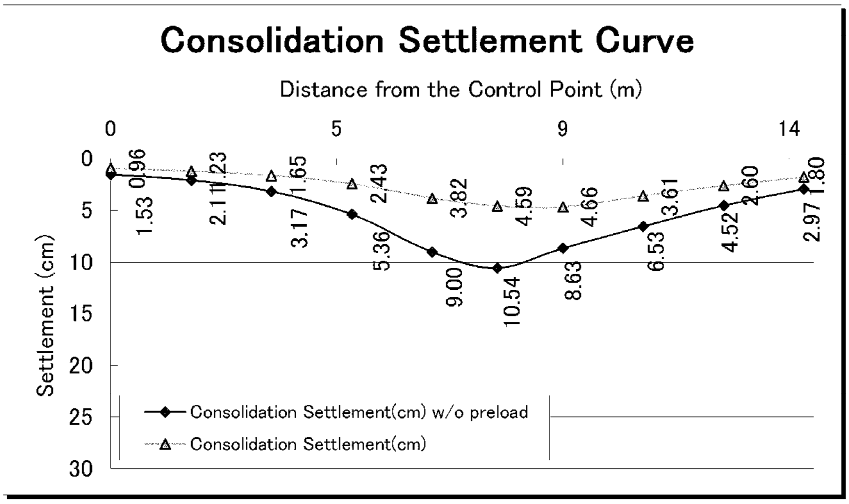
Consolidation Data		AC1	DC
PC kgf/cm^2		0.55	1.245
	p kgf/cm2		
e-logp	0.1	1.3171	1.0932
	0.2	1.2814	1.0584
	0.4	1.2272	1.0226
	0.8	1.1378	0.9826
	1.6	1.0319	0.9299
	3.2	0.9017	0.8488
	6.4	0.7770	0.7508
	12.8	0.6490	0.6381
Cv 10^{-3} cm^2/sec	0.05	4.838	21.94
	0.15	4.073	16.938
	0.3	3.628	10.956
	0.6	2.322	15.764
	1.2	2.047	12.185
	2.4	1.500	11.365
	4.8	1.643	9.376
	9.6	1.498	2.987

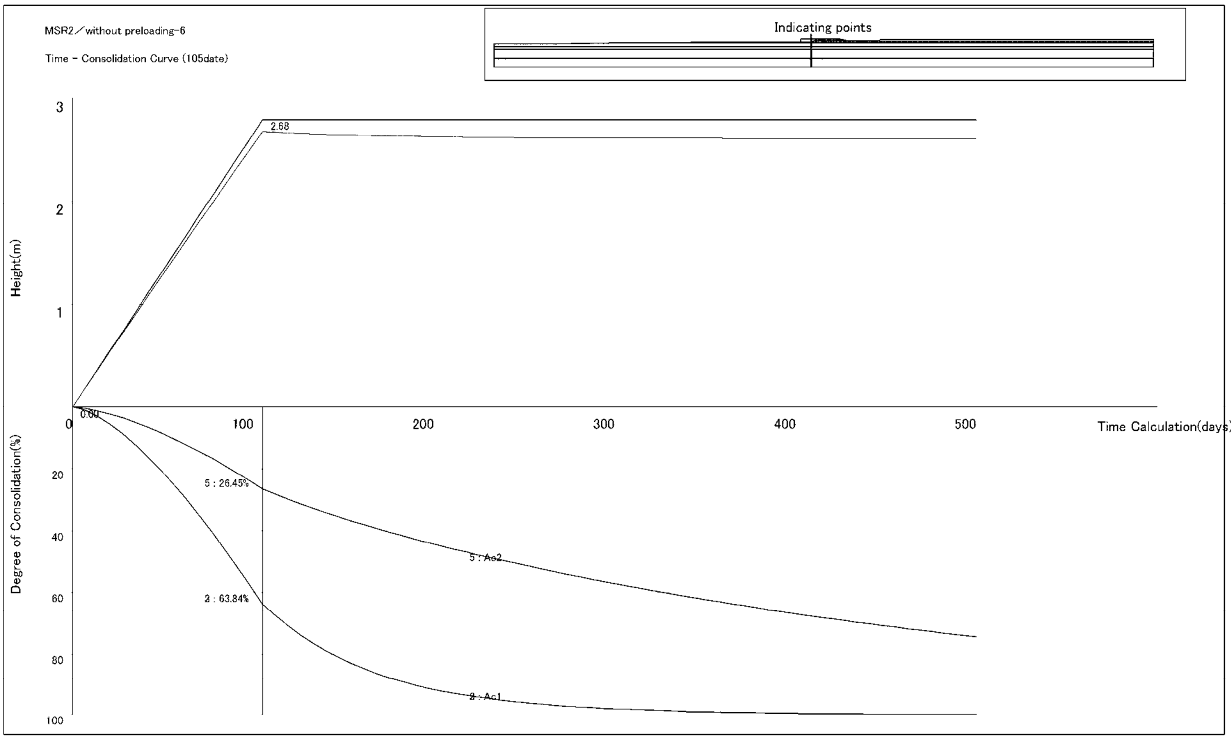
AC1 : Average of BHLM25 3m & 4m
DC : BHLM24 14m

7.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

Control Point Location of Calculation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Consolidation Settlement(cm) w/o preload	1.53	2.11	3.17	5.36	9.00	10.54	8.63	6.53	4.52	2.97
after 105days	0.58	0.88	1.51	2.92	5.18	5.95	3.97	2.93	1.93	1.17
Consolidation Settlement(cm)	0.96	1.23	1.65	2.43	3.82	4.59	4.66	3.61	2.60	1.80





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

E'	:	Modulus of Deformation of the Ground after Pre-Load（kN/m^2）
E_0	:	Original Modulus of Deformation of the Ground（kN/m^2）(= 700 x N)
C	:	Cohesion after Pre-Load（kN/m^2）
C_0	:	Original Cohesion（kN/m^2）

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

m	:	Strength Increase Ratio（Cohesive soil：m＝0.25）
ΔP	:	Load Increase（kN/m^2）
U	:	Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BHLM-24)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
F	0	24	-	-	-	-	-	-	-
Ac1	24	0	2.80	0.25	19.2	0.6	27	1.12	3.14
As	0	26	9.10	-	-	-	-	-	9.10
Ac2	36	0	4.20	0.25	6.0	0.2	36	1.01	4.24

7.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BMLM-24)

Loading Width	B (m)	13.800
Loading Length	L (m)	5.600
Boring		BMLM-24

No.	Classification	E_i (MN/m^2)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	Ac1	2.80	3.75	3.75	-0.10706	
2	As	9.10	3.00	6.75	-0.01367	
3	Ac2	4.20	10.05	16.80	-0.04618	
4						
5						
6						
7						
8						
9						
10						
11						
	Total		16.80		-0.16691	-0.61810
E_m (MN/m^2)						3.7
(kN/m^2)						3703
(tf/m^2)						370

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment	Sign	Load					Loading Width	1/2 Loading Width	Location of the Center of the Load from the Control Point				
(E.L. m)		(kN/m3)		(m)		(kN/m2)	(m)	(m)	(m)				
14.70	q1	18.0	×	1.1	=	19.8	13.80	6.90	6.90			=	6.90
	q2	18.0	×	1.0	=	18.0	7.10	3.55	6.70	+	3.55	=	10.25
	q3	18.0	×	1.0	=	18.0	7.10	3.55	6.70	+	3.55	=	10.25
	q4	18.0	×	1.0	=	18.0	4.40	2.20	6.70	+	2.20	=	8.90

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	19.8	6.90	6.90
q2	Load 2	18.0	3.55	10.25
q3	Load 3	18.0	3.55	10.25
q4	Load 4	18.0	2.20	8.90

● Ground Conditon BMLM-24

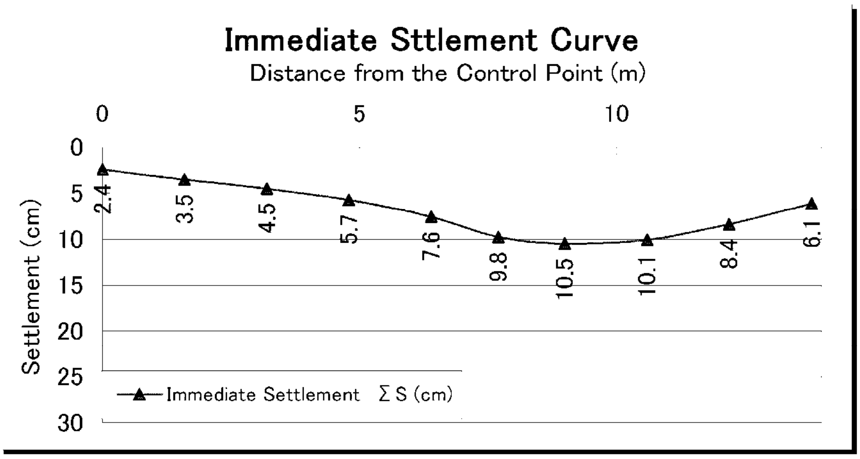
Thickness of Layer 16.8 m

Average Modulus of Elasticity 3703 kN/m2

$$Six = \sum_{i-t}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

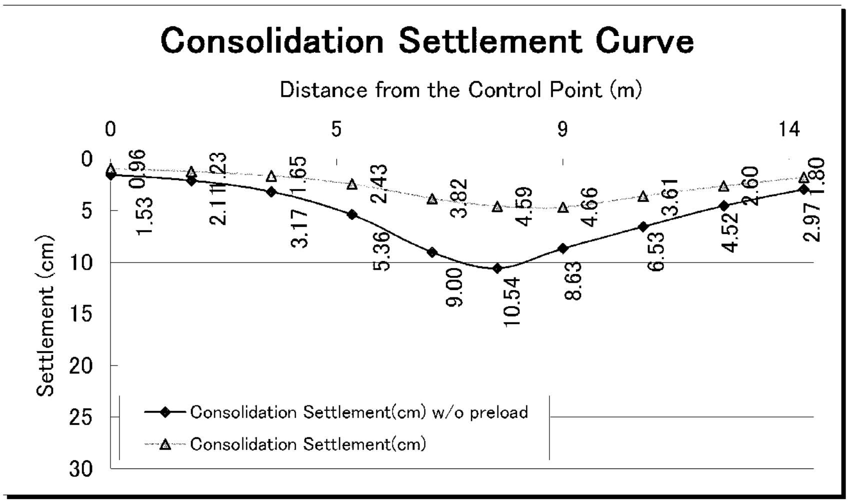
$$= \sum_{i-t}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \{1.0 - 0.2387\,[\alpha \log \alpha + \beta \log \beta]\}$$

	X	(m)	Location of Caluculation	Control Point (1) 0.0	(2) 1.6	(3) 3.2	(4) 4.8	(5) 6.4	(6) 7.7	(7) 9.0	(8) 10.6	(9) 12.2	(10) 13.8
q1	X1	(m)	Xo1-X	6.9	5.3	3.7	2.1	0.5	0.8	2.1	3.7	5.3	6.9
	S1	(cm)		2.28	2.87	3.16	3.33	3.41	3.40	3.33	3.16	2.87	2.28
q2	X2	(m)	Xo2-X	10.3	8.7	7.1	5.5	3.9	2.6	1.3	0.4	2.0	3.6
	S2	(cm)		0.07	0.29	0.56	0.93	1.52	2.24	2.52	2.59	2.40	1.74
q3	X3	(m)	Xo3-X	10.3	8.7	7.1	5.5	3.9	2.6	1.3	0.4	2.0	3.6
	S3	(cm)		0.07	0.29	0.56	0.93	1.52	2.24	2.52	2.59	2.40	1.74
q4	X4	(m)	Xo4-X	8.9	7.3	5.7	4.1	2.5	1.2	0.1	1.7	3.3	4.9
	S4	(cm)		0.00	0.00	0.17	0.52	1.14	1.93	2.08	1.75	0.77	0.33
Immediate Settlement			**ΣS (cm)**	**2.4**	**3.5**	**4.5**	**5.7**	**7.6**	**9.8**	**10.5**	**10.1**	**8.4**	**6.1**



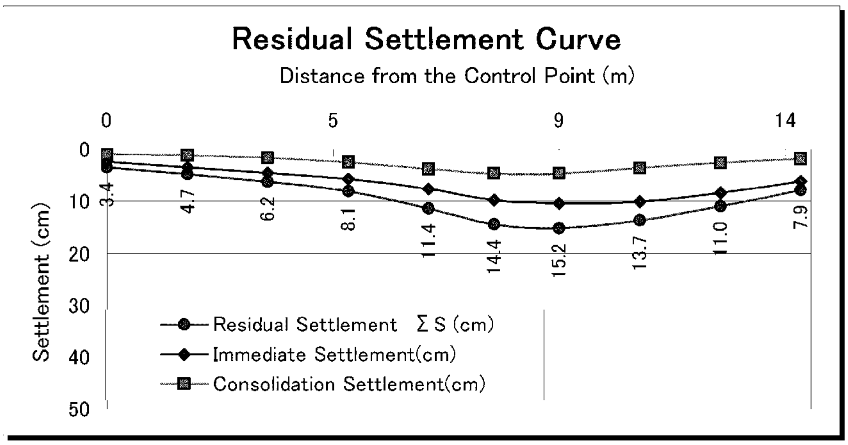
7.5 Consolidation Settlement

Control Point Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Consolidation Settlement(cm) w/o preload	1.53	2.11	3.17	5.36	9.00	10.54	8.63	6.53	4.52	2.97
after 105days	0.58	0.88	1.51	2.92	5.18	5.95	3.97	2.93	1.93	1.17
Consolidation Settlement(cm)	0.96	1.23	1.65	2.43	3.82	4.59	4.66	3.61	2.60	1.80



7.6 Residual Settlement

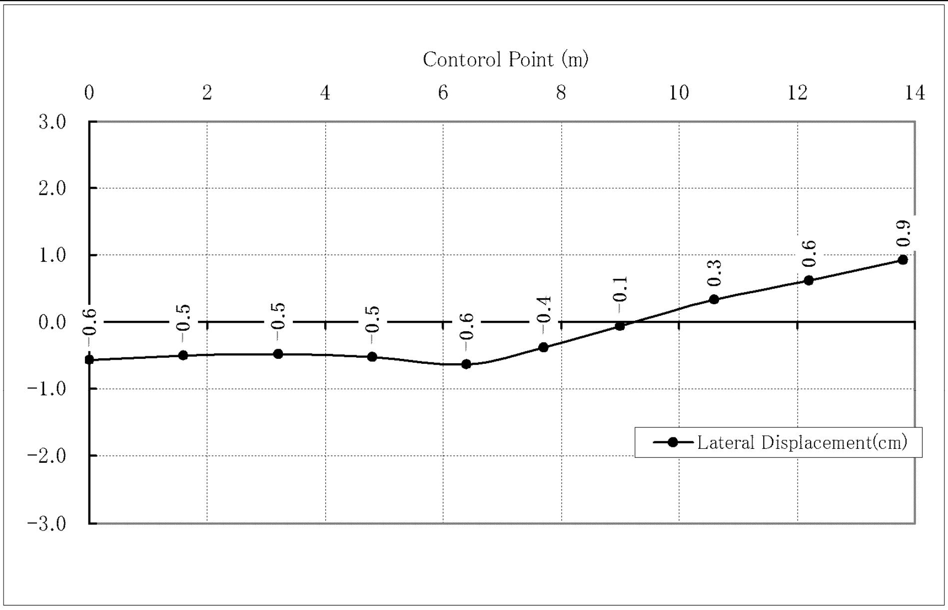
Control Point Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Immediate Settlement(cm)	2.40	3.50	4.50	5.70	7.60	9.80	10.50	10.10	8.40	6.10
Consolidation Settlement(cm)	0.96	1.23	1.65	2.43	3.82	4.59	4.66	3.61	2.60	1.80
Residual Settlement ΣS (cm)	**3.4**	**4.7**	**6.2**	**8.1**	**11.4**	**14.4**	**15.2**	**13.7**	**11.0**	**7.9**



7.7 Lateral Displacement

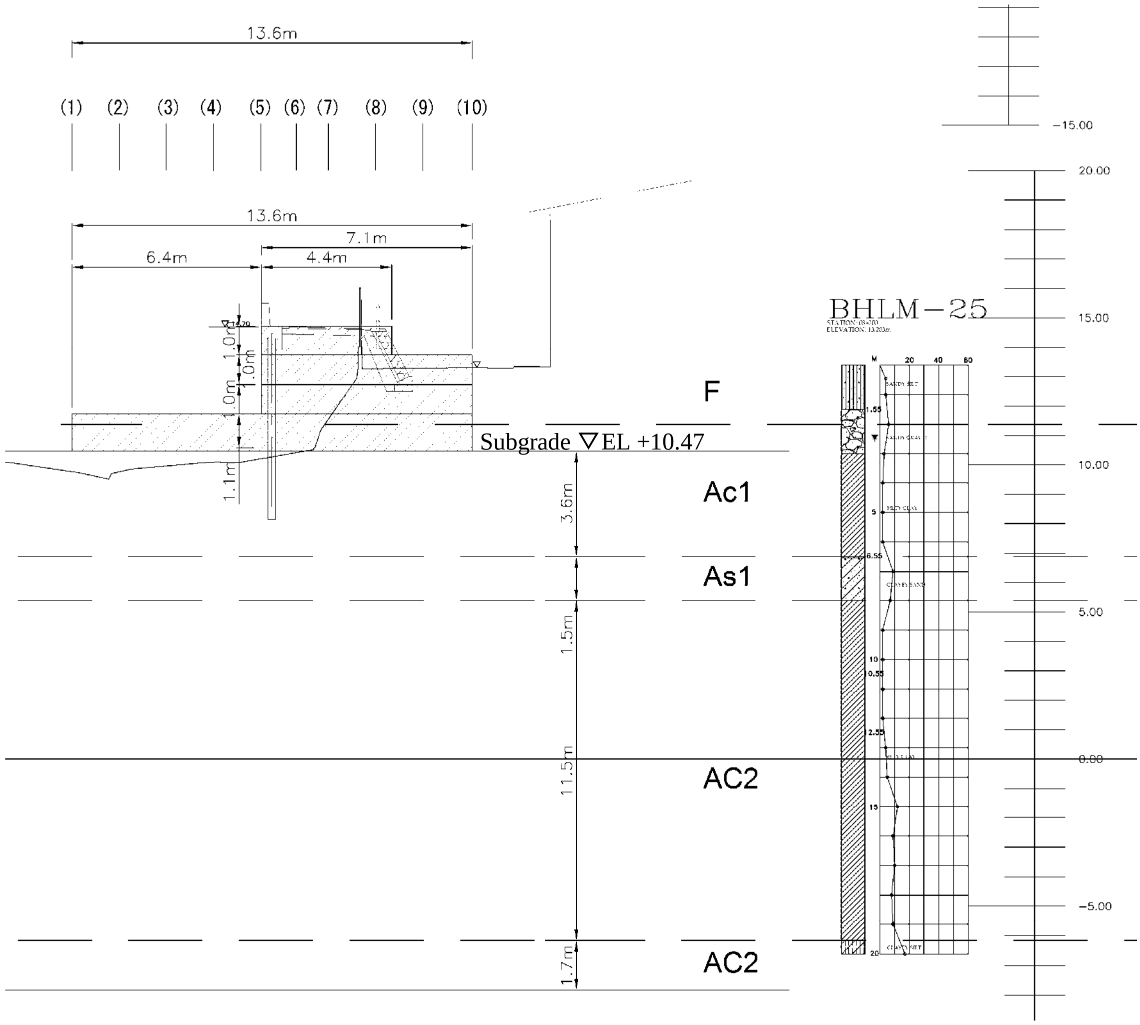
Lateral Displacement (BMLM-24)

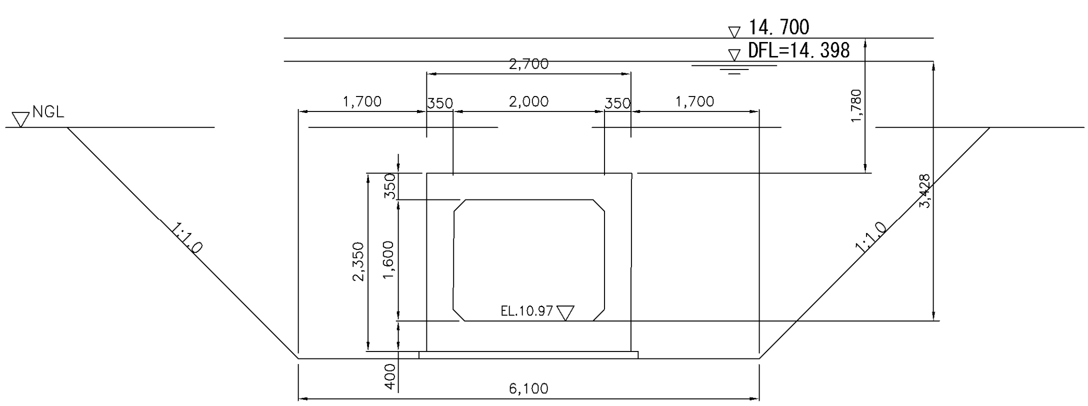
			q1		q2		q3		q4		Total
q_i (kN/m^2)			19.800		18.000		18.000		18.000		
a_i (m)			6.900		3.550		3.550		2.200		
b_i (m)			2.800		2.800		2.800		2.800		
ν			0.400		0.400		0.400		0.400		
x_i (m)			6.900		10.250		10.250		8.900		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3703	6.9	−0.35	10.2	−0.09	10.2	−0.09	8.9	−0.06	−0.6
(2)	1.6	3703	5.3	−0.23	8.7	−0.10	8.7	−0.10	7.3	−0.07	−0.5
(3)	3.2	3703	3.7	−0.15	7.1	−0.12	7.1	−0.12	5.7	−0.09	−0.5
(4)	4.8	3703	2.1	−0.08	5.5	−0.16	5.5	−0.16	4.1	−0.12	−0.5
(5)	6.4	3703	0.5	−0.02	3.9	−0.22	3.9	−0.22	2.5	−0.17	−0.6
(6)	7.7	3703	−0.8	0.03	2.6	−0.16	2.6	−0.16	1.2	−0.10	−0.4
(7)	9.0	3703	−2.1	0.08	1.3	−0.07	1.3	−0.07	−0.1	0.01	−0.1
(8)	10.6	3703	−3.7	0.15	−0.4	0.02	−0.4	0.02	−1.7	0.14	0.3
(9)	12.2	3703	−5.3	0.23	−2.0	0.12	−2.0	0.12	−3.3	0.14	0.6
(10)	13.8	3703	−6.9	0.35	−3.5	0.24	−3.5	0.24	−4.9	0.10	0.9



8 Calculation of MSR-3

8.1 Calculation Model





8.2 Results of Consolidation test

Sluiceway No **MSR-3**

Geological Data	
Borehole	BHLM-25
station	R 3+240
elevation	13.283
northing	
easting	

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	2.0	F	17.0
	6.5	AC1	17.3
	8.0	AS1	18.0
	19.5	DC	18.0
	20.0	DC	18.0

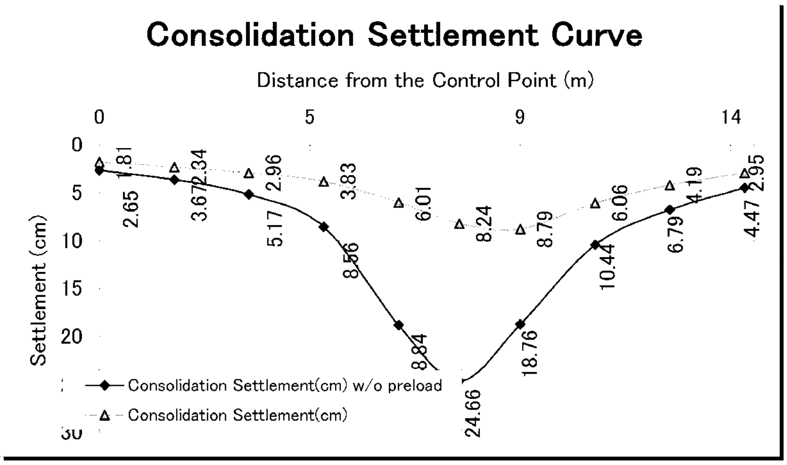
Consolidation Data		AC1	DC
PC kgf/cm^2		0.55	1.245
	p kgf/cm2		
e-logp	0.1	1.3171	1.0932
	0.2	1.2814	1.0584
	0.4	1.2272	1.0226
	0.8	1.1378	0.9826
	1.6	1.0319	0.9299
	3.2	0.9017	0.8488
	6.4	0.7770	0.7508
	12.8	0.6490	0.6381
Cv 10^{-3} cm^2/sec	0.05	4.838	21.94
	0.15	4.073	16.938
	0.3	3.628	10.956
	0.6	2.322	15.764
	1.2	2.047	12.185
	2.4	1.500	11.365
	4.8	1.643	9.376
	9.6	1.498	2.987

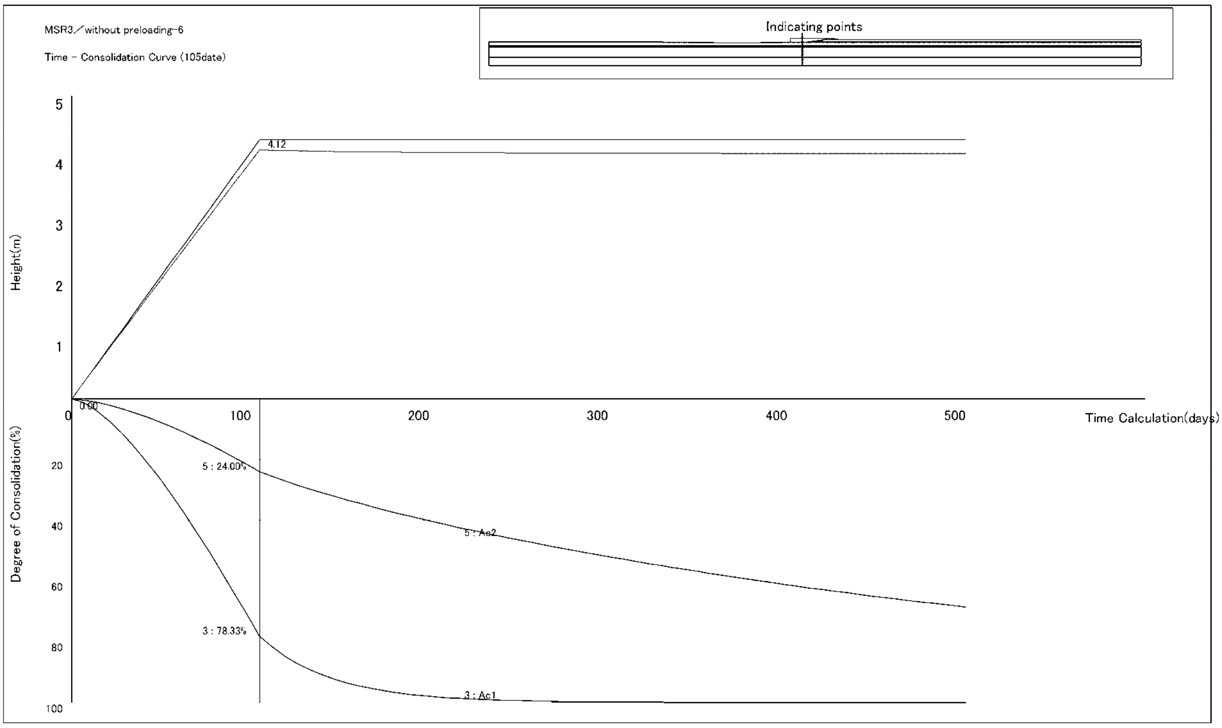
AC1 : Average of BHLM25 3m & 4m
DC : BHLM24 14m

8.3 Considering Pre-Load Effect

In this site, pre-load effect darling 105 days embankment works is considered. Consolidation settlement without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Consolidation Settlement(cm) w/o preload	2.65	3.67	5.17	8.56	18.84	24.66	18.76	10.44	6.79	4.47
after 105days	0.84	1.33	2.21	4.73	12.83	16.42	9.98	4.38	2.60	1.52
Consolidation Settlement(cm)	1.81	2.34	2.96	3.83	6.01	8.24	8.79	6.06	4.19	2.95





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

- E' : Modulus of Deformation of the Ground after Pre-Load (kN/m^2)
- E_0 : Original Modulus of Deformation of the Ground (kN/m^2) (= 700 x N)
- C : Cohesion after Pre-Load (kN/m^2)
- C_0 : Original Cohesion (kN/m^2)

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

- m : Strength Increase Ratio (Cohesive soil : m＝0.25)
- ΔP : Load Increase (kN/m^2)
- U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BHLM-25)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
F	-	-	-	-	-	-	-	-	-
Ac1	12	0	2.80	0.25	46.9	0.7	20	1.68	4.72
As	-	-	1.40	-	-	-	-	-	1.40
Ac2	30	0	5.60	0.25	11.6	0.2	31	1.02	5.71
Ac2	102	0	11.90	0.25			102	1.00	11.90

8.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BHLM-25)

Loading Width	B (m)	13.600
Loading Length	L (m)	6.100
Boring		BHLM-25

No.	Classification	E_i (MN/m^2)	Thickness of Laver (m)	h_i (m)	Denominator	Numerator
1	Ac1	4.72	3.59	3.59	-0.05352	
2	As	1.40	1.50	5.09	-0.04504	
3	Ac2	5.71	11.55	16.64	-0.03973	
4	Ac2	11.90	1.66	18.30	-0.00136	
5						
6						
7						
8						
9						
10						
11						
	Total		18.30		-0.13965	-0.55840
E_m (MN/m^2)						4.0
(kN/m^2)						3999
(tf/m^2)						400

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment (E.L. m)	Sign	Load (kN/m3)		(m)		(kN/m2)	Loading Width (m)	1/2 Loading Width (m)	Location of the Center of the Load from the Control Point (m)				
14.70	q1	18.0	×	1.1	=	19.8	13.80	6.90	6.90			=	6.90
	q2	18.0	×	1.0	=	18.0	7.10	3.55	6.70	+	3.55	=	10.25
	q3	18.0	×	1.0	=	18.0	7.10	3.55	6.70	+	3.55	=	10.25
	q4	18.0	×	1.0	=	18.0	4.40	2.20	6.70	+	2.20	=	8.90

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load qi(kN/m2)	1/2 Loading Width ai (m)	Distance Xoi (m)
q1	Load 1	19.8	6.90	6.90
q2	Load 2	18.0	3.55	10.25
q3	Load 3	18.0	3.55	10.25
q4	Load 4	18.0	2.20	8.90

● Ground Conditon BHLM-25

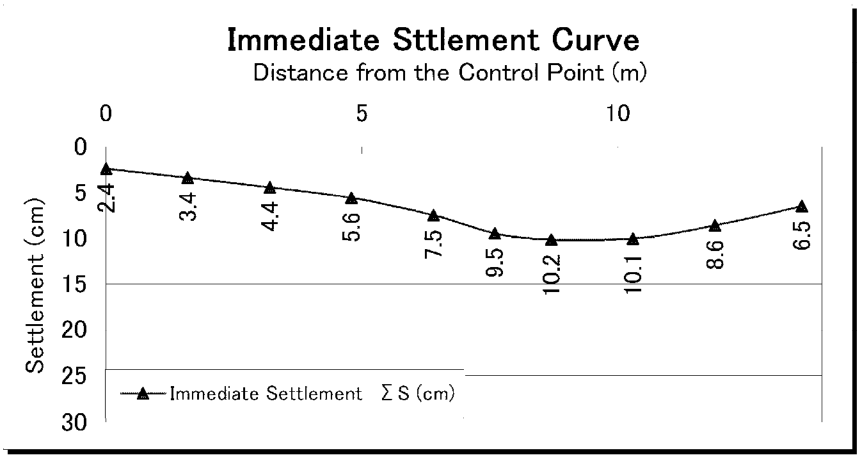
Thickness of Layer 18.3 m

Average Modulus of Elasticity 3999 kN/m2

$$Six = \sum_{i=1}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

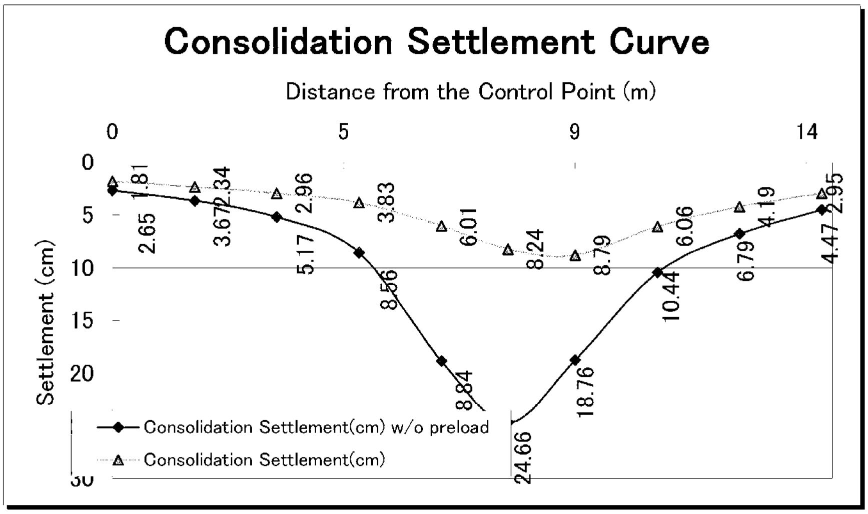
$$= \sum_{i=1}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - 0.2387\left[\alpha \log \alpha + \beta \log|\beta|\right]\right\}$$

	X	(m)	Location of Caluculation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
			Control Point	0.0	1.6	3.2	4.8	6.4	7.6	8.7	10.3	11.9	13.6
q1	X1	(m)	Xo1-X	6.9	5.3	3.7	2.1	0.5	0.7	1.8	3.4	5.0	6.7
	S1	(cm)		2.27	2.86	3.15	3.32	3.40	3.39	3.34	3.19	2.92	2.40
q2	X2	(m)	Xo2-X	10.3	8.7	7.1	5.5	3.9	2.7	1.6	0.1	1.7	3.4
	S2	(cm)		0.07	0.28	0.55	0.90	1.48	2.15	2.41	2.53	2.40	1.85
q3	X3	(m)	Xo3-X	10.3	8.7	7.1	5.5	3.9	2.7	1.6	0.1	1.7	3.4
	S3	(cm)		0.07	0.28	0.55	0.90	1.48	2.15	2.41	2.53	2.40	1.85
q4	X4	(m)	Xo4-X	8.9	7.3	5.7	4.1	2.5	1.3	0.2	1.4	3.0	4.7
	S4	(cm)		0.00	0.00	0.16	0.50	1.10	1.83	2.01	1.80	0.86	0.36
Immediate Settlement			**ΣS (cm)**	**2.4**	**3.4**	**4.4**	**5.6**	**7.5**	**9.5**	**10.2**	**10.1**	**8.6**	**6.5**



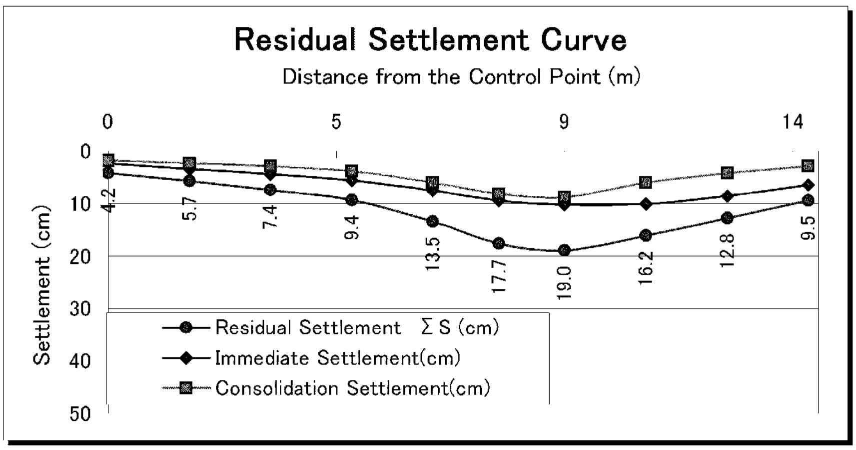
8.5 Consolidation Settlement

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Consolidation Settlement(cm) w/o preload	2.65	3.67	5.17	8.56	18.84	24.66	18.76	10.44	6.79	4.47
after 105days	0.84	1.33	2.21	4.73	12.83	16.42	9.98	4.38	2.60	1.52
Consolidation Settlement(cm)	1.81	2.34	2.96	3.83	6.01	8.24	8.79	6.06	4.19	2.95



8.6 Residual Settlement

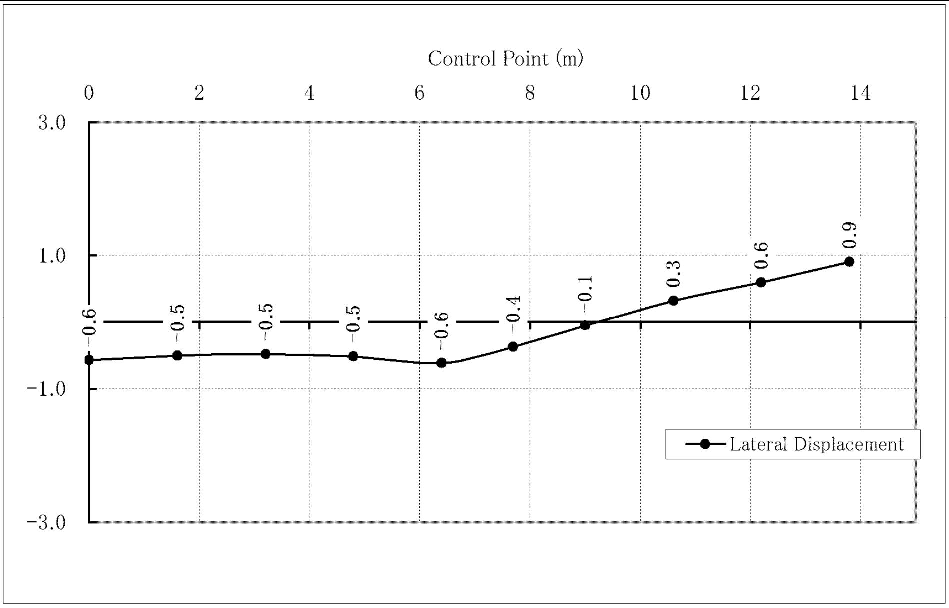
	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.4	7.7	9.0	10.6	12.2	13.8
Immediate Settlement(cm)	2.40	3.40	4.40	5.60	7.50	9.50	10.20	10.10	8.60	6.50
Consolidation Settlement(cm)	1.81	2.34	2.96	3.83	6.01	8.24	8.79	6.06	4.19	2.95
Residual Settlement ΣS (cm)	**4.2**	**5.7**	**7.4**	**9.4**	**13.5**	**17.7**	**19.0**	**16.2**	**12.8**	**9.5**



8.7 Lateral Displacement

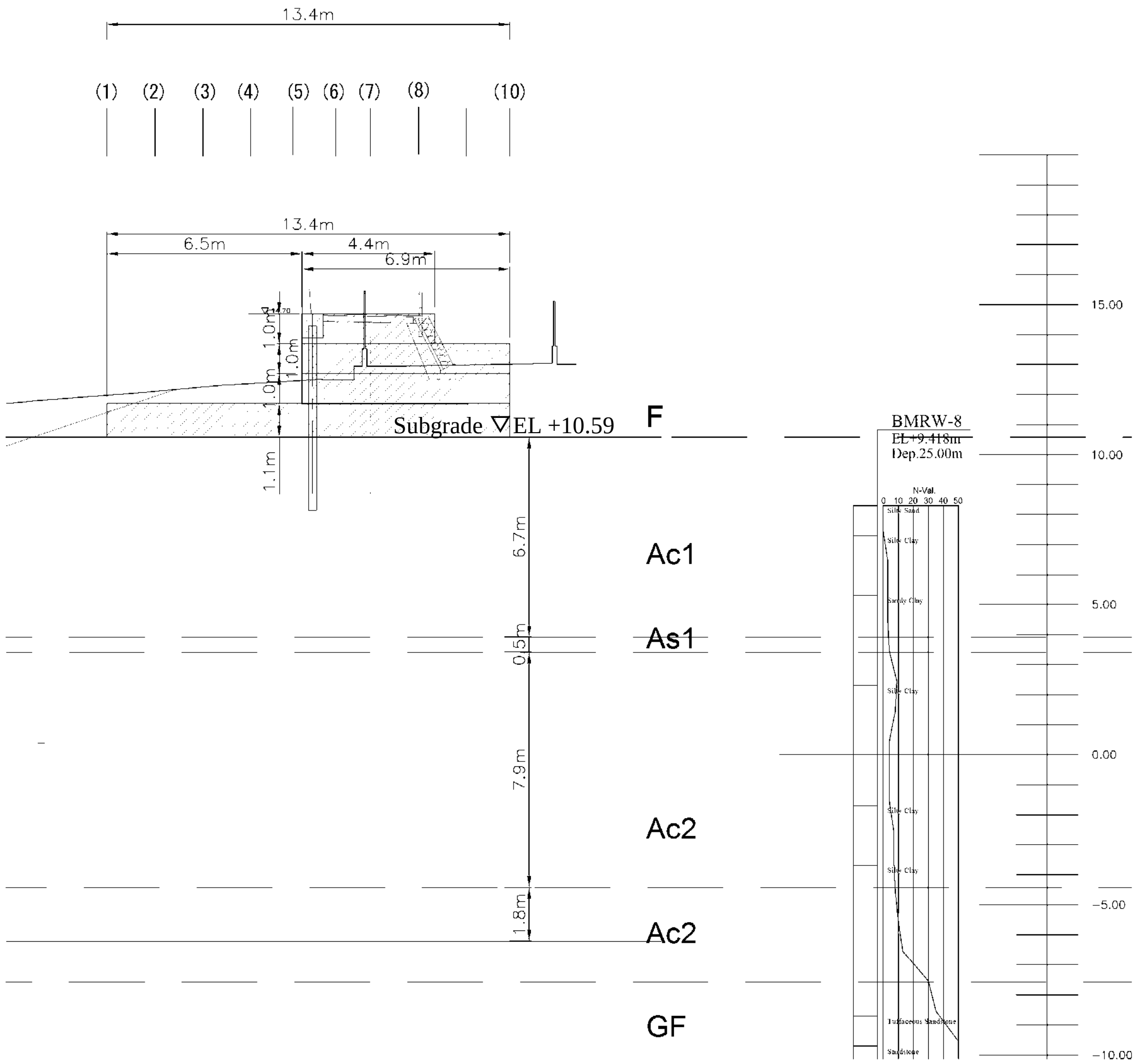
Lateral Displacement (BHLM-25)

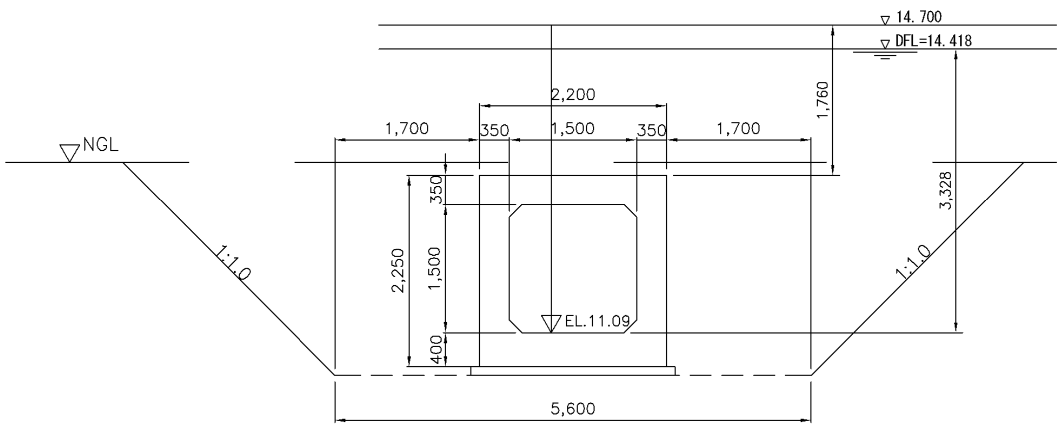
			q1		q2		q3		q4		Total
q_i (kN/m^2)				19.800		18.000		18.000		18.000	
a_i (m)				6.900		3.550		3.550		2.200	
b_i (m)				3.050		3.050		3.050		3.050	
ν				0.400		0.400		0.400		0.400	
x_i (m)				6.900		10.250		10.250		8.900	
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3999	6.9	−0.34	10.2	−0.09	10.2	−0.09	8.9	−0.06	−0.6
(2)	1.6	3999	5.3	−0.23	8.7	−0.10	8.7	−0.10	7.3	−0.07	−0.5
(3)	3.2	3999	3.7	−0.15	7.1	−0.12	7.1	−0.12	5.7	−0.09	−0.5
(4)	4.8	3999	2.1	−0.08	5.5	−0.16	5.5	−0.16	4.1	−0.12	−0.5
(5)	6.4	3999	0.5	−0.02	3.9	−0.22	3.9	−0.22	2.5	−0.16	−0.6
(6)	7.7	3999	−0.8	0.03	2.6	−0.16	2.6	−0.16	1.2	−0.09	−0.4
(7)	9.0	3999	−2.1	0.08	1.3	−0.07	1.3	−0.07	−0.1	0.01	−0.1
(8)	10.6	3999	−3.7	0.15	−0.4	0.02	−0.4	0.02	−1.7	0.13	0.3
(9)	12.2	3999	−5.3	0.23	−2.0	0.12	−2.0	0.12	−3.3	0.14	0.6
(10)	13.8	3999	−6.9	0.34	−3.5	0.23	−3.5	0.23	−4.9	0.10	0.9



9 Calculation of MSR-4

9.1 Calculation Model





9.2 Results of Consolidation test

Sluiceway No **MSR-4**

Geological Data	
Borehole	BMRW-8
station	R 3+450
elevation	9.418
northing	
easting	

	BOUNDARY	SOIL TYPE	UNIT WEIGHT
formation	5.5	AC1	17.3
	6.0	AS1	18.0
	17.0	DC	18.0
	20.0	GF	15.5

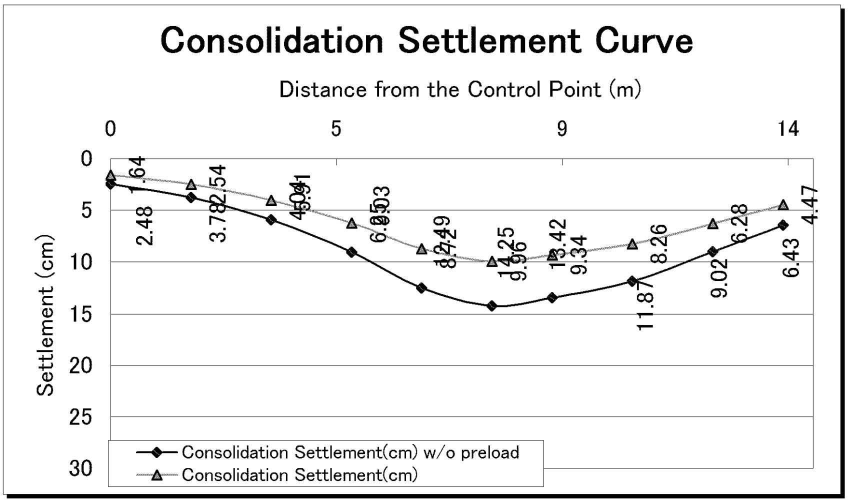
Consolidation Data		AC1	DC
PC kgf/cm^2		0.628	1.245
	p kgf/cm2		
e-logp	0.1	1.5997	1.0932
	0.2	1.5656	1.0584
	0.4	1.5066	1.0226
	0.8	1.3912	0.9826
	1.6	1.2443	0.9299
	3.2	1.0345	0.8488
	6.4	0.8732	0.7508
	12.8	0.7159	0.6381
Cv 10^{-3} cm^2/sec	0.05	4.324	21.94
	0.15	3.263	16.938
	0.3	3.661	10.956
	0.6	1.690	15.764
	1.2	2.281	12.185
	2.4	1.118	11.365
	4.8	0.966	9.376
	9.6	0.943	2.987

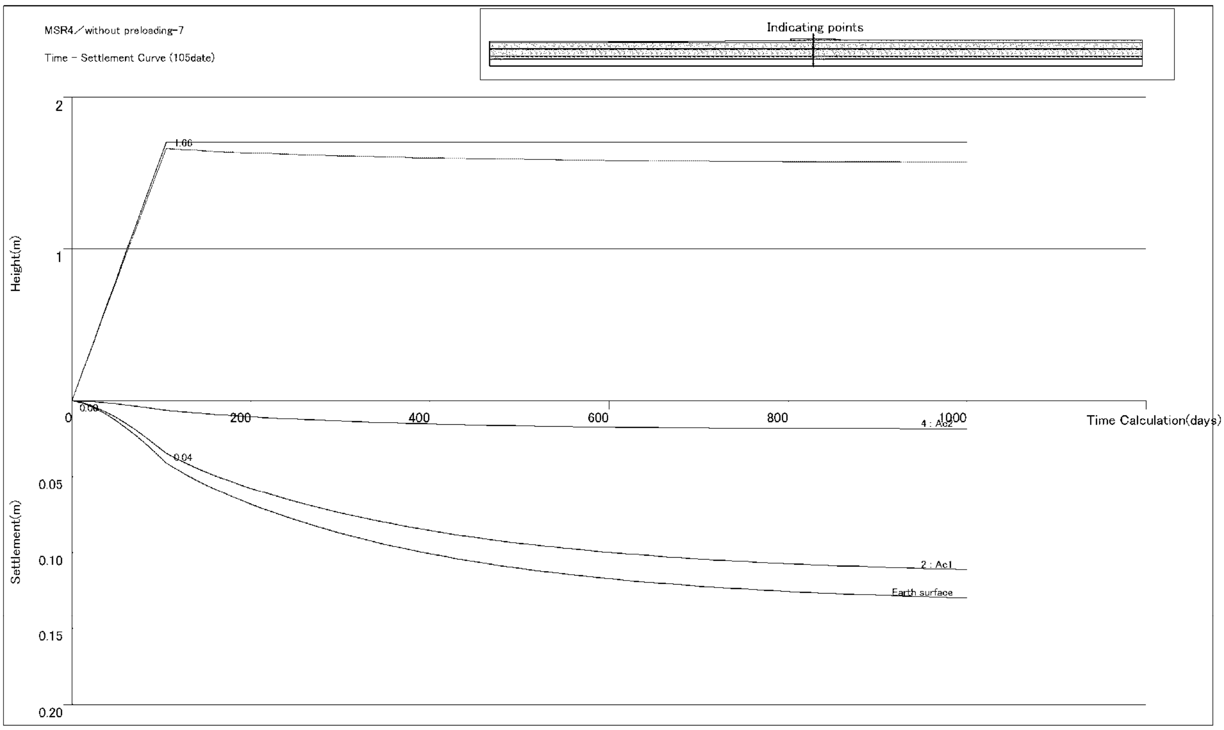
AC1 : BHLM27 2m
DC : BHLM24 14m

9.3 Considerng Pre-Load Effect

In this site, pre-load effect durling 105 days enmbankment works is considerd. Consolidation settlemet without pre-load effect and consolidation settlement after finishing embankment works is shown in the figure below.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.0	1.6	3.2	4.8	6.2	7.6	8.8	10.4	12.0	13.4
Consolidation Settlement(cm) w/o preload	2.48	3.78	5.91	9.03	12.49	14.25	13.42	11.87	9.02	6.43
after 105 days	0.84	1.24	1.87	2.78	3.78	4.29	4.08	3.60	2.74	1.96
Consolidation Settlement(cm)	1.64	2.54	4.04	6.25	8.72	9.96	9.34	8.26	6.28	4.47





Modulus of Deformation of the ground with considering pre-load effect is calculated as follows.

$E' = E_0 \times C_P / C_0$

Where,

E' : Modulus of Deformation of the Ground after Pre-Load (kN/m^2)

E_0 : Original Modulus of Deformation of the Ground (kN/m^2) (= 700 x N)

C : Cohesion after Pre-Load (kN/m^2)

C_0 : Original Cohesion (kN/m^2)

$C = C_0 + m \cdot \Delta P \cdot U$

Where,

m : Strength Increase Ratio (Cohesive soil : m = 0.25)

ΔP : Load Increase (kN/m^2)

U : Percentage of Consolidation

Cohesion and moduls of Deformation after Strength Incrasing by Preloading Effect (BMRW-8)

	c_0 (kN/m2)	φ(°)	E_0 (MN/m^2)	strength increase ratio m	ΔP (kN/m^2)	Degree of Consolidation U	C_P (kN/m^2)	C_P/C_0	E' (MN/m2)
Ac1	13	0	2.28	0.25	15.5	0.3	14	1.09	2.49
As1	0	26	6.30	-	-	-	-	-	6.30
Ac2	72	0	4.55	0.25	7.2	0.3	72	1.01	4.58
Ac2	72	0	15.40	-	-	-	-	-	15.40

9.4 Immediate Settlement

(1) Calculation of Converted Modules of Deformation

Modulus of Elasticity of Ground (BMRW-8)

Loading Width	B (m)	18.500
Loading Length	L (m)	5.600
Boring		BMRW-8

No.	Classification	E_i (MN/m^2)	Thickness of Layer (m)	h_i (m)	Denominator	Numerator
1	Ac1	2.49	6.67	6.67	-0.20759	
2	As1	6.30	0.50	7.17	-0.00328	
3	Ac2	4.58	7.85	15.02	-0.04605	
4	Ac2	15.40	1.78	16.80	-0.00195	
5						
6						
7						
8						
9						
10						
11						
	Total		16.80		-0.25888	-0.77875
E_m (MN/m^2)						3.0
(kN/m^2)						3008
(tf/m^2)						301

(2) Calculation of Load

Caluculation of the Load, qi

Proposed Elevation of Embankment	Sign	Load						Loading Width	1/2 Loading Width	Location of the Center of the Load from the Control Point				
(E.L. m)		(kN/m3)		(m)		(kN/m2)		(m)	(m)	(m)				
14.70	q1	18.0	×	1.1	=	19.8		13.40	6.70	6.70			=	6.70
	q2	18.0	×	1.0	=	18.0		6.90	3.45	6.50	+	3.45	=	9.95
	q3	18.0	×	1.0	=	18.0		6.90	3.45	6.50	+	3.45	=	9.95
	q4	18.0	×	1.0	=	18.0		4.40	2.20	6.50	+	2.20	=	8.70

(3) Calculation of immediate settlement

● Load Conditon

Sign	Load No	Load	1/2 Loading Width	Distance
		qi(kN/m2)	ai (m)	Xoi (m)
q1	Load 1	19.8	6.70	6.70
q2	Load 2	18.0	3.45	9.95
q3	Load 3	18.0	3.45	9.95
q4	Load 4	18.0	2.20	8.70

● Ground Conditon BMRW-8

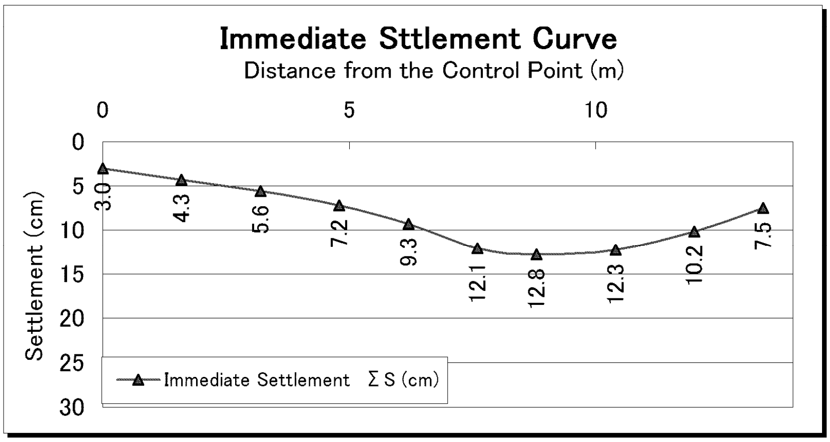
Thickness of Layer 16.8 m

Average Modulus of Elasticity 3008 kN/m2

$$Six = \sum_{i=\ell}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

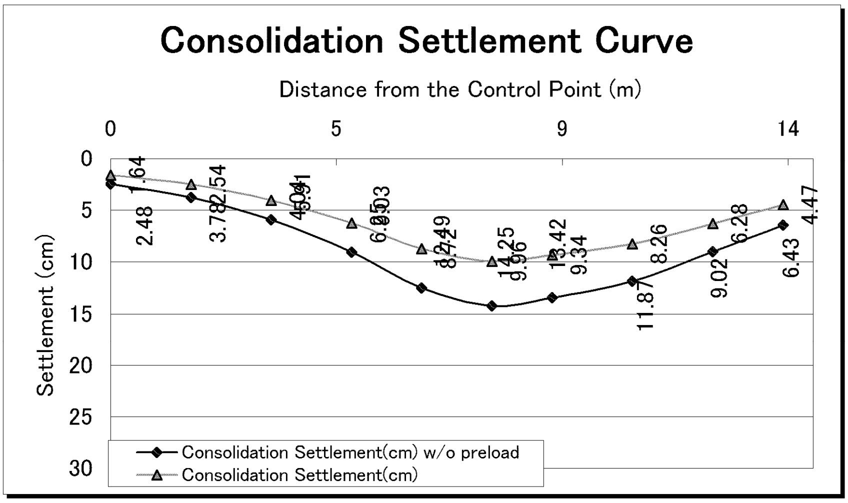
$$= \sum_{i=\ell}^{n} \frac{-0.9550\, a_i \cdot q_i}{E_m}\log \sin\left(\tan^{-1}\frac{a_i}{H}\right)\cdot\left\{1.0-0.2387\left[\alpha\log|\alpha|+\beta\log|\beta|\right]\right\}$$

				Control Point									
			Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	X	(m)	Caluculation	0.0	1.6	3.2	4.8	6.2	7.6	8.8	10.4	12.0	13.4
q1	X1	(m)	Xo1−X	6.7	5.1	3.5	1.9	0.5	0.9	2.1	3.7	5.3	6.7
	S1	(cm)		2.80	3.53	3.90	4.10	4.18	4.16	4.08	3.86	3.47	2.80
q2	X2	(m)	Xo2−X	10.0	8.4	6.8	5.2	3.8	2.4	1.2	0.5	2.1	3.5
	S2	(cm)		0.08	0.36	0.71	1.18	1.84	2.78	3.08	3.15	2.88	2.12
q3	X3	(m)	Xo3−X	10.0	8.4	6.8	5.2	3.8	2.4	1.2	0.5	2.1	3.5
	S3	(cm)		0.08	0.36	0.71	1.18	1.84	2.78	3.08	3.15	2.88	2.12
q4	X4	(m)	Xo4−X	8.7	7.1	5.5	3.9	2.5	1.1	0.1	1.7	3.3	4.7
	S4	(cm)		0.00	0.00	0.25	0.71	1.41	2.41	2.57	2.15	0.95	0.46
Immediate Settlement			ΣS (cm)	3.0	4.3	5.6	7.2	9.3	12.1	12.8	12.3	10.2	7.5



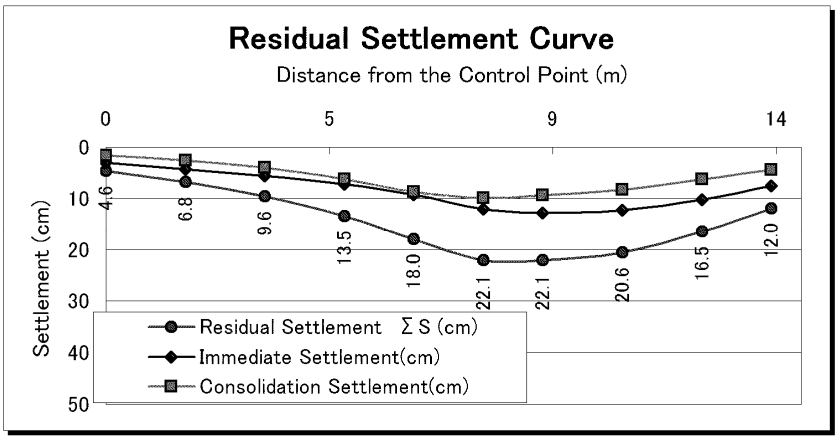
9.5 Consolidation Settlement

		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.0	1.6	3.2	4.8	6.2	7.6	8.8	10.4	12.0	13.4
Consolidation Settlement(cm) w/o preload		2.48	3.78	5.91	9.03	12.49	14.25	13.42	11.87	9.02	6.43
	after 105 days	0.84	1.24	1.87	2.78	3.78	4.29	4.08	3.60	2.74	1.96
Consolidation Settlement(cm)		1.64	2.54	4.04	6.25	8.72	9.96	9.34	8.26	6.28	4.47



9.6 Residual Settlement

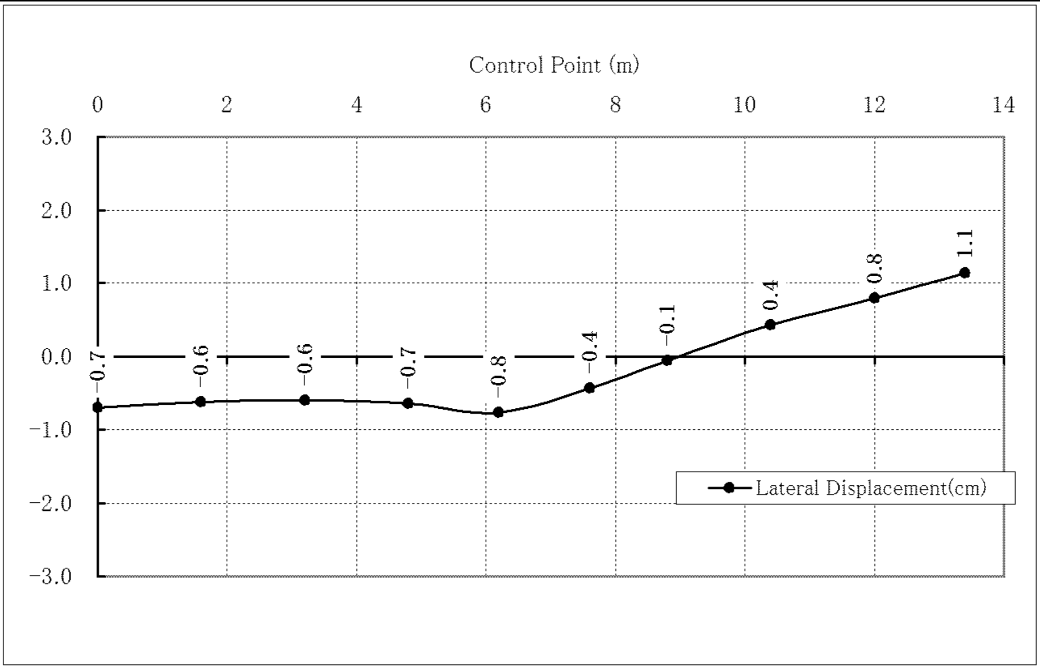
		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.0	1.6	3.2	4.8	6.2	7.6	8.8	10.4	12.0	13.4
Immediate Settlement(cm)		3.00	4.30	5.60	7.20	9.30	12.10	12.80	12.30	10.20	7.50
Consolidation Settlement(cm)		1.64	2.54	4.04	6.25	8.72	9.96	9.34	8.26	6.28	4.47
Residual Settlement	ΣS (cm)	**4.6**	**6.8**	**9.6**	**13.5**	**18.0**	**22.1**	**22.1**	**20.6**	**16.5**	**12.0**



9.7 Lateral Displacement

Lateral Displacement (BMRW-8)

			q1		q2		q3		q4		Total
q_i (kN/m^2)			19.800		18.000		18.000		18.000		
a_i (m)			6.700		3.450		3.450		2.200		
b_i (m)			2.800		2.800		2.800		2.800		
ν			0.400		0.400		0.400		0.400		
x_i (m)			6.700		9.950		9.950		8.700		
No.	x(m)	E (kN/m^2)	x_1(m)	R_1(cm)	x_2(m)	R_2(cm)	x_3(m)	R_3(cm)	x_4(m)	R_4(cm)	R(cm)
(1)	0.0	3008	6.7	−0.42	9.9	−0.10	9.9	−0.10	8.7	−0.07	−0.7
(2)	1.6	3008	5.1	−0.28	8.4	−0.13	8.4	−0.13	7.1	−0.09	−0.6
(3)	3.2	3008	3.5	−0.18	6.8	−0.16	6.8	−0.16	5.5	−0.11	−0.6
(4)	4.8	3008	1.9	−0.09	5.2	−0.20	5.2	−0.20	3.9	−0.15	−0.7
(5)	6.2	3008	0.5	−0.02	3.8	−0.27	3.8	−0.27	2.5	−0.21	−0.8
(6)	7.6	3008	−0.9	0.04	2.4	−0.18	2.4	−0.18	1.1	−0.11	−0.4
(7)	8.8	3008	−2.1	0.10	1.2	−0.09	1.2	−0.09	−0.1	0.01	−0.1
(8)	10.4	3008	−3.7	0.19	−0.5	0.03	−0.5	0.03	−1.7	0.17	0.4
(9)	12.0	3008	−5.3	0.30	−2.1	0.16	−2.1	0.16	−3.3	0.18	0.8
(10)	13.4	3008	−6.7	0.42	−3.4	0.29	−3.4	0.29	−4.7	0.13	1.1



CHAPTER 11 STRUCTURAL CALCULATION OF SLUICEWAY STRUCTURE

Regarding Structural Calculation of Sluiceway, the detail calculation sheet which is chosen as representative cases from table is shown.

Table R 11.1 Calculation Case

Sluiceway Number	Location /Station		Proposed Dimension (m x m)	Calculation Case
MSL-1	Left Bank	1+104	1.4 x 1.4	
MSL-2		1+333	1.5 x 1.5	
MSL-3		3+945	2 x 1.2 x 1.2	○
MSL-4		4+233	1.6 x 1.6	○
MSL-5		4+406	1.0 x 1.0	
MSL-6		4+503	1.2 x 1.2	
MSR-2	Right Bank	3+157	1.4 x 1.4	
MSR-3		3+258	2.0 x 1.6	○
MSR-4		3+438	1.5 x 1.5	

Note: "○" means that calculation sheet is shown in this report.

11.1 Box Culvert (Cross Sectional Direction)

(1) Design Condition

The condition of concrete and reinforcing bar is same as manhole. Only the condition which is different from other structure is indicated as follows

(a) Live Load

(i) On Top Slab and Cover

Vehicle : T-25, 250 (kN)

(ii) On the Ground

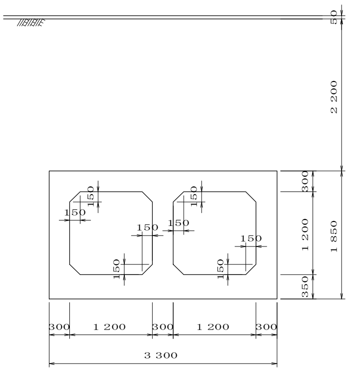
Live Load: 10.0 (kN/m^2)

(2) Structural Calculation

The detail of structural calculation of box culvert in cross sectional direction is indicated from the following page.

1 MSL3 Box Culvert (Cross Section)

1.1 Design Condition

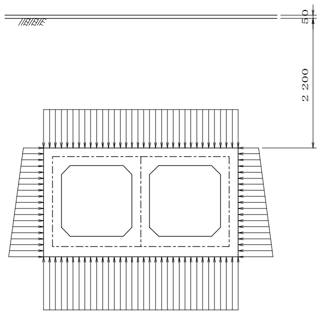


［ T-25 ］

Thickness of Embankment is less than 4.0m

Intensity of uniformly distributed load q = 10.00 (kN/m^2)

1.1.1 Dead Load



Weight of Body

(1) Top Slab

w	=	0.300 × 3.300 × 24.00	=	23.76 (kN/m)
w	=	1/2(0.150 × 0.150 + 0.150 × 0.150) ×24.00	=	0.54 (kN/m)
w	=	1/2(0.150 × 0.150 + 0.150 × 0.150) ×24.00	=	0.54 (kN/m)
		ΣW	=	24.84 (kN/m)

$$w = \frac{24.84}{3.000} = 8.28 \ (kN/m^2)$$

(2) Left Side Wall

w = 0.300 × 24.00 = 7.20 (kN/m^2)

(3) Right Side Wall

w = 0.300 × 24.00 = 7.20 (kN/m^2)

Load

(1) Pavement and Embankment

α

Pavement	=	1.000 × 0.050 × 22.50	=	1.13 (kN/m^2)
Embankment	=	1.000 × 2.200 × 18.00	=	39.60 (kN/m^2)
		Σwd	=	40.73 (kN/m^2)

(2) Load Effecting on Top Slab

Uniformly Distributed Load

w = 40.73 + 0.00 = 40.73 (kN/m^2)

Water Pressure and Earth Pressure

Intensity of Earth Pressure and Water Pressure

pi = Ko×(qd + Yo×γa + Zo×γ)

Ko :	Coefficient of Earth Pressure at Rest	Left	=	0.500
		Right	=	0.500
qd :	Load Effecting top of Embankment		=	0.00 (kN/m^2)
Yo :	Thickness of Pavement		=	0.050 (m)
γa:	Unit Weight of Pavement		=	22.50 (kN/m^3)
γ :	Unit Weight of Soil		=	18.00 (kN/m^3)
Zo:	Depth (m)			

(1) Left Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	20.36
p_2	Axis Line of Top Slab	2.350	21.71
p_3	Axis Line of Bottom Slab	3.875	35.44
p_4	Under side of Bottom Slab	4.050	37.01

(2) Right Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	2.200	20.36
p_2	Axis Line of Top Slab	2.350	21.71
p_3	Axis Line of Bottom Slab	3.875	35.44
p_4	Under side of Bottom Slab	4.050	37.01

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Weight of Body	Top Slab	24.84		1.650		40.99
	Left Side Wall	8.64		0.150		1.30
	Center Side	8.64		1.650		14.26
	Right Side Wall	8.64		3.150		27.22
Load		134.39		1.650		221.75
Earth Pressure	Left Side Wall		53.07		0.836	44.34
	Right Side Wall		-53.07		0.836	-44.34
Total		185.15				305.50

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 1.650 \ (m)$$

$$e = \frac{B}{2} - X = 0.000 \ (m)$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

$$ql = \frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} = 56.11 \ (kN/m^2)$$

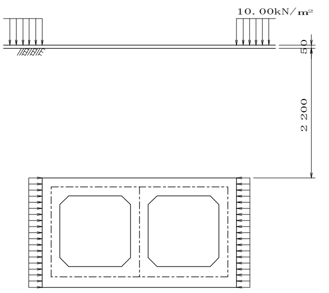
$$qr = \frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} = 56.11 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 56.11 \ (kN/m^2)$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 56.11 \ (kN/m^2)$$

1.1.2 Live Load (Case 1)

［Lateral Pressure］



Load

(1) Horizontal Load Effecting on Left Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

(2) Horizontal Load Effecting on Right Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

Summary of Effecting Force

Item		H (kN/m)	y (m)	M (kN.m/m)
Left Side Wall	Distribute	9.25	0.925	8.56
Right Side Wall	Distribute	-9.25	0.925	-8.56
Total				0.00

Subgrade Reaction

(1) Subgrade Reaction

$$ql = \pm \left(\frac{6 \times Me}{B^2} \right) \times 1.000 = 0.00 \ (kN/m^2)$$

$$qr = 0.00 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 0.00 \ (kN/m^2)$$

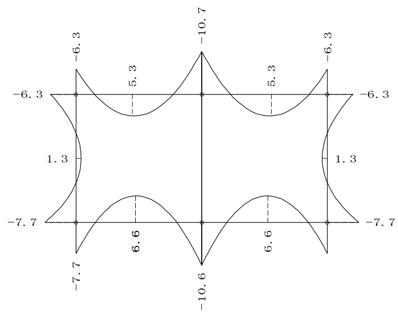
$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 0.00 \ (kN/m^2)$$

1.1.3 Calculation Case

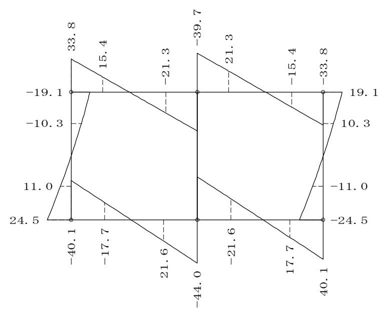
No	
1	Dead Load
2	Dead Load + Live Load 1
3	Dead Load + Live Load 2

1.2 Cross Section Force

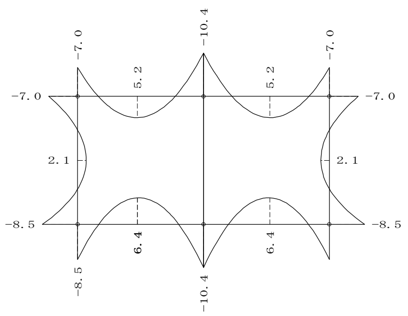
(1) Bending Moment (Case 1)



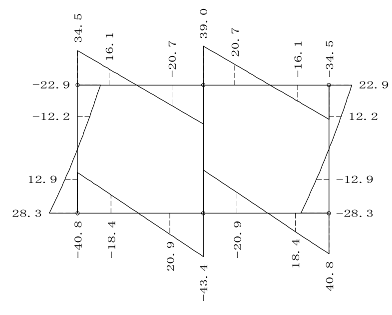
(2) Shear Force (Case 1)



(3) Bending Moment (Case 2)



(4) Shear Force (Case2)



1.3 Stress Calculation

1.3.1 Intensity of Bending Stress

(1) Top Slab

a) Outside Tensile

Item		Unit	Left Edge	Center	Center Wall	Center	Right
Bending Moment	M	kN.m	-7.0	0.0	-10.7	0.0	-7.0
Axial Force	N	kN	22.9	0.0	19.1	0.0	22.9
Width of Member	b	cm	100.0	100.0	100.0	100.0	100.0
Height of Member	h	cm	30.0	30.0	30.0	30.0	30.0
Effective Width	d	cm	21.0	21.0	21.0	21.0	21.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	1.34	0.00	2.92	0.00	1.34
	Inside	cm^2	0.00	0.00	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
	Inside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
Neutral Axis	X	cm	5.223	0.000	4.435	0.000	5.223
Stress Intensity	σc	N/mm^2	1.66	0.00	2.74	0.00	1.66
	σs	N/mm^2	45.00	0.00	92.24	0.00	45.00
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00	140.00	140.00
Case	―	――	2	―	1	―	2

b) Inside Tensile

Item		Unit	Left Edge	Center	Center Wall	Center	Right
Bending Moment	M	kN.m	0.0	5.3	0.0	5.3	0.0
Axial Force	N	kN	0.0	19.1	0.0	19.1	0.0
Width of Member	b	cm	100.0	100.0	100.0	100.0	100.0
Height of Member	h	cm	30.0	30.0	30.0	30.0	30.0
Effective Width	d	cm	21.0	21.0	21.0	21.0	21.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	0.00	0.00	0.00	0.00	0.00
	Inside	cm^2	0.00	0.94	0.00	0.94	0.00
Reinforcement	Outside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
	Inside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
Neutral Axis	X	cm	0.000	5.406	0.000	5.406	0.000
Stress Intensity	σc	N/mm^2	0.00	1.25	0.00	1.25	0.00
	σs	N/mm^2	0.00	32.47	0.00	32.47	0.00
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00	140.00	140.00
Case	―	――	―	1	―	1	―

(2) Left Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-7.0	2.1	-8.5
Axial Force	N	kN	34.5	38.9	43.1
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	30.0	30.0	30.0
Effective Width	d	cm	21.0	21.0	21.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	0.77	0.00	0.90
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
	Inside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
Neutral Axis	X	cm	6.492	28.866	6.630
Stress Intensity	σc	N/mm^2	1.48	0.27	1.77
	σs	N/mm^2	29.68	-0.66	34.57
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	―	――	2	2	2

(3) Center Wall

Item		Unit	Corner Edge	Bottom Edge
Bending Moment	M	kN.m	0.0	0.0
Axial Force	N	kN	0.0	88.0
Width of Member	b	cm	100.0	100.0
Height of Member	h	cm	30.0	30.0
Effective Width	d	cm	21.0	21.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	0.00	0.00
	Inside	cm^2	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
	Inside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
Neutral Axis	X	cm	0.000	941.810
Stress Intensity	σc	N/mm^2	0.00	0.29
	σs	N/mm^2	0.00	-2.59
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20
	σsa	N/mm^2	140.00	-
Case	―	――	―	1

(4) Right Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-7.0	2.1	-8.5
Axial Force	N	kN	34.5	38.9	43.1
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	30.0	30.0	30.0
Effective Width	d	cm	21.0	21.0	21.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	0.77	0.00	0.90
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
	Inside	cm^2	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525	D12 @250 D― @―― 4.525
Neutral Axis	X	cm	6.492	28.866	6.630
Stress Intensity	σc	N/mm^2	1.48	0.27	1.77
	σs	N/mm^2	29.68	-0.66	34.57
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	―	――	2	2	2

(5) Bottom Slab

a) Outside Tensile

Item		Unit	Left Edge	Center	Center Wall	Center	Right
Bending Moment	M	kN.m	-8.5	0.0	-10.6	0.0	-8.5
Axial Force	N	kN	28.3	0.0	24.5	0.0	28.3
Width of Member	b	cm	100.0	100.0	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0	35.0	35.0
Effective Width	d	cm	23.5	23.5	23.5	23.5	23.5
Thickness of Cover Concrete (Outside)	d1	cm	11.5	11.5	11.5	11.5	11.5
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	1.22	0.00	2.14	0.00	1.22
	Inside	cm^2	0.00	0.00	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
	Inside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
Neutral Axis	X	cm	5.975	0.000	5.164	0.000	5.975
Stress Intensity	σc	N/mm^2	1.58	0.00	2.15	0.00	1.58
	σs	N/mm^2	41.71	0.00	68.86	0.00	41.71
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00	140.00	140.00
Case	—	—	2	—	1	—	2

b) Inside Tensile

Item		Unit	Left Edge	Center	Center Wall	Center	Right
Bending Moment	M	kN.m	0.0	6.6	0.0	6.6	0.0
Axial Force	N	kN	0.0	24.5	0.0	24.5	0.0
Width of Member	b	cm	100.0	100.0	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	11.5	11.5	11.5	11.5	11.5
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	0.00	0.00	0.00	0.00	0.00
	Inside	cm^2	0.00	0.75	0.00	0.75	0.00
Reinforcement	Outside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
	Inside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
Neutral Axis	X	cm	0.000	6.842	0.000	6.842	0.000
Stress Intensity	σc	N/mm^2	0.00	1.07	0.00	1.07	0.00
	σs	N/mm^2	0.00	27.03	0.00	27.03	0.00
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00	140.00	140.00
Case	—	—	—	1	—	1	—

1.3.2 Intensity of Shear Stress

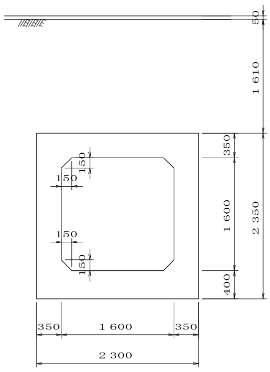
$$\tau m = \frac{S}{b \times d} \leqq \tau a$$

b = 100.0 (cm)

Member	Position	S (kN)	d (cm)	τm (N/mm^2)	τa (N/mm^2)	Case	L (m)
Top Slab	Left Edge	16.1	21.0	0.077	0.360	2	0.375
	Left Side τ	-21.3	21.0	0.102	0.360	1	0.375
	Right Side τ	21.3	21.0	0.102	0.360	1	0.375
	Right Edge	-16.1	21.0	0.077	0.360	2	0.375
Left Side Wall	Upper Side Edge	-12.2	21.0	0.058	0.360	2	0.375
	Upper Side τ	12.9	21.0	0.061	0.360	2	0.400
Center Wall	Upper Side Edge	0.0	21.0	0.000	0.360	1	0.450
	Upper Side τ	0.0	21.0	0.000	0.360	1	0.475
Right Side Wall	Upper Side Edge	12.2	21.0	0.058	0.360	2	0.375
	Upper Side τ	-12.9	21.0	0.061	0.360	2	0.400
Bottom Slab	Left Edge	-18.4	23.5	0.078	0.360	2	0.400
	Left Side τ	21.6	23.5	0.092	0.360	1	0.400
	Right Side τ	-21.6	23.5	0.092	0.360	1	0.400
	Right Edge	18.4	23.5	0.078	0.360	2	0.400

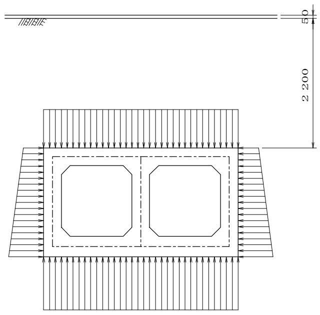
2 MSL4 Box Culvert (Cross Section)

2.1 Design Condition



[T-25]
Thickness of Embankment is less than 4.0m
Intensity of uniformly distributed load q = 10.00 (kN/m^2)

2.1.1 Dead Load



Weight of Body

(1) Top Slab

w = 0.350 × 2.300 × 24.00 = 19.32 (kN/m)
w = 1/2(0.150 × 0.150 + 0.150 × 0.150) × 24.00 = 0.54 (kN/m)
ΣW = 19.86 (kN/m)

$$w = \frac{19.86}{1.950} = 10.18 \ (kN/m^2)$$

(2) Left Side Wall

w = 0.350 × 24.00 = 8.40 (kN/m^2)

(3) Right Side Wall

w = 0.350 × 24.00 = 8.40 (kN/m^2)

Load

(1) Pavement and Embankment

	α			
Pavement =	1.000 ×	0.050 × 22.50	=	1.13 (kN/m^2)
Embankment =	1.000 ×	1.610 × 18.00	=	28.98 (kN/m^2)
		Σwd	=	30.10 (kN/m^2)

(2) Load Effecting on Top Slab
Uniformly Distributed Load
w = 30.10 + 0.00 = 30.10 (kN/m^2)

Water Pressure and Earth Pressure
Intensity of Earth Pressure and Water Pressure
pi = Ko×(qd + Yo×γa + Zo×γ)
Ko : Coefficient of Earth Pressure at Rest Left = 0.500
Right = 0.500
qd : Load Effecting top of Embankment = 0.00 (kN/m^2)
Yo : Thickness of Pavement = 0.050 (m)
γa: Unit Weight of Pavement = 22.50 (kN/m^3)
γ : Unit Weight of Soil = 18.00 (kN/m^3)
Zo: Depth (m)

(1) Left Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	1.610	15.05
p_2	Axis Line of Top Slab	1.785	16.63
p_3	Axis Line of Bottom Slab	3.760	34.40
p_4	Under side of Bottom Slab	3.960	36.20

(2) Right Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	1.610	15.05
p_2	Axis Line of Top Slab	1.785	16.63
p_3	Axis Line of Bottom Slab	3.760	34.40
p_4	Under side of Bottom Slab	3.960	36.20

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Weight of Body	Top Slab	19.86		1.150		22.84
	Left Side Wall	13.44		0.175		2.35
	Center Side	13.44		2.125		28.56
	Right Side Wall	19.86		1.150		22.84
Load		69.24		1.150		79.63
Earth Pressure	Left Side Wall		60.22		1.013	61.03
	Right Side Wall		-60.22		1.013	-61.03
Total		115.98				133.38

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 1.150 \ (m)$$

$$e = \frac{B}{2} - X = 0.000 \ (m)$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

$$ql = \frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} = 50.43 \ (kN/m^2)$$

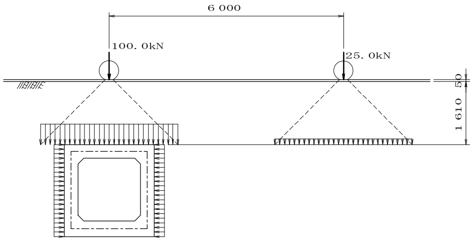
$$qr = \frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} = 50.43 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 50.43 \ (kN/m^2)$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 50.43 \ (kN/m^2)$$

2.1.2 Live Load (Case 1)

[2 Axis 250 (kN)]



Intensity of Wheel Load

$$P_{l+i} = \frac{2 \times P \times (1+i)}{2.75}$$

$$P_{vl} = \frac{(P_{l+i}) \times \beta}{2 \times D + D_o}$$

Pl+I	:	Live Load at Unit Length (kN/m)
P	:	Wheel Load (kN)
i	:	Impact Coefficient
Pvl	:	Converted Uniformly Distributed Load (kN/m^2)
D	:	Depth = 1.850 (m)
Do	:	Width of Wheel (m)
β	:	Coefficient of Reduction

Rear wheel

$$P_{l+i} = \frac{2 \times 100.0 \times (1 + 0.300)}{2.75} = 94.55 \text{ (kN/m)}$$

$$P_{vl} = \frac{94.55 \times 0.900}{2 \times 1.660 + 0.20} = 24.17 \text{ (kN/m}^2\text{)}$$

Front wheel

$$P_{l+i} = \frac{2 \times 25.0 \times (1 + 0.300)}{2.75} = 23.64 \text{ (kN/m)}$$

$$P_{vl} = \frac{23.64 \times 1.000}{2 \times 1.660 + 0.20} = 6.71 \text{ (kN/m}^2\text{)}$$

Load

(1) Vertical Load Effecting Top Slab

	Intensity (kN/m^2)	Position (m)	Width (m)
Rear Wheel	24.17	0.000	1.950
Front Wheel	6.71	0.000	0.000

(2) Horizontal Load Effecting on Left Side Wall

Converted Uniformly Distributed Load

wl = 24.17 (kN/m^2)

p = Ko×wl = 0.500 × 24.17 = 12.09 (kN/m^2)

(3) Horizontal Load Effecting on Right Side Wall

Converted Uniformly Distributed Load

wl = 24.17 (kN/m^2)

p = Ko×wl = 0.500 × 24.17 = 12.09 (kN/m^2)

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Top Slab	Distributed 1	55.60		1.150		63.94
	Distributed 2	0.00		0.000		0.00
Left Side Wall	Distributed		28.40		1.175	33.37
Right Side Wall	Distributed		-28.40		1.175	-33.37
Total		55.60				63.94

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 1.150 \text{ (m)}$$

$$e = \frac{B}{2} - X = 0.000 \text{ (m)}$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

$$q_l = \left(\frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} \right) \times 1.000 = 24.17 \text{ (kN/m}^2\text{)}$$

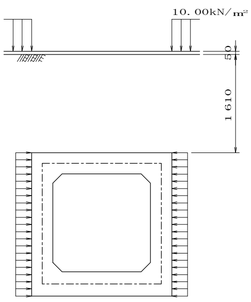
$$q_r = \left(\frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} \right) \times 1.000 = 24.17 \text{ (kN/m}^2\text{)}$$

$$q_l' = q_l + \frac{q_r - q_l}{B} \times \frac{T}{2} = 24.17 \text{ (kN/m}^2\text{)}$$

$$q_r' = q_r + \frac{q_l - q_r}{B} \times \frac{T}{2} = 24.17 \text{ (kN/m}^2\text{)}$$

2.1.3 Live Load (Case 2)

[Lateral Pressure]



Load

(1) Horizontal Load Effecting on Left Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

(2) Horizontal Load Effecting on Right Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

Summary of Effecting Force

Item		H (kN/m)	y (m)	M (kN.m/m)
Left Side Wall	Distribute	11.75	1.175	13.81
Right Side Wall	Distribute	-11.75	1.175	-13.81
Total				0.00

Subgrade Reaction

(1) Subgrade Reaction

$$q_l = \pm \left(\frac{6 \times Me}{B^2} \right) \times 1.000 = 0.00 \text{ (kN/m}^2\text{)}$$

$$q_r = 0.00 \text{ (kN/m}^2\text{)}$$

$$q_l' = q_l + \frac{q_r - q_l}{B} \times \frac{T}{2} = 0.00 \text{ (kN/m}^2\text{)}$$

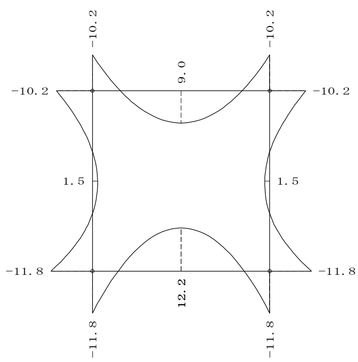
$$q_r' = q_r + \frac{q_l - q_r}{B} \times \frac{T}{2} = 0.00 \text{ (kN/m}^2\text{)}$$

2.1.4 Calculation Case

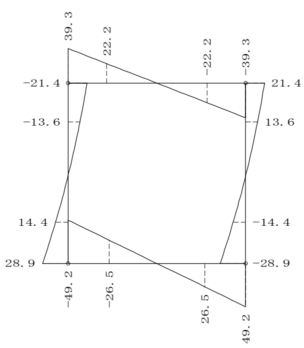
No	
1	Dead Load
2	Dead Load + Live Load 1
3	Dead Load + Live Load 2

2.2 Cross Section Force

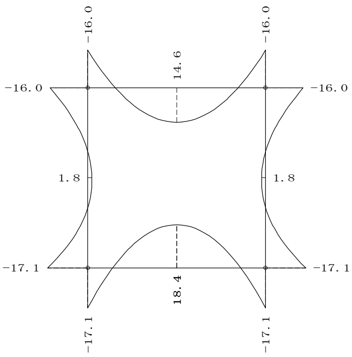
(1) Bending Moment (Case 1)



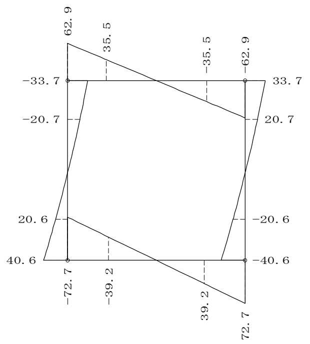
(2) Shear Force (Case 1)



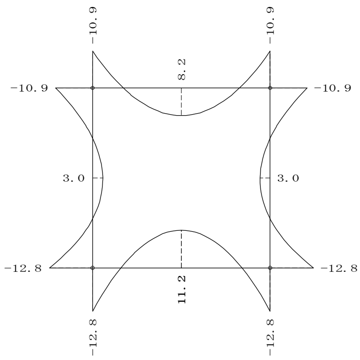
(3) Bending Moment (Case 2)



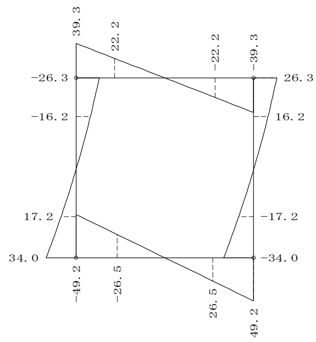
(4) Shear Force (Case2)



(5) Bending Moment (Case 3)



(6) Shear Force (Case3)



2.3 Stress Calculation

2.3.1 Intensity of Bending Stress

(1) Top Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-16.0	14.6	-16.0
Axial Force	N	kN	33.7	33.7	33.7
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	3.11	0.00	3.11
	Inside	cm^2	0.00	2.69	0.00
Reinforcement	Outside	cm^2	D12 @250 D— @—	D12 @250 D— @—	D12 @250 D— @—
	Inside	cm^2	D12 @250 D— @—	D12 @250 D— @—	D12 @250 D— @—
Neutral Axis	X	cm	5.427	5.576	5.427
Stress Intensity	σc	N/mm^2	2.88	2.60	2.88
	σs	N/mm^2	98.16	85.58	98.16
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	—	—	2	2	2

(2) Left Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-16.0	1.8	-17.1
Axial Force	N	kN	62.9	69.7	76.3
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	1.77	0.00	1.48
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D— @—	D12 @250 D— @—	D12 @250 D— @—
	Inside	cm^2	D12 @250 D— @—	D12 @250 D— @—	D12 @250 D— @—
Neutral Axis	X	cm	7.104	56.072	7.810
Stress Intensity	σc	N/mm^2	2.55	0.29	2.58
	σs	N/mm^2	60.95	-1.38	54.09
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	—	—	2	2	2

(3) Right Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-16.0	1.8	-17.1
Axial Force	N	kN	62.9	69.7	76.3
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	1.77	0.00	1.48
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D12 @250 D─ @──	D12 @250 D─ @──	D12 @250 D─ @──
	Inside	cm^2	D12 @250 D─ @──	D12 @250 D─ @──	D12 @250 D─ @──
Neutral Axis	X	cm	7.104	56.072	7.810
Stress Intensity	σc	N/mm^2	2.55	0.29	2.58
	σs	N/mm^2	60.95	-1.38	54.09
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	──	──	2	2	2

(4) Bottom Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-17.1	18.4	-17.1
Axial Force	N	kN	40.6	40.6	40.6
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	40.0	40.0	40.0
Effective Width	d	cm	28.5	31.0	28.5
Thickness of Cover Concrete (Outside)	d1	cm	11.5	11.5	11.5
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	2.56	0.00	2.56
	Inside	cm^2	0.00	2.67	0.00
Reinforcement	Outside	cm^2	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525
	Inside	cm^2	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525	D12 @250 D─ @── 4.525
Neutral Axis	X	cm	6.172	6.414	6.172
Stress Intensity	σc	N/mm^2	2.52	2.47	2.52
	σs	N/mm^2	81.97	85.11	81.97
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	──	──	2	2	2

2.3.2 Intensity of Shear Stress

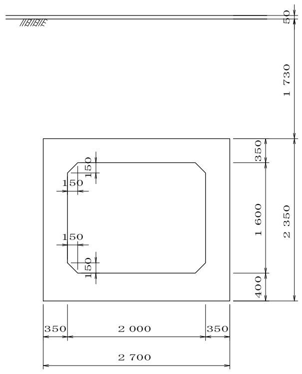
$$\tau m = \frac{S}{b \times d} \leqq \tau a$$

b = 100.0 (cm)

Member	Position	S (kN)	d (cm)	τm (N/mm^2)	τa (N/mm^2)	Case	L (m)
Top Slab	Left Side τ	35.5	26.0	0.136	0.360	2	0.425
	Right Side τ	-35.5	26.0	0.136	0.360	2	0.425
Left Side Wall	UpperSide	-20.7	26.0	0.079	0.360	2	0.425
	UnderSide	20.6	26.0	0.079	0.360	2	0.450
Right Side Wall	UpperSide	20.7	26.0	0.079	0.360	2	0.425
	UnderSide	-20.6	26.0	0.079	0.360	2	0.450
Bottom Slab	Left Side τ	-39.2	28.5	0.137	0.360	2	0.450
	Right Side τ	39.2	28.5	0.137	0.360	2	0.450

3 MSR3 Box Culvert (Cross Section)

3.1 Design Condition

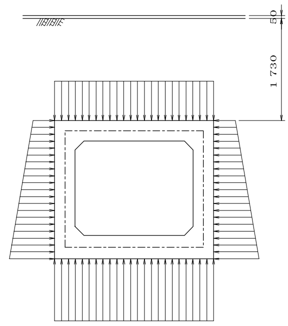


［ T-25 ］

Thickness of Embankment is less than 4.0m

Intensity of uniformly distributed load q = 10.00 (kN/m^2)

3.1.1 Dead Load



Weight of Body

(1) Top Slab

w	=	0.350 × 2.700 × 24.00	=	22.68 (kN/m)
w	=	1/2(0.150 × 0.150 + 0.150 × 0.150) × 24.00	=	0.54 (kN/m)
		ΣW	=	23.22 (kN/m)

$$w = \frac{23.22}{2.350} = 9.88 \ (kN/m^2)$$

(2) Left Side Wall

w = 0.350 × 24.00 = 8.40 (kN/m^2)

(3) Right Side Wall

w = 0.350 × 24.00 = 8.40 (kN/m^2)

Load

(1) Pavement and Embankment

		α		
Pavement	=	1.000 × 0.050 × 22.50	=	1.13 (kN/m^2)
Embankment	=	1.000 × 1.730 × 18.00	=	31.14 (kN/m^2)
		Σwd	=	32.26 (kN/m^2)

(2) Load Effecting on Top Slab

Uniformly Distributed Load

w = 32.26 + 0.00 = 32.26 (kN/m^2)

Water Pressure and Earth Pressure

Intensity of Earth Pressure and Water Pressure

pi = Ko×(qd + Yo×γa + Zo×γ)

	Ko : Coefficient of Earth Pressure at Rest	Left	=	0.500
		Right	=	0.500
qd :	Load Effecting top of Embankment		=	0.00 (kN/m^2)
Yo :	Thickness of Pavement		=	0.050 (m)
γa:	Unit Weight of Pavement		=	22.50 (kN/m^3)
γ :	Unit Weight of Soil		=	18.00 (kN/m^3)
Zo:	Depth (m)			

(1) Left Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	1.730	16.13
p_2	Axis Line of Top Slab	1.905	17.71
p_3	Axis Line of Bottom Slab	3.880	35.48
p_4	Under side of Bottom Slab	4.080	37.28

(2) Right Side Wall

	Position	Zo (m)	p (kN/m^2)
p_1	Upper Side of Top Slab	1.730	16.13
p_2	Axis Line of Top Slab	1.905	17.71
p_3	Axis Line of Bottom Slab	3.880	35.48
p_4	Under side of Bottom Slab	4.080	37.28

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Weight of Body	Top Slab	23.22		1.350		31.35
	Left Side Wall	13.44		0.175		2.35
	Right Side Wall	13.44		2.525		33.94
Load		87.12		1.350		117.61
Earth Pressure	Left Side Wall		62.76		1.020	64.01
	Right Side Wall		-62.76		1.020	-64.01
Total		137.22				185.24

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 1.350 \ (m)$$

$$e = \frac{B}{2} - X = 0.000 \ (m)$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

$$ql = \frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} = 50.82 \ (kN/m^2)$$

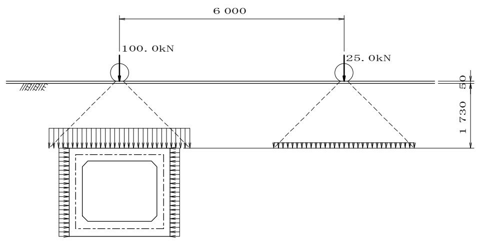
$$qr = \frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} = 50.82 \ (kN/m^2)$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 50.82 \ (kN/m^2)$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 50.82 \ (kN/m^2)$$

3.1.2 Live Load (Case 1)

［ 2 Axis 250 (kN) ］



Intensity of Wheel Load

$$Pl+i = \frac{2 \times P \times (1+i)}{2.75}$$

$$Pvl = \frac{(Pl+i) \times \beta}{2 \times D + Do}$$

Pl+I	:	Live Load at Unit Length (kN/m)
P	:	Wheel Load (kN)
i	:	Impact Coefficient
Pvl	:	Converted Uniformly Distributed Load (kN/m^2)
D	:	Depth = 1.850 (m)
Do	:	Width of Wheel (m)
β	:	Coefficient of Reduction

Rear wheel

$$Pl+i = \frac{2 \times 100.0 \times (1 + 0.300)}{2.75} = 94.55 \ \text{(kN/m)}$$

$$Pvl = \frac{94.55 \times 0.900}{2 \times 1.780 + 0.20} = 22.63 \ \text{(kN/m}^2\text{)}$$

Front wheel

$$Pl+i = \frac{2 \times 25.0 \times (1 + 0.300)}{2.75} = 23.64 \ \text{(kN/m)}$$

$$Pvl = \frac{23.64 \times 1.000}{2 \times 1.780 + 0.20} = 6.29 \ \text{(kN/m}^2\text{)}$$

Load

(1) Vertical Load Effecting Top Slab

	Intensity (kN/m^2)	Position (m)	Width (m)
Rear Wheel	22.63	0.000	2.350
Front Wheel	6.29	0.000	0.000

(2) Horizontal Load Effecting on Left Side Wall

Converted Uniformly Distributed Load

wl = 22.63 (kN/m^2)

p = Ko×wl = 0.500 × 22.63 = 11.32 (kN/m^2)

(3) Horizontal Load Effecting on Right Side Wall

Converted Uniformly Distributed Load

wl = 22.63 (kN/m^2)

p = Ko×wl = 0.500 × 22.63 = 11.32 (kN/m^2)

Summary of Effecting Force

Item		V (kN/m)	H (kN/m)	x (m)	y (m)	M (kN.m/m)
Top Slab	Distributed 1	61.10		1.350		82.49
	Distributed 2	0.00		0.000		0.00
Left Side Wall	Distributed		26.59		1.175	31.24
Right Side Wall	Distributed		-26.59		1.175	-31.24
Total		61.10				82.49

Subgrade Reaction

(1) Force Effecting Position and Displacement Distance

$$X = \frac{\Sigma M}{\Sigma V} = 1.350 \ \text{(m)}$$

$$e = \frac{B}{2} - X = 0.000 \ \text{(m)}$$

(2) Intensity of Subgrade Reaction

Me = ΣV × e = 0.00 (kN.m/m)

$$ql = \left(\frac{\Sigma V}{B} + \frac{6 \times Me}{B^2} \right) \times 1.000 = 22.63 \ \text{(kN/m}^2\text{)}$$

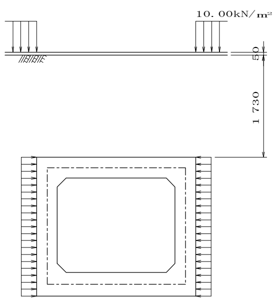
$$qr = \left(\frac{\Sigma V}{B} - \frac{6 \times Me}{B^2} \right) \times 1.000 = 22.63 \ \text{(kN/m}^2\text{)}$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 22.63 \ \text{(kN/m}^2\text{)}$$

$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 22.63 \ \text{(kN/m}^2\text{)}$$

3.1.3 Live Load (Case 2)

［Lateral Pressure］



Load

(1) Horizontal Load Effecting on Left Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

(2) Horizontal Load Effecting on Right Side Wall

p Ko×wl = 0.500 × 10.00 = 5.00 (kN/m^2)

Summary of Effecting Force

Item		H (kN/m)	y (m)	M (kN.m/m)
Left Side Wall	Distribute	11.75	1.175	13.81
Right Side Wall	Distribute	-11.75	1.175	-13.81
Total				0.00

Subgrade Reaction

(1) Subgrade Reaction

$$ql = \pm \left(\frac{6 \times Me}{B^2} \right) \times 1.000 = 0.00 \ \text{(kN/m}^2\text{)}$$

$$qr = 0.00 \ \text{(kN/m}^2\text{)}$$

$$ql' = ql + \frac{qr - ql}{B} \times \frac{T}{2} = 0.00 \ \text{(kN/m}^2\text{)}$$

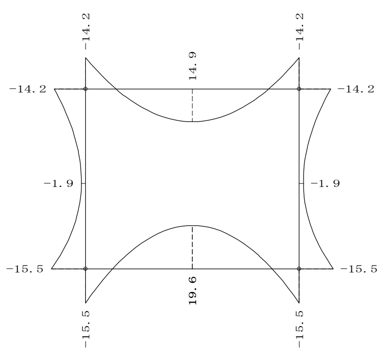
$$qr' = qr + \frac{ql - qr}{B} \times \frac{T}{2} = 0.00 \ \text{(kN/m}^2\text{)}$$

3.1.4 Calculation Case

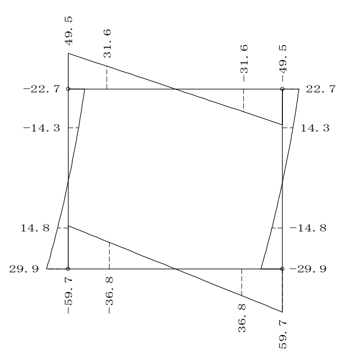
No	
1	Dead Load
2	Dead Load + Live Load 1
3	Dead Load + Live Load 2

3.2 Cross Section Force

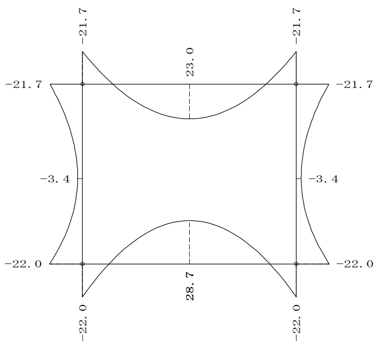
(1) Bending Moment (Case 1)



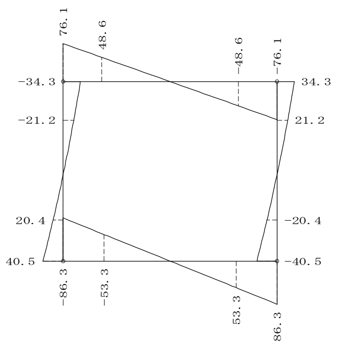
(2) Shear Force (Case 1)



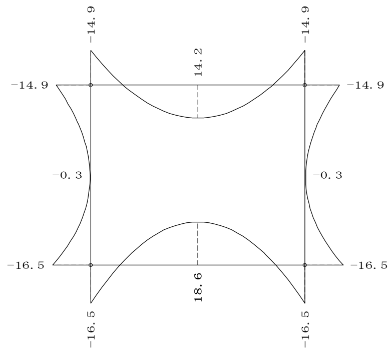
(3) Bending Moment (Case 2)



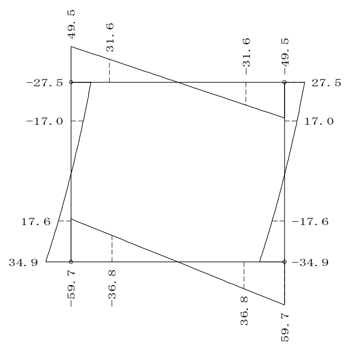
(4) Shear Force (Case2)



(5) Bending Moment (Case 3)



(6) Shear Force (Case3)



3.3 Stress Calculation

3.3.1 Intensity of Bending Stress

(1) Top Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-21.7	23.0	-21.7
Axial Force	N	kN	34.3	34.3	34.3
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	4.79	0.00	4.79
	Inside	cm^2	0.00	5.20	0.00
Reinforcement	Outside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
	Inside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
Neutral Axis	X	cm	6.502	6.434	6.502
Stress Intensity	σc	N/mm^2	3.18	3.38	3.18
	σs	N/mm^2	85.72	92.54	85.72
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	—	—	2	2	2

(2) Left Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-21.7	-3.4	-22.0
Axial Force	N	kN	76.1	83.4	89.6
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	2.89	0.00	2.38
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
	Inside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
Neutral Axis	X	cm	8.298	41.870	8.989
Stress Intensity	σc	N/mm^2	2.92	0.40	2.87
	σs	N/mm^2	56.08	-1.38	48.80
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	—	—	2	2	2

(3) Right Side Wall

Item		Unit	Upper Side Edge	Center	Underside Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-21.7	-3.4	-22.0
Axial Force	N	kN	76.1	83.4	89.6
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	35.0	35.0	35.0
Effective Width	d	cm	26.0	26.0	26.0
Thickness of Cover Concrete (Outside)	d1	cm	9.0	9.0	9.0
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	2.89	0.00	2.38
	Inside	cm^2	0.00	0.00	0.00
Reinforcement	Outside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
	Inside	cm^2	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525	D12 @250 D— @— 4.525
Neutral Axis	X	cm	8.298	41.870	8.989
Stress Intensity	σc	N/mm^2	2.92	0.40	2.87
	σs	N/mm^2	56.08	-1.38	48.80
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	-	140.00
Case	—	—	2	2	2

(4) Bottom Slab

Item		Unit	Left Edge	Center	Right Edge
			Outside Tensile	Inside Tensile	Outside Tensile
Bending Moment	M	kN.m	-22.0	28.7	-22.0
Axial Force	N	kN	40.5	40.5	40.5
Width of Member	b	cm	100.0	100.0	100.0
Height of Member	h	cm	40.0	40.0	40.0
Effective Width	d	cm	28.5	31.0	28.5
Thickness of Cover Concrete (Outside)	d1	cm	11.5	11.5	11.5
Thickness of Cover Concrete (Inside)	d2	cm	9.0	9.0	9.0
Required Area of Reinforcement	Outside	cm^2	3.91	0.00	3.91
	Inside	cm^2	0.00	5.28	0.00
Reinforcement	Outside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
	Inside	cm^2	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040	D16 @250 D— @— 8.040
Neutral Axis	X	cm	7.252	7.266	7.252
Stress Intensity	σc	N/mm^2	2.69	3.19	2.69
	σs	N/mm^2	70.99	93.86	70.99
Allowable Stress Intensity	σca	N/mm^2	8.20	8.20	8.20
	σsa	N/mm^2	140.00	140.00	140.00
Case	—	—	2	2	2

3.3.2 Intensity of Shear Stress

$$\tau m = \frac{S}{b \times d} \leqq \tau a$$

b = 100.0 (cm)

Member	Position	S (kN)	d (cm)	τm (N/mm^2)	τa (N/mm^2)	Case	L (m)
Top Slab	Left Side τ	48.6	26.0	0.187	0.360	2	0.425
	Right Side τ	-48.6	26.0	0.187	0.360	2	0.425
Left Side Wall	UpperSide	-21.2	26.0	0.082	0.360	2	0.425
	UnderSide	20.4	26.0	0.078	0.360	2	0.450
Right Side Wall	UpperSide	21.2	26.0	0.082	0.360	2	0.425
	UnderSide	-20.4	26.0	0.078	0.360	2	0.450
Bottom Slab	Left Side τ	-53.3	28.5	0.187	0.360	2	0.450
	Right Side τ	53.3	28.5	0.187	0.360	2	0.450

11.2 Box Culvert (Longitudinal Direction)

The detail of structural calculation of box culvert in longitudinal direction is indicated from the following page.

Structural Calculation of Box Culvert (Longitudinal Direction)

MSL-3

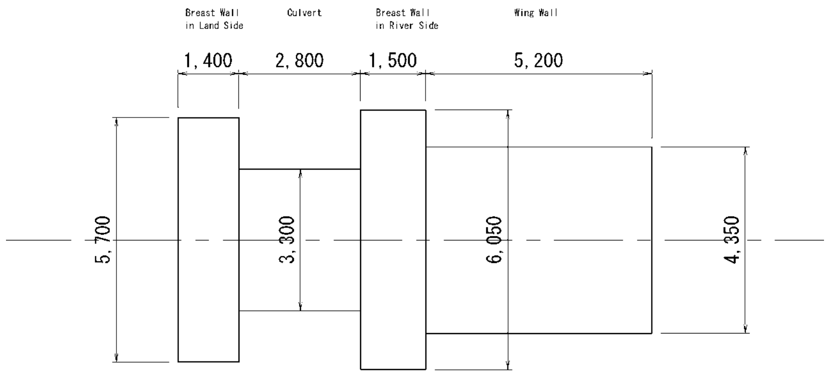
1 Desing Condition

(1) Load and Case

Case	Load Condition	Water Level		Load Condition							
		Inside	Outside (Only Breast wall and Wing Wall)	Direction of Seismic	Weight of Body	Live Load (Surcharge)	Load of Breast Wall (Incruding Earth Pressure and Water	Weight of Seepage Cut off Wall	Weight of Water	Weight of Soil (Embankment)	Uplift
1	Normal	Full	-	-	○	○	○	○	○	△	-
2-1	Seismic 1	-	-	→	○	○	◎	○	-	△	-
2-2	Seismic 2	-	-	←	○	○	◎	○	-	△	-

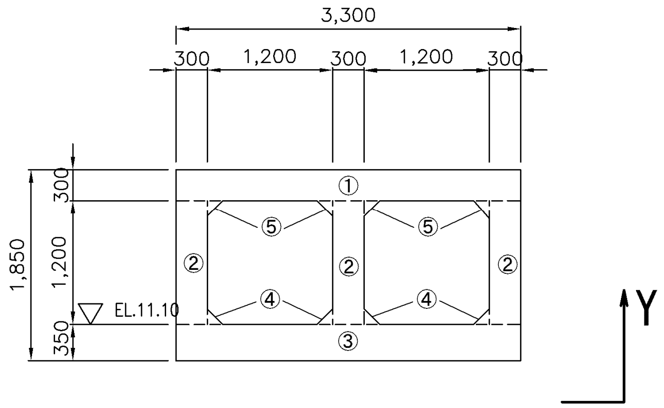
→ : Land Side to River Silde
← : River Side to Land Silde
○ : Considered
◎ : Seismic Force is Considered
△ : Considered as Residual Settlement and Lateral displacement
－ : Not Considered

(2) Properties of Each Section



		Wing Wall	Breast Wall in River Side	Culvert Section	Breast Wall in Land Side
Second Moment of Area	m^4	0.4402	5.8535	1.4196	4.0243
Area	m^2	2.950	8.028	3.315	6.350

Cross Section of Typical Cross Section of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	3.300	0.300	1.0	1	0.990	1.700	1.683
②	0.300	1.200	1.0	3	1.080	0.950	1.026
③	3.300	0.350	1.0	1	1.155	0.175	0.202
④	0.150	0.150	0.5	4	0.045	1.500	0.068
⑤	0.150	0.150	0.5	4	0.045	0.400	0.018
Total					3.315		2.997

α; Triangle: 0.5, Square: 1.0

n; Number of Element

$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 0.904 \quad m$

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix ; Second Moment of Area at Centroid (m^4)

ix ; Second Moment of Area Rrgarding Each Member (m^4)

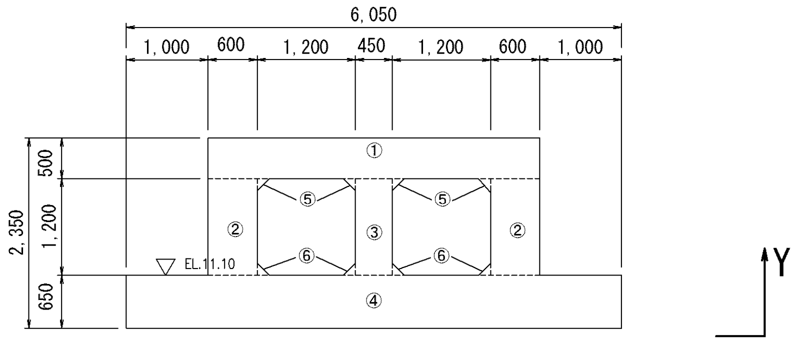
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.00743	0.990	-0.796	0.634	0.6277	0.6351
②	0.12960	1.080	-0.046	0.002	0.0022	0.1318
③	0.01179	1.155	0.729	0.531	0.6133	0.6251
④	0.00006	0.045	-0.596	0.355	0.0160	0.0161
⑤	0.00006	0.045	0.504	0.254	0.0114	0.0115
Total		3.315				1.4196

Area	**A=**	3.315	m^2
Second Moment	**Ix=**	1.4196	m^4

Breastwall in River Side



	Dimension				A(m^2)	Y(m)	A・Y(m^3)
	Width (B)	Height (h)	α	n			
①	4.050	0.500	1.0	1	2.025	2.150	4.354
②	0.600	1.200	1.0	2	1.440	1.300	1.872
③	0.450	1.200	1.0	1	0.540	1.300	0.702
④	6.050	0.650	1.0	1	3.933	0.350	1.377
⑤	0.150	0.150	0.5	4	0.045	1.850	0.083
⑥	0.150	0.150	0.5	4	0.045	0.750	0.034
Total					8.028		8.422

α; Triangle: 0.5, Square: 1.0
n; Number of Element

Y = ΣA・Y／ΣA = 1.049 m

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

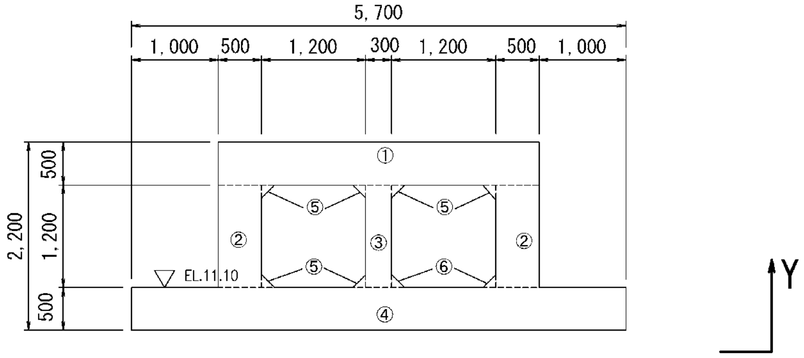
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.04219	2.025	-1.101	1.212	2.4543	2.4965
②	0.17280	1.440	-0.251	0.063	0.0907	0.2635
②	0.06480	0.540	-1.300	1.690	0.9126	0.9774
③	0.13846	3.933	0.699	0.489	1.9232	2.0617
④	0.00006	0.045	-0.801	0.642	0.0289	0.0290
④	0.00006	0.045	-0.750	0.563	0.0253	0.0254
Total		8.028				5.8535

Area	**A=**	8.028	m^2
Second Moment	**Ix=**	5.8535	m^4

Breast wall in Land Side



	Dimension				A(m^2)	Y(m)	A・Y(m^3)
	Width (B)	Height (h)	α	n			
①	3.700	0.500	1.0	1	1.850	1.950	3.608
②	0.500	1.200	1.0	2	1.200	1.100	1.320
③	0.300	1.200	1.0	1	0.360	1.100	0.396
④	5.700	0.500	1.0	1	2.850	0.250	0.713
⑤	0.150	0.150	0.5	4	0.045	1.650	0.074
⑥	0.150	0.150	0.5	4	0.045	0.550	0.025
計					6.350		6.136

α; Triangle: 0.5, Square: 1.0
n; Number of Element

Y＝ ΣA・Y／ΣA ＝ 0.966 m

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix ; Second Moment of Area at Centroid (m^4)

ix ; Second Moment of Area Rrgarding Each Member (m^4)

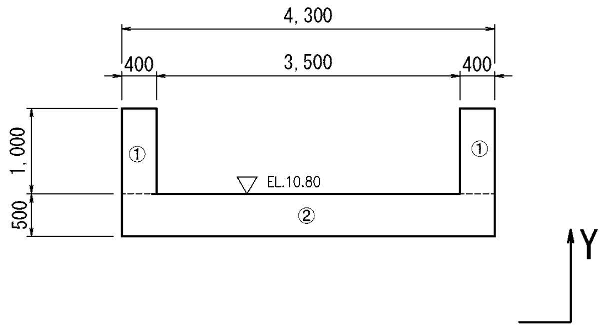
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.03854	1.850	-0.984	0.968	1.7908	1.8293
②	0.14400	1.200	-0.134	0.018	0.0216	0.1656
③	0.04320	0.360	-1.100	1.210	0.4356	0.4788
④	0.05938	2.850	0.716	0.513	1.4621	1.5215
⑤	0.00006	0.045	-0.684	0.468	0.0211	0.0212
⑥	0.00006	0.045	0.416	0.173	0.0078	0.0079
Total		6.350				4.0243

Area	**A=**	6.350	m^2
Second Moment	**Ix=**	4.0243	m^4

Wing Wall



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	0.400	1.000	1.0	2	0.800	1.000	0.800
②	4.300	0.500	1.0	1	2.150	0.250	0.538
計					2.950		1.338

α;Triangle: 0.5, Square: 1.0

n; Number of Element

$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 0.454 \quad m$

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix ; Second Moment of Area at Centroid (m^4)

ix ; Second Moment of Area Rrgarding Each Member (m^4)

A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.06667	0.800	−0.546	0.298	0.2384	0.3051
②	0.04479	2.150	0.204	0.042	0.0903	0.1351
Total		2.950				0.4402

Area	**A=**	2.950	m^2
Second Moment	**Ix=**	0.4402	m^4

(3) Calculation of Load

1. Weight of Mail Body

(a) Wing Wall

A= 2.950 m^2

Vertical Force

Wc1= 2.950 × 24.0 kN/m^3 = 70.80 kN/m

(b) Sliceway (Breast Wall in River Side)

A= 8.028 8.028 m^2

Vertical Force

Wc2= 8.028 × 24.0 kN/m^3 = 192.67 kN/m

(c) Sliceway (Box Culvert)

A= 3.315 m^2

Vertical Force

Wc3= 3.315 × 24.0 kN/m^3 = 79.56 kN/m

(d) Sliceway (Breast Wall in Land Side)

A= 6.350 6.350 m^2

Vertical Force

Wc4= 6.350 × 24.0 kN/m^3 = 152.40 kN/m

(e) Weight of Seepage Cut off Wall

Vo= (5.3×2.85-3.3×1.85)×0.50 = 4.50 m^3

V= 4.50 × 24.0 kN/m^3 = 108.00 kN

2. Water Weight

Only Normal Condition

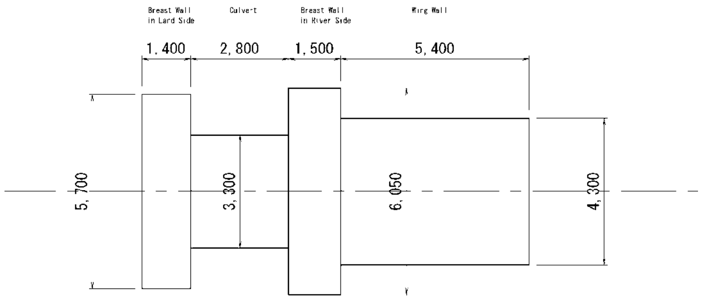
(a) Wing Wall

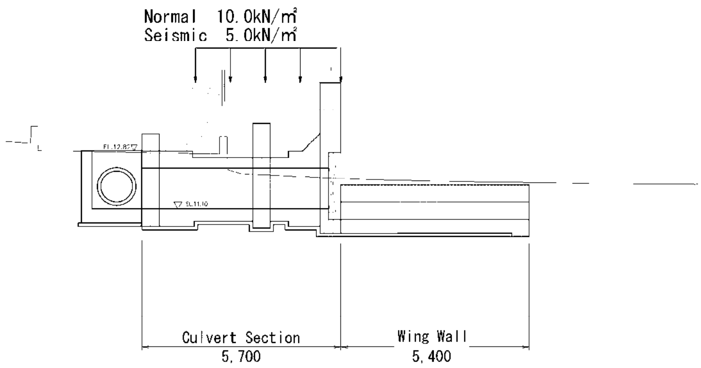
Wo1= $1.0 \times 3.55 \times 9.8$ = 34.79 kN/m

(b) Sliceway Main Body

Wo2= $2 \times (1.2 \times 1.2 - 1/2 \times 0.15 \times 0.15 \times 4) \times 9.8$ = 27.34 kN/m

3. Vertical Load





Nomal Condition

q1=	10.0×3.3	=	33.00	kN/m
q2=	10.0×6.05	=	60.50	kN/m

Seismic Condition

q1=	5.0×3.3	=	16.50	kN/m
q2=	5.0×6.05	=	30.25	kN/m

4. Calculation of the Force Effecting on Breast Wall

(a) River Side (River ← Land, +)

Nomal Condition

V=	127.12	×	1.00	×	2	=	254.24	kN
H=	133.45	×	1.00	×	2	=	266.90	kN
M=	-12.02	×	1.00	×	2	=	-24.04	kNm
x=	0.690	m						

Seismic Condition, from Land side to River side

V=	154.05	×	1.00	×	2	=	308.10	kN
H=	117.58	×	1.00	×	2	=	235.16	kN
M=	-33.59	×	1.00	×	2	=	-67.18	kNm
x=	0.810	m						

Seismic Condition, from River side to Land side

V=	154.05	×	1.00	×	2	=	308.10	kN
H=	62.42	×	1.00	×	2	=	124.84	kN
M=	-100.51	×	1.00	×	2	=	-201.02	kNm
x=	0.810	m						

(b) Land Side (River → Land, +)

Nomal Condition

V=	67.06	×	1.00	×	2	=	134.12	kN
H=	93.76	×	1.00	×	2	=	187.52	kN
M=	-23.56	×	1.00	×	2	=	-47.12	kNm
x=	0.620	m						

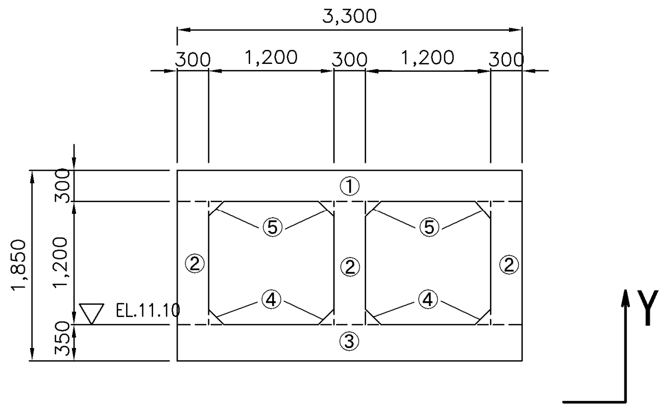
Seismic Condition, from Land side to River side

V=	88.28	×	1.00	×	2	=	176.56	kN
H=	44.23	×	1.00	×	2	=	88.46	kN
M=	-54.14	×	1.00	×	2	=	-108.28	kNm
x=	0.780	m						

Seismic Condition, from River side to Land side

V=	88.28	×	1.00	×	2	=	176.56	kN
H=	76.37	×	1.00	×	2	=	152.74	kN
M=	-41.12	×	1.00	×	2	=	-82.24	kNm
x=	0.780	m						

5. Centroid of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	3.300	0.300	1.0	1	0.990	1.700	1.683
②	0.300	1.200	1.0	3	1.080	0.950	1.026
③	3.300	0.350	1.0	1	1.155	0.175	0.202
④	0.150	0.150	0.5	4	0.045	1.500	0.068
⑤	0.150	0.150	0.5	4	0.045	0.400	0.018
Total					3.315		2.997

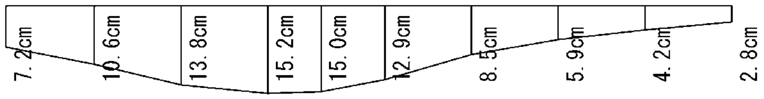
α; Triangle: 0.5, Square: 1.0

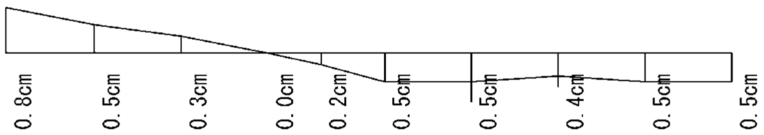
n; Number of Element

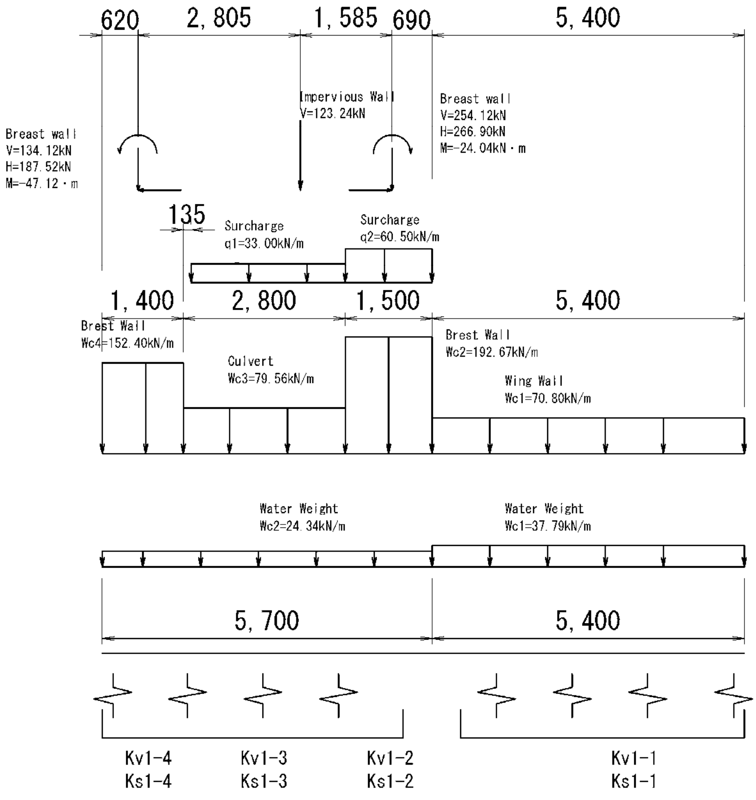
$Y = \Sigma A \cdot Y / \Sigma A$ = 0.904 m

6 Summary of Load

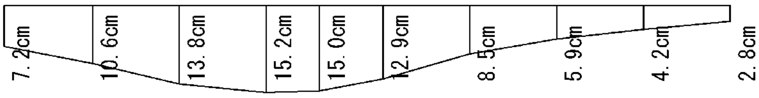
a) Normal Condition

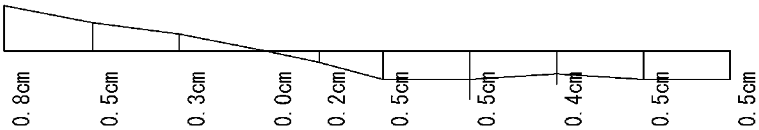


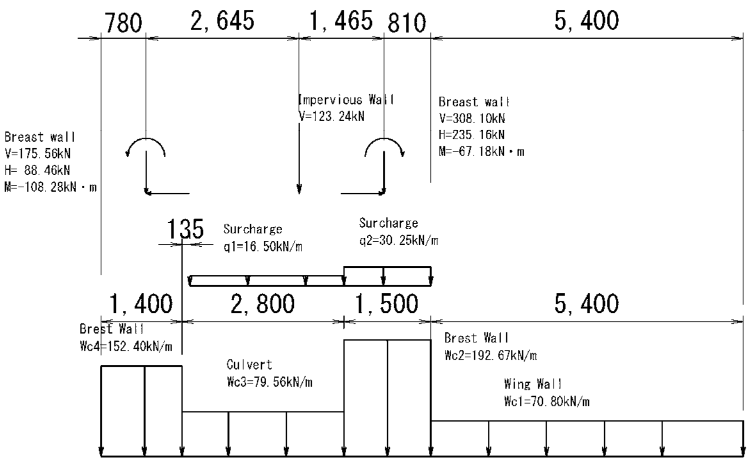


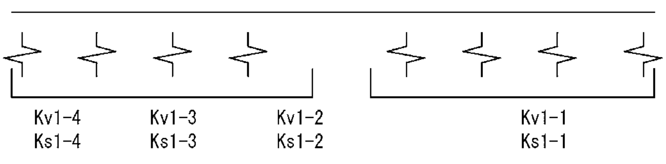


b) Seismic Condition (Land Side → River Side)

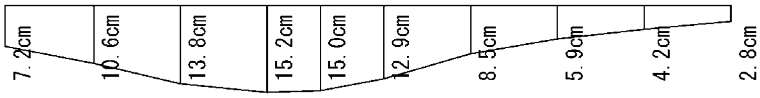


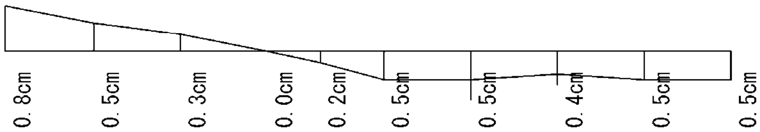


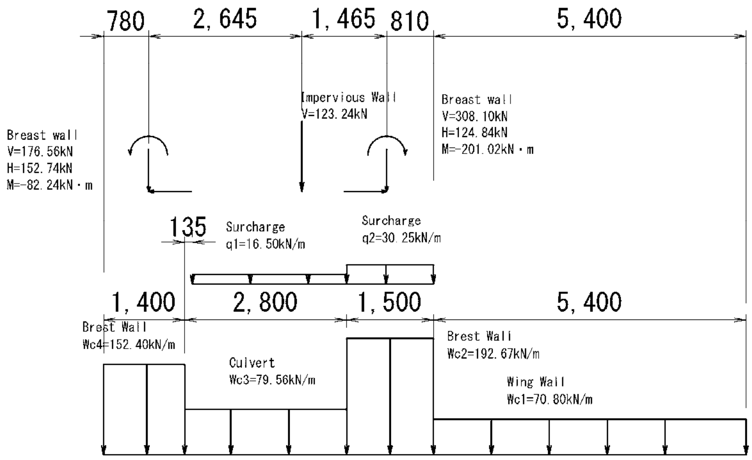


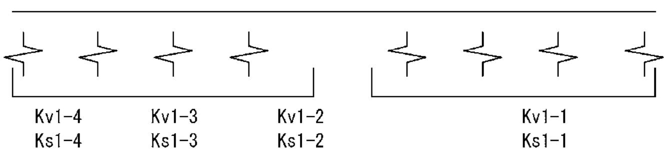


c) Seismic Condition (Land Side ← River Side)

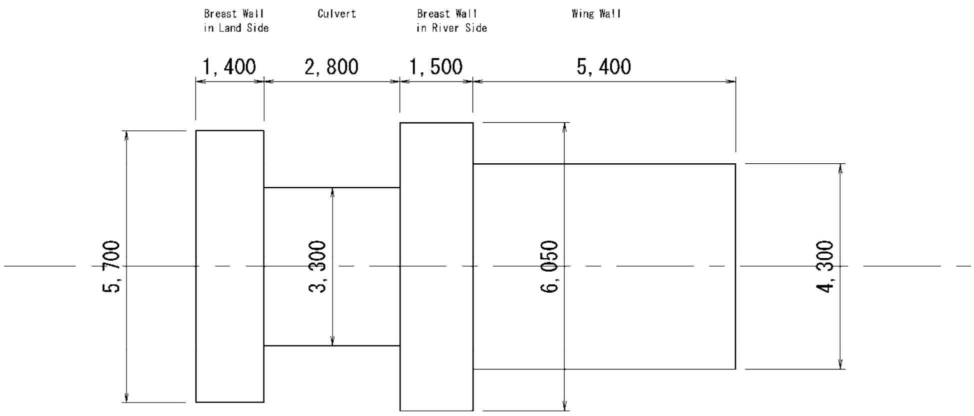








(4) Modulus of Subgrade Reaction and Modulus of Subgrade Reaction for Horizontal Shea



Results of Calculation

		Wing Walll	Breast Wall in River side	Culvert Section	Breast Wall in Lnd side
		1	2	3	4
Vertical Kv (kN/m^2)	Nomal	24398	48642	24347	47458
	Seismic	48796	97284	48695	94916
Horizontal Shear Ks(kN/m^2)	Nomal	8133	16214	8116	15819
	Seismic	16265	32428	16232	31639

a. Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo\left(\frac{Bv}{0.3}\right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvc : Coefficient of Vertical Reaction of soil (kN/m^3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E_{0m} obtained by various soil tests and investigations:

$$Kvo = \frac{1}{0.3} \quad \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eon : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a) Wing Wall

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 0.4402 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3414.68 = 45529.1 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.15421

β・L= 0.833 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv=\sqrt{L \cdot D} = \sqrt{5.40 \times 4.30} = \boxed{4.819}$ m

Case2 Elastic body $Bv=\sqrt{D/\beta} = \sqrt{4.30 / 0.15421} = 5.281$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 45529.1 \times (4.819 / 0.3)^{-3/4} = 5674$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 5674 × 4.30 = 24398 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 11348 (kN/m^3) 〔= 2 × 5674 (kN/m^3)〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 11348 × 4.30 = 48796 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 24398

= 8133 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 48796

= 16265 (kN/m^2)

(Wing wall)
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L		(kN/m^2)
B ;	Loading Width = Width of Wing Wall	B=	4.300 m
L ;	Loading Length = Length of a span	L=	5.400 m
h_n ;	Depth for Calculation	hn=	12.900 m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)		
Ei ;	onverted Modulus of Deformation of Each Layer (kN/m^2)		
θ ;	Angle of Load Dispersion (θ=30°)		

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3414.68	4.300	5.400	12.900	4.070	4.070	2830	30	0.1721	0.112	0.0000396
2					2.000	6.070	14700			0.022	0.0000015
3					3.500	9.570	2840			0.024	0.0000085
4					3.330	12.900	16800			0.013	0.0000008
5											
6											
7											
8											
9											
10											
				Total	12.900					Total	0.0000504

b) Breast wall in River Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 5.8535 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3401.07 = 45347.6 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.09596

β・L= 0.144 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv=\sqrt{L \cdot D} = \sqrt{1.50 \times 6.05} = \boxed{3.012}$ m

Case2 Elastic body $Bv=\sqrt{D/\beta} = \sqrt{6.05 / 0.09596} = 7.940$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 45347.6 \times (3.012 / 0.3)^{-3/4} = 8040$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 8040 × 6.05 = 48642 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 16080 (kN/m^3) 〔 2倍 × 8040 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 16080 × 6.05 = 97284 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 48642

= 16214 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 97284

= 32428 (kN/m^2)

(Breast wall in River Side)
Calculation Results of Converted Modulus of Deformation

$$Eom=\frac{\log\dfrac{(B+2h_n\cdot\tan\theta)\,L}{(L+2h_n\cdot\tan\theta)\,B}}{\sum_{i=1}^{n}\dfrac{1}{Ei}\log\dfrac{(B+2h_i\cdot\tan\theta)\,(L+2\,h_{i-1}\cdot\tan\theta)}{(L+2h_i\cdot\tan\theta)\,(B+2\,h_{i-1}\cdot\tan\theta)}}=\frac{f\,n}{\sum_{i=1}^{n}\dfrac{1}{Ei}\times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L			(kN/m^2)
B ;	Loading Width = Width of Culvert	B=	6.050	m
L ;	Loading Length = Length of a span	L=	1.500	m
h_n ;	Depth for Calculation	hn=	18.150	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each I (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3401.07	6.050	1.500	18.150	4.070	4.070	2830	30	-1.2101	-0.844	-0.0002982
2					2.000	6.070	14700			-0.122	-0.0000083
3					3.500	9.570	2840			-0.119	-0.0000419
4					8.580	18.150	16800			-0.125	-0.0000074
5											
6											
7											
8											
				Total	18.150					Total	-0.0003558

c) Culvert Section

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert (kN・m^2)

E= 2.45E+07 kN/m^2

I= 1.4196 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3142.86 = 41904.8 kN/m^3

β= ｛（Kv・D）/(4・E・I)｝ $^{1/4}$

= 0.11502

β・L= 0.322 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body Bv= $\sqrt{L \cdot D}$ = $\sqrt{2.80 \times 3.30}$ = $\boxed{3.040}$ m

Case2 Elastic body Bv= $\sqrt{D/\beta}$ = $\sqrt{3.30 / 0.11502}$ = 5.356 m

Kv = Kvo × $(Bv/0.3)^{-3/4}$

= 41904.8 × (3.040 / 0.3 $)^{-3/4}$ = 7378 (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 7378 × 3.30 = 24347 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 14756 (kN/m^3) 〔 2倍 × 7378 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 14756 × 3.30 = 48695 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 24347

= 8116 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 48695

= 16232 (kN/m^2)

(Culvert Section)
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\,L}{(L+2h_n \cdot \tan\theta)\,B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom	;	Converted Modulus of Deformation in B≠L		(kN/m^2)
B	;	Loading Width = Width of Culvert	B=	3.300 m
L	;	Loading Length = Length of a span	L=	2.800 m
h_n	;	Depth for Calculation	hn=	9.900 m
h_i	;	Depth from the ground to the bottom Elevation of Each Layer (m)		
Ei	;	onverted Modulus of Deformation of Each I (kN/m^2)		
θ	;	Angle of Load Dispersion (θ=30°)		

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3142.86	3.300	2.800	9.900	4.070	4.070	2830	30	-0.1298	-0.100	-0.0000353
2					2.000	6.070	14700			-0.015	-0.0000010
3					3.500	9.570	2840			-0.014	-0.0000049
4					0.330	9.900	16800			-0.001	-0.0000001
5											
6											
7											
8											
				Total	9.900					Total	-0.0000413

d) Breast Wall in Land Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert (kN・m^2)

E= 2.45E+07 kN/m^2

I= 4.0243 m^4

◇ Nomal Condition

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3356.93 = 44759.1 kN/m^3

β= ｛（Kv・D）/(4・E・I)｝ $^{1/4}$

= 0.10474

β・L= 0.147 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body Bv=$\sqrt{L \cdot D}$ = $\sqrt{1.40 \times 5.70}$ = $\boxed{2.825}$ m

Case2 Elastic body Bv=$\sqrt{D/\beta}$ = $\sqrt{5.70 / 0.10474}$ = 7.377 m

Kv = Kvo × $(Bv/0.3)^{-3/4}$

= 44759.1 × (2.825 / 0.3 $)^{-3/4}$ = 8326 (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 8326 × 5.70 = 47458 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 16652 (kN/m^3) 〔 2倍 × 8326 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 16652 × 5.70 = 94916 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 47458

= 15819 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 94916

= 31639 (kN/m^2)

(Breast Wall in Land Side)
Calculation Results of Converted Modulus of Deformation

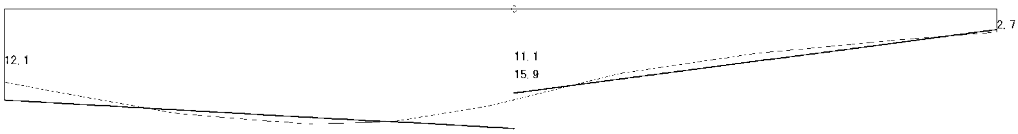
$$Eom=\frac{\log \dfrac{(B+2h_n\cdot\tan\theta)\ L}{(L+2h_n\cdot\tan\theta)\ B}}{\sum_{i=1}^{n}\dfrac{1}{Ei}\log\dfrac{(B+2h_i\cdot\tan\theta)\ (L+2\ h_{i-1}\cdot\tan\theta)}{(L+2h_i\cdot\tan\theta)\ (B+2\ h_{i-1}\cdot\tan\theta)}}=\frac{f\ n}{\sum_{i=1}^{n}\dfrac{1}{Ei}\times f\ i}$$

Eom ;	Converted Modulus of Deformation in B≠L		（kN/m^2）	
B ;	Loading Width = Width of Culvert	B=	5.700	m
L ;	Loading Length = Length of a span	L=	1.400	m
h_n ;	Depth for Calculation	hn=	17.100	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer（m）			
Ei ;	onverted Modulus of Deformation of Each La（kN/m^2）			
θ ;	Angle of Load Dispersion（θ=30°）			

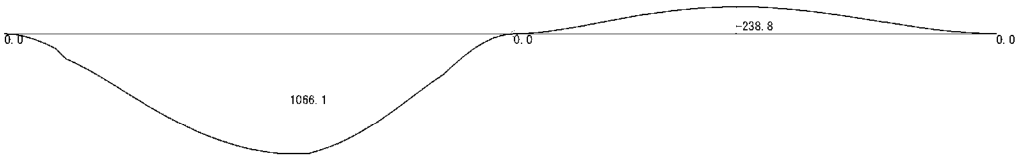
	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3356.93	5.700	1.400	17.100	4.070	4.070	2830	30	-1.2189	-0.870	-0.0003074
2					2.000	6.070	14700			-0.121	-0.0000082
3					3.500	9.570	2840			-0.116	-0.0000408
4					7.530	17.100	16800			-0.112	-0.0000067
5											
6											
7											
8											
				Total	17.100					Total	-0.0003631

2 Cross Section Force

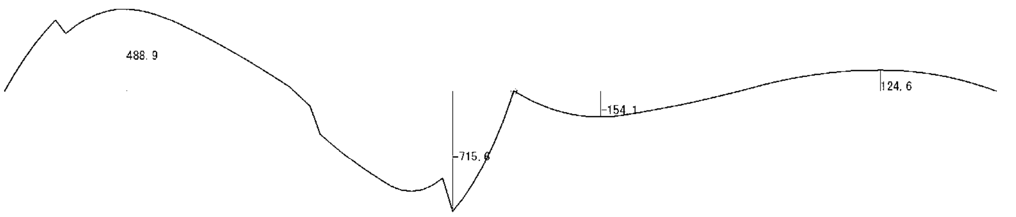
(1) Case 1 Normal Condition



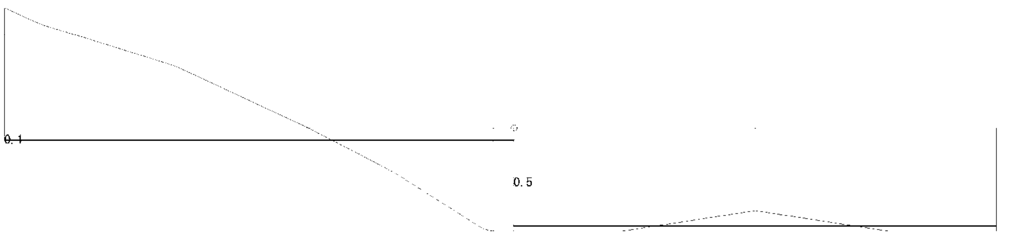
Settlement of Culvert and Ground (cm)



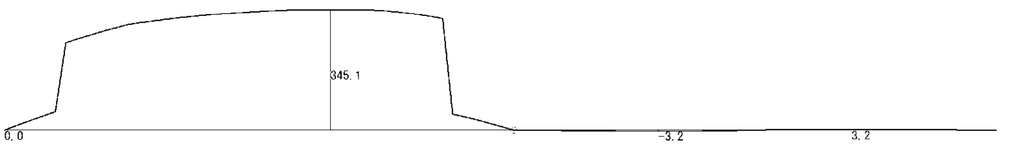
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

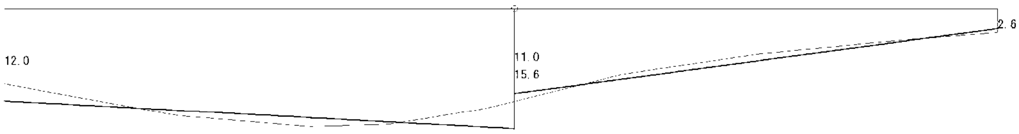


Axial Force (kN)

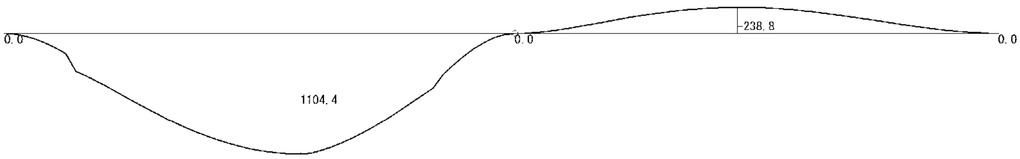
1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	0.1	9.7	12.1	0.00	0.00	0.00	1129.72
	1.140	-0.4	0.1	12.2	12.8	284.99	476.37	413.47	311.22
	2.280	-0.2	0.1	14.2	13.6	328.97	312.35	907.59	-142.40
B-M max	3.192	0.0	0.1	15.1	14.2	343.13	23.93	1066.06	-216.29
	3.420	0.0	0.1	15.3	14.4	344.55	-91.17	1058.59	-228.27
F-F max	3.648	0.1	0.1	15.2	14.5	345.09	-309.80	1000.87	-172.72
	4.560	0.3	0.1	14.5	15.1	332.91	-597.80	560.54	298.08
S-F max	5.016	0.4	0.1	13.6	15.4	45.71	-715.56	283.57	869.53
	5.700	0.5	0.1	12.0	15.9	0.00	0.03	0.17	1876.16

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.5	0.5	12.0	11.1	0.00	0.00	0.00	-212.88
S-F max	0.972	0.5	0.5	9.2	9.6	-2.20	-154.13	-100.56	112.92
	1.080	0.5	0.5	8.9	9.5	-2.44	-151.71	-117.11	149.12
	2.160	0.4	0.5	6.8	7.8	-2.38	-47.37	-230.11	230.05
B-M max	2.484	0.4	0.5	6.3	7.3	-1.14	-5.79	-238.85	243.79
	3.240	0.4	0.5	5.3	6.1	2.38	87.19	-205.38	196.79
F-F max	3.780	0.5	0.5	4.7	5.3	3.18	119.66	-148.17	140.64
	4.320	0.5	0.5	4.1	4.4	2.44	121.45	-81.62	78.63
	5.400	0.5	0.5	3.1	2.7	0.00	0.00	0.05	-86.37

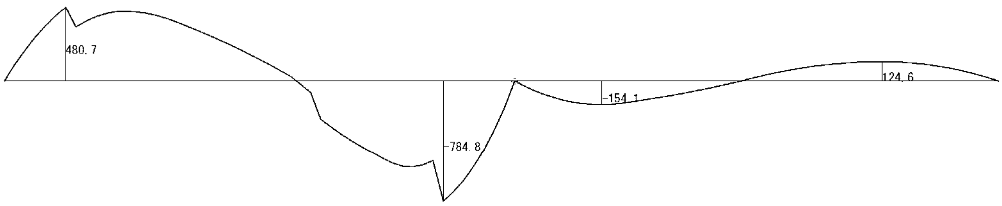
(2) Case 2 Seismic Condition (Land Side to River Side)



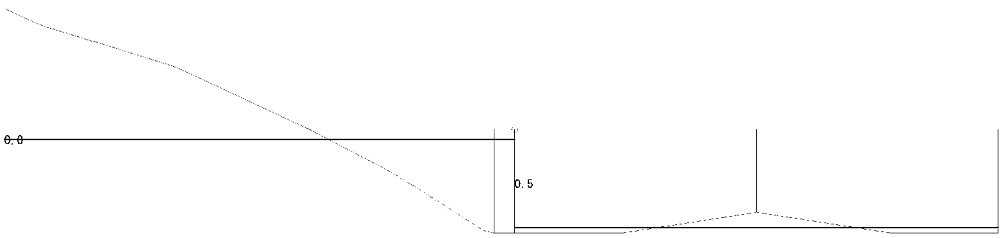
Settlement of Culvert and Ground (cm)



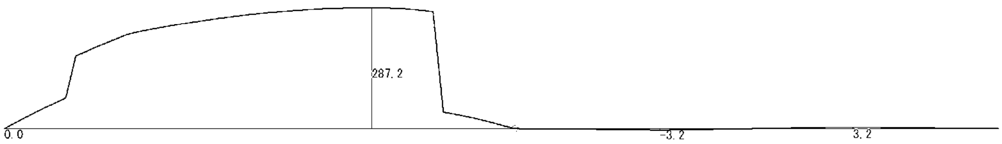
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

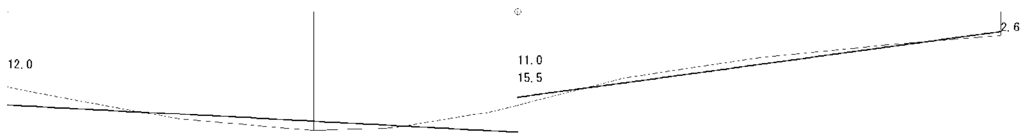


Axial Force (kN)

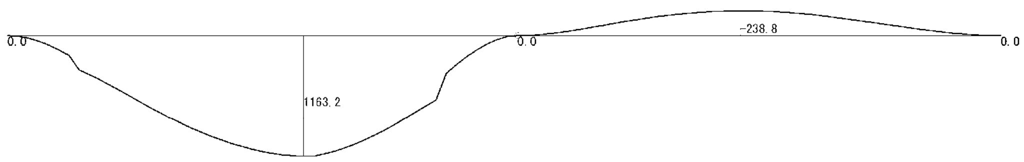
1-SPAN	X (m)	GRD-U (c m)	DIS-U (c m)	GRD-W (c m)	DIS-W (c m)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	0.0	9.7	12.0	0.00	0.00	0.00	1112.27
	1.140	-0.4	0.0	12.2	12.7	203.48	440.44	484.76	289.67
	2.280	-0.2	0.0	14.2	13.4	258.41	298.63	947.47	-157.82
	3.420	0.0	0.0	15.3	14.2	282.99	-74.72	1099.00	-246.84
	4.560	0.3	0.0	14.5	14.9	283.19	-560.76	624.96	263.10
	5.700	0.5	0.0	12.0	15.6	0.00	0.03	0.16	1831.26

2-SPAN	X (m)	GRD-U (c m)	DIS-U (c m)	GRD-W (c m)	DIS-W (c m)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.5	0.5	12.0	11.0	0.00	0.00	0.00	-250.67
	1.080	0.5	0.5	8.9	9.3	-2.44	-151.71	-117.11	111.33
	2.160	0.4	0.5	6.8	7.6	-2.38	-47.37	-230.11	192.26
	3.240	0.4	0.5	5.3	5.9	2.38	87.19	-205.38	159.00
	4.320	0.5	0.5	4.1	4.3	2.44	121.45	-81.62	40.84
	5.400	0.5	0.5	3.1	2.6	0.00	0.00	0.05	-124.16

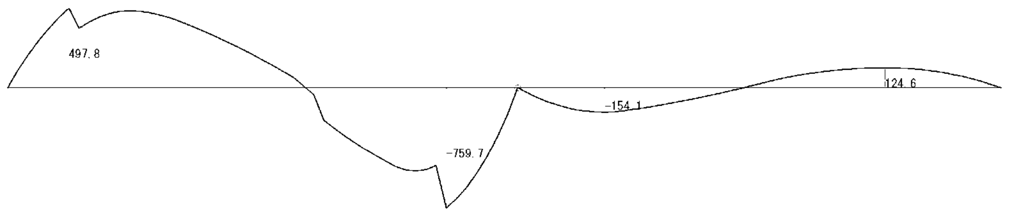
(3) Case 2-2 Seismic Condition (River Side to Land Side)



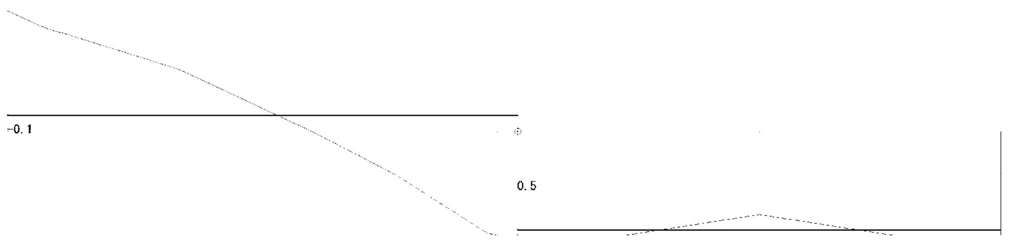
Settlement of Culvert and Ground (m)



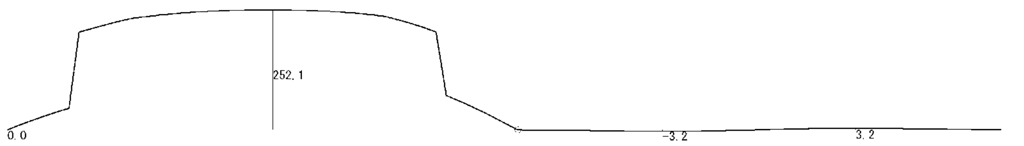
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)



Axial Force (kN)

1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	-0.1	9.7	12.0	0.00	0.00	0.00	1141.15
	1.140	-0.4	-0.1	12.2	12.7	222.25	466.07	474.71	305.76
	2.280	-0.2	-0.1	14.2	13.4	248.78	331.78	972.47	-156.13
	3.420	0.0	-0.1	15.3	14.1	250.02	-43.39	1161.45	-251.72
	4.560	0.3	-0.1	14.5	14.9	219.51	-527.23	735.19	240.24
	5.700	0.5	-0.1	12.0	15.6	0.00	0.03	0.16	1795.29

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.5	0.5	12.0	11.0	0.00	0.00	0.00	-250.67
	1.080	0.5	0.5	8.9	9.3	-2.44	-151.71	-117.11	111.33
	2.160	0.4	0.5	6.8	7.6	-2.38	-47.37	-230.11	192.26
	3.240	0.4	0.5	5.3	5.9	2.38	87.19	-205.38	159.00
	4.320	0.5	0.5	4.1	4.3	2.44	121.45	-81.62	40.84
	5.400	0.5	0.5	3.1	2.6	0.00	0.00	0.05	-124.16

3 Stress Calculation

		Case1 Normal Condition		Seismic Condition (L to R)		Case2-2 Seismic Condition (R to L)	
		Mmax	Smax	Mmax	Smax	Mmax	Smax
MSL3	Tensile Side	Bottom	-	Bottom	-	Bottom	-
	M (kNm)	1066.1	-	1104.4	-	1163.2	-
	S (kN)	23.9	715.56	74.7	784.8	43.4	921.2
	N (kN)	343.1	-	283.0	-	250.0	-
	Re-Bar	D16 x 28	-	D16 x 28	-	D16 x 28	-
	Area (m^2)	56.3	-	56.3	-	56.3	-
	σ_c (N/mm^2)	2.05	-	2.12	-	2.22	-
	σ_s (N/mm^2)	96.9	-	104.8	-	113.3	-
	τ (N/mm^2)	0.01	0.04	0.03	0.05	0.02	0.06
	σ_{ca} (N/mm^2)	8.2	8.2	12.3	12.3	12.3	12.3
	σ_{sa} (N/mm^2)	140.0	140.0	210.0	210.0	210.0	210.0
	τa (N/mm^2)	0.36	0.36	0.54	0.54	0.54	0.54
	Evaluation	OK	OK	OK	OK	OK	OK

L : Land Side
R : River Side

(1) Case1 Normal Condition (Mmax)

cgs		SI	
M (tfxm)	=	1066.06	kNm
S (tf)	=	23.93	kN
N (tf)	=	343.13	kN
b (cm)	=	330	cm
H (cm)	=	185	cm
t (cm)	=	30	cm
bw (cm)	=	90	cm
d (cm)	=	166	cm
d' (cm)	=	0	cm
n (Es/Ec)	=	9	
σca	=	8.20	N/mm^2
σsa	=	140	N/mm^2
τca	=	0.36	N/mm^2

$$y_1 = \frac{\frac{b \cdot t^2}{2} + \frac{b_w}{2} \cdot (h-t) \cdot (h+t)}{b \cdot t + b_w \cdot (h-t)}$$

$\frac{b \cdot t^2}{2}$	148500	cm^3
$\frac{b_w}{2} \cdot (h-t) \cdot (h+t)$	1499625	cm^3
$b \cdot t + b_w \cdot (h-t)$	23850	cm^2

$$x^3 + 3 \cdot e' \cdot x^2 + \frac{3}{b_w} \cdot [t \cdot (b-b_w) \cdot (t + 2 \cdot e') + 2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}] \cdot x$$
$$- \frac{3 \cdot (b-b_w) \cdot t^2}{b_w} \cdot \left(\frac{2}{3} \cdot t + e'\right) - \frac{6 \cdot n}{b_w}[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s'] = 0$$

$\frac{3}{b_w}$	0.03	cm
$t \cdot (b-b_w) \cdot (t + 2 \cdot e')$	3694797.19	
$2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}$	413103.44	
$-\frac{3 \cdot (b-b_w) \cdot t^2}{b_w}$	-7200	
$\left(\frac{2}{3} \cdot t + e'\right)$	261.58	
$-\frac{6 \cdot n}{b_w}$	-0.6	
$[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s']$	3809731.76	

	dia.	piece	cm2
Re-BarAs	16	28	56.308
As'	12	0	0

e0 = 3 m
y1 = 69 cm
e' = 242 cm

x^3+ 725 x^2 + 136930 x + -4169238 = y

x = 26.57 cm
y = 0.00 OK

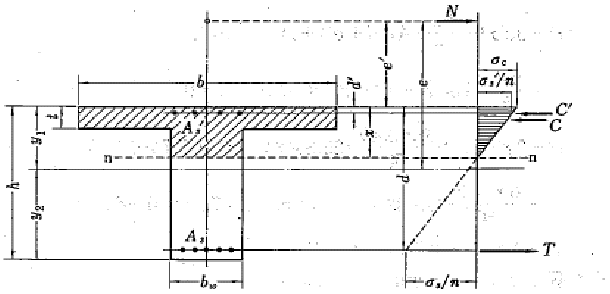
$$\sigma_c = \frac{N \cdot x}{\frac{b \cdot x^2}{2} - \frac{(b-b_w)}{2} \cdot (x-t)^2 + n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

$$\sigma_s' = n \cdot \sigma_c \cdot \frac{x-d'}{x}$$

$\frac{b \cdot x^2}{2} - \frac{(b-b_w)}{2} \cdot (x-t)^2$	115104.93
$n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}$	-70657.50

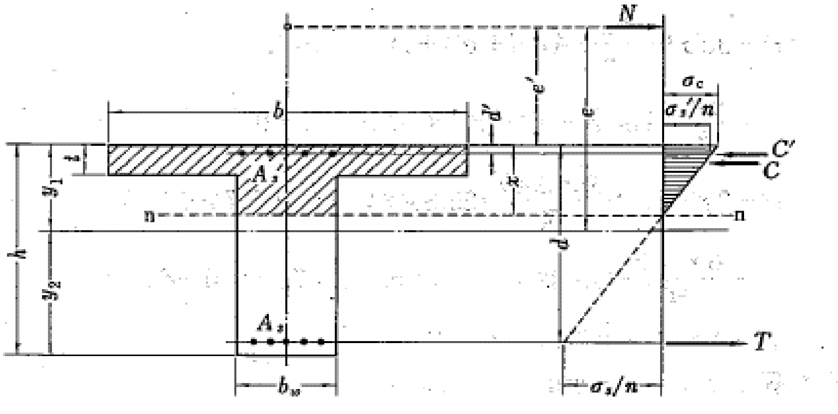
σc =	2.05	N/mm^2	OK
σs =	97	N/mm^2	OK
τc =	0.01	N/mm^2	OK



(2) Case1 Normal Condition (Smax)

	cgs		SI		
	M	=	-	kNm	
	S	=	715.56	kN	
	N	=	-	kN	
	b	=	405	cm	(Breast Wall Riverside Section)
	H	=	240	cm	
	t	=	500	cm	
	bw	=	165	cm	
	d	=	203	cm	
	d'	=	0	cm	
	n (Es/Ec)	=	9		
	σca	=	8.20	N/mm^2	
	σsa	=	140	N/mm^2	
	τca	=	0.36	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.04	N/mm^2	OK



(3) Case2-1 Seismic Condition (Mmax)

	cgs		SI	
	M	=	1104.4	kNm
	S	=	74.72	kN
	N	=	282.99	kN
	b	=	330	cm
	H	=	185	cm
	t	=	30	cm
	bw	=	90	cm
	d	=	166	cm
	d'	=	0	cm
	n (Es/Ec)	=	9	
	σca	=	12.30	N/mm²
	σsa	=	210.00	N/mm²
	τca	=	0.54	N/mm²

	dia.	piece	cm2
Re-BarAs	16	28	56.308
As'	12	0	0

e0 = 4 m
y1 = 69 cm
e' = 321 cm

$x^3 + 963\, x^2 + 177814\, x + -5188446 = y$

x = 25.55 cm
y = 0.00 OK

$$y_1 = \frac{\dfrac{b \cdot t^2}{2} + \dfrac{b_w}{2} \cdot (h-t) \cdot (h+t)}{b \cdot t + b_w \cdot (h-t)}$$

$\dfrac{b \cdot t^2}{2}$	148500	cm³
$\dfrac{b_w}{2} \cdot (h-t) \cdot (h+t)$	1499625	cm³
$b \cdot t + b_w \cdot (h-t)$	23850	cm²

$$x^3 + 3 \cdot e' \cdot x^2 + \frac{3}{b_w} \cdot \left[t \cdot (b-b_w) \cdot (t + 2 \cdot e') + 2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}\right] \cdot x$$
$$- \frac{3 \cdot (b-b_w) \cdot t^2}{b_w} \cdot \left(\frac{2}{3} \cdot t + e'\right) - \frac{6 \cdot n}{b_w}\left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s'\right] = 0$$

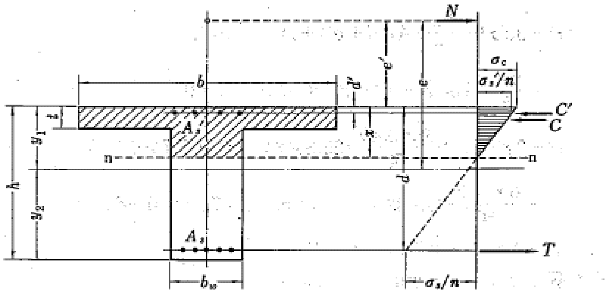
$\dfrac{3}{b_w}$	0.03	cm
$t \cdot (b-b_w) \cdot (t + 2 \cdot e')$	4840666	
$2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}$	493755	
$-\dfrac{3 \cdot (b-b_w) \cdot t^2}{b_w}$	-7200	
$\left(\dfrac{2}{3} \cdot t + e'\right)$	341	
$-\dfrac{6 \cdot n}{b_w}$	-0.6	
$\left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s'\right]$	4553522	
$\dfrac{b \cdot x^2}{2} - \dfrac{(b-b_w)}{2} \cdot (x-t)^2$	105322	
$n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}$	-71177	

$$\sigma_c = \frac{N \cdot x}{\dfrac{b \cdot x^2}{2} - \dfrac{(b-b_w)}{2} \cdot (x-t)^2 + n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

$$\sigma_s' = n \cdot \sigma_c \cdot \frac{x-d'}{x}$$

σc =	2.12	N/mm²	OK
σs =	105	N/mm²	OK
τc =	0.03	N/mm²	OK



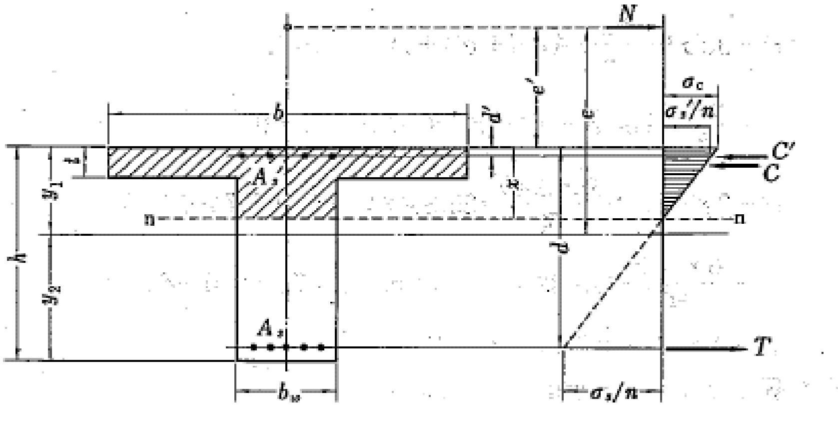
(4) Case2-1 Seismic Condition (Smax)

cgs		SI		
M	=	-	kNm	
S	=	784.8	kN	
N	=	-	kN	
b	=	405	cm	(Breast Wall Riverside Section)
H	=	240	cm	
t	=	500	cm	
bw	=	165	cm	
d	=	203	cm	
d'	=	0	cm	
n (Es/Ec)	=	9		
σca	=	12.30	N/mm^2	
σsa	=	210.00	N/mm^2	
τca	=	0.54	N/mm^2	

σc = - N/mm^2

σs = - N/mm^2

τc = 0.05 N/mm^2 OK



(5) Case2-2 Seismic Condition (Mmax)

cgs		SI	
M	=	1163.2	kNm
S	=	43.39	kN
N	=	250.02	kN
b	=	330	cm
H	=	185	cm
t	=	30	cm
bw	=	90	cm
d	=	166	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	12.30	N/mm^2
σsa	=	210.00	N/mm^2
τca	=	0.54	N/mm^2

$$y_1 = \frac{\frac{b\cdot t^2}{2} + \frac{b_w}{2}\cdot(h-t)\cdot(h+t)}{b\cdot t + b_w\cdot(h-t)}$$

$\frac{b\cdot t^2}{2}$	148500	cm^3
$\frac{b_w}{2}\cdot(h-t)\cdot(h+t)$	1499625	cm^3
$b\cdot t + b_w\cdot(h-t)$	23850	cm^2

$$x^3 + 3\cdot e'\cdot x^2 + \frac{3}{b_w}\cdot[t\cdot(b-b_w)\cdot(t+2\cdot e') + 2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}]\cdot x$$

$$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}\cdot\left(\frac{2}{3}\cdot t + e'\right) - \frac{6\cdot n}{b_w}[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'] = 0$$

	dia.	piece	cm2
Re-BarAs	16	28	56.308
As'	12	0	0

$\frac{3}{b_w}$	0.03	cm
$t\cdot(b-b_w)\cdot(t+2\cdot e')$	5920402	
$2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}$	569753	
$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}$	-7200	
$\left(\frac{2}{3}\cdot t + e'\right)$	416	
$-\frac{6\cdot n}{b_w}$	-0.6	
$[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s']$	5254385	
$\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2$	99512	
$n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}$	-71488	

e0 = 5 m
y1 = 69 cm
e' = 396 cm

x^3+ 1188 x^2+ 216338 x + -6148832 = y

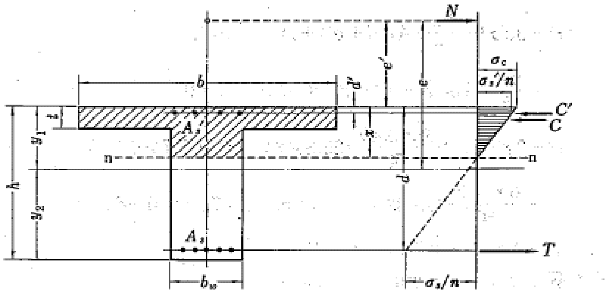
x = 24.94 cm
y = 0.00 OK

$$\sigma_c = \frac{N\cdot x}{\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2 + n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

$$\sigma_s' = n\cdot\sigma_c\cdot\frac{x-d'}{x}$$

σc =	2.22	N/mm^2	OK
σs =	113	N/mm^2	OK
τc =	0.02	N/mm^2	OK



(6) Case2-2 Seismic Condition (Smax)

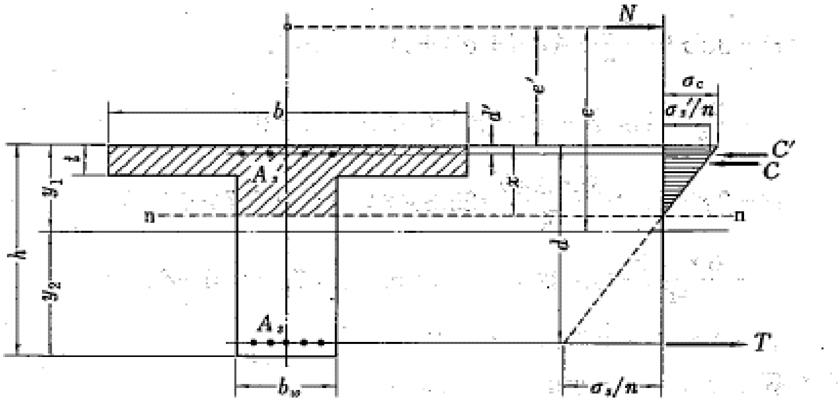
cgs		SI	
M	=	-	kNm
S	=	921.2	kN
N	=	-	kN
b	=	405	cm
H	=	240	cm
t	=	500	cm
bw	=	165	cm
d	=	203	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	12.30	N/mm^2
σsa	=	210.00	N/mm^2
τca	=	0.54	N/mm^2

(Breast Wall Riverside Section)

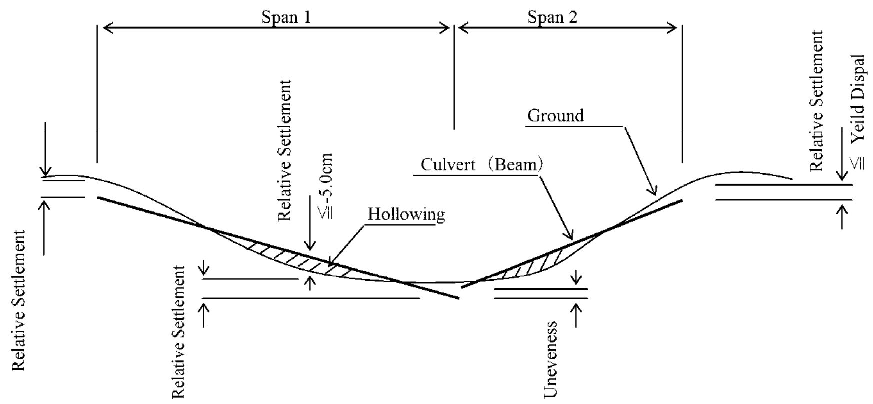
σc = - N/mm^2

σs = - N/mm^2

τc = 0.06 N/mm^2 OK



4. Relative Displacement (Stability Against Bearing)



Positive Relative Displacement

Amount which culvert dent into ground : Yielding Displacement of Ground

Yielding Displacement of Grc ≦ 5.0 cm

≦ 1.0% of Width of Foundation

	① Width of Foundation (cm)	② 1 % of ① (cm)		Allowable Relative Displacement (c m)
River Side	605.0	6.1	＞ 5.0 cm	5.0
Land Side	570.0	5.7	＞ 5.0 cm	5.0

Negative Relative Displacement

Amount of Hollowing ≦ -5.0 cm

	X (m)	Case 1 Normal Condition				
		Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
Land Side	0.000	9.7	12.1	2.4	5.0	ok
	3.420	15.3	14.4	-0.9	-5.0	ok
River Side	5.700	12.0	15.9	3.9	5.0	ok

	X (m)	Case 2-1,-2 Seismic Condition				
		Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
Land Side	0.000	9.7	12.0	2.3	5.0	ok
	3.420	15.3	14.1	-1.2	-5.0	ok
River Side	5.700	12.0	15.6	3.6	5.0	ok

Structural Calculation of Box Culvert (Longitudinal Direction)

MSL-4

1 Desing Condition

(1) Load and Case

Case	Load Condition	Water Level		Load Condition							
		Inside	Outside (Only Breast wall and Wing Wall)	Direction of Seismic	Weight of Body	Live Load (Surcharge)	Load of Breast Wall (Incruding Earth Pressure and Water	Weight of Seepage Cut off Wall	Weight of Water	Weight of Soil (Embankment)	Uplift
1	Normal	Full	-	-	○	○	○	○	○	△	-
2-1	Seismic 1	-	-	→	○	○	◎	○	-	△	-
2-2	Seismic 2	-	-	←	○	○	◎	○	-	△	-

→ : Land Side to River Silde

← : River Side to Land Silde

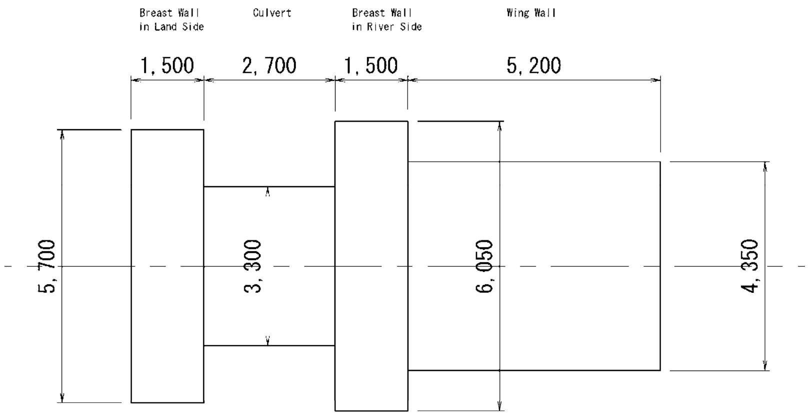
○ : Considered

◎ : Seismic Force is Considered

△ : Considered as Residual Settlement and Lateral displacement

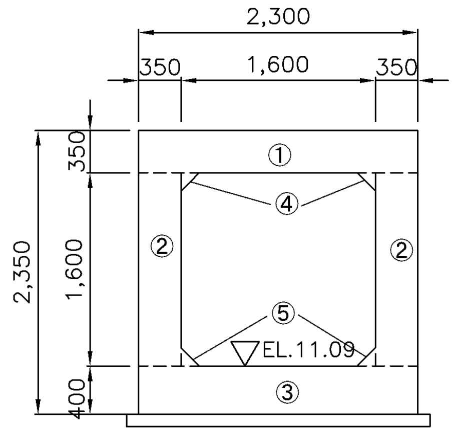
－ : Not Considered

(2) Properties of Each Section



		Wing Wall	Breast Wall in River Side	Culvert Section	Breast Wall in Land Side
Second Moment of Area	m^4	0.5650	5.6188	1.9634	4.2029
Area	m^2	2.240	6.761	2.891	5.246

Cross Section of Typical Cross Section of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	2.300	0.350	1.0	1	0.805	2.175	1.751
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.300	0.400	1.0	1	0.920	0.200	0.184
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
Total					2.891		3.334

α;Triangle: 0.5, Square: 1.0
n; Number of Element

$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 1.153 \quad m$

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix ; Second Moment of Area at Centroid (m^4)

ix ; Second Moment of Area Rrgarding Each Member (m^4)

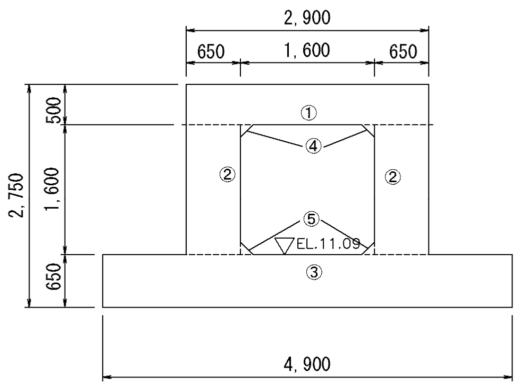
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.00822	0.805	-1.022	1.044	0.8404	0.8486
②	0.23893	1.120	-0.047	0.002	0.0022	0.2411
③	0.01227	0.920	0.953	0.908	0.8354	0.8477
④	0.00003	0.023	-0.797	0.635	0.0146	0.0146
⑤	0.00003	0.023	0.703	0.494	0.0114	0.0114
Total		2.891				1.9634

Area	**A=**	2.891	m^2
Second Moment	**Ix=**	1.9634	m^4

Breastwall in River Side



	Dimension				A(m^2)	Y(m)	A･Y(m^3)
	Width (B)	Height (h)	α	n			
①	2.900	0.500	1.0	1	1.450	2.500	3.625
②	0.650	1.600	1.0	2	2.080	1.450	3.016
③	4.900	0.650	1.0	1	3.185	0.325	1.035
④	0.150	0.150	0.5	2	0.023	2.200	0.051
⑤	0.150	0.150	0.5	2	0.023	0.700	0.016
Total					6.761		7.743

α ; Triangle: 0.5, Square: 1.0

n ; Number of Element

Y＝ ΣA･Y／ΣA ＝ 1.145 m

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

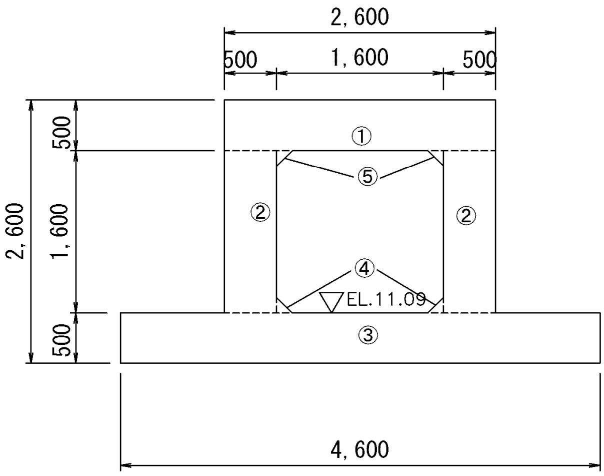
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.03021	1.450	-1.355	1.836	2.6622	2.6924
②	0.44373	2.080	-0.305	0.093	0.1934	0.6371
③	0.11214	3.185	0.820	0.672	2.1403	2.2524
④	0.00003	0.023	-1.055	1.113	0.0256	0.0256
⑤	0.00003	0.023	-0.700	0.490	0.0113	0.0113
Total		6.761				5.6188

Area	**A=**	6.761	m^2
Second Moment	**Ix=**	5.6188	m^4

Breast wall in Land Side



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	2.600	0.500	1.0	1	1.300	2.350	3.055
②	0.500	1.600	1.0	2	1.600	1.300	2.080
③	4.600	0.500	1.0	1	2.300	0.250	0.575
④	0.150	0.150	0.5	2	0.023	2.050	0.047
⑤	0.150	0.150	0.5	2	0.023	0.550	0.013
計					5.246		5.770

α; Triangle: 0.5, Square: 1.0

n; Number of Element

$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 1.100 \quad m$

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

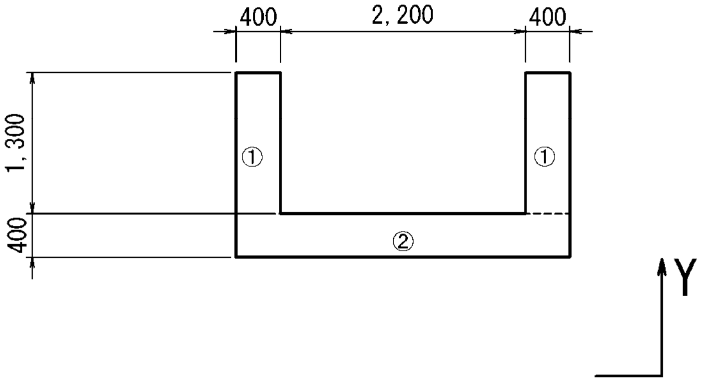
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.02708	1.300	-1.250	1.563	2.0319	2.0590
②	0.34133	1.600	-0.200	0.040	0.0640	0.4053
③	0.04792	2.300	0.850	0.723	1.6629	1.7108
④	0.00003	0.023	-0.950	0.903	0.0208	0.0208
⑤	0.00003	0.023	0.550	0.303	0.0070	0.0070
Total		5.246				4.2029

Area	**A=**	5.246	m^2
Second Moment	**Ix=**	4.2029	m^4

Wing Wall



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	0.400	1.300	1.0	2	1.040	1.050	1.092
②	3.000	0.400	1.0	1	1.200	0.200	0.240
計					2.240		1.332

α; Triangle: 0.5, Square: 1.0

n; Number of Element

$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 0.595 \quad m$

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix ; Second Moment of Area at Centroid (m^4)

ix ; Second Moment of Area Rrgarding Each Member (m^4)

A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.14647	1.040	−0.455	0.207	0.2153	0.3618
②	0.01600	1.200	0.395	0.156	0.1872	0.2032
Total		2.240				0.5650

Area	A=	2.240	m^2
Second Moment	Ix=	0.5650	m^4

(3) Calculation of Load

1. Weight of Mail Body

(a) Wing Wall

A= 2.240 m^2

Vertical Force

Wc1= 2.240 × 24.0 kN/m^3 = 53.76 kN/m

(b) Sliceway (Breast Wall in River Side)

A= 6.761 6.761 m^2

Vertical Force

Wc2= 6.761 × 24.0 kN/m^3 = 162.26 kN/m

(c) Sliceway (Box Culvert)

A= 2.891 m^2

Vertical Force

Wc3= 2.891 × 24.0 kN/m^3 = 69.38 kN/m

(d) Sliceway (Breast Wall in Land Side)

A= 5.246 5.246 m^2

Vertical Force

Wc4= 5.246 × 24.0 kN/m^3 = 125.90 kN/m

(e) Weight of Impervious Wall

Vo= (4.3×3.35-2.3×2.35)×0.50 = 4.50 m^3

V= 4.50 × 24.0 kN/m^3 = 108.00 kN

2. Water Weight

Only Normal Condition

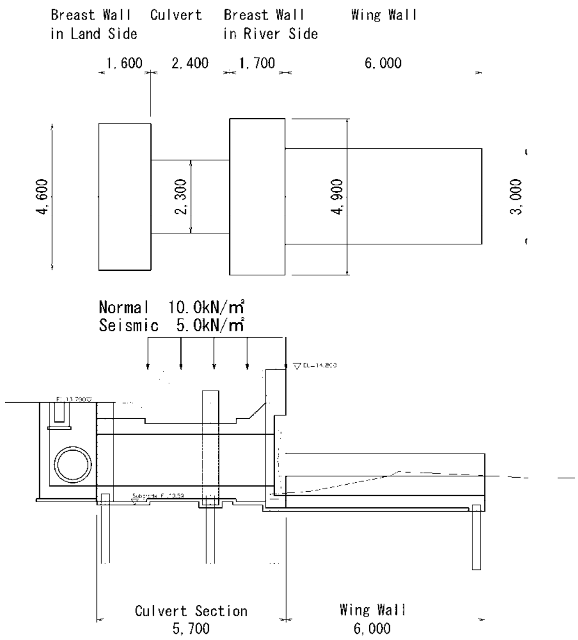
(a) Wing Wall

Wo1= $1.3 \times 2.20 \times 9.8$ = 28.03 kN/m

(b) Sliceway Main Body

Wo2= $(1.6 \times 1.6 - 1/2 \times 0.15 \times 0.15 \times 4) \times 9.8$ = 24.65 kN/m

3. Vertical Load



Nomal Condition

q1= 10.0×4.9	=	49.00	kN/m
q2= 10.0×2.3	=	23.00	kN/m

Seismic Condition

q1= 5.0×4.9	=	24.50	kN/m
q2= 5.0×2.3	=	11.50	kN/m

4. Calculation of the Force Effecting on Breast Wall

(a) River Side (River ← Land, +)

Nomal Condition

V=	143.88	×	1.00	×	2	=	287.76	kN
H=	131.09	×	1.00	×	2	=	262.18	kN
M=	-72.89	×	1.00	×	2	=	-145.78	kNm
x=	0.790	m						

Seismic Condition, from Land side to River side

V=	172.36	×	1.00	×	2	=	344.72	kN
H=	122.08	×	1.00	×	2	=	244.16	kN
M=	-106.40	×	1.00	×	2	=	-212.80	kNm
x=	0.910	m						

Seismic Condition, from River side to Land side

V=	172.36	×	1.00	×	2	=	344.72	kN
H=	61.26	×	1.00	×	2	=	122.52	kN
M=	-164.00	×	1.00	×	2	=	-328.00	kNm
x=	0.910	m						

(b) Land Side (River → Land, +)

Nomal Condition

V=	93.54	×	1.00	×	2	=	187.08	kN
H=	100.21	×	1.00	×	2	=	200.42	kN
M=	-62.35	×	1.00	×	2	=	-124.70	kNm
x=	0.740	m						

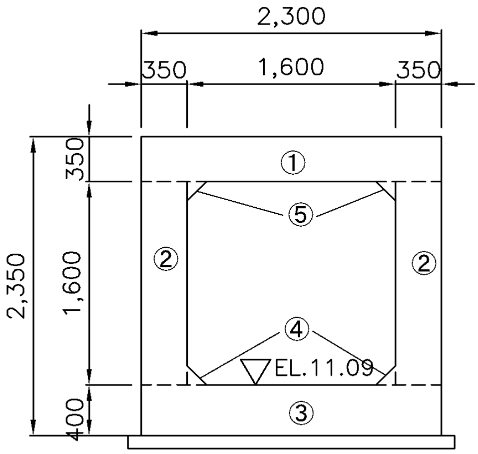
Seismic Condition, from Land side to River side

V=	113.20	×	1.00	×	2	=	226.40	kN
H=	17.45	×	1.00	×	2	=	34.90	kN
M=	-109.96	×	1.00	×	2	=	-219.92	kNm
x=	0.840	m						

Seismic Condition, from River side to Land side

V=	113.20	×	1.00	×	2	=	226.40	kN
H=	60.27	×	1.00	×	2	=	120.54	kN
M=	-90.44	×	1.00	×	2	=	-180.88	kNm
x=	0.840	m						

5. Centroid of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A\cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	2.300	0.350	1.0	1	0.805	2.175	1.751
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.300	0.400	1.0	1	0.920	0.200	0.184
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
Total					2.891		3.334

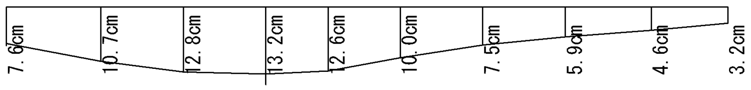
α; Triangle: 0.5, Square: 1.0

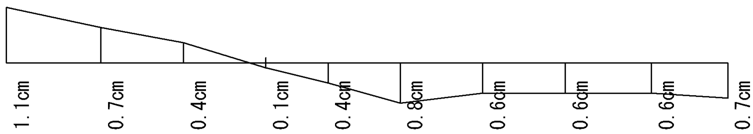
n; Number of Element

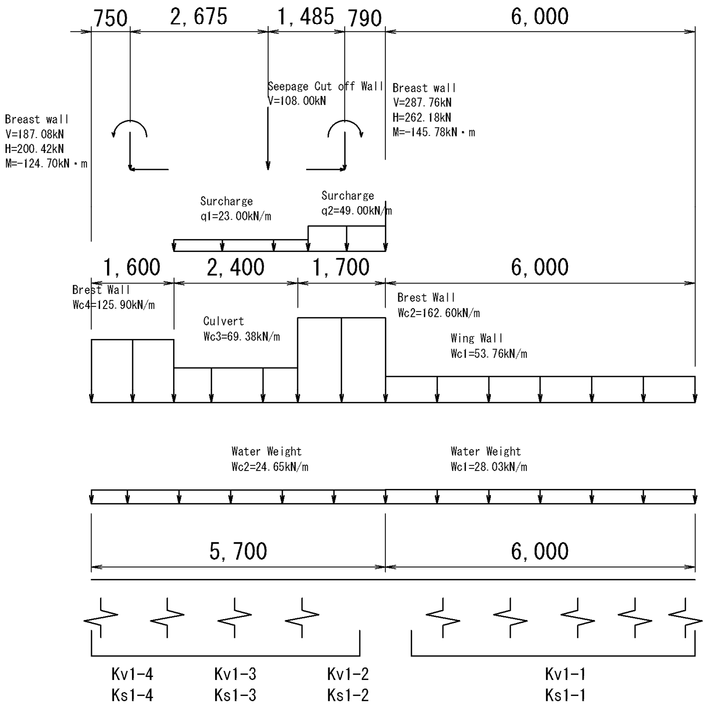
$Y = \Sigma A\cdot Y / \Sigma A \quad = \quad 1.153 \quad m$

6 Summary of Load

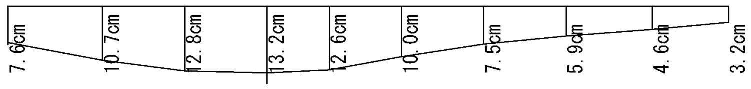
a) Normal Condition

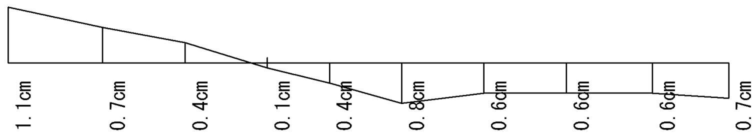


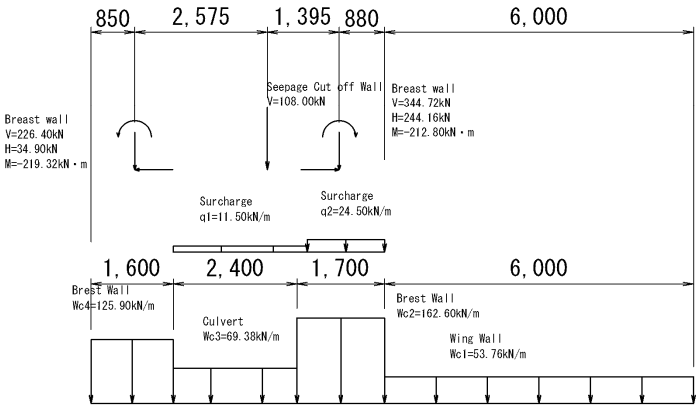


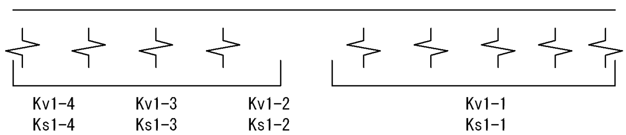


b) Seismic Condition (Land Side → River Side)

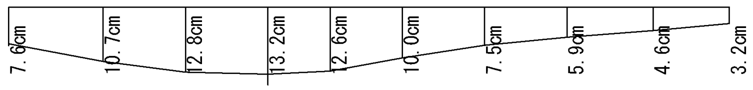


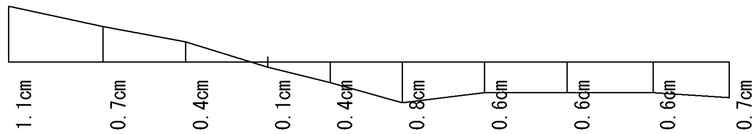


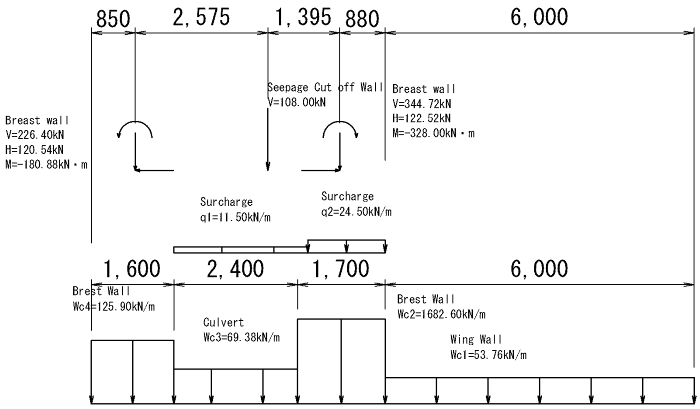


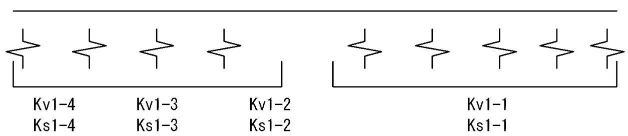


b) Seismic Condition (Land Side ← River Side)

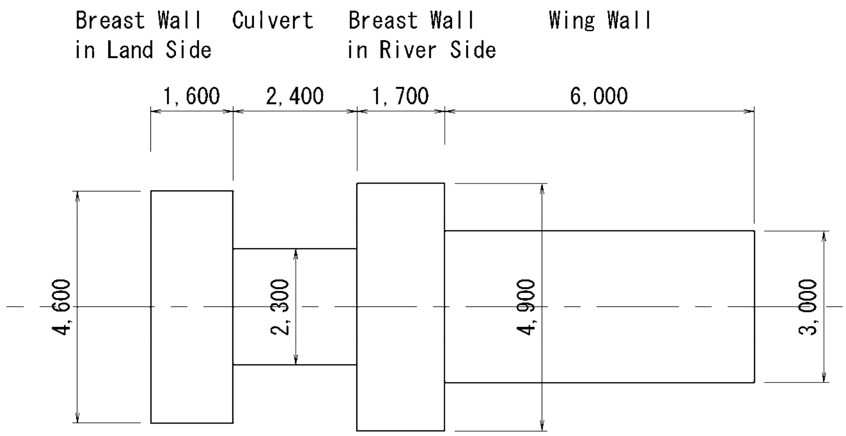








(4) Modulus of Subgrade Reaction and Modulus of Subgrade Reaction for Horizontal Shea



Results of Calculation

		Wing Walll	Breast Wall in River side	Culvert Section	Breast Wall in Lnd side
		1	2	3	4
Vertical Kv (kN/m^2)	Nomal	12348	28053	13471	27062
	Seismic	24696	56105	26942	54124
Horizontal Shear Ks(kN/m^2)	Nomal	4116	9351	4490	9021
	Seismic	8232	18702	8981	18041

a. Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvo : Coefficient of Vertical Reaction of soil (kN/m^3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E_{0m} obtained by various soil tests and investigations:

$$Kvo= \frac{1}{0.3} \quad \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eom : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{ (Kv \cdot D) / (4 \cdot E \cdot I) \}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a) Wing Wall

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert $(kN \cdot m^2)$

E= 2.45E+07 kN/m^2

I= 0.5650 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 2251.26 = 30016.8 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.12220

β・L= 0.733 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{6.00 \times 3.00} = \boxed{4.243}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{3.00 / 0.12220} = 4.955$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 30016.8 \times (4.243 / 0.3)^{-3/4} = 4116$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 4116 × 3.00 = 12348 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 8232 (kN/m^3) 〔= 2 × 4116 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 8232 × 3.00 = 24696 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 12348

= 4116 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 24696

= 8232 (kN/m^2)

(Wing wall)
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ; Converted Modulus of Deformation in B≠L （kN/m^2）
B ; Loading Width = Width of Wing Wall B= 3.000 m
L ; Loading Length = Length of a span L= 6.000 m
h_n ; Depth for Calculation hn= 9.000 m
h_i ; Depth from the ground to the bottom Elevation of Each Layer（m）
Ei ; onverted Modulus of Deformation of Each Layer （kN/m^2）
θ ; Angle of Load Dispersion（θ=30°）

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	2251.26	3.000	6.000	9.000	4.070	4.070	1880	30	0.4910	0.364	0.0001936
2					2.000	6.070	4200			0.067	0.0000160
3					2.930	9.000	7050			0.060	0.0000085
4											
5											
6											
7											
8											
9											
10											
				Total	9.000					Total	0.0002181

b) Breast wall in River Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert $(kN \cdot m^2)$

E= 2.45E+07 kN/m^2

I= 5.6188 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 2345.40 = 31272.0 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.08448

β・L= 0.144 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{1.70 \times 4.90} = \boxed{2.886}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{4.90 / 0.08448} = 7.616$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 31272.0 \times (2.886 / 0.3)^{-3/4} = 5725$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 5725 × 4.90 = 28053 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 11450 (kN/m^3) 〔 2 × 5725 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 11450 × 4.90 = 56105 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 28053

= 9351 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 56105

= 18702 (kN/m^2)

(Breast wall in River Side)
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\displaystyle\sum_{i=1}^{n} \frac{1}{Ei} \log \frac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\displaystyle\sum_{i=1}^{n} \frac{1}{Ei} \times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L		（kN/m^2）	
B ;	Loading Width = Width of Culvert	B=	4.900	m
L ;	Loading Length = Length of a span	L=	1.700	m
h_n ;	Depth for Calculation	hn=	14.700	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer（m）			
Ei ;	onverted Modulus of Deformation of Each La（kN/m^2）			
θ ;	Angle of Load Dispersion（θ=30°）			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	2345.40	4.900	1.700	14.700	4.070	4.070	1880	30	-0.9004	-0.653	-0.0003473
2					2.000	6.070	4200			-0.093	-0.0000221
3					3.500	9.570	7050			-0.089	-0.0000126
4					5.330	14.900	35000			-0.068	-0.0000019
5											
6											
7											
8											
				Total	14.900					Total	-0.0003839

c) Culvert Section

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert $(kN \cdot m^2)$

E= 2.45E+07 kN/m^2

I= 1.9634 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 2056.25 = 27416.7 kN/m^3

β= ｛（Kv・D）/(4・E・I)｝$^{1/4}$

= 0.09147

β・L= 0.220 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv=\sqrt{L \cdot D}$ $=\sqrt{2.40 \times 2.30}$ = $\boxed{2.349}$ m

Case2 Elastic body $Bv=\sqrt{D/\beta}$ $=\sqrt{2.30 / 0.09147}$ = 5.014 m

Kv = Kvo × $(Bv/0.3)^{-3/4}$

= 27416.7 × (2.349 / 0.3)$^{-3/4}$ = 5857 (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 5857 × 2.30 = <u>13471</u> (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 11714 (kN/m^3) 〔 2 × 5857 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 11714 × 2.30 = <u>26942</u> (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 13471

= <u>4490</u> (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 26942

= <u>8981</u> (kN/m^2)

(Culvert Section)
Calculation Results of Converted Modulus of Deformation

$$Eom=\frac{\log\dfrac{(B+2h_n\cdot\tan\theta)\,L}{(L+2h_n\cdot\tan\theta)\,B}}{\displaystyle\sum_{i=1}^{n}\frac{1}{Ei}\log\frac{(B+2h_i\cdot\tan\theta)\,(L+2\,h_{i-1}\cdot\tan\theta)}{(L+2h_i\cdot\tan\theta)\,(B+2\,h_{i-1}\cdot\tan\theta)}}=\frac{f\,n}{\displaystyle\sum_{i=1}^{n}\frac{1}{Ei}\times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L		(kN/m^2)	
B ;	Loading Width = Width of Culvert	B=	2.300	m
L ;	Loading Length = Length of a span	L=	2.400	m
h_n ;	Depth for Calculation	hn=	6.900	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each La (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	2056.25	2.300	2.400	6.900	4.070	4.070	1880	30	0.0329	0.028	0.0000149
2					2.000	6.070	4200			0.004	0.0000010
3					0.830	6.900	7050			0.001	0.0000001
4											
5											
6											
7											
8											
				Total	6.900					Total	0.0000160

d) Breast Wall in Land Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 4.2029 m^4

◇ Nomal Condition

$Kvo = 1/0.3 \times \alpha \times Eom$

$= 1 / 0.3 \times 4 \times 2301.05 = 30680.7$ kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

$= 0.09003$

$\beta \cdot L = 0.144 \leqq 1.5$ Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{1.60 \times 4.60} = \boxed{2.713}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{4.60 / 0.09003} = 7.148$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 30680.7 \times (2.713 / 0.3)^{-3/4} = 5883$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

$kv = Kv \cdot D = 5883 \times 4.60 = \underline{27062}$ (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 11766 (kN/m^3) 〔 2×5883 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

$kv = Kv \cdot D = 11766 \times 4.60 = \underline{54124}$ (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

$Ks = \lambda \cdot kv$

$= 1/3 \times 27062$

$= \underline{9021}$ (kN/m^2)

◇ Seismic Condition

$Ks = \lambda \cdot kv$

$= 1/3 \times 54124$

$= \underline{18041}$ (kN/m^2)

(Breast Wall in Land Side)
Calculation Results of Converted Modulus of Deformation

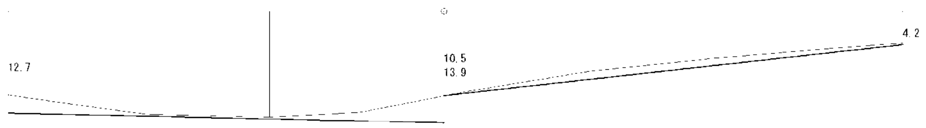
$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ; Converted Modulus of Deformation in B≠L (kN/m^2)
B ; Loading Width = Width of Culvert B= 4.600 m
L ; Loading Length = Length of a span L= 1.600 m
h_n ; Depth for Calculation hn= 13.800 m
h_i ; Depth from the ground to the bottom Elevation of Each Layer (m)
Ei ; onverted Modulus of Deformation of Each Lay (kN/m^2)
θ ; Angle of Load Dispersion (θ=30°)

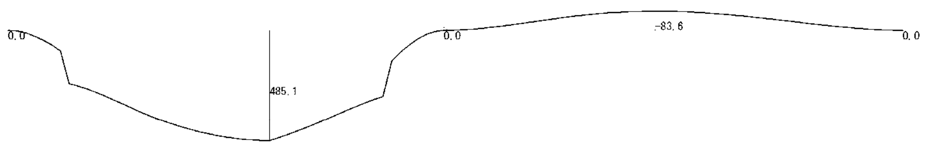
	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	2301.05	4.600	1.600	13.800	4.070	4.070	1880	30	-0.8981	-0.667	-0.0003548
2					2.000	6.070	4200			-0.091	-0.0000217
3					3.500	9.570	7050			-0.086	-0.0000122
4					4.230	13.800	35000			-0.055	-0.0000016
5											
6											
7											
8											
				Total	13.800					Total	-0.0003903

2 Cross Section Force

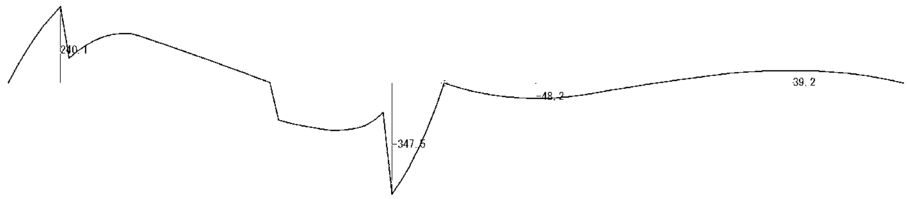
(1) Case 1 Normal Condition



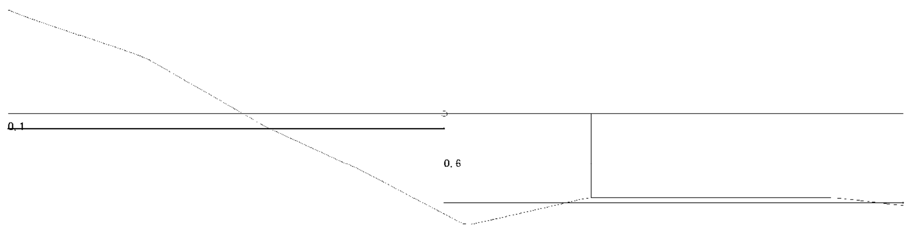
Settlement of Culvert and Ground (cm)



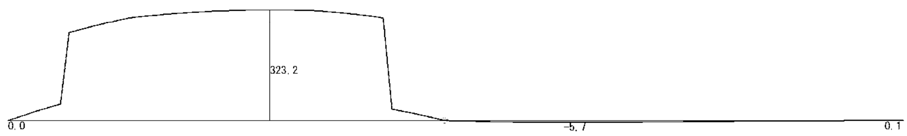
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

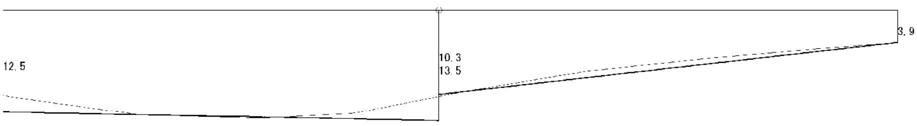


Axial Force (kN)

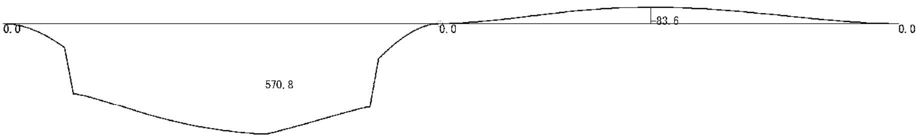
	1-SPAN X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.7	0.1	10.4	12.7	0.00	0.00	0.00	620.86
	1.140	-0.5	0.1	11.9	12.9	276.05	133.52	270.37	262.80
	2.280	-0.2	0.1	12.9	13.2	314.25	100.39	426.14	33.20
F-F max	3.420	0.1	0.1	13.2	13.4	323.20	2.26	485.12	31.65
B-M max	3.420	0.1	0.1	13.2	13.4	323.20	2.26	485.12	31.65
	4.560	0.4	0.1	12.6	13.7	309.95	-140.13	332.04	296.23
S-F max	5.016	0.5	0.1	11.8	13.8	32.83	-347.51	134.39	550.55
	5.700	0.7	0.1	10.5	13.9	0.00	0.02	0.09	949.14

	2-SPAN X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.7	0.6	10.5	10.5	0.00	0.00	0.00	-5.13
S-F max	1.200	0.7	0.6	8.6	9.2	-5.34	-48.17	-38.67	80.43
F-F max	1.560	0.6	0.6	8.0	8.8	-5.72	-44.61	-55.62	102.93
	2.400	0.6	0.6	7.0	8.0	-4.66	-13.84	-80.99	120.01
B-M max	2.760	0.6	0.6	6.6	7.6	-4.09	-0.53	-83.55	117.50
	3.600	0.6	0.6	5.8	6.7	-2.75	26.88	-72.09	109.12
	4.800	0.6	0.6	4.8	5.4	-0.84	37.99	-28.83	72.99
	6.000	0.7	0.6	4.0	4.2	0.00	0.00	0.03	24.22

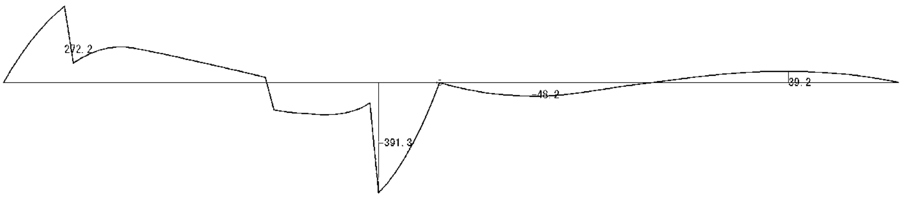
(2) Case 2 Seismic Condition (Land Side to River Side)



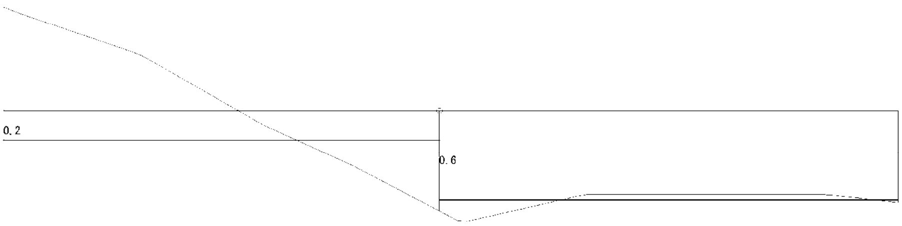
Settlement of Culvert and Ground (cm)



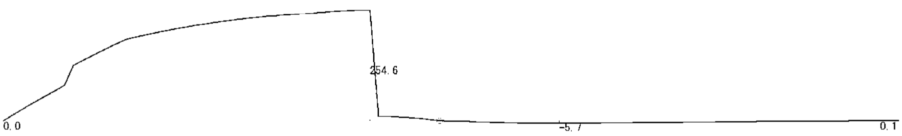
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

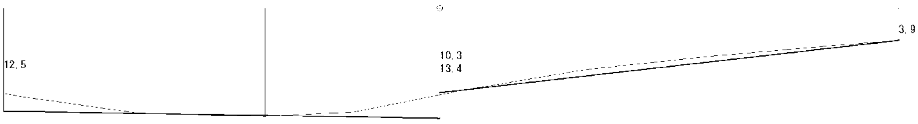


Axial Force (kN)

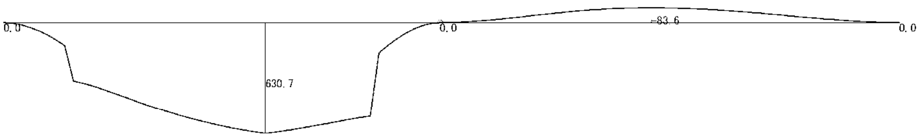
1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.7	0.3	10.4	12.5	0.00	0.00	0.00	604.77
	1.140	-0.5	0.3	11.9	12.7	147.43	103.41	379.54	243.20
	2.280	-0.2	0.3	12.9	12.9	211.48	89.39	508.70	21.70
	3.420	0.1	0.3	13.2	13.0	238.80	18.36	570.76	18.40
	4.560	0.4	0.3	12.6	13.3	253.68	-101.49	448.97	264.98
	5.700	0.7	0.3	10.5	13.5	0.00	0.02	0.09	914.25

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.7	0.6	10.5	10.3	0.00	0.00	0.00	-33.16
	1.200	0.7	0.6	8.6	9.0	-5.34	-48.17	-38.67	52.40
	2.400	0.6	0.6	7.0	7.7	-4.66	-13.84	-80.99	91.98
	3.600	0.6	0.6	5.8	6.5	-2.75	26.88	-72.09	81.09
	4.800	0.6	0.6	4.8	5.2	-0.84	37.99	-28.83	44.96
	6.000	0.7	0.6	4.0	3.9	0.00	0.00	0.03	-3.81

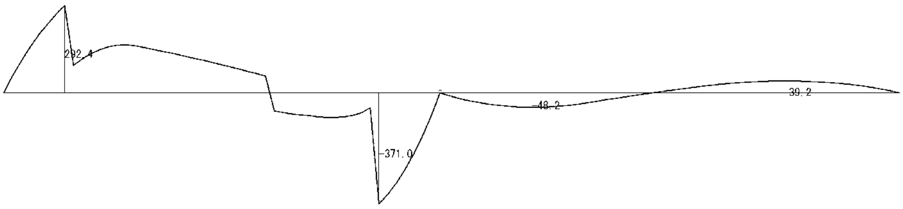
(3) Case 2-2 Seismic Condition (River Side to Land Side)



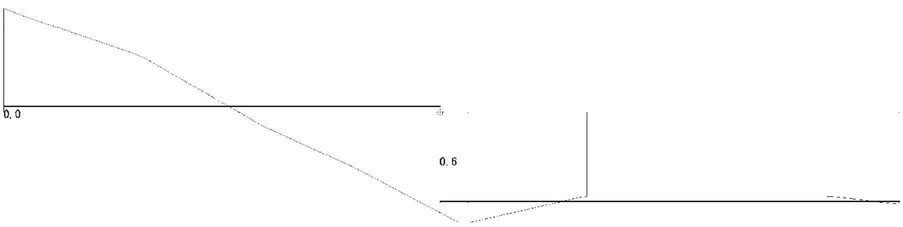
Settlement of Culvert and Ground (cm)



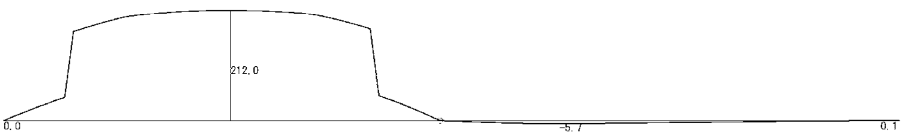
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)



Axial Force (kN)

1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.7	0.0	10.4	12.5	0.00	0.00	0.00	634.17
	1.140	-0.5	0.0	11.9	12.7	181.22	130.32	357.69	261.00
	2.280	-0.2	0.0	12.9	12.9	208.94	126.67	525.32	24.79
	3.420	0.1	0.0	13.2	13.0	210.45	55.88	630.65	15.71
	4.560	0.4	0.0	12.6	13.2	185.80	-74.58	547.53	247.38
	5.700	0.7	0.0	10.5	13.5	0.00	0.02	0.09	884.63

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	0.7	0.6	10.5	10.3	0.00	0.00	0.00	-33.16
	1.200	0.7	0.6	8.6	9.0	-5.34	-48.17	-38.67	52.40
	2.400	0.6	0.6	7.0	7.7	-4.66	-13.84	-80.99	91.98
	3.600	0.6	0.6	5.8	6.5	-2.75	26.88	-72.09	81.09
	4.800	0.6	0.6	4.8	5.2	-0.84	37.99	-28.83	44.96
	6.000	0.7	0.6	4.0	3.9	0.00	0.00	0.03	-3.81

3 Stress Calculation

		Case1 Normal Condition		Case2-1 Seismic Condition (L to R)		Case2-2 Seismic Condition (R to L)	
		Mmax	Smax	Mmax	Smax	Mmax	Smax
MSL4	Tensile Side	Bottom	-	Bottom	-	Bottom	-
	M (kNm)	485.1	-	570.8	-	630.7	-
	S (kN)	2.3	347.51	18.4	391.3	55.9	371.0
	N (kN)	323.2	-	238.8	-	210.5	-
	Re-Bar	D12 x 20	-	D12 x 20	-	D12 x 20	-
	Area (m^2)	22.6	-	22.6	-	22.6	-
	σ_c (N/mm^2)	1.05	-	1.31	-	1.47	-
	σ_s (N/mm^2)	48.3	-	78.1	-	94.5	-
	τ (N/mm^2)	0.00	0.03	0.01	0.03	0.03	0.03
	σ_{ca} (N/mm^2)	8.2	8.2	12.3	12.3	12.3	12.3
	σ_{sa} (N/mm^2)	140.0	140.0	210.0	210.0	210.0	210.0
	τa (N/mm^2)	0.36	0.36	0.54	0.54	0.54	0.54
	Evaluation	OK	OK	OK	OK	OK	OK

L : Land Side
R : River Side

(1) Case1 Normal Condition (Mmax)

cgs		SI	
M (tfxm)	=	485.12	kNm
S (tf)	=	2.26	kN
N (tf)	=	323.2	kN
b (cm)	=	230	cm
H (cm)	=	235	cm
t (cm)	=	35	cm
bw (cm)	=	70	cm
d (cm)	=	219	cm
d' (cm)	=	0	cm
n (Es/Ec)	=	9	
σca	=	8.20	N/mm^2
σsa	=	140	N/mm^2
τca	=	0.36	N/mm^2

$$y_1 = \frac{\frac{b\cdot t^2}{2} + \frac{b_w}{2}\cdot(h-t)\cdot(h+t)}{b\cdot t + b_w\cdot(h-t)}$$

$\frac{b\cdot t^2}{2}$	140875	cm^3
$\frac{b_w}{2}\cdot(h-t)\cdot(h+t)$	1890000	cm^3
$b\cdot t + b_w\cdot(h-t)$	22050	cm^2

$$x^3 + 3\cdot e'\cdot x^2 + \frac{3}{b_w}\cdot\left[t\cdot(b-b_w)\cdot(t+2\cdot e') + 2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}\right]\cdot x$$
$$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}\cdot\left(\frac{2}{3}\cdot t + e'\right) - \frac{6\cdot n}{b_w}\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right] = 0$$

$\frac{3}{b_w}$	0.04	cm
$t\cdot(b-b_w)\cdot(t+2\cdot e')$	845553.36	
$2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}$	112781.62	
$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}$	-8400	
$\left(\frac{2}{3}\cdot t + e'\right)$	81.33	
$-\frac{6\cdot n}{b_w}$	-0.771428571	
$\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right]$	1372176.43	

	dia.	piece	cm2
Re-BarAs	12	20	22.62
As'	12	0	0

e0 = 2 m

y1 = 92 cm

e' = 58 cm

x^3+ 174 x^2 + 41071 x + -1741701 = y

x = 35.84 cm

y = 0.00 OK

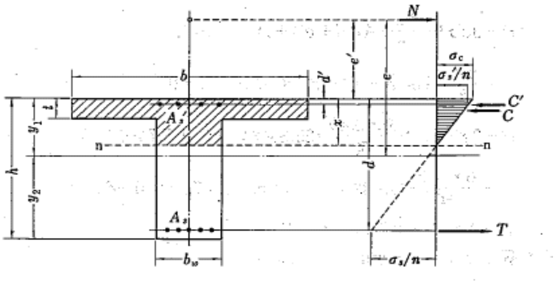
$$\sigma_c = \frac{N\cdot x}{\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2 + n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

$$\sigma_s' = n\cdot\sigma_c\cdot\frac{x-d'}{x}$$

$\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2$	147686.28	
$n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}$	-37287.10	

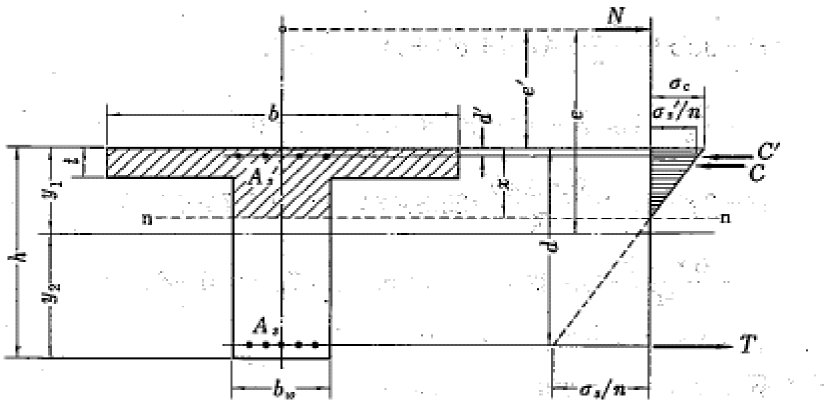
σc =	1.05	N/mm^2	OK
σs =	48	N/mm^2	OK
τc =	0.00	N/mm^2	OK



(2) Case1 Normal Condition (Smax)

cgs		SI		
M	=	-	kNm	
S	=	347.51	kN	
N	=	-	kN	
b	=	290	cm	(Breast Wall Riverside Section)
H	=	280	cm	
t	=	500	cm	
bw	=	130	cm	
d	=	243	cm	
d'	=	0	cm	
n (Es/Ec)	=	9		
σca	=	8.20	N/mm^2	
σsa	=	140	N/mm^2	
τca	=	0.36	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.03	N/mm^2	OK



(3) Case2-1 Seismic Condition (Mmax)

cgs		SI	
M	=	570.8	kNm
S	=	18.36	kN
N	=	238.8	kN
b	=	230	cm
H	=	235	cm
t	=	35	cm
bw	=	70	cm
d	=	219	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	12.30	N/mm^2
σsa	=	210.00	N/mm^2
τca	=	0.54	N/mm^2

	dia.	piece	cm2
Re-BarAs	12	20	22.62
As'	12	0	0

e0 = 2 m

y1 = 92 cm

e' = 147 cm

x^3+ 441 x^2 + 85309 x + -2828551 = y

x = 28.64 cm

y = 0.00 OK

$$y_1 = \frac{\frac{b \cdot t^2}{2} + \frac{b_w}{2} \cdot (h-t) \cdot (h+t)}{b \cdot t + b_w \cdot (h-t)}$$

$\frac{b \cdot t^2}{2}$ 140875 cm^3

$\frac{b_w}{2} \cdot (h-t) \cdot (h+t)$ 1890000 cm^3

$b \cdot t + b_w \cdot (h-t)$ 22050 cm^2

$$x^3 + 3 \cdot e' \cdot x^2 + \frac{3}{b_w} \cdot \left[t \cdot (b-b_w) \cdot (t + 2 \cdot e') + 2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}\right] \cdot x$$
$$- \frac{3 \cdot (b-b_w) \cdot t^2}{b_w} \cdot \left(\frac{2}{3} \cdot t + e'\right) - \frac{6 \cdot n}{b_w}\left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s'\right] = 0$$

$\frac{3}{b_w}$ 0.04 cm

$t \cdot (b-b_w) \cdot (t + 2 \cdot e')$ 1841563

$2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}$ 148990

$-\frac{3 \cdot (b-b_w) \cdot t^2}{b_w}$ -8400

$\left(\frac{2}{3} \cdot t + e'\right)$ 170

$-\frac{6 \cdot n}{b_w}$ -0.771428571

$\left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s'\right]$ 1812713

$$\sigma_c = \frac{N \cdot x}{\frac{b \cdot x^2}{2} - \frac{(b-b_w)}{2} \cdot (x-t)^2 + n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}}$$

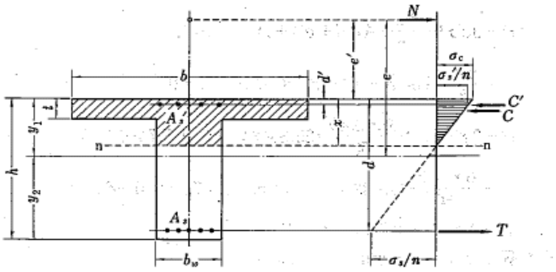
$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

$$\sigma_s' = n \cdot \sigma_c \cdot \frac{x-d'}{x}$$

$\frac{b \cdot x^2}{2} - \frac{(b-b_w)}{2} \cdot (x-t)^2$ 91110

$n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}$ -38753

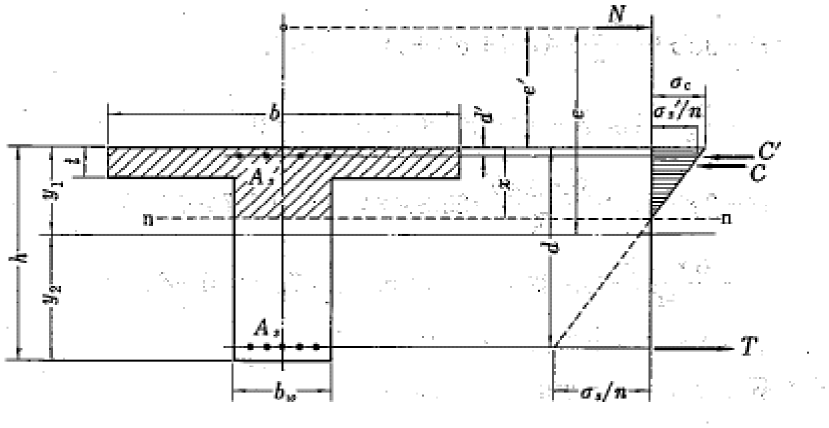
σc =	1.31	N/mm^2	OK
σs =	78	N/mm^2	OK
τc =	0.01	N/mm^2	OK



(4) Case2-1 Seismic Condition (Smax)

cgs		SI		
M	=	-	kNm	
S	=	391.3	kN	
N	=	-	kN	
b	=	290	cm	(Breast Wall Riverside Section)
H	=	280	cm	
t	=	500	cm	
bw	=	130	cm	
d	=	243	cm	
d'	=	0	cm	
n (Es/Ec)	=	9		
σca	=	12.30	N/mm^2	
σsa	=	210.00	N/mm^2	
τca	=	0.54	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.03	N/mm^2	OK



(5) Case2-2 Seismic Condition (Mmax)

cgs		SI	
M	=	630.65	kNm
S	=	55.88	kN
N	=	210.45	kN
b	=	230	cm
H	=	235	cm
t	=	35	cm
bw	=	70	cm
d	=	219	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	12.30	N/mm²
σsa	=	210.00	N/mm²
τca	=	0.54	N/mm²

$$y_1 = \frac{\frac{b\cdot t^2}{2} + \frac{b_w}{2}\cdot(h-t)\cdot(h+t)}{b\cdot t + b_w\cdot(h-t)}$$

$\frac{b\cdot t^2}{2}$	140875	cm^3
$\frac{b_w}{2}\cdot(h-t)\cdot(h+t)$	1890000	cm^3
$b\cdot t + b_w\cdot(h-t)$	22050	cm^2

$$x^3 + 3\cdot e'\cdot x^2 + \frac{3}{b_w}\cdot\left[t\cdot(b-b_w)\cdot(t+2\cdot e') + 2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}\right]\cdot x$$
$$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}\cdot\left(\frac{2}{3}\cdot t + e'\right) - \frac{6\cdot n}{b_w}\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right] = 0$$

	dia.	piece	cm2
Re-BarAs	12	20	22.62
As'	12	0	0

$\frac{3}{b_w}$	0.04	cm
$t\cdot(b-b_w)\cdot(t+2\cdot e')$	2520719	
$2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}$	173680	
$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}$	-8400	
$\left(\frac{2}{3}\cdot t + e'\right)$	231	
$-\frac{6\cdot n}{b_w}$	-0.771428571	
$\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right]$	2113105	
$\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2$	77637	
$n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}$	-39117	

e0 = 3 m
y1 = 92 cm
e' = 208 cm

x^3+ 623 x^2 + 115474 x + -3569649 = y

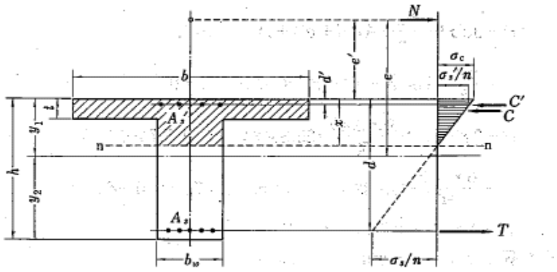
x = 26.86 cm
y = 0.00 OK

$$\sigma_c = \frac{N\cdot x}{\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2 + n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

$$\sigma_s' = n\cdot\sigma_c\cdot\frac{x-d'}{x}$$

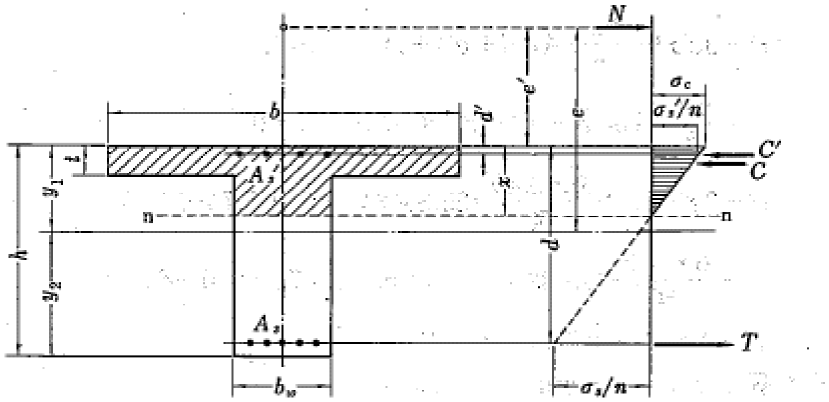
σc =	1.47	N/mm²	OK
σs =	94	N/mm²	OK
τc =	0.03	N/mm²	OK



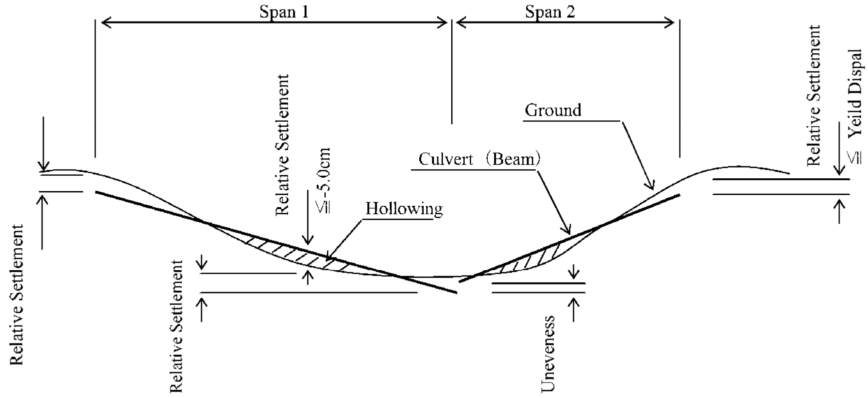
(6) Case2-2 Seismic Condition (Smax)

	cgs		SI		
M	=	-		kNm	
S	=		371	kN	
N	=	-		kN	
b	=		290	cm	(Breast Wall Riverside Section)
H	=		280	cm	
t	=		500	cm	
bw	=		130	cm	
d	=		243	cm	
d'	=		0	cm	
n (Es/Ec)	=		9		
σca	=		12.30	N/mm^2	
σsa	=		210.00	N/mm^2	
τca	=		0.54	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.03	N/mm^2	OK



4. Relative Displacement （Stability Against Bearing）



Positive Relative Displacement

Amount which culvert dent into ground : Yielding Displacement of Ground

Yielding Displacement of Grc ≦ 5.0 cm

≦ 1.0% of Width of Foundation

	① Width of Foundation (cm)	② 1 % of ① (cm)		Allowable Relative Displacement (ｃｍ)
River Side	605.0	6.1	＞ 5.0 cm	5.0
Land Side	570.0	5.7	＞ 5.0 cm	5.0

Negative Relative Displacement

Amount of Hollowing ≦ -5.0 cm

	X	Case 1 Normal Condition				
	(m)	Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
Land Side	0.000	10.4	12.7	2.3	5.0	ok
	3.420	13.2	13.4	0.2	-	-
River Side	5.700	10.5	13.9	3.4	5.0	ok

	X	Case 2-1,-2 Seismic Condition				
	(m)	Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
Land Side	0.000	10.4	12.5	2.1	5.0	ok
	3.420	13.2	13.0	-0.2	-5.0	ok
River Side	5.700	10.5	13.5	3.0	5.0	ok

Structural Calculation of Box Culvert
(Longitudinal Direction)

MSR-3

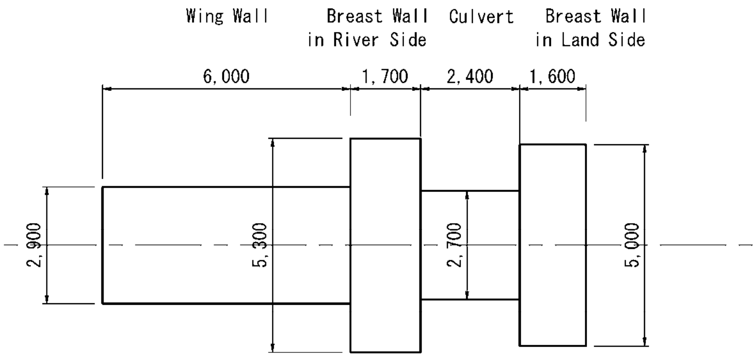
1 Desing Condition

(1) Load and Case

Case	Load Condition	Water Level		Load Condition							
		Inside	Outside (Only Breast wall and Wing Wall)	Direction of Seismic	Weight of Body	Live Load (Surcharge)	Load of Breast Wall (Incruding Earth Pressure and Water	Weight of Seepage Cut off Wall	Weight of Water	Weight of Soil (Embankment)	Uplift
1	Normal	Full	-	-	○	○	○	○	○	△	-
2-1	Seismic 1	-	-	→	○	○	◎	○	-	△	-
2-2	Seismic 2	-	-	←	○	○	◎	○	-	△	-

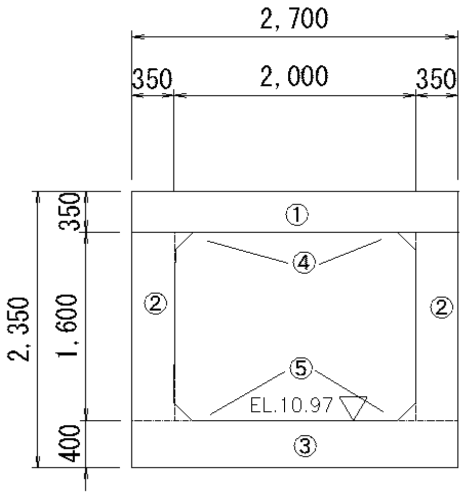
→ : Land Side to River Silde
← : River Side to Land Silde
○ : Considered
◎ : Seismic Force is Considered
△ : Considered as Residual Settlement and Lateral displacement
－ : Not Considered

(2) Properties of Each Section



		Wing Wall	Breast Wall in River Side	Culvert Section	Breast Wall in Land Side
Second Moment of Area	m^4	0.0603	6.3287	2.2608	4.6651
Area	m^2	1.420	7.206	3.191	5.646

Cross Section of Typical Cross Section of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A\cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	2.700	0.350	1.0	1	0.945	2.175	2.055
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.700	0.400	1.0	1	1.080	0.200	0.216
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
Total					3.191		3.670

α;Triangle: 0.5, Square: 1.0
n; Number of Element

$Y = \Sigma A\cdot Y / \Sigma A \quad = \quad 1.150 \quad m$

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

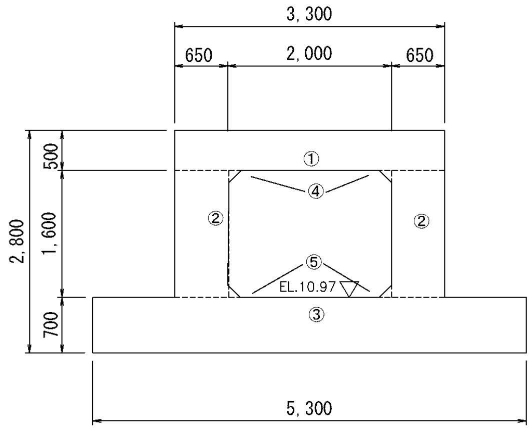
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.00965	0.945	-1.025	1.051	0.9932	1.0029
②	0.23893	1.120	-0.050	0.003	0.0034	0.2423
③	0.01440	1.080	0.950	0.903	0.9752	0.9896
④	0.00003	0.023	-0.800	0.640	0.0147	0.0147
⑤	0.00003	0.023	0.700	0.490	0.0113	0.0113
Total		3.191				2.2608

Area	**A=**	3.191	m^2
Second Moment	**Ix=**	2.2608	m^4

Breastwall in River Side



	Dimension				A(m^2)	Y(m)	A・Y(m^3)
	Width (B)	Height (h)	α	n			
①	3.300	0.500	1.0	1	1.650	2.550	4.208
②	0.650	1.600	1.0	2	2.080	1.500	3.120
③	4.900	0.700	1.0	1	3.430	0.350	1.201
④	0.150	0.150	0.5	2	0.023	2.250	0.052
⑤	0.150	0.150	0.5	2	0.023	0.750	0.017
Total					7.206		8.598

α;Triangle: 0.5, Square: 1.0
n; Number of Element

Y＝ ΣA･Y／ΣA ＝ 1.193 m

Second Moment of Area

$$Ix = ix + A \cdot Y^2$$

Where,

Ix;Second Moment of Area at Centroid（m^4）

ix;Second Moment of Area Rrgarding Each Member（m^4）

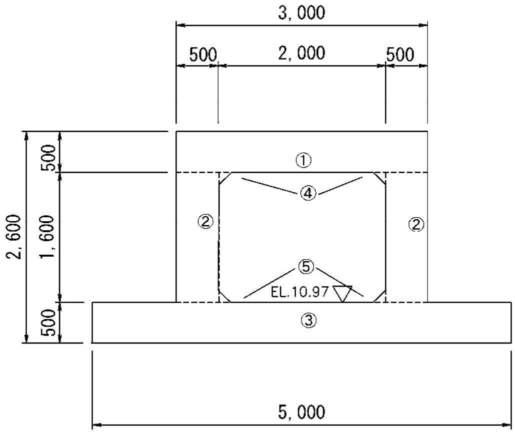
A;Area of Each Member（m^2）

Y;Distance between Each Centroid and Combined Centroid（m）

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.03438	1.650	-1.357	1.841	3.0377	3.0721
②	0.44373	2.080	-0.307	0.094	0.1955	0.6392
③	0.14006	3.430	0.843	0.711	2.4387	2.5788
④	0.00003	0.023	-1.057	1.117	0.0257	0.0257
⑤	0.00003	0.023	-0.750	0.563	0.0129	0.0129
Total		7.206				6.3287

Area	**A=**	7.206	m^2
Second Moment	**Ix=**	6.3287	m^4

Breast wall in Land Side



	Dimension				A(m^2)	Y(m)	A･Y(m^3)
	Width (B)	Height (h)	α	n			
①	3.000	0.500	1.0	1	1.500	2.350	3.525
②	0.500	1.600	1.0	2	1.600	1.300	2.080
③	5.000	0.500	1.0	1	2.500	0.250	0.625
④	0.150	0.150	0.5	2	0.023	2.050	0.047
⑤	0.150	0.150	0.5	2	0.023	0.550	0.013
計					5.646		6.290

α; Triangle: 0.5, Square: 1.0

n; Number of Element

Y= ΣA･Y／ΣA = 1.114 m

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

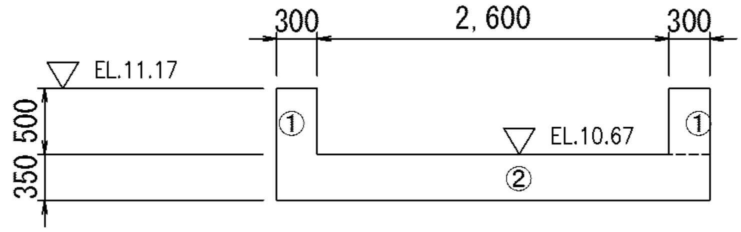
A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.03125	1.500	-1.236	1.528	2.2920	2.3233
②	0.34133	1.600	-0.186	0.035	0.0560	0.3973
③	0.05208	2.500	0.864	0.746	1.8650	1.9171
④	0.00003	0.023	-0.936	0.876	0.0201	0.0201
⑤	0.00003	0.023	0.564	0.318	0.0073	0.0073
Total		5.646				4.6651

Area	**A=**	5.646	m^2
Second Moment	**Ix=**	4.6651	m^4

Wing Wall



	Dimension				A(m^2)	Y(m)	A・Y(m^3)
	Width (B)	Height (h)	α	n			
①	0.300	0.500	1.0	2	0.300	0.600	0.180
②	3.200	0.350	1.0	1	1.120	0.175	0.196
計					1.420		0.376

α;Triangle: 0.5, Square: 1.0
ｎ;　Number of Element

Y＝ ΣA・Y／ΣA　　＝　　0.265　m

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Rrgarding Each Member (m^4)

A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.00625	0.300	−0.335	0.112	0.0336	0.0399
②	0.01143	1.120	0.090	0.008	0.0090	0.0204
Total		1.420				0.0603

Area	**A=**	1.420	m^2
Second Moment	**Ix=**	0.0603	m^4

(3) Calculation of Load

1. Weight of Mail Body

(a) Wing Wall

A= 1.420 m^2

Vertical Force

Wc1= 1.420 × 24.0 kN/m^3 = 34.08 kN/m

(b) Sliceway (Breast Wall in River Side)

A= 7.206 7.206 m^2

Vertical Force

Wc2= 7.206 × 24.0 kN/m^3 = 172.94 kN/m

(c) Sliceway (Box Culvert)

A= 3.191 m^2

Vertical Force

Wc3= 3.191 × 24.0 kN/m^3 = 76.58 kN/m

(d) Sliceway (Breast Wall in Land Side)

A= 5.646 5.646 m^2

Vertical Force

Wc4= 5.646 × 24.0 kN/m^3 = 135.50 kN/m

(e) Weight of Impervious Wall

Vo= (4.7×3.35-2.7×2.35)×0.50 = 4.70 m^3

V= 4.70 × 24.0 kN/m^3 = 112.80 kN

2. Water Weight

Only Normal Condition

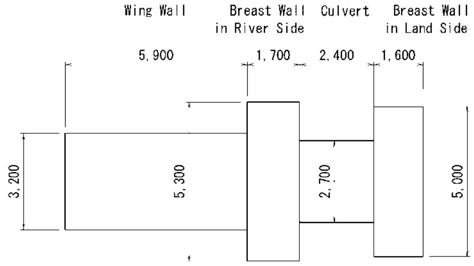
(a) Wing Wall

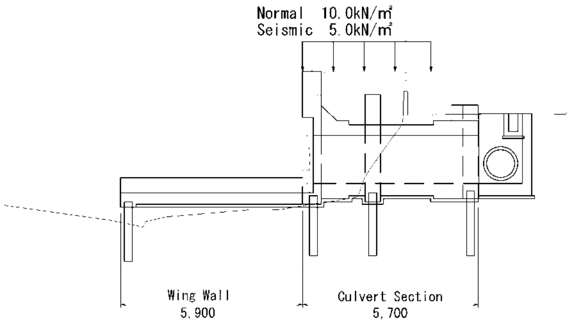
Wo1= 0.5×2.20×9.8 = 10.78 kN/m

(b) Sliceway Main Body

Wo2= (2.0×1.6-1/2×0.15×0.15×4)×9.8 = 30.92 kN/m

3. Vertical Load





Nomal Condition

q1= 10.0×5.3	=	53.00	kN/m
q2= 10.0×2.7	=	27.00	kN/m

Seismic Condition

q1= 5.0×5.3	=	26.50	kN/m
q2= 5.0×2.7	=	13.50	kN/m

4. Calculation of the Force Effecting on Breast Wall

(a) River Side (River ← Land, +)

Nomal Condition

V=	142.75	×	1.00	×	2	=	285.50	kN
H=	144.86	×	1.00	×	2	=	289.72	kN
M=	-61.75	×	1.00	×	2	=	-123.50	kNm
x=	0.780	m						

Seismic Condition, from Land side to River side

V=	166.26	×	1.00	×	2	=	332.52	kN
H=	111.90	×	1.00	×	2	=	223.80	kN
M=	-10.66	×	1.00	×	2	=	-21.32	kNm
x=	0.900	m						

Seismic Condition, from River side to Land side

V=	166.26	×	1.00	×	2	=	332.52	kN
H=	49.04	×	1.00	×	2	=	98.08	kN
M=	-72.00	×	1.00	×	2	=	-144.00	kNm
x=	0.900	m						

(b) Land Side (River → Land, +)

Nomal Condition

V=	92.80	×	1.00	×	2	=	185.60	kN
H=	113.24	×	1.00	×	2	=	226.48	kN
M=	-60.48	×	1.00	×	2	=	-120.96	kNm
x=	0.740	m						

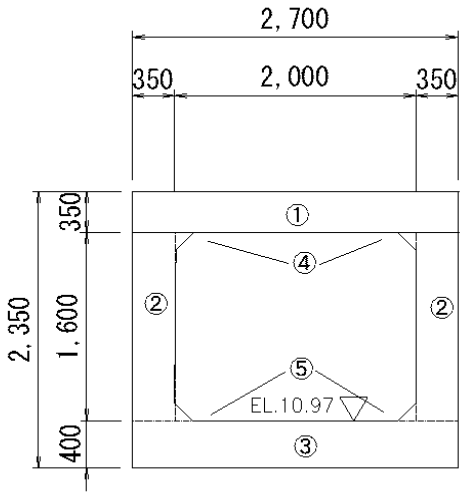
Seismic Condition, from Land side to River side

V=	115.02	×	1.00	×	2	=	230.04	kN
H=	97.47	×	1.00	×	2	=	194.94	kN
M=	-65.54	×	1.00	×	2	=	-131.08	kNm
x=	0.890	m						

Seismic Condition, from River side to Land side

V=	115.02	×	1.00	×	2	=	230.04	kN
H=	52.57	×	1.00	×	2	=	105.14	kN
M=	-87.72	×	1.00	×	2	=	-175.44	kNm
x=	0.890	m						

5. Centroid of Box Culvert



	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width (B)	Height (h)	α	n			
①	2.700	0.350	1.0	1	0.945	2.175	2.055
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.700	0.400	1.0	1	1.080	0.200	0.216
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
Total					3.191		3.670

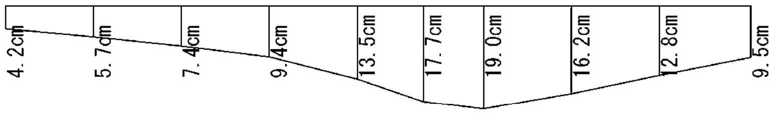
α; Triangle: 0.5, Square: 1.0

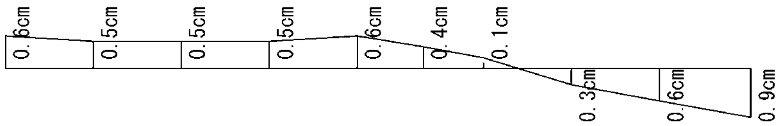
n; Number of Element

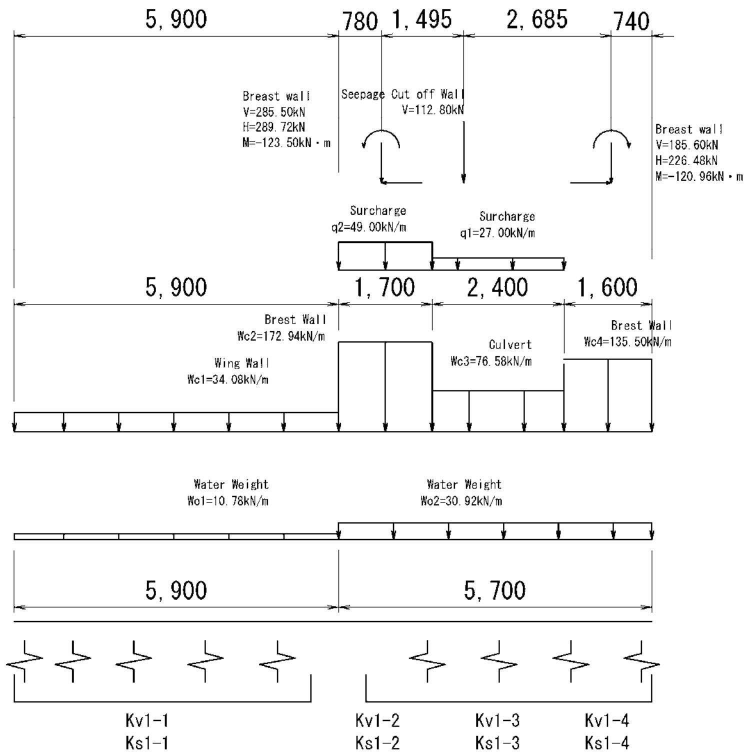
$Y = \Sigma A \cdot Y / \Sigma A \quad = \quad 1.150 \quad m$

(4) Summary of Load

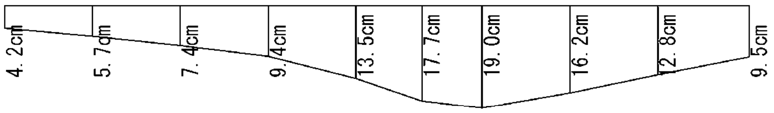
a) Normal Condition

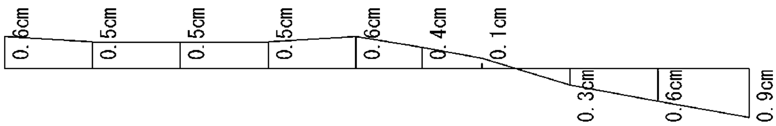


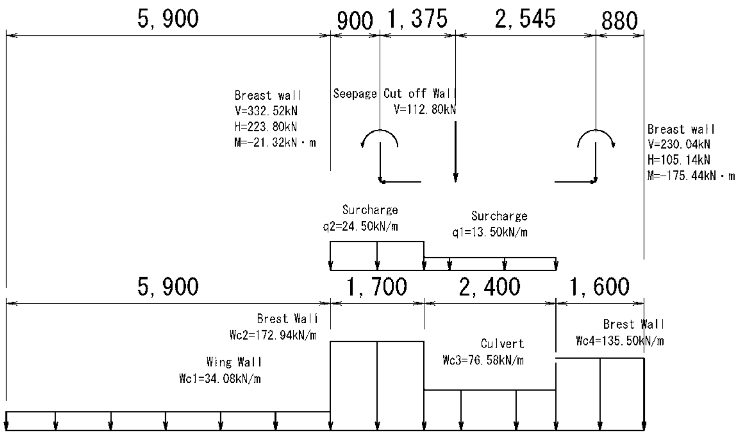


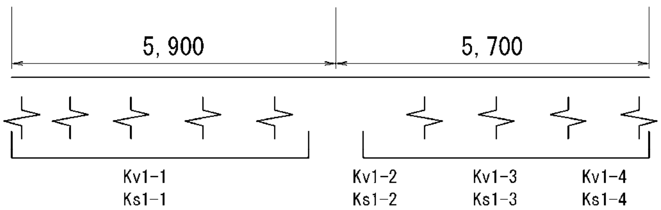


b) Seismic Condition (River Side ← Land Side)

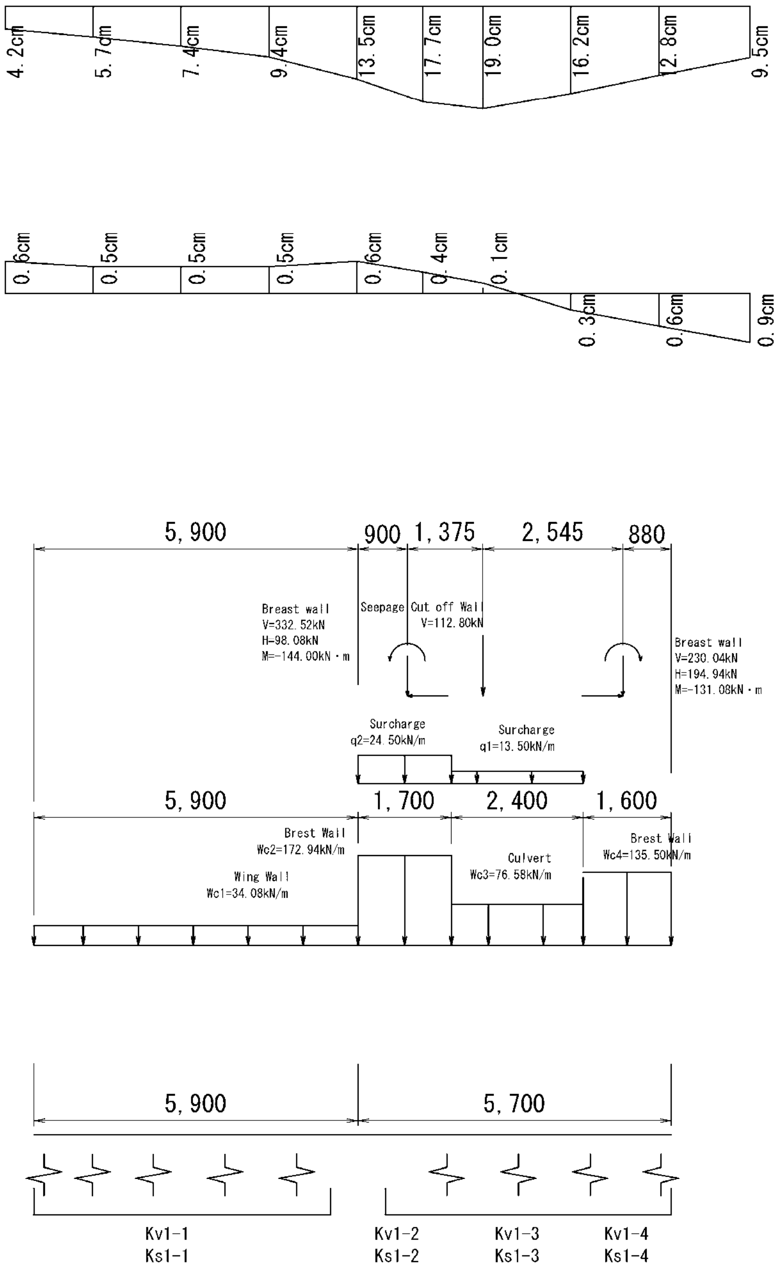




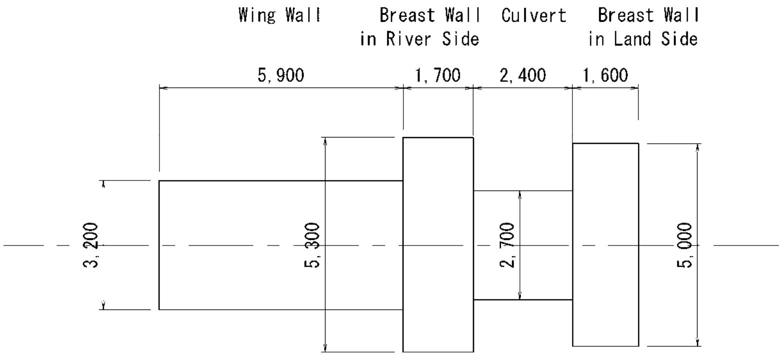




b) Seismic Condition (River Side → Land Side)



(5) Modulus of Subgrade Reaction and Modulus of Subgrade Reaction for Horizontal Shea



Results of Calculation

		Wing Walll	Breast Wall in River side	Culvert Section	Breast Wall in Lnd side
		1	2	3	4
Vertical Kv (kN/m^2)	Nomal	21661	50159	28288	49455
	Seismic	43322	100318	56576	98910
Horizontal Shear Ks(kN/m^2)	Nomal	7220	16720	9429	16485
	Seismic	14441	33439	18859	32970

a. Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvo : Coefficient of Vertical Reaction of soil (kN/m^3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E_{0m} obtained by various soil tests and investigations:

$$Kvo = \frac{1}{0.3} \quad \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eoп : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{ (Kv \cdot D) / (4 \cdot E \cdot I) \}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	√Av	β・L≦1.5
Elastic Body	√D/β	β・L＞1.5

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a) Wing Wall

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert $(kN \cdot m^2)$

E= 2.45E+07 kN/m^2

I= 0.0603 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

$= 1 / 0.3 \times 4 \times 3768.90 = 50252.0 \quad kN/m^3$

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

$= 0.24606$

$\beta \cdot L = 1.452 \leqq 1.5$ Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{5.90 \times 3.20} = \boxed{4.345}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{3.20 / 0.24606} = 3.606$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 50252.0 \times (4.345 / 0.3)^{-3/4} = 6769 \quad (kN/m^3)$

Coefficient of Reaction of Soil in Width of Foundatiuon

$kv = Kv \cdot D = 6769 \times 3.20 = \underline{21661} \quad (kN/m^2)$

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 13538 (kN/m^3) 〔 = 2 × 6769 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

$kv = Kv \cdot D = 13538 \times 3.20 = \underline{43322} \quad (kN/m^2)$

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

$Ks = \lambda \cdot kv$

$= 1/3 \times 21661$

$= \underline{7220} \quad (kN/m^2)$

◇ Seismic Condition

$Ks = \lambda \cdot kv$

$= 1/3 \times 43322$

$= \underline{14441} \quad (kN/m^2)$

(Wing wall)

Calculation Results of Converted Modulus of Deformation

$$E_{om} = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\,L}{(L+2h_n \cdot \tan\theta)\,B}}{\displaystyle\sum_{i=1}^{n} \frac{1}{E_i} \log \frac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f_n}{\displaystyle\sum_{i=1}^{n} \frac{1}{E_i} \times f_i}$$

Eom	;	Converted Modulus of Deformation in B≠L			(kN/m²)
B	;	Loading Width = Width of Wing Wall	B=	3.200	m
L	;	Loading Length = Length of a span	L=	5.900	m
h_n	;	Depth for Calculation	hn=	9.600	m
h_i	;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei	;	converted Modulus of Deformation of Each Layer (kN/m²)			
θ	;	Angle of Load Dispersion (θ=30°)			

	Eom (kN/m²)	B (m)	L (m)	h_n (m)	Tickness of Layer (m)	h i (m)	Ei (kN/m²)	θ (°)	f n	f i	1 / E i × f i
1	3768.90	3.200	5.900	9.600	3.590	3.590	4720	30	0.4387	0.299	0.0000633
2					1.500	5.090	1400			0.053	0.0000379
3					4.510	9.600	5710			0.087	0.0000152
4											
5											
6											
7											
8											
9											
10											
				Total	9.600					Total	0.0001164

b) Breast wall in River Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert (kN・m^2)

E= 2.45E+07 kN/m^2

I= 6.3287 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3993.43 = 53245.7 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.09483

β・L= 0.161 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{1.70 \times 5.30} = \boxed{3.002}$ m

Case2 Elastic body $Bv = \sqrt{D/\beta} = \sqrt{5.30 / 0.09483} = 7.476$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 53245.7 \times (3.002 / 0.3)^{-3/4} = 9464$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 9464 × 5.30 = 50159 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 18928 (kN/m^3) 〔 2 × 9464 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 18928 × 5.30 = 100318 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 50159

= 16720 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 100318

= 33439 (kN/m^2)

(Breast wall in River Side)
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ; Converted Modulus of Deformation in B≠L (kN/m^2)
B ; Loading Width = Width of Culvert B= 5.300 m
L ; Loading Length = Length of a span L= 1.700 m
h_n ; Depth for Calculation hn= 15.900 m
h_i ; Depth from the ground to the bottom Elevation of Each Layer (m)
Ei ; converted Modulus of Deformation of Each L (kN/m^2)
θ ; Angle of Load Dispersion (θ=30°)

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3993.43	5.300	1.700	15.900	3.590	3.590	4720	30	-0.9720	-0.657	-0.0001392
2					1.500	5.090	1400			-0.091	-0.0000650
3					10.810	15.900	5710			-0.224	-0.0000392
4						15.900	11900			0.000	0.0000000
5											
6											
7											
8											
				Total	15.900					Total	-0.0002434

c) Culvert Section

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN・m^2$)

E= 2.45E+07 kN/m^2

I= 2.2608 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3907.17 = 52095.6 kN/m^3

β= ｛（Kv・D）/(4・E・I)｝$^{1/4}$

= 0.10630

β・L= 0.255 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{2.40 \times 2.70} = \boxed{2.546}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{2.70 / 0.10630} = 5.040$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 52095.6 \times (2.546 / 0.3)^{-3/4} = 10477$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 10477 × 2.70 = 28288 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 20954 (kN/m^3) 〔 2 × 10477 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 20954 × 2.70 = 56576 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 28288

= 9429 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 56576

= 18859 (kN/m^2)

(Culvert Section)
Calculation Results of Converted Modulus of Deformation

$$Eom=\frac{\log \dfrac{(B+2h_n\cdot\tan\theta)\ L}{(L+2h_n\cdot\tan\theta)\ B}}{\sum_{i=1}^{n}\dfrac{1}{Ei}\log\dfrac{(B+2h_i\cdot\tan\theta)\ (L+2\ h_{i-1}\cdot\tan\theta)}{(L+2h_i\cdot\tan\theta)\ (B+2\ h_{i-1}\cdot\tan\theta)}}=\frac{f\,n}{\sum_{i=1}^{n}\dfrac{1}{Ei}\times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L		(kN/m^2)	
B ;	Loading Width = Width of Culvert	B=	2.700	m
L ;	Loading Length = Length of a span	L=	2.400	m
h_n ;	Depth for Calculation	hn=	8.100	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	converted Modulus of Deformation of Each L (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3907.17	2.700	2.400	8.100	3.590	3.590	4720	30	-0.0926	-0.073	-0.0000155
2					1.500	5.090	1400			-0.009	-0.0000064
3					3.010	8.100	5710			-0.010	-0.0000018
4											
5											
6											
7											
8											
				Total	8.100					Total	-0.0000237

d) Breast Wall in Land Side

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 4.6651 m^4

◇ Nomal Condition

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3990.99 = 53213.2 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.10198

β・L= 0.163 ≦ 1.5 Threrefore, this calvert is assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{1.60 \times 5.00} = \boxed{2.828}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{5.00 / 0.10198} = 7.002$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 53213.2 \times (2.828 / 0.3)^{-3/4} = 9891$ (kN/m^3)

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 9891 × 5.00 = 49455 (kN/m^2)

◇ Seismic Condition

In Seismic Condition, Kv is assumed as twise of the one in Nomal condition.

Kv= 19782 (kN/m^3) 〔 2 × 9891 (kN/m^3) 〕

Coefficient of Reaction of Soil in Width of Foundatiuon

kv= Kv・D = 19782 × 5.00 = 98910 (kN/m^2)

b. Coefficient of Lateral Shear Reaction of Soil

◇ Nomal Condition

Ks=λ・kv

= 1/3× 49455

= 16485 (kN/m^2)

◇ Seismic Condition

Ks=λ・kv

= 1/3× 98910

= 32970 (kN/m^2)

(Breast Wall in Land Side)
Calculation Results of Converted Modulus of Deformation

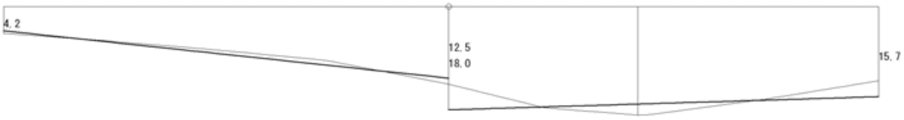
$$E_{om} = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{E_i} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{E_i} \times f\,i}$$

Eom ; Converted Modulus of Deformation in B≠L (kN/m^2)
B ; Loading Width = Width of Culvert B= 5.000 m
L ; Loading Length = Length of a span L= 1.600 m
h_n ; Depth for Calculation hn= 15.000 m
h_i ; Depth from the ground to the bottom Elevation of Each Layer (m)
Ei ; converted Modulus of Deformation of Each La (kN/m^2)
θ ; Angle of Load Dispersion (θ=30°)

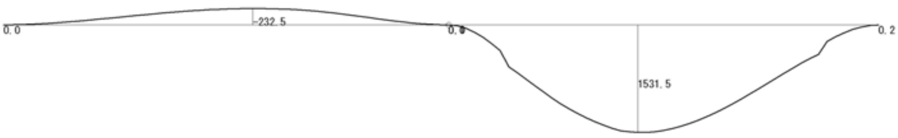
	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	3990.99	5.000	1.600	15.000	3.590	3.590	4720	30	-0.9742	-0.675	-0.0001430
2					1.500	5.090	1400			-0.090	-0.0000643
3					9.910	15.000	5710			-0.210	-0.0000368
4											
5											
6											
7											
8											
				Total	15.000					Total	-0.0002441

2 Cross Section Force

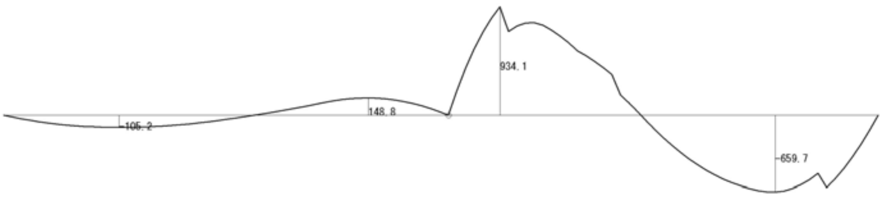
(1) Case 1 Normal Condition



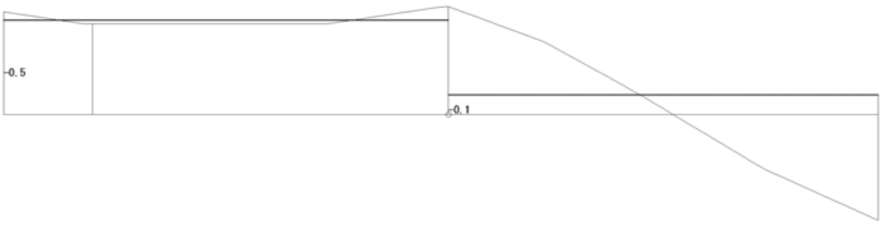
Settlement of Culvert and Ground (cm)



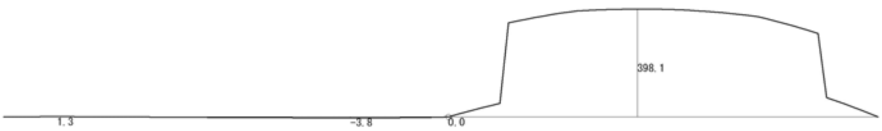
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

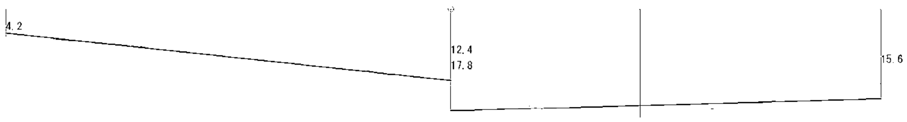


Axial Force (kN)

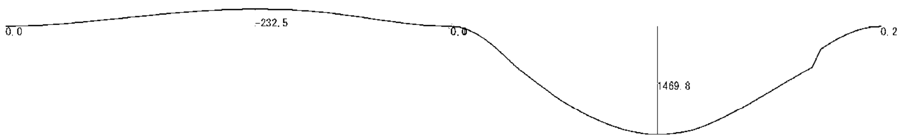
	1-SPAN X (m)	GRD-U (c m)	DIS-U (c m)	GRD-W (c m)	DIS-W (c m)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0. 000	-0. 6	-0. 5	4. 7	4. 2	0. 00	0. 00	0. 00	-98. 82
	1. 180	-0. 5	-0. 5	5. 8	5. 9	0. 92	-99. 85	-72. 57	16. 83
	2. 360	-0. 5	-0. 5	7. 1	7. 5	-0. 79	-81. 93	-189. 85	103. 25
B-M max	3. 304	-0. 5	-0. 5	8. 2	8. 9	-2. 16	-2. 09	-232. 55	147. 04
	3. 540	-0. 5	-0. 5	8. 5	9. 2	-2. 50	22. 94	-230. 13	154. 74
F-F max	4. 602	-0. 5	-0. 5	10. 2	10. 7	-3. 80	143. 28	-138. 87	97. 84
	4. 720	-0. 5	-0. 5	10. 5	10. 8	-3. 77	147. 78	-121. 67	68. 14
S-F max	4. 838	-0. 5	-0. 5	10. 8	11. 0	-3. 68	148. 78	-104. 14	38. 44
	5. 900	-0. 6	-0. 5	13. 6	12. 5	0. 00	0. 02	0. 06	-231. 74

	2-SPAN X (m)	GRD-U (c m)	DIS-U (c m)	GRD-W (c m)	DIS-W (c m)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0. 000	-0. 6	-0. 1	13. 6	18. 0	0. 00	0. 00	0. 00	2242. 23
S-F max	0. 684	-0. 5	-0. 1	15. 8	17. 8	49. 88	934. 07	368. 09	994. 69
	1. 140	-0. 4	-0. 1	17. 2	17. 6	366. 17	797. 22	863. 05	163. 00
	2. 280	-0. 2	-0. 1	18. 7	17. 1	397. 35	175. 19	1507. 55	-449. 01
F-F max	2. 508	-0. 1	-0. 1	18. 9	17. 0	398. 10	31. 83	1531. 54	-539. 51
B-M max	2. 508	-0. 1	-0. 1	18. 9	17. 0	398. 10	31. 83	1531. 54	-539. 51
	3. 420	0. 1	-0. 1	17. 5	16. 7	389. 14	-473. 50	1306. 93	-248. 08
	4. 560	0. 4	-0. 1	15. 4	16. 2	336. 65	-629. 45	616. 43	396. 03
	5. 700	0. 6	-0. 1	13. 0	15. 7	0. 00	0. 04	0. 22	1366. 71

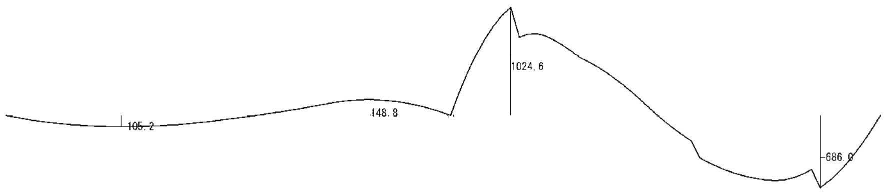
(2) Case 2 Seismic Condition (Land Side to River Side)



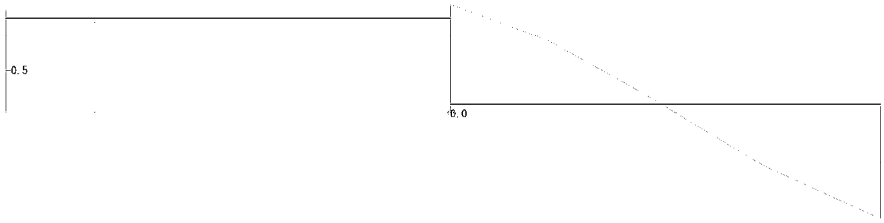
Settlement of Culvert and Ground (cm)



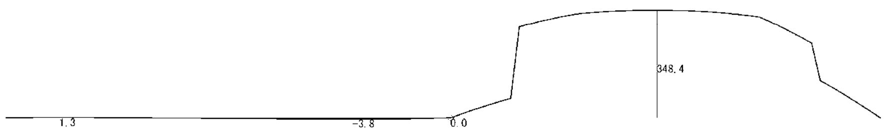
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)

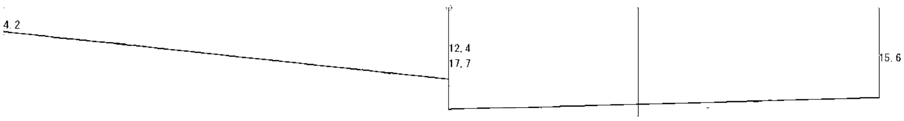


Axial Force (kN)

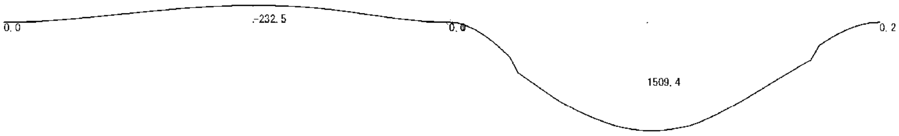
1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	-0.5	4.7	4.2	0.00	0.00	0.00	-109.60
	1.180	-0.5	-0.5	5.8	5.8	0.92	-99.85	-72.57	6.05
	2.360	-0.5	-0.5	7.1	7.5	-0.79	-81.93	-189.85	92.47
	3.540	-0.5	-0.5	8.5	9.1	-2.50	22.94	-230.13	143.96
	4.720	-0.5	-0.5	10.5	10.8	-3.77	147.78	-121.67	57.36
	5.900	-0.6	-0.5	13.6	12.4	0.00	0.02	0.06	-242.52

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	0.0	13.6	17.8	0.00	0.00	0.00	2208.72
	1.140	-0.4	0.0	17.2	17.4	308.55	775.80	765.07	130.71
	2.280	-0.2	0.0	18.7	16.9	346.17	292.36	1400.62	-471.28
	3.420	0.1	0.0	17.5	16.5	342.64	-443.40	1328.41	-269.51
	4.560	0.4	0.0	15.4	16.1	270.63	-583.96	684.74	364.21
	5.700	0.6	0.0	13.0	15.7	0.00	0.04	0.22	1334.87

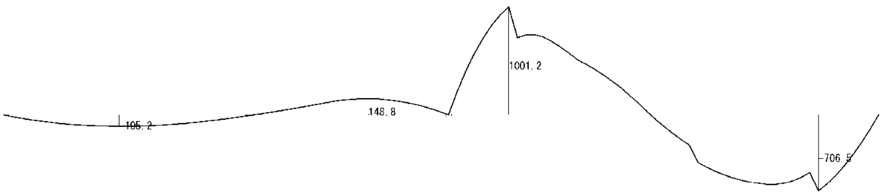
(3) Case 2-2 Seismic Condition (River Side to Land Side)



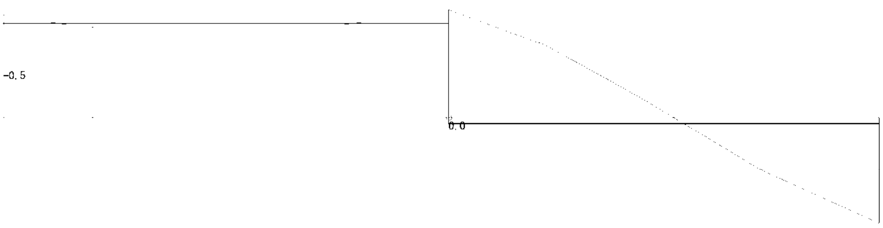
Settlement of Culvert and Ground (cm)



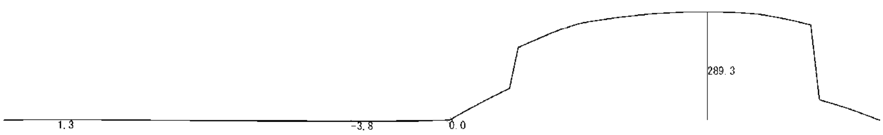
Bending Moment (kNm)



Shear Force (kN)



Horizontal Displacement of Culvert and Ground (cm)



Axial Force (kN)

1-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	-0.5	4.7	4.2	0.00	0.00	0.00	-109.60
	1.180	-0.5	-0.5	5.8	5.8	0.92	-99.85	-72.57	6.05
	2.360	-0.5	-0.5	7.1	7.5	-0.79	-81.93	-189.85	92.47
	3.540	-0.5	-0.5	8.5	9.1	-2.50	22.94	-230.13	143.96
	4.720	-0.5	-0.5	10.5	10.8	-3.77	147.78	-121.67	57.36
	5.900	-0.6	-0.5	13.6	12.4	0.00	0.02	0.06	-242.52

2-SPAN	X (m)	GRD-U (cm)	DIS-U (cm)	GRD-W (cm)	DIS-W (cm)	F-FORCE (kN)	S-FORCE (kN)	B-MOMENT (kN·m)	REACTION (kN/m)
	0.000	-0.6	0.1	13.6	17.8	0.00	0.00	0.00	2174.92
	1.140	-0.4	0.1	17.2	17.3	213.72	744.60	868.57	109.77
	2.280	-0.2	0.1	18.7	16.9	275.39	249.99	1459.39	-471.73
	3.420	0.1	0.1	17.5	16.5	289.29	-482.64	1339.97	-263.57
	4.560	0.4	0.1	15.4	16.1	265.58	-611.03	657.18	381.61
	5.700	0.6	0.1	13.0	15.7	0.00	0.04	0.22	1364.95

3 Stress Calculation

		Case1 Normal Condition		Case2-1 Seismic Condition (L to R)		Case2-2 Seismic Condition (R to L)	
		M_{max}	S_{max}	M_{max}	S_{max}	M_{max}	S_{max}
MSR3	Tensile Side	Bottom	-	Bottom	-	Bottom	-
	M (kNm)	1531.5	-	1469.8	-	1509.4	-
	S (kN)	31.8	934.07	292.4	1024.6	250.0	1001.2
	N (kN)	398.1	-	346.2	-	275.4	-
	Re-Bar	D16 x 24	-	D16 x 24	-	D16 x 24	-
	Area (m^2)	48.3	-	48.3	-	48.3	-
	σ_c (N/mm^2)	2.39	-	2.29	-	2.35	-
	σ_s (N/mm^2)	125.0	-	122.5	-	131.5	-
	τ (N/mm^2)	0.01	0.20	0.12	0.22	0.11	0.21
	σ_{ca} (N/mm^2)	8.2	8.2	12.3	12.3	12.3	12.3
	σ_{sa} (N/mm^2)	140.0	140.0	210.0	210.0	210.0	210.0
	τa (N/mm^2)	0.36	0.36	0.54	0.54	0.54	0.54
	Evaluation	OK	OK	OK	OK	OK	OK

L : Land Side
R : River Side

(1) Case1 Normal Condition (Mmax)

cgs		SI	
M	=	1531.5	kNm
S	=	31.83	kN
N	=	398.1	kN
b	=	270	cm
H	=	235	cm
t	=	35	cm
bw	=	70	cm
d	=	213	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	8.20	N/mm^2
σsa	=	140	N/mm^2
τca	=	0.36	N/mm^2

	dia.	piece	cm2
Re-BarAs	16	24	48.264
As'	12	0	0

$$y_1 = \frac{\dfrac{b \cdot t^2}{2} + \dfrac{b_w}{2} \cdot (h-t) \cdot (h+t)}{b \cdot t + b_w \cdot (h-t)}$$

$\dfrac{b \cdot t^2}{2}$	165375 cm^3
$\dfrac{b_w}{2} \cdot (h-t) \cdot (h+t)$	1890000 cm^3
$b \cdot t + b_w \cdot (h-t)$	23450 cm^2

$$x^3 + 3 \cdot e' \cdot x^2 + \frac{3}{b_w} \cdot \left[t \cdot (b-b_w) \cdot (t + 2 \cdot e') + 2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\} \right] \cdot x$$
$$- \frac{3 \cdot (b-b_w) \cdot t^2}{b_w} \cdot \left(\frac{2}{3} \cdot t + e' \right) - \frac{6 \cdot n}{b_w} \left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s' \right] = 0$$

$\dfrac{3}{b_w}$	0.04 cm
$t \cdot (b-b_w) \cdot (t + 2 \cdot e')$	4403743.15
$2 \cdot n \cdot \{(d+e') \cdot A_s + (d'+e') \cdot A_s'\}$	443109.64
$-\dfrac{3 \cdot (b-b_w) \cdot t^2}{b_w}$	-10500
$\left(\dfrac{2}{3} \cdot t + e' \right)$	320.39
$-\dfrac{6 \cdot n}{b_w}$	-0.771428571
$\left[d \cdot (d+e') \cdot A_s + d' \cdot (e'+d') \cdot A_s' \right]$	5243464.02
$\dfrac{b \cdot x^2}{2} - \dfrac{(b-b_w)}{2} \cdot (x-t)^2$	131012.79
$n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}$	-78920.28

e0 = 4 m
y1 = 88 cm
e' = 297 cm

x^3+ 891 x^2 + 207722 x + -7409015 = y

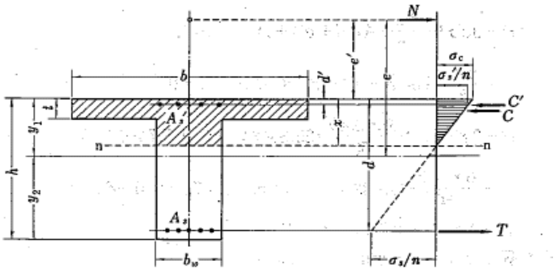
x = 31.31 cm
y = 0.00 OK

$$\sigma_c = \frac{N \cdot x}{\dfrac{b \cdot x^2}{2} - \dfrac{(b-b_w)}{2} \cdot (x-t)^2 + n \cdot \{-(d-x) \cdot A_s + (x-d') \cdot A_s'\}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

$$\sigma_s' = n \cdot \sigma_c \cdot \frac{x-d'}{x}$$

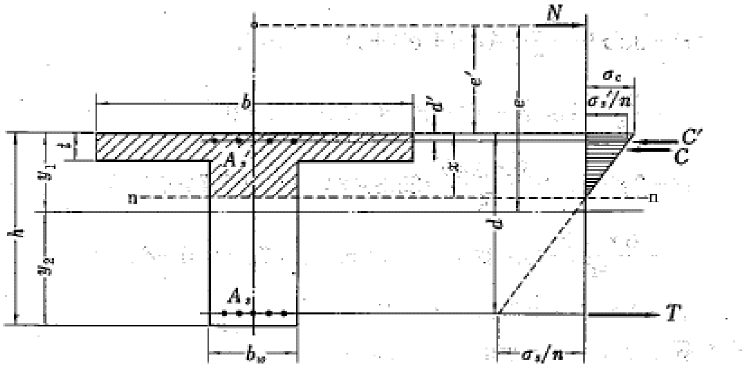
σc =	2.39	N/mm^2	OK
σs =	125	N/mm^2	OK
τc =	0.01	N/mm^2	OK



(2) Case1 Normal Condition (Smax)

cgs		SI	
M	=	-	kNm
S	=	934.07	kN
N	=	-	kN
b	=	370	cm (Breast Wall Riverside Section)
H	=	270	cm
t	=	50	cm
bw	=	130	cm
d	=	243	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	8.20	N/mm^2
σsa	=	140	N/mm^2
τca	=	0.36	N/mm^2

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.20	N/mm^2	OK



(3) Case2-1 Seisimic Condition (Mmax)

	cgs		SI	
	M	=	1469.8	kNm
	S	=	292.36	kN
	N	=	346.17	kN
	b	=	270	cm
	H	=	235	cm
	t	=	35	cm
	bw	=	70	cm
	d	=	213	cm
	d'	=	0	cm
	n (Es/Ec)	=	9	
	σca	=	12.30	N/mm^2
	σsa	=	210.00	N/mm^2
	τca	=	0.54	N/mm^2

	dia.	piece	cm2
Re-BarAs	16	24	48.264
As'	12	0	0

e0 = 4 m
y1 = 88 cm
e' = 337 cm

x^3+ 1011 x^2 + 233139 x + -8144146 = y

x = 30.72 cm
y = 0.00 OK

$$y_1 = \frac{\frac{b\cdot t^2}{2} + \frac{b_w}{2}\cdot(h-t)\cdot(h+t)}{b\cdot t + b_w\cdot(h-t)}$$

$\frac{b\cdot t^2}{2}$	165375	cm^3
$\frac{b_w}{2}\cdot(h-t)\cdot(h+t)$	1890000	cm^3
$b\cdot t + b_w\cdot(h-t)$	23450	cm^2

$$x^3 + 3\cdot e'\cdot x^2 + \frac{3}{b_w}\cdot\left[t\cdot(b-b_w)\cdot(t+2\cdot e') + 2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}\right]\cdot x$$
$$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}\cdot\left(\frac{2}{3}\cdot t + e'\right) - \frac{6\cdot n}{b_w}\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right] = 0$$

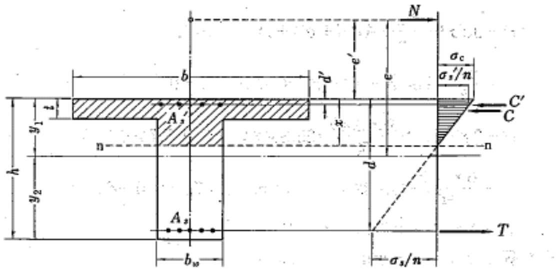
$\frac{3}{b_w}$	0.04	cm
$t\cdot(b-b_w)\cdot(t+2\cdot e')$	4962157	
$2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}$	477761	
$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}$	-10500	
$\left(\frac{2}{3}\cdot t + e'\right)$	360	
$-\frac{6\cdot n}{b_w}$	-0.771428571	
$\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right]$	5653509	
$\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2$	125545	
$n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}$	-79179	

$$\sigma_c = \frac{N\cdot x}{\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2 + n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

$$\sigma_s' = n\cdot\sigma_c\cdot\frac{x-d'}{x}$$

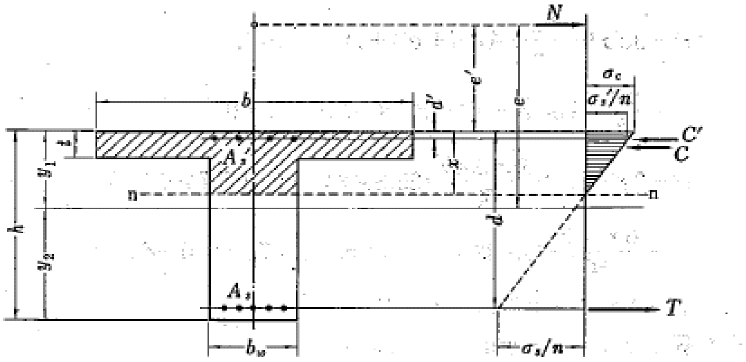
σc =	2.29	N/mm^2	OK
σs =	122	N/mm^2	OK
τc =	0.12	N/mm^2	OK



(4) Case2-1 Seisimic Condition (Smax)

cgs		SI		
M	=	-	kNm	
S	=	1024.6	kN	
N	=	-	kN	
b	=	370	cm	(Breast Wall Riverside Section)
H	=	270	cm	
t	=	50	cm	
bw	=	130	cm	
d	=	243	cm	
d'	=	0	cm	
n (Es/Ec)	=	9		
σca	=	12.30	N/mm^2	
σsa	=	210.00	N/mm^2	
τca	=	0.54	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.22	N/mm^2	OK



(5) Case2-2 Seisimic Condition (Mmax)

cgs		SI	
M	=	1509.4	kNm
S	=	249.99	kN
N	=	275.39	kN
b	=	270	cm
H	=	235	cm
t	=	35	cm
bw	=	70	cm
d	=	213	cm
d'	=	0	cm
n (Es/Ec)	=	9	
σca	=	12.30	N/mm^2
σsa	=	210.00	N/mm^2
τca	=	0.54	N/mm^2

$$y_1 = \frac{\frac{b\cdot t^2}{2} + \frac{b_w}{2}\cdot(h-t)\cdot(h+t)}{b\cdot t + b_w\cdot(h-t)}$$

$\frac{b\cdot t^2}{2}$	165375	cm^3
$\frac{b_w}{2}\cdot(h-t)\cdot(h+t)$	1890000	cm^3
$b\cdot t+b_w\cdot(h-t)$	23450	cm^2

$$x^3 + 3\cdot e'\cdot x^2 + \frac{3}{b_w}\cdot\left[t\cdot(b-b_w)\cdot(t+2\cdot e') + 2\cdot n\cdot\{(d+e')\cdot A_s + (d'+e')\cdot A_s'\}\right]\cdot x$$
$$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}\cdot\left(\frac{2}{3}\cdot t+e'\right) - \frac{6\cdot n}{b_w}\left[d\cdot(d+e')\cdot A_s + d'\cdot(e'+d')\cdot A_s'\right] = 0$$

	dia.	piece	cm2
Re-BarAs	16	24	48.264
As'	12	0	0

$\frac{3}{b_w}$	0.04	cm
$t\cdot(b-b_w)\cdot(t+2\cdot e')$	6691246	
$2\cdot n\cdot\{(d+e')\cdot A_s+(d'+e')\cdot A_s'\}$	585058	
$-\frac{3\cdot(b-b_w)\cdot t^2}{b_w}$	-10500	
$\left(\frac{2}{3}\cdot t+e'\right)$	484	
$-\frac{6\cdot n}{b_w}$	-0.771428571	
$\left[d\cdot(d+e')\cdot A_s+d'\cdot(e'+d')\cdot A_s'\right]$	6923183	
$\frac{b\cdot x^2}{2}-\frac{(b-b_w)}{2}\cdot(x-t)^2$	114306	
$n\cdot\{-(d-x)\cdot A_s+(x-d')\cdot A_s'\}$	-79715	

e0 = 5 m

y1 = 88 cm

e' = 460 cm

x^3+ 1381 x^2 + 311842 x + -10420426 = y

x = 29.48 cm

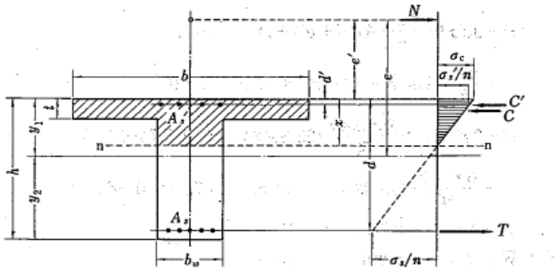
y = 0.00 OK

$$\sigma_c = \frac{N\cdot x}{\frac{b\cdot x^2}{2} - \frac{(b-b_w)}{2}\cdot(x-t)^2 + n\cdot\{-(d-x)\cdot A_s + (x-d')\cdot A_s'\}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

$$\sigma_s' = n\cdot\sigma_c\cdot\frac{x-d'}{x}$$

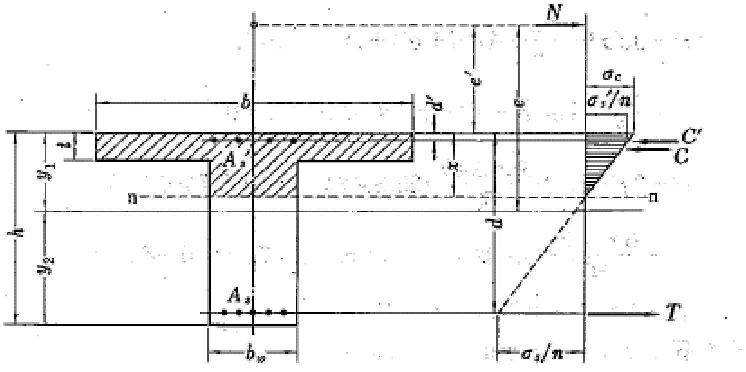
σc =	2.35	N/mm^2	OK
σs =	131	N/mm^2	OK
τc =	0.11	N/mm^2	OK



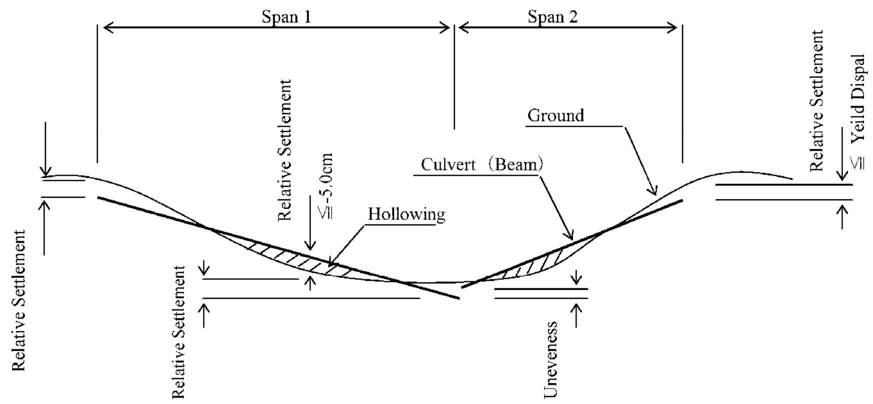
(6) Case2-2 Seismic Condition (Smax)

cgs		SI		
M	=	-	kNm	
S	=	1001.2	kN	
N	=	-	kN	
b	=	370	cm	(Brest wall Riverside Section)
H	=	270	cm	
t	=	50	cm	
bw	=	130	cm	
d	=	243	cm	
d'	=	0	cm	
n (Es/Ec)	=	9		
σca	=	12.30	N/mm^2	
σsa	=	210.00	N/mm^2	
τca	=	0.54	N/mm^2	

σc =	-	N/mm^2	
σs =	-	N/mm^2	
τc =	0.21	N/mm^2	OK



4. Relative Displacement （Stability Against Bearing）



Positive Relative Displacement

Amount which culvert dent into ground : Yielding Displacement of Ground

Yielding Displacement of Grc ≦ 5.0 cm

≦ 1.0% of Width of Foundation

	① Width of Foundation (cm)	② 1 % of ① (cm)				Allowable Relative Displacement (c m)
River Side	530.0	5.3	＞	5.0	cm	5.0
Land Side	500.0	5.0	＞	5.0	cm	5.0

Negative Relative Displacement

Amount of Hollowing ≦ -5.0 cm

	X (m)	Case 1 Normal Condition				
		Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
River Side	0.000	13.6	18.0	4.4	5.0	ok
	2.508	18.9	17.0	-1.9	-5.0	ok
Land Side	5.700	13.0	15.7	2.7	5.0	ok

	X (m)	Case 2-1,-2 Seismic Condition				
		Ground Settlement ① GRD-W(cm)	Culvert Settlement ② DIS-W(cm)	Relative Displacement ②-① (cm)	Allowable Relative Displacement (cm)	Evaluation
River Side	0.000	13.6	17.8	4.2	5.0	ok
	2.280	18.7	16.9	-1.8	-5.0	ok
Land Side	5.700	13.0	15.7	2.7	5.0	ok

11.3 Flexible Joint

The detail of design calculation of flexible joint is indicated from the following page.

Design Calculation of Flexible Joint

Displacement of joint is express by relative displacement between culvert which is next to each other. And it is difined as opening ⊿u, uneveness ⊿w and bend angle ⊿ θ , which come from the results of structural calculation of box culvert in longitudinal direction..

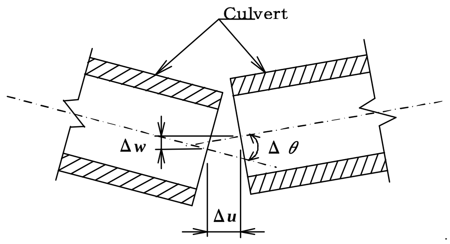


Figure 1 Displacement of Joint

Capability of flexible Joint is expressed by settlement and expansion.
These 2 values inversely relate. In case that settlement and expansion occurs at the same time, capability of flexible joint is the range which is shown in Figure 2.

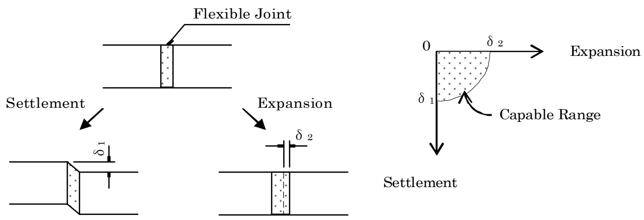


Figure 2 Capability of Flexible Joint

Opening between culverts is converted to settlement, and unevenness is converted to expansion.
Ability of flexible joint is evaluated based on these 2 elements.
Angle Δθ is converted to opening Δu_θ by the following formula.

$$\Delta u_\theta = 2 \times H \tan \frac{\Delta\theta}{2}$$

Δu_θ : Converted Opening (cm)
H : Height of Culvert (cm)

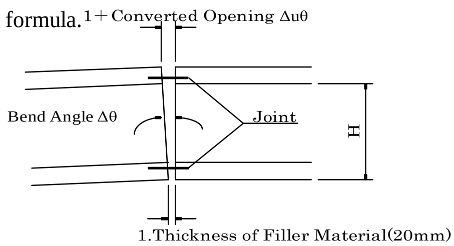


Figure 3 Converted opening by Bend Angel

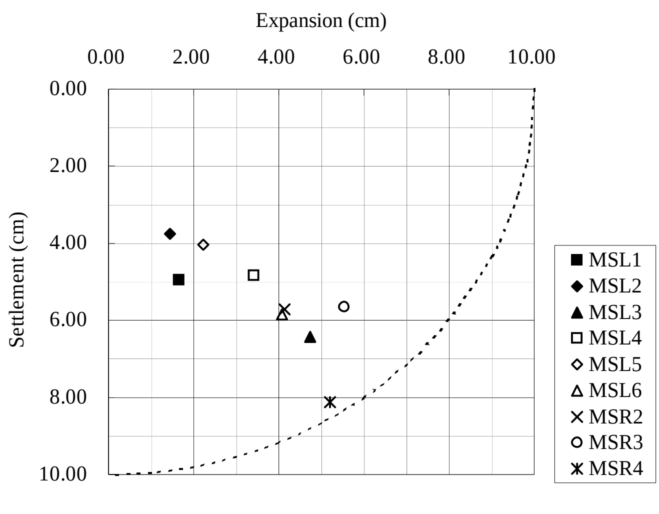


Figure 4 Relation between Expansion and Settlement

Allowable expansion and settlement of flexible joint : 100 mm

From the Figure above, expansion and settlement is less than allowable value.
Hence, Flexible Joint for 100mm settlemet would be applied.

Table 1 Calculation Results

	H(cm)	Case	Opening	Unevenness	Bend Angle		Settlement	Expansion
					Angle	Opening		
			Δu' (cm)	Δw' (cm)	Δθ(rad)	Δuθ (cm)	Δw (cm)	Δu (cm)
MSL1	180.0	Normal	0.28	-1.75	-0.0149	4.68	1.75	4.96
		Seismic (→)	0.29	-1.66	-0.0148	4.66	1.66	4.95
		Seismic (←)	0.37	-1.64	-0.0147	4.65	**1.64**	**5.02**
MSL2	190.0	Normal	0.44	-1.61	-0.0069	3.24	1.61	3.68
		Seismic (→)	0.43	-1.56	-0.0070	3.26	1.56	3.69
		Seismic (←)	0.59	-1.44	-0.0065	3.17	**1.44**	**3.76**
MSL3	155.0	Normal	0.41	-4.73	-0.0223	6.01	**4.73**	**6.42**
		Seismic (→)	0.42	-4.59	-0.0219	5.94	4.59	6.36
		Seismic (←)	0.55	-4.50	-0.0218	5.92	4.50	6.47
MSL4	205.0	Normal	0.53	-3.42	-0.0128	4.30	**3.42**	**4.83**
		Seismic (→)	0.43	-3.22	-0.0124	4.23	3.22	4.66
		Seismic (←)	0.68	-3.17	-0.0122	4.20	3.17	4.88
MSL5	135.0	Normal	-0.08	-2.38	-0.0102	3.84	2.38	3.76
		Seismic (→)	0.13	-2.26	-0.0100	3.80	2.26	3.93
		Seismic (←)	0.28	-2.22	-0.0098	3.76	**2.22**	**4.04**
MSL6	150.0	Normal	0.35	-4.08	-0.0194	5.49	**4.08**	**5.84**
		Seismic (→)	0.37	-3.88	-0.0189	5.40	3.88	5.77
		Seismic (←)	0.63	-3.38	-0.0186	5.35	3.38	5.98
MSR2	180.0	Normal	0.44	4.02	-0.0179	5.22	4.02	5.66
		Seismic (←)	0.42	4.14	-0.0183	5.29	**4.14**	**5.71**
		Seismic (→)	-0.59	4.10	-0.0182	5.28	4.10	4.69
MSR3	200.0	Normal	0.41	5.54	-0.0180	5.24	**5.54**	**5.65**
		Seismic (←)	0.47	5.33	-0.0178	5.20	5.33	5.67
		Seismic (→)	0.56	5.30	-0.0177	5.19	5.30	5.75
MSR4	190.0	Normal	0.65	5.21	-0.0304	7.47	**5.21**	**8.12**
		Seismic (←)	0.66	5.00	-0.0303	7.45	5.00	8.11
		Seismic (→)	0.82	4.86	0.0297	7.35	4.86	8.17

*Note : **Bold Type** is indicated value in **Figure 4***

11.4 SSP with Flexible Joint

The capability of SSP with flexible joint is shown in **Table R 11.4.1** in accordance with the difference of settlement between sluiceway site and dike. The difference of settlement in each sluiceway site is indicated from the following page

Table R 11.4.1 Capability of SSP with flexible joint

Sluiceway No.	Breast Wall in River Side (mm)	Seepage Cut Off Wall (mm)	Breast Wall in Land Side (mm)	Remarks
MSL1	100	100	100	
MSL2	100	100	100	
MSL3	100	100	100	
MSL4	200	200	200	
MSL5	100	100	100	
MSL6	100	100	100	
MSR2	100	200	100	
MSR3	100	200	100	
MSR4	200	200	200	

Note: Capacity is the deference of settlement

Design of SSP with Flexible Joint

(1) MSL-1

Residual Settlement of Dike

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	5.80	6.80	8.40	10.00	11.60	13.20
Immediate Settlement(cm)	3.40	4.60	5.50	5.80	5.70	5.30	4.30	3.70	3.10	2.30
Consolidation Settlement(cm)	1.93	3.05	4.35	5.41	5.97	9.07	5.68	2.36	1.32	0.79
Residual Settlement ΣS (cm)	**5.3**	**7.6**	**9.8**	**11.2**	**11.7**	**14.4**	**10.0**	**6.1**	**4.4**	**3.1**

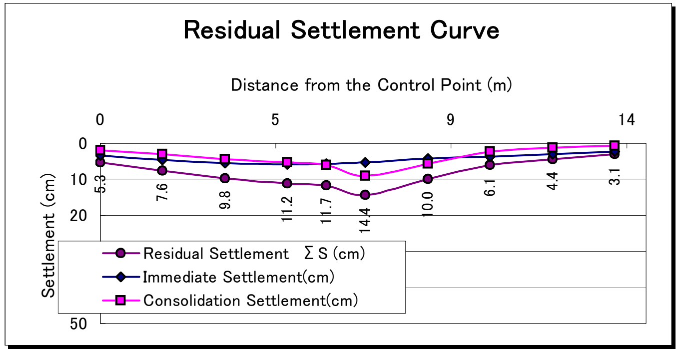
Settlement during embankment work, 90days

→ After 90 days, dike is excaved and suluiceway is installed.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	5.80	6.80	8.40	10.00	11.60	13.20
Immediate Settlement(cm)	3.40	4.60	5.50	5.80	5.70	5.30	4.30	3.70	3.10	2.30
Consolidation Settlement(cm)	1.24	1.86	2.54	3.14	3.43	5.35	3.64	1.68	0.88	0.54
Residual Settlement ΣS (cm)	**4.6**	**6.5**	**8.0**	**8.9**	**9.1**	**10.6**	**7.9**	**5.4**	**4.0**	**2.8**

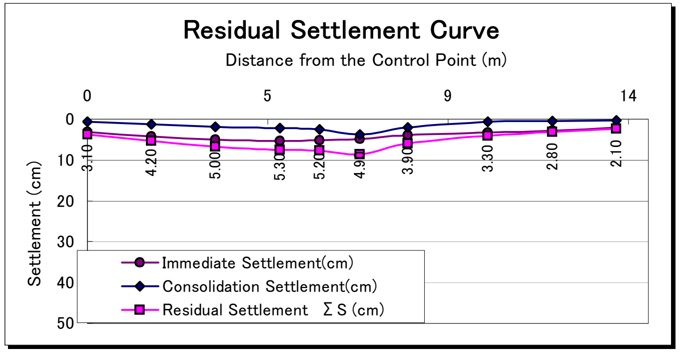
Settlement of Dike After Embankment Work

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	5.80	6.80	8.40	10.00	11.60	13.20
Immediate Settlement(cm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)	0.70	1.19	1.81	2.27	2.54	3.72	2.04	0.68	0.44	0.25
Residual Settlement ΣS (cm)	**0.7**	**1.2**	**1.8**	**2.3**	**2.5**	**3.7**	**2.0**	**0.7**	**0.4**	**0.2**



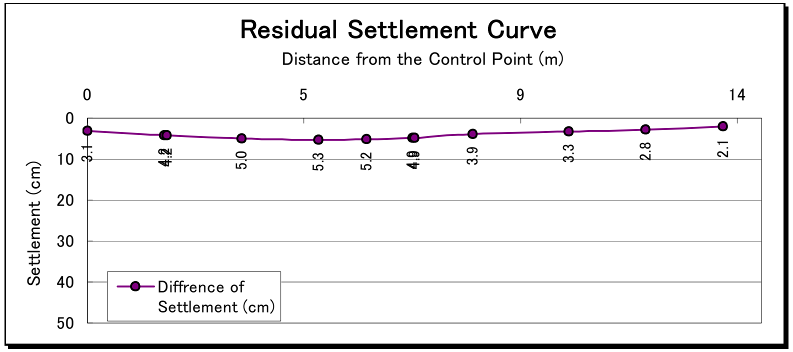
Residual Settlment at Sluiceway Site

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	5.80	6.80	8.00	10.00	11.60	13.20
Immediate Settlement(cm)	3.10	4.20	5.00	5.30	5.20	4.90	3.90	3.30	2.80	2.10
Consolidation Settlement(cm)	0.70	1.19	1.81	2.27	2.54	3.72	2.04	0.68	0.44	0.25
Residual Settlement ΣS (cm)	**3.8**	**5.4**	**6.8**	**7.6**	**7.7**	**8.6**	**5.9**	**4.0**	**3.2**	**2.3**



Difference of Settlment

X (m) Location of Caluculation	Control Point (1)	(2)	Breast Wall in Land SIde SSP	(3)	Seepage Cut Off Wall SSP (4)	(5)	Breast Wall in River Side SSP	(6)	(7)	(8)	(9)	(10)
	0.00	1.60	1.65	3.20	4.80	5.80	6.75	6.80	8.00	10.00	11.60	13.20
Diffrence of Settlement (cm)	**3.1**	**4.2**	**4.2**	**5.0**	**5.3**	**5.2**	**4.9**	**4.9**	**3.9**	**3.3**	**2.8**	**2.1**
			↓		↓		↓					
Applied Capability of SSP with Flexible Joint (cm)			**10.0**		**10.0**		**10.0**					



Design of SSP with Flexible Joint
(2) MSL-2

Residual Settlement of Dike

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.70	3.30	4.90	5.90	7.00	8.60	10.20	11.80	13.40
Immediate Settlement(cm)	7.10	9.50	10.30	10.40	10.10	9.20	7.40	6.20	5.20	3.80
Consolidation Settlement(cm)	1.44	1.44	1.37	1.27	1.19	1.09	0.82	0.66	0.59	0.40
Residual Settlement ΣS (cm)	**8.5**	**10.9**	**11.7**	**11.7**	**11.3**	**10.3**	**8.2**	**6.9**	**5.8**	**4.2**

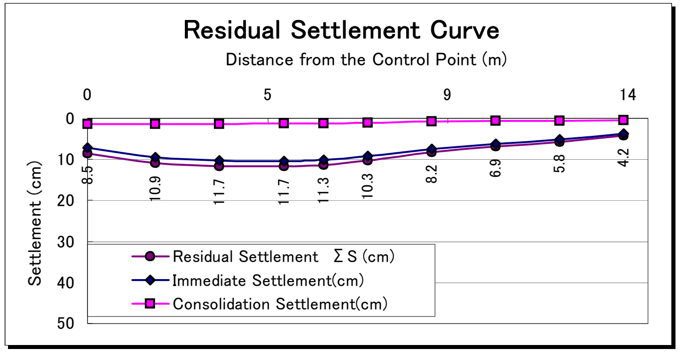
Settlement During Embankment Work, 90days

→ After 90 days, dike is excaved and suluiceway is installed.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.70	3.30	4.90	5.90	7.00	8.60	10.20	11.80	13.40
Immediate Settlement(cm)	7.10	9.50	10.30	10.40	10.10	9.20	7.40	6.20	5.20	3.80
Consolidation Settlement(cm)	0.40	0.40	0.38	0.35	0.33	0.31	0.24	0.20	0.15	0.12
Residual Settlement ΣS (cm)	**7.5**	**9.9**	**10.7**	**10.8**	**10.4**	**9.5**	**7.6**	**6.4**	**5.3**	**3.9**

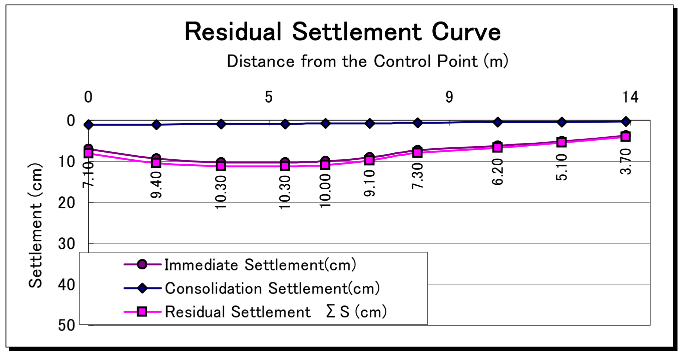
Settlement of Dike After Embankment Work

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.70	3.30	4.90	5.90	7.00	8.60	10.20	11.80	13.40
Immediate Settlement(cm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)	1.04	1.04	0.99	0.92	0.86	0.79	0.58	0.46	0.45	0.28
Residual Settlement ΣS (cm)	**1.0**	**1.0**	**1.0**	**0.9**	**0.9**	**0.8**	**0.6**	**0.5**	**0.4**	**0.3**



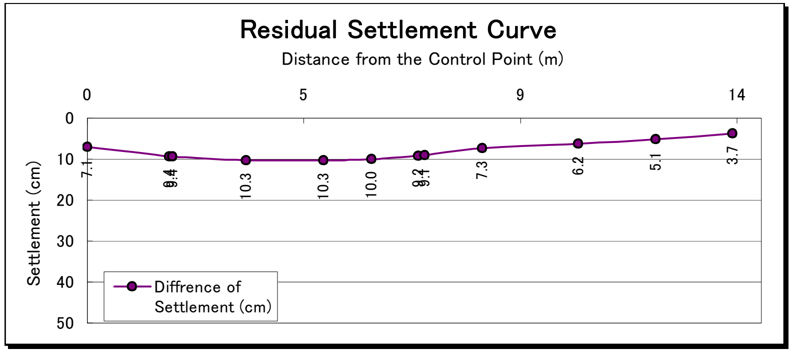
Residual Settlment at Sluiceway Site

	Control Point			SSP						
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.70	3.30	4.90	5.90	7.00	8.20	10.20	11.80	13.40
Immediate Settlement(cm)	7.10	9.40	10.30	10.30	10.00	9.10	7.30	6.20	5.10	3.70
Consolidation Settlement(cm)	1.04	1.04	0.99	0.92	0.86	0.79	0.58	0.46	0.45	0.28
Residual Settlement ΣS (cm)	**8.1**	**10.4**	**11.3**	**11.2**	**10.9**	**9.9**	**7.9**	**6.7**	**5.5**	**4.0**



Difference of Settlment

	Control Point		Breast Wall in Land SIde SSP		Seepage Cut Off Wall SSP		Breast Wall in River Side SSP					
Location of Caluculation	(1)	(2)		(3)	(4)	(5)		(6)	(7)	(8)	(9)	(10)
X (m)	0.00	1.70	1.77	3.30	4.90	5.90	6.87	7.00	8.20	10.20	11.80	13.40
Diffrence of Settlement (cm)	**7.1**	**9.4**	**9.4**	**10.3**	**10.3**	**10.0**	**9.2**	**9.1**	**7.3**	**6.2**	**5.1**	**3.7**
			↓		↓		↓					
Applied Capability of SSP with Flexible Joint (cm)			**10.0**		**10.0**		**10.0**					



Design of SSP with Flexible Joint
(3) MSL-3

Residual Settlement of Dike

Location of Calculation		Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	0.00	1.50	3.00	4.50	5.40	6.50	8.00	9.50	11.00	12.50
Immediate Settlement(cm)		5.60	7.80	9.30	9.60	9.50	8.30	5.80	4.50	3.40	2.40
Consolidation Settlement(cm)		4.02	7.21	11.23	14.45	14.31	12.01	7.25	3.81	2.07	1.22
Residual Settlement ΣS (cm)		**9.6**	**15.0**	**20.5**	**24.0**	**23.8**	**20.3**	**13.0**	**8.3**	**5.5**	**3.6**

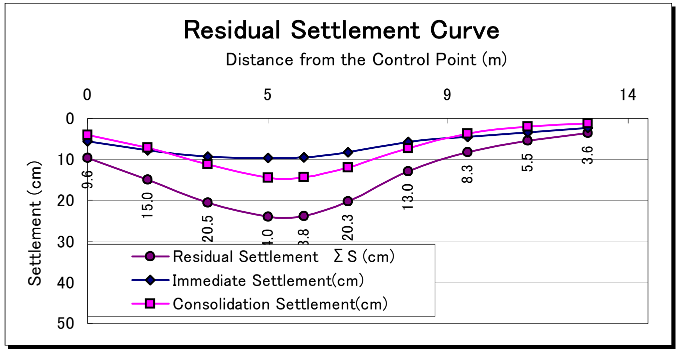
Settlement During Embankment Work, 105days

→ **After 105 days, dike is excaved and suluiceway is installed.**

Location of Calculation		Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	0.00	1.50	3.00	4.50	5.40	6.50	8.00	9.50	11.00	12.50
Immediate Settlement(cm)		5.60	7.80	9.30	9.60	9.50	8.30	5.80	4.50	3.40	2.40
Consolidation Settlement(cm)		2.24	4.12	6.47	8.43	8.36	7.10	4.34	2.26	1.19	0.67
Residual Settlement ΣS (cm)		**7.8**	**11.9**	**15.8**	**18.0**	**17.9**	**15.4**	**10.1**	**6.8**	**4.6**	**3.1**

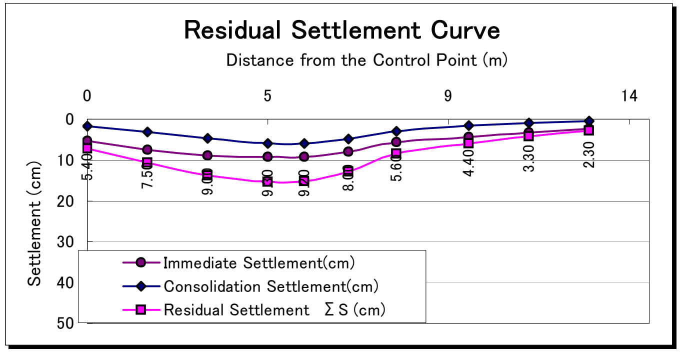
Settlement of Dike After Embankment Work

Location of Calculation		Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	0.00	1.50	3.00	4.50	5.40	6.50	8.00	9.50	11.00	12.50
Immediate Settlement(cm)		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)		1.79	3.09	4.76	6.02	5.95	4.91	2.90	1.55	0.88	0.55
Residual Settlement ΣS (cm)		**1.8**	**3.1**	**4.8**	**6.0**	**6.0**	**4.9**	**2.9**	**1.5**	**0.9**	**0.5**



Residual Settlment at Sluiceway Site

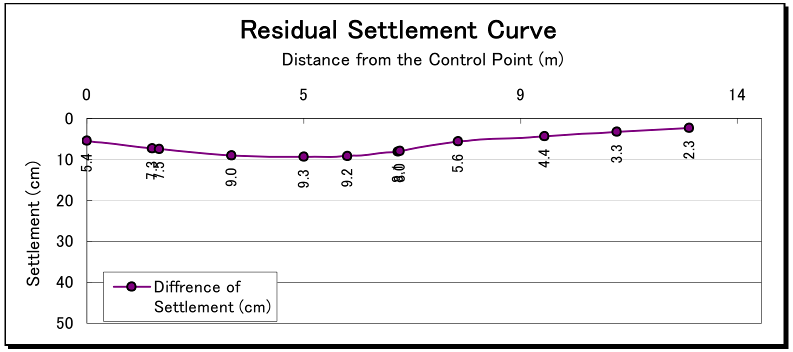
Location of Calculation		Control Point (1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	0.00	1.50	3.00	4.50	5.40	6.50	7.70	9.50	11.00	12.50
Immediate Settlement(cm)		5.40	7.50	9.00	9.30	9.20	8.00	5.60	4.40	3.30	2.30
Consolidation Settlement(cm)		1.79	3.09	4.76	6.02	5.95	4.91	2.90	1.55	0.88	0.55
Residual Settlement ΣS (cm)		**7.2**	**10.6**	**13.8**	**15.3**	**15.2**	**12.9**	**8.5**	**5.9**	**4.2**	**2.8**



Difference of Settlment

	Breast Wall in Land SIde			Seepage Cut Off Wall		Breast Wall in River Side					
	Control Point	SSP			SSP		SSP					

Location of Calculation	(1)		(2)	(3)	(4)	(5)		(6)	(7)	(8)	(9)	(10)
	Control Point	SSP (Breast Wall in Land SIde)			SSP (Seepage Cut Off Wall)		SSP (Breast Wall in River Side)					
X (m)	0.00	1.35	1.50	3.00	4.50	5.40	6.45	6.50	7.70	9.50	11.00	12.50
Diffrence of Settlement (cm)	**5.4**	**7.3**	**7.5**	**9.0**	**9.3**	**9.2**	**8.1**	**8.0**	**5.6**	**4.4**	**3.3**	**2.3**
		↓			↓		↓					
Applied Capability of SSP with Flexible Joint (cm)		**10.0**			**10.0**		**10.0**					



Design of SSP with Flexible Joint
(4) MSL-4

Residual Settlement of Dike

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.80	3.40	5.00	6.20	7.60	9.20	10.80	12.40	14.00
Immediate Settlement(cm)	8.50	12.10	13.80	14.80	14.10	10.70	8.10	6.30	4.90	3.50
Consolidation Settlement(cm)	1.17	1.83	2.50	3.11	3.87	6.25	2.79	1.47	1.00	0.71
Residual Settlement ΣS (cm)	**9.7**	**13.9**	**16.3**	**17.9**	**18.0**	**17.0**	**10.9**	**7.8**	**5.9**	**4.2**

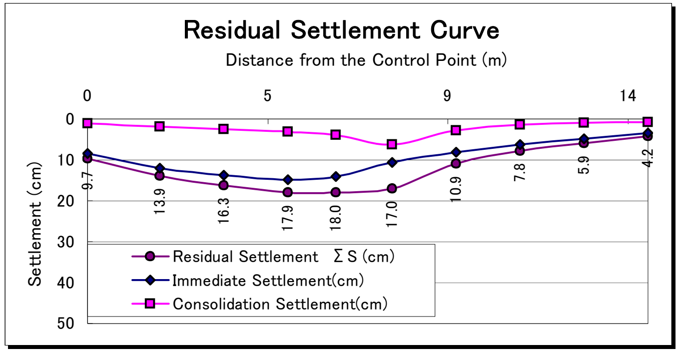
Settlement During Embankment Work, 105days

→ After 105 days, dike is excaved and suluiceway is installed.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.80	3.40	5.00	6.20	7.60	9.20	10.80	12.40	14.00
Immediate Settlement(cm)	8.50	12.10	13.80	14.80	14.10	10.70	8.10	6.30	4.90	3.50
Consolidation Settlement(cm)	0.77	1.29	1.84	2.36	3.02	5.25	2.05	0.93	0.59	0.40
Residual Settlement ΣS (cm)	**9.3**	**13.4**	**15.6**	**17.2**	**17.1**	**16.0**	**10.1**	**7.2**	**5.5**	**3.9**

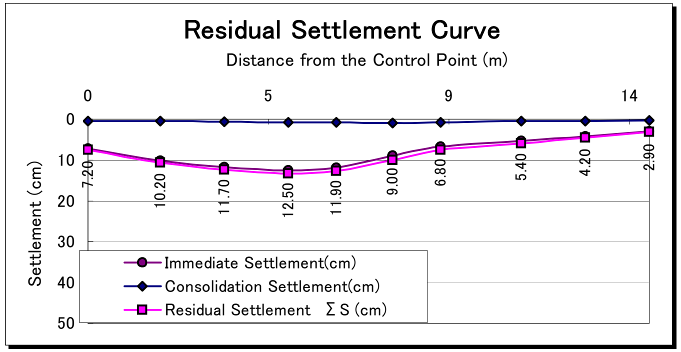
Settlement of Dike After Embankment Work

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.80	3.40	5.00	6.20	7.60	9.20	10.80	12.40	14.00
Immediate Settlement(cm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)	0.41	0.54	0.66	0.75	0.85	1.00	0.74	0.54	0.41	0.31
Residual Settlement ΣS (cm)	**0.4**	**0.5**	**0.7**	**0.8**	**0.8**	**1.0**	**0.7**	**0.5**	**0.4**	**0.3**



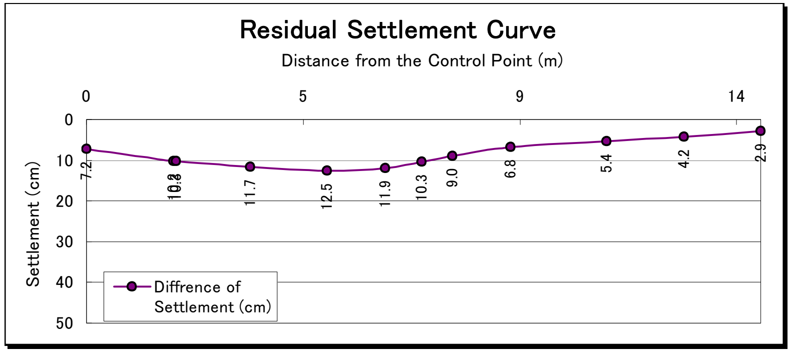
Residual Settlment at Sluiceway Site

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.80	3.40	5.00	6.20	7.60	8.80	10.80	12.40	14.00
Immediate Settlement(cm)	7.20	10.20	11.70	12.50	11.90	9.00	6.80	5.40	4.20	2.90
Consolidation Settlement(cm)	0.41	0.54	0.66	0.75	0.85	1.00	0.74	0.54	0.41	0.31
Residual Settlement ΣS (cm)	**7.6**	**10.7**	**12.4**	**13.3**	**12.7**	**10.0**	**7.5**	**5.9**	**4.6**	**3.2**



Difference of Settlment

	Control Point		SSP Breast Wall in Land SIde		SSP Seepage Cut Off Wall		SSP Breast Wall in River Side					
Location of	(1)	(2)		(3)	(4)	(5)		(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.80	1.86	3.40	5.00	6.20	6.96	7.60	8.80	10.80	12.40	14.00
Diffrence of Settlement (cm)	**7.2**	**10.2**	**10.3**	**11.7**	**12.5**	**11.9**	**10.3**	**9.0**	**6.8**	**5.4**	**4.2**	**2.9**
			↓		↓		↓					
Applied Capability of SSP with Flexible Joint (cm)			**20.0**		**20.0**		**20.0**					



Design of SSP with Flexible Joint
(5) MSL-5

Residual Settlement of Dike

		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.00	1.70	3.20	4.70	5.60	6.60	8.10	9.60	11.10	12.60
Immediate Settlement(cm)		4.20	6.00	7.20	7.40	7.10	6.20	4.10	3.10	2.20	1.50
Consolidation Settlement(cm)		2.19	4.48	7.68	11.39	11.18	8.96	4.50	2.04	1.01	0.58
Residual Settlement ΣS (cm)		**6.4**	**10.5**	**14.9**	**18.8**	**18.3**	**15.2**	**8.6**	**5.1**	**3.2**	**2.1**

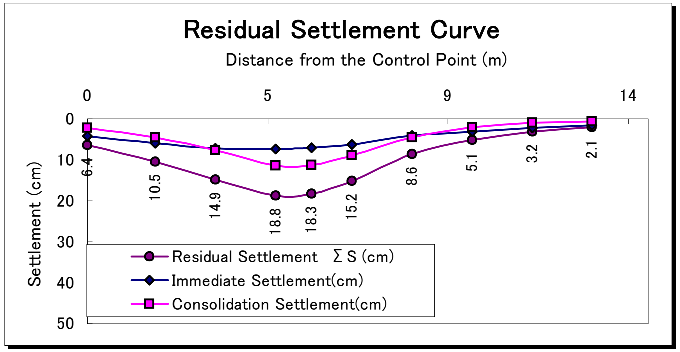
Settlement During Embankment Work, 105days

→ After 105 days, dike is excaved and suluiceway is installed.

		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.00	1.70	3.20	4.70	5.60	6.60	8.10	9.60	11.10	12.60
Immediate Settlement(cm)		4.20	6.00	7.20	7.40	7.10	6.20	4.10	3.10	2.20	1.50
Consolidation Settlement(cm)		1.46	3.01	5.35	8.78	8.83	7.15	3.40	1.44	0.67	0.38
Residual Settlement ΣS (cm)		**5.7**	**9.0**	**12.5**	**16.2**	**15.9**	**13.3**	**7.5**	**4.5**	**2.9**	**1.9**

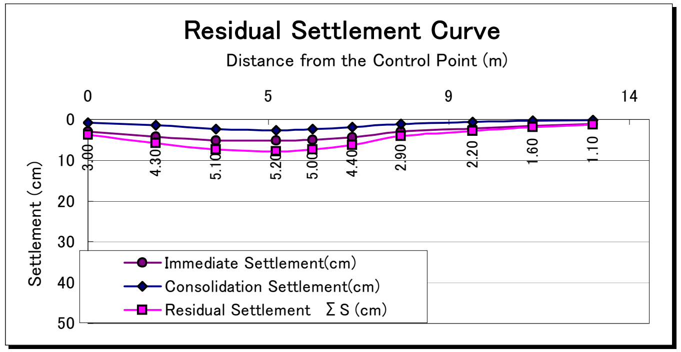
Settlement of Dike After Embankment Work

		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.00	1.70	3.20	4.70	5.60	6.60	8.10	9.60	11.10	12.60
Immediate Settlement(cm)		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)		0.73	1.46	2.34	2.61	2.35	1.81	1.10	0.60	0.34	0.21
Residual Settlement ΣS (cm)		**0.7**	**1.5**	**2.3**	**2.6**	**2.3**	**1.8**	**1.1**	**0.6**	**0.3**	**0.2**



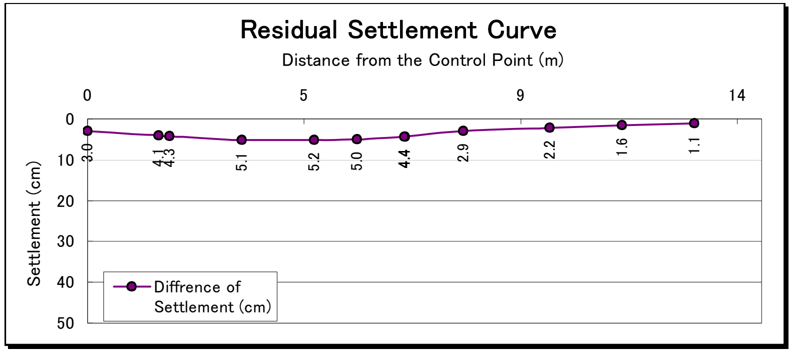
Residual Settlment at Sluiceway Site

		Control Point									
	Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)	Caluculation	0.00	1.70	3.20	4.70	5.60	6.60	7.80	9.60	11.10	12.60
Immediate Settlement(cm)		3.00	4.30	5.10	5.20	5.00	4.40	2.90	2.20	1.60	1.10
Consolidation Settlement(cm)		0.73	1.46	2.34	2.61	2.35	1.81	1.10	0.60	0.34	0.21
Residual Settlement ΣS (cm)		**3.7**	**5.8**	**7.4**	**7.8**	**7.3**	**6.2**	**4.0**	**2.8**	**1.9**	**1.3**



Difference of Settlment

	Control Point	SSP (Breast Wall in Land SIde)			SSP (Seepage Cut Off Wall)			SSP (Breast Wall in River Side)				
Location of	(1)		(2)	(3)	(4)	(5)	(6)		(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.48	1.70	3.20	4.70	5.60	6.60	6.58	7.80	9.60	11.10	12.60
Diffrence of Settlement (cm)	**3.0**	**4.1**	**4.3**	**5.1**	**5.2**	**5.0**	**4.4**	**4.4**	**2.9**	**2.2**	**1.6**	**1.1**
		↓			↓			↓				
Applied Capability of SSP with Flexible Joint (cm)		**10.0**			**10.0**			**10.0**				



Design of SSP with Flexible Joint
(6) MSL-6
Residual Settlement of Dike

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Calculation	0.00	1.70	3.20	4.70	5.90	7.20	8.70	10.20	11.70	13.20
Immediate Settlement(cm)	6.50	9.30	11.10	11.40	10.60	8.00	5.80	4.40	3.30	2.30
Consolidation Settlement(cm)	2.48	4.41	6.61	8.90	8.71	6.74	3.94	2.19	1.30	1.30
Residual Settlement ΣS (cm)	**9.0**	**13.7**	**17.7**	**20.3**	**19.3**	**14.7**	**9.7**	**6.6**	**4.6**	**3.6**

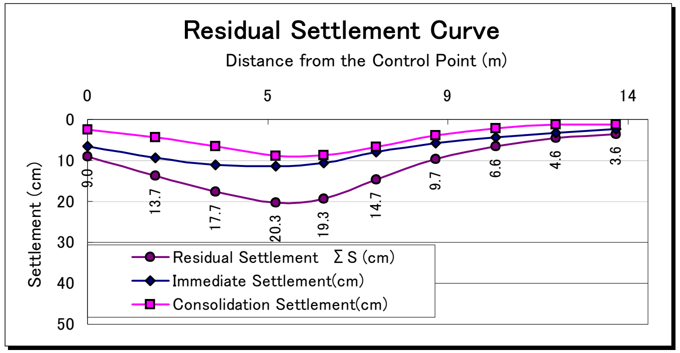
Settlement During Embankment Work, 105days

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Calculation	0.00	1.70	3.20	4.70	5.90	7.20	8.70	10.20	11.70	13.20
Immediate Settlement(cm)	6.50	9.30	11.10	11.40	10.60	8.00	5.80	4.40	3.30	2.30
Consolidation Settlement(cm)	1.25	2.40	3.70	4.90	4.78	3.62	2.01	1.03	0.57	0.34
Residual Settlement ΣS (cm)	**7.7**	**11.7**	**14.8**	**16.3**	**15.4**	**11.6**	**7.8**	**5.4**	**3.9**	**2.6**

→ **After 105 days, dike is excaved and suluiceway is installed.**

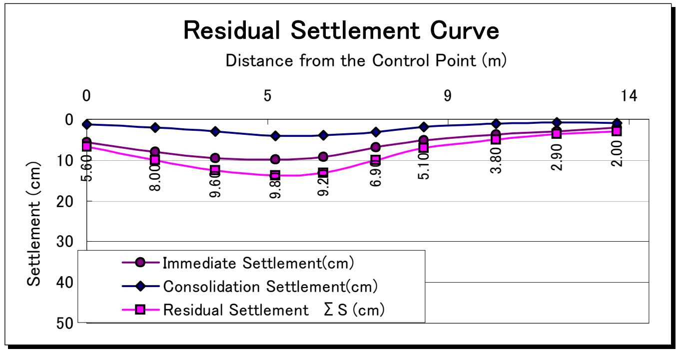
Settlement of Dike After Embankment Work

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Calculation	0.00	1.70	3.20	4.70	5.90	7.20	8.70	10.20	11.70	13.20
Immediate Settlement(cm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)	1.23	2.02	2.91	4.00	3.93	3.12	1.93	1.16	0.74	0.96
Residual Settlement ΣS (cm)	**1.2**	**2.0**	**2.9**	**4.0**	**3.9**	**3.1**	**1.9**	**1.2**	**0.7**	**1.0**



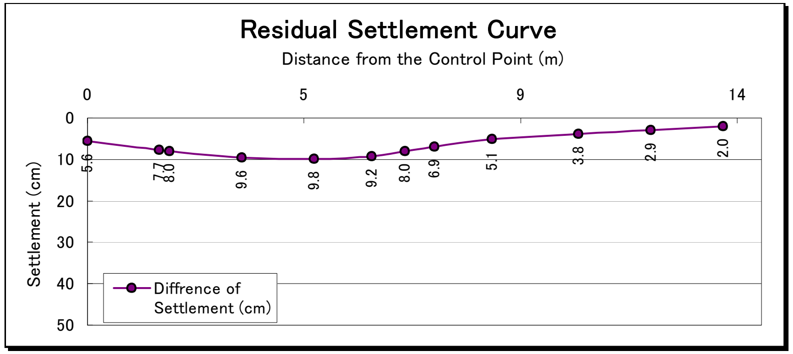
Residual Settlment at Sluiceway Site

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Calculation	0.00	1.70	3.20	4.70	5.90	7.20	8.40	10.20	11.70	13.20
Immediate Settlement(cm)	5.60	8.00	9.60	9.80	9.20	6.90	5.10	3.80	2.90	2.00
Consolidation Settlement(cm)	1.23	2.02	2.91	4.00	3.93	3.12	1.93	1.16	0.74	0.96
Residual Settlement ΣS (cm)	**6.8**	**10.0**	**12.5**	**13.8**	**13.1**	**10.0**	**7.0**	**5.0**	**3.6**	**3.0**



Difference of Settlment

Location of Calculation	Control Point	SSP Breast Wall in Land SIde			SSP Seepage Cut Off Wall		SSP Breast Wall in River Side					
	(1)		(2)	(3)	(4)	(5)		(6)	(7)	(8)	(9)	(10)
X (m)	0.00	1.49	1.70	3.20	4.70	5.90	6.59	7.20	8.40	10.20	11.70	13.20
Diffrence of Settlement (cm)	**5.6**	**7.7**	**8.0**	**9.6**	**9.8**	**9.2**	**8.0**	**6.9**	**5.1**	**3.8**	**2.9**	**2.0**
		↓			↓		↓					
Applied Capability of SSP with Flexible Joint (cm)		**10.0**			**10.0**		**10.0**					



Design of SSP with Flexible Joint
(7) MSR-2

Residual Settlement of Dike

	Control Point Location of Calculuation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)		0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)		2.40	3.50	4.50	5.70	7.60	9.80	10.50	10.10	8.40	6.10
Consolidation Settlement(cm)		1.53	2.11	3.17	5.36	9.00	10.54	8.63	6.53	4.52	2.97
Residual Settlement ΣS (cm)		**3.9**	**5.6**	**7.7**	**11.1**	**16.6**	**20.3**	**19.1**	**16.6**	**12.9**	**9.1**

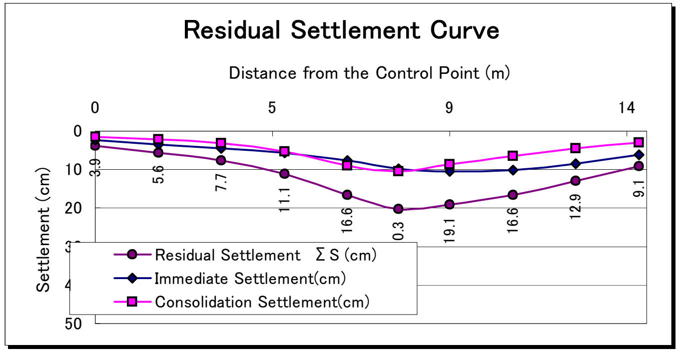
Settlement During Embankment Work, 105days

→ **After 105 days, dike is excaved and suluiceway is installed.**

	Control Point Location of Calculuation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)		0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)		2.40	3.50	4.50	5.70	7.60	9.80	10.50	10.10	8.40	6.10
Consolidation Settlement(cm)		0.58	0.88	1.51	2.92	5.18	5.95	3.97	2.93	1.93	1.17
Residual Settlement ΣS (cm)		**3.0**	**4.4**	**6.0**	**8.6**	**12.8**	**15.8**	**14.5**	**13.0**	**10.3**	**7.3**

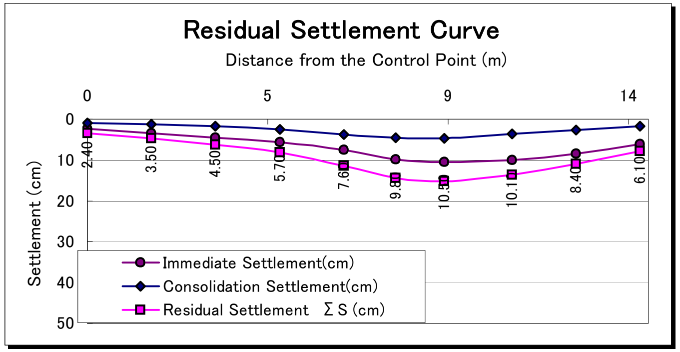
Settlement of Dike After Embankment Work

	Control Point Location of Calculuation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)		0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)		0.96	1.23	1.65	2.43	3.82	4.59	4.66	3.61	2.60	1.80
Residual Settlement ΣS (cm)		**1.0**	**1.2**	**1.7**	**2.4**	**3.8**	**4.6**	**4.7**	**3.6**	**2.6**	**1.8**



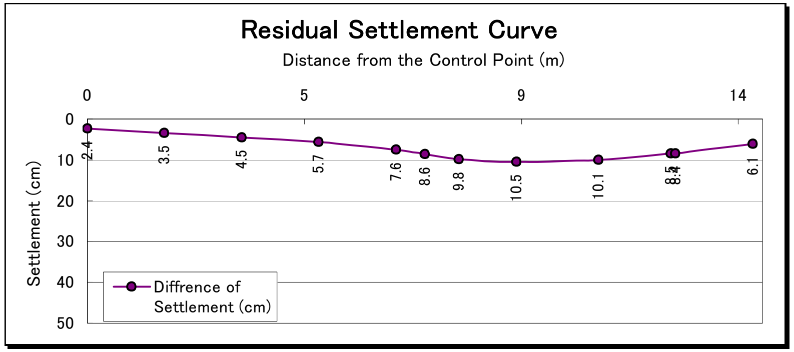
Residual Settlment at Sluiceway Site

	Control Point Location of Calculuation	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m)		0.00	1.60	3.20	4.80	6.40	7.70	8.90	10.60	12.20	13.80
Immediate Settlement(cm)		2.40	3.50	4.50	5.70	7.60	9.80	10.50	10.10	8.40	6.10
Consolidation Settlement(cm)		0.96	1.23	1.65	2.43	3.82	4.59	4.66	3.61	2.60	1.80
Residual Settlement ΣS (cm)		**3.4**	**4.7**	**6.2**	**8.1**	**11.4**	**14.4**	**15.2**	**13.7**	**11.0**	**7.9**



Difference of Settlment

						Breast Wall in River Side		Seepage Cut Off Wall		Breast Wall in Land SIde		
	Control Point					SSP		SSP		SSP		
Location of	(1)	(2)	(3)	(4)	(5)		(6)	(7)	(8)		(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	7.00	7.70	8.90	10.60	12.10	12.20	13.80
Diffrence of Settlement (cm)	**2.4**	**3.5**	**4.5**	**5.7**	**7.6**	**8.6**	**9.8**	**10.5**	**10.1**	**8.5**	**8.4**	**6.1**
						↓		↓		↓		
Applied Capability of SSP with Flexible Joint (cm)						**10.0**		**20.0**		**10.0**		



Design of SSP with Flexible Joint
(8) MSR-3

Residual Settlement of Dike

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)	3.00	4.30	5.60	7.20	9.50	12.10	12.90	12.80	10.90	8.20
Consolidation Settlement(cm)	2.65	3.67	5.17	8.56	18.84	24.66	18.76	10.44	6.79	4.47
Residual Settlement ΣS (cm)	**5.7**	**8.0**	**10.8**	**15.8**	**28.3**	**36.8**	**31.7**	**23.2**	**17.7**	**12.7**

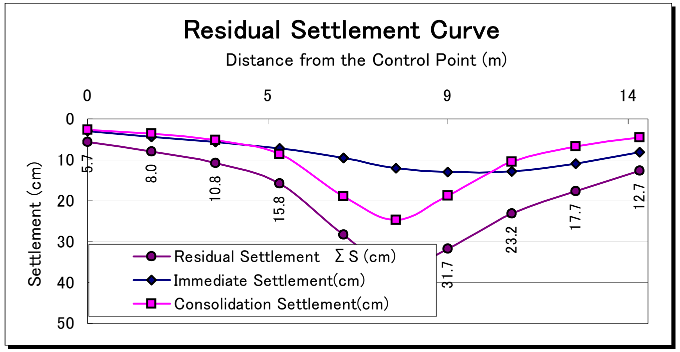
Settlement During Embankment Work, 105days

→ After 105 days, dike is excaved and suluiceway is installed.

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)	3.00	4.30	5.60	7.20	9.50	12.10	12.90	12.80	10.90	8.20
Consolidation Settlement(cm)	0.84	1.33	2.21	4.73	12.83	16.42	9.98	4.38	2.60	1.52
Residual Settlement ΣS (cm)	**3.8**	**5.6**	**7.8**	**11.9**	**22.3**	**28.5**	**22.9**	**17.2**	**13.5**	**9.7**

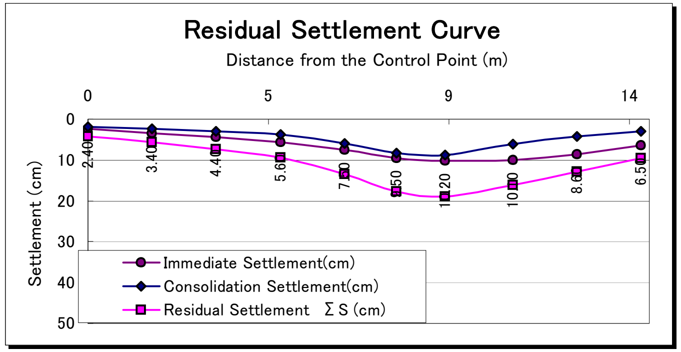
Settlement of Dike After Embankment Work

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	7.70	9.00	10.60	12.20	13.80
Immediate Settlement(cm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)	1.81	2.34	2.96	3.83	6.01	8.24	8.79	6.06	4.19	2.95
Residual Settlement ΣS (cm)	**1.8**	**2.3**	**3.0**	**3.8**	**6.0**	**8.2**	**8.8**	**6.1**	**4.2**	**3.0**



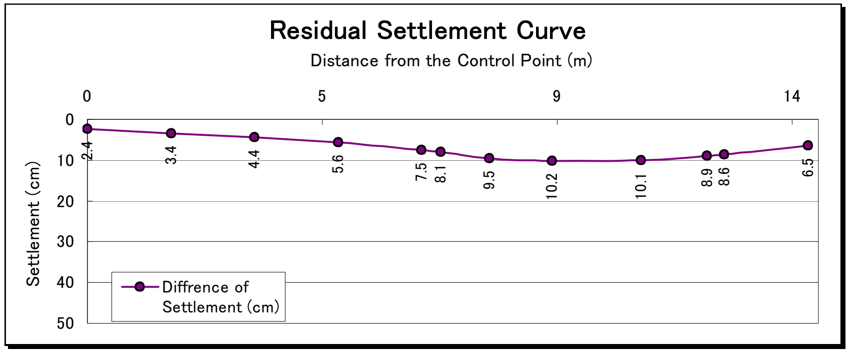
Residual Settlment at Sluiceway Site

	Control Point									
Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	7.70	8.90	10.60	12.20	13.80
Immediate Settlement(cm)	2.40	3.40	4.40	5.60	7.50	9.50	10.20	10.10	8.60	6.50
Consolidation Settlement(cm)	1.81	2.34	2.96	3.83	6.01	8.24	8.79	6.06	4.19	2.95
Residual Settlement ΣS (cm)	**4.2**	**5.7**	**7.4**	**9.4**	**13.5**	**17.7**	**19.0**	**16.2**	**12.8**	**9.5**



Difference of Settlment

						Breast Wall in River Side		Seepage Cut Off Wall		Breast Wall in Land SIde		
	Control Point					SSP		SSP		SSP		
Location of	(1)	(2)	(3)	(4)	(5)		(6)	(7)	(8)		(9)	(10)
X (m) Caluculation	0.00	1.60	3.20	4.80	6.40	6.77	7.70	8.90	10.60	11.87	12.20	13.80
Diffrence of Settlement (cm)	**2.4**	**3.4**	**4.4**	**5.6**	**7.5**	**8.1** ↓	**9.5**	**10.2** ↓	**10.1**	**8.9** ↓	**8.6**	**6.5**
Applied Capability of SSP with Flexible Joint (cm)						**10.0**		**20.0**		**10.0**		



Design of SSP with Flexible Joint
(9) MSR-4

Residual Settlement of Dike

			Control Point									
		Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	Caluculation	0.00	1.60	3.20	4.80	6.20	7.60	8.80	10.40	12.00	13.40
Immediate Settlement(cm)			3.50	5.00	6.50	8.40	10.80	14.10	14.90	14.40	11.90	8.70
Consolidation Settlement(cm)			2.48	3.78	5.91	9.03	12.49	14.25	13.42	11.87	9.02	6.43
Residual Settlement ΣS (cm)			**6.0**	**8.8**	**12.4**	**17.4**	**23.3**	**28.4**	**28.3**	**26.3**	**20.9**	**15.1**

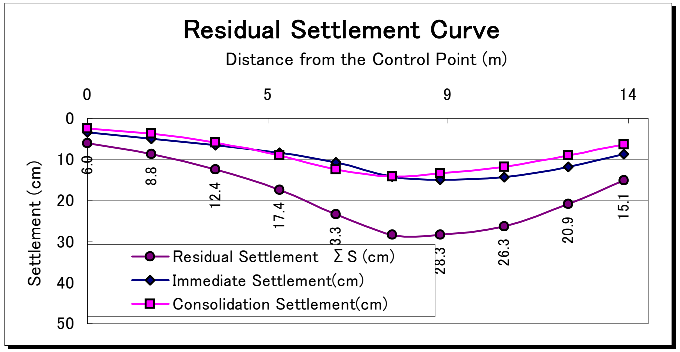
Settlement During Embankment Work, 105days

→ **After 105 days, dike is excaved and suluiceway is installed.**

			Control Point									
		Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	Caluculation	0.00	1.60	3.20	4.80	6.20	7.60	8.80	10.40	12.00	13.40
Immediate Settlement(cm)			3.50	5.00	6.50	8.40	10.80	14.10	14.90	14.40	11.90	8.70
Consolidation Settlement(cm)			0.84	1.24	1.87	2.78	3.78	4.29	4.08	3.60	2.74	1.96
Residual Settlement ΣS (cm)			**4.3**	**6.2**	**8.4**	**11.2**	**14.6**	**18.4**	**19.0**	**18.0**	**14.6**	**10.7**

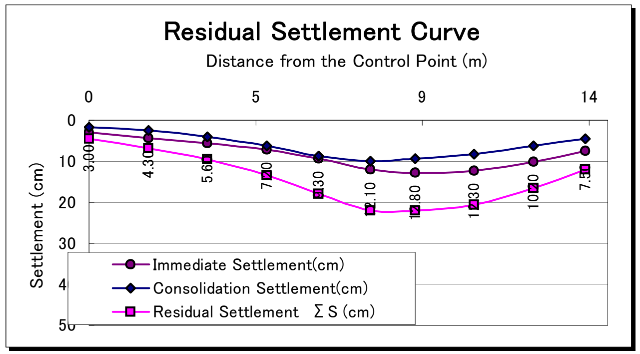
Settlement of Dike After Embankment Work

			Control Point									
		Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	Caluculation	0.00	1.60	3.20	4.80	6.20	7.60	8.80	10.40	12.00	13.40
Immediate Settlement(cm)			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Consolidation Settlement(cm)			1.64	2.54	4.04	6.25	8.72	9.96	9.34	8.26	6.28	4.47
Residual Settlement ΣS (cm)			**1.6**	**2.5**	**4.0**	**6.3**	**8.7**	**10.0**	**9.3**	**8.3**	**6.3**	**4.5**



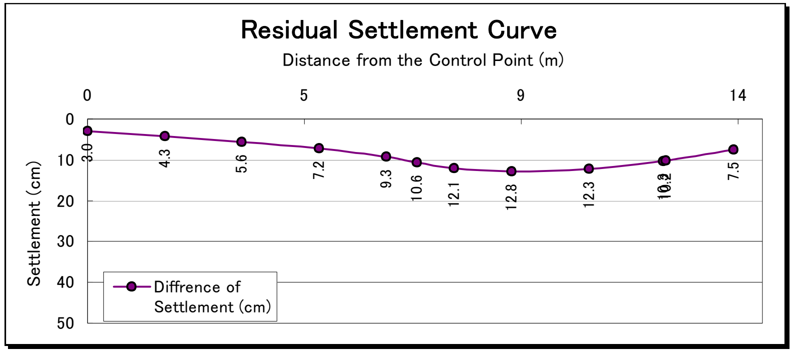
Residual Settlment at Sluiceway Site

			Control Point									
		Location of	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
X	(m)	Caluculation	0.00	1.60	3.20	4.80	6.20	7.60	8.80	10.40	12.00	13.40
Immediate Settlement(cm)			3.00	4.30	5.60	7.20	9.30	12.10	12.80	12.30	10.20	7.50
Consolidation Settlement(cm)			1.64	2.54	4.04	6.25	8.72	9.96	9.34	8.26	6.28	4.47
Residual Settlement ΣS (cm)			**4.6**	**6.8**	**9.6**	**13.5**	**18.0**	**22.1**	**22.1**	**20.6**	**16.5**	**12.0**



Difference of Settlment

		(1)	(2)	(3)	(4)	(5)		(6)	(7)	(8)		(9)	(10)
		Control Point					SSP Breast Wall in River Side		SSP Seepage Cut Off Wall		SSP Breast Wall in Land SIde		
X (m)	Location of Caluculation	0.00	1.60	3.20	4.80	6.20	6.84	7.60	8.80	10.40	11.94	12.00	13.40
Diffrence of Settlement (cm)		**3.0**	**4.3**	**5.6**	**7.2**	**9.3**	**10.6** ↓	**12.1**	**12.8** ↓	**12.3**	**10.3** ↓	**10.2**	**7.5**
Applied Capability of SSP with Flexible Joint (cm)							**20.0**		**20.0**		**20.0**		



11.5 Breast Wall

(1) Design Condition

Breast Wall is Calculated as a cantilever which is fixed on the box culvert.

The condition of concrete and reinforcing bar is same as manhole. Only the condition which is different from other structure is indicated as follows

(a) Load

Normal Condition :10.0kN/m^2)

Seismic Condition : 5.0 (kN/m^2)

(b) Water Level Condition

	Front Side	Back Side	Remarks
Normal	Underside of	RWL	
Seismic	Bottom Slab	GWL or MWL	Higher Level should be Applied

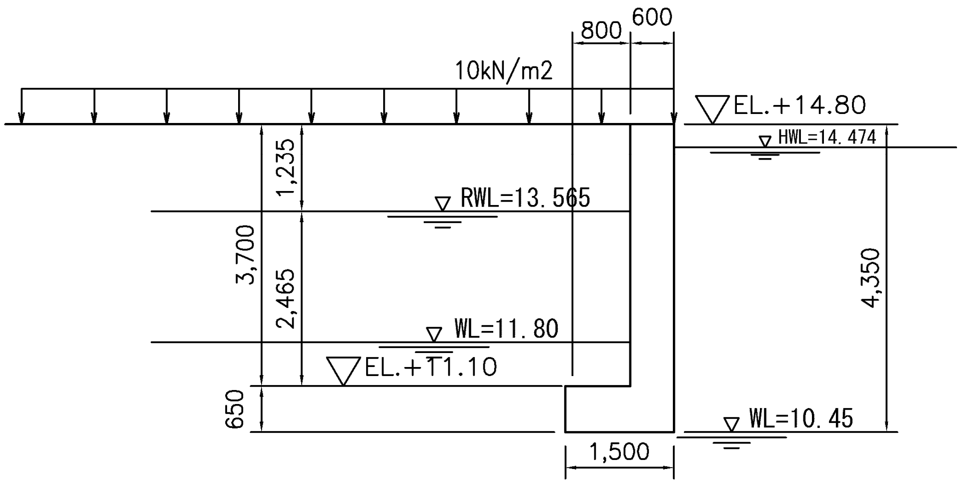
(2) Structural Calculation

The detail of structural calculation of breast wall in longitudinal direction is indicated from the following page.

Structural Culculation of Breast Wall

MSL3 River Side

1. Calculation Model



・Dimension

Height of Vertical wall	3.70	m
Tickness of Vertical Wall	0.60	m
Width of Bottom Slab	1.40	m
Tickness of Bottom Slab	0.65	m
Height of Embankment	0.00	m

・Elevation

Top of the wall	EL+	14.80	m
Upperside of Bottom Slab	EL+	11.10	m
Underside of Bottom Slab	EL+	10.45	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	10.45	13.57	—	○	○	○	○	○	○
2	Seismic	10.45	11.80	←	◎	◎	○	○	○	○
3	Seismic	10.45	11.80	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

— : Without Considering

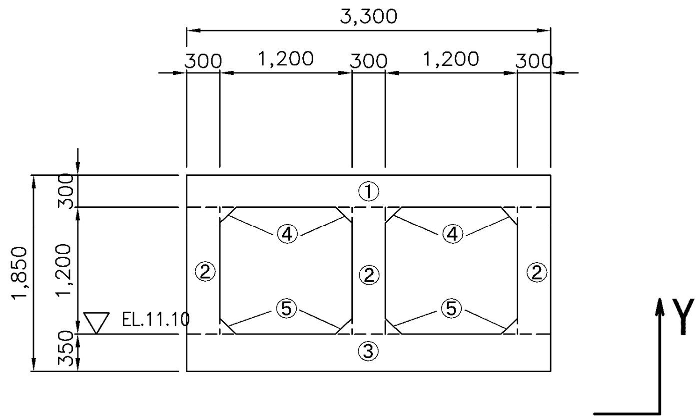
2. Design Condition

Unit Weight of Concrete		24.0	kN/m^3
Unit Weight of Water		9.8	kN/m^3
Unit Weight of Soil			
Saturated		20.0	kN/m^3
Wet		18.0	kN/m^3
Submerged		11.0	kN/m^3
Backfill Material		Sandy Soil	
internal Friction Angle of Soil		30.00	°
Seismic Lad		0.20	
Water Level			
Forth	Normal	10.45	m
	Seismic	10.45	m
Back	Normal	13.57	m
	Seismic	11.80	m
Difference	Normal	3.12	m
	Seismic	1.35	m
Earth Pressure	Normal	Earth Pressure at Rest	
	Seismic	Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane		0.00	°
Surchage	Normal	10.0	kN/m^2 (Pedestrian Load)
	Seismic	5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height（h）	α	n	A（m^2）	Y（m）	A・Y（m^3）
①	3.300	0.300	1.0	1	0.990	1.700	1.683
②	0.300	1.200	1.0	3	1.080	0.950	1.026
③	3.300	0.350	1.0	1	1.155	0.175	0.202
④	0.150	0.150	0.5	4	0.045	1.500	0.068
⑤	0.150	0.150	0.5	4	0.045	0.400	0.018
計					3.315		2.997

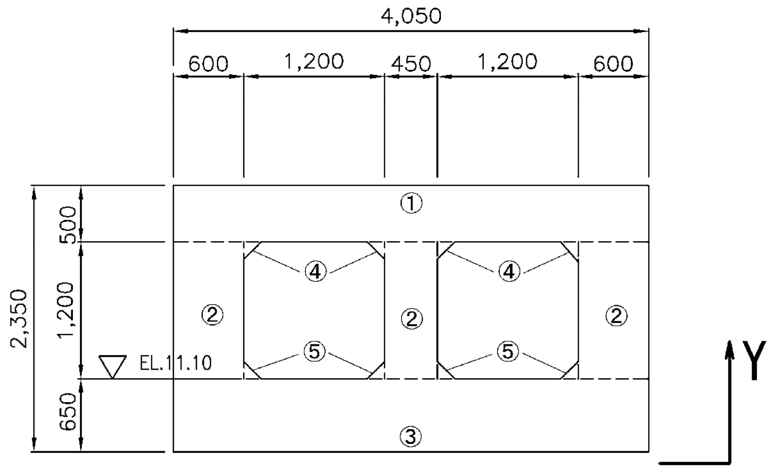
α；Triangle 0.5, Rectangle 1.0

ｎ；　Number

Centroid of Box Culvet

Y＝ ΣA・Y／ΣA　　　＝　0.904　m

Breast Wal Section



	Width (B)	Height（h）	α	n	A（m^2）
①	4.050	0.500	1.0	1	2.025
②	0.600	1.200	1.0	2	1.440
③	0.450	1.200	1.0	1	0.540
③	4.050	0.650	1.0	1	2.633
④	0.150	0.150	0.5	4	0.045
⑤	0.150	0.150	0.5	4	0.045
計					6.728

α；Triangle 0.5, Rectangle 1.0

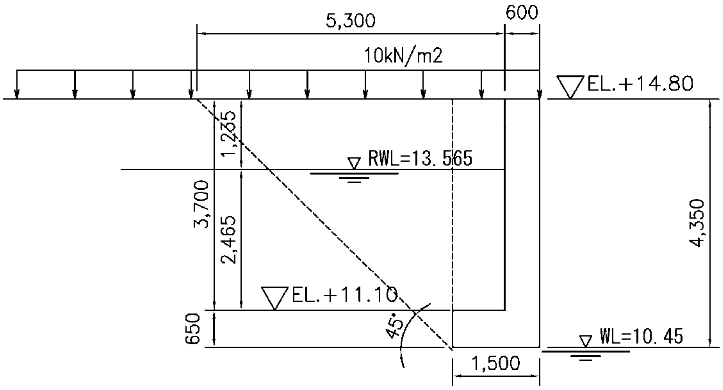
ｎ； Number

Weight of Body

W = $\gamma \times A \times 1.0$ ＝ 161.472 kN

3.2 Converted Load of Embankment

(1) Normal Condition



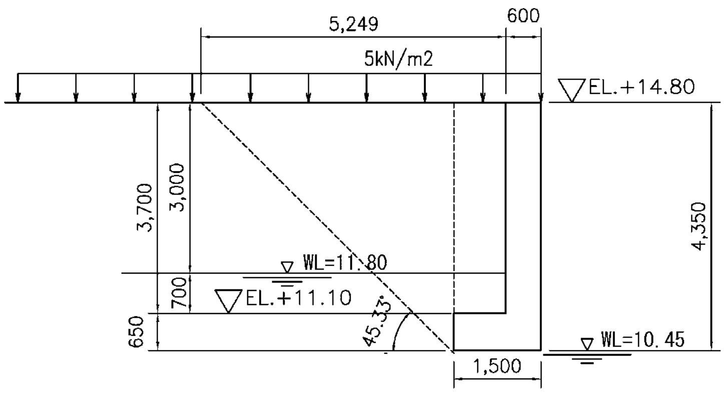
A= 1/2×(0 + 5.3)× 0 = 0.00 m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.3	×	10.0	)／	5.300

= 10.00 kN/m^2

(2) Seismic Condition



A= 1/2×(0.000 + 5.249)× 0 = 0.00 m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.249	×	5.0	)／	5.249

= 5.00 kN/m^2

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

$\varphi =$ 30 ° $\theta =$ 0.000 ° $\delta = \Phi / 2 =$ 15.0 ° $\alpha =$ 0 °

$kh =$ 0.20 $\theta o = \tan^{-1} kh =$ 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta o - \theta)}{\cos\theta o \times \cos^2\theta \times \cos(\theta + \theta o + \delta) \times \left\{1 + \sqrt{\dfrac{\sin(\varphi + \delta) \times \sin(\varphi - \alpha - \theta o)}{\cos(\theta + \theta o + \delta) \times \cos(\theta - \alpha)}}\right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{1 + \sqrt{\dfrac{0.2266}{0.8964}}\right\}^2} = \mathbf{0.452}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °

$\delta =$ $\varphi/2 =$ 15 ° (Between soil and soil)

$\theta =$ 0

$\alpha =$ 0

$\theta o =$ $\tan^{-1} Kh =$ 11.310 °

$kh =$ 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi + \delta + \theta - \alpha) + \sec(\varphi + \delta + \theta - \alpha) \cdot X$$

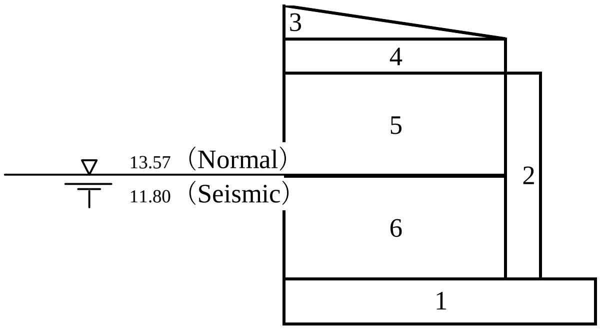
$$X = \sqrt{\frac{\cos(\theta + \delta + \theta o) \cdot \sin(\varphi + \delta)}{\cos(\theta - \alpha) \cdot \sin(\varphi - \alpha - \theta o)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1} 1 / 0.9892 = \boxed{45.311 °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.500	0.650	1.00		24.0	23.40
2	0.600	3.700	1.00		24.0	53.28

No	V (kN)	X (m)	V・X (kNm)	H (kN)	Y (m)	H・Y (kNm)
1	23.40	0.750	17.55	4.68	-0.579	-2.71
2	53.28	0.300	15.98	10.66	1.596	17.01
Total	76.68		33.53	15.34		14.30

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.900	0.000	1.00		18.0	0.00
4	0.900	0.000	1.00		18.0	0.00
5	0.900	1.235	1.00		18.0	20.01
6	0.900	2.465	1.00		20.0	44.37

No	V (kN)	X (m)	V・X (kNm)
3	0.00	1.050	0.00
4	0.00	1.050	0.00
5	20.01	1.050	21.01
6	44.37	1.050	46.59
Total	64.38		67.60

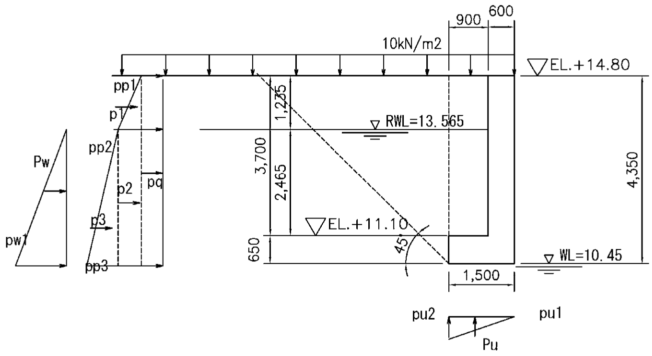
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.900	0.000	1.00		18.0	0.00
4	0.900	0.000	1.00		18.0	0.00
5	0.900	3.000	1.00		18.0	48.60
6	0.900	0.700	1.00		20.0	12.60

No	V (kN)	X (m)	V・X (kNm)	H (kN)	Y (m)	H・Y (kNm)
3	0.00	1.050	0.00	0.00	3.446	0.00
4	0.00	1.050	0.00	0.00	3.446	0.00
5	48.60	1.050	51.03	9.72	1.946	18.92
6	12.60	1.050	13.23	2.52	0.096	0.24
Total	61.20		64.26	12.24		19.16

(5) Earth Pressure

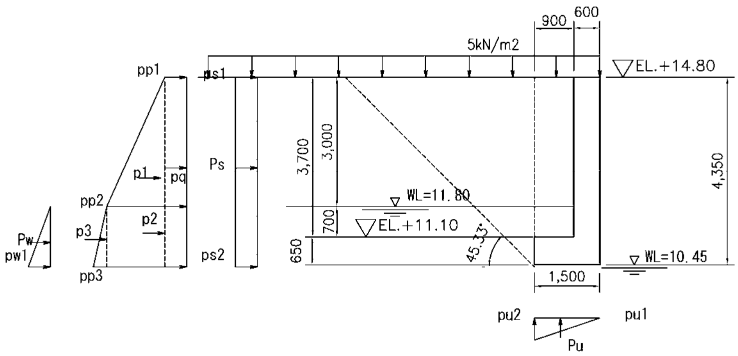
1) Normal Condition



φ	:	internal Friction Angle of Soil	30
Ko	:	Earth Pressure at Rest	0.5
q	:	Converted Load of Embankment	10.00 kN/m^2

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	6.86	2.623	17.99
Earth Pressure (p2)	0.00	0.000	0.00	30.46	0.654	19.92
Earth Pressure (p3)	0.00	0.000	0.00	26.68	0.134	3.58
Earth Pressure (pq)	0.00	0.000	0.00	21.75	1.271	27.64
Total	0.00		0.00	85.75		69.13

2) Seismic Condition

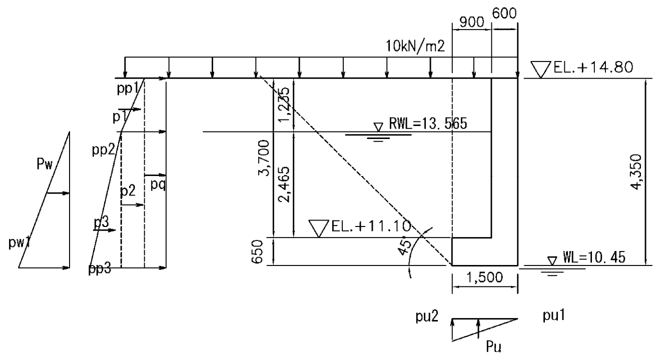


φ	:	internal Friction Angle of Soil	30	°
δ	:	Angle of Wall Friction	15	°
		δ=φ/2（Seismic Condition）		
α	:	Angle Between Ground Surface and Horizontal Plane	0	°
θ	:	Angle Between Back side Surface of Wall and Vertical Plane	0	°
Ka	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	5.0	kN/m^2

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	9.48	1.500	14.22	35.36	1.446	51.13
Earth Pressure (p2)	8.53	1.500	12.80	31.83	-0.229	-7.29
Earth Pressure (p3)	1.04	1.500	1.56	4.38	-0.454	-1.99
Earth Pressure (pq)	2.54	1.500	3.81	9.50	1.271	12.07
Total	21.59		32.39	81.07		53.92

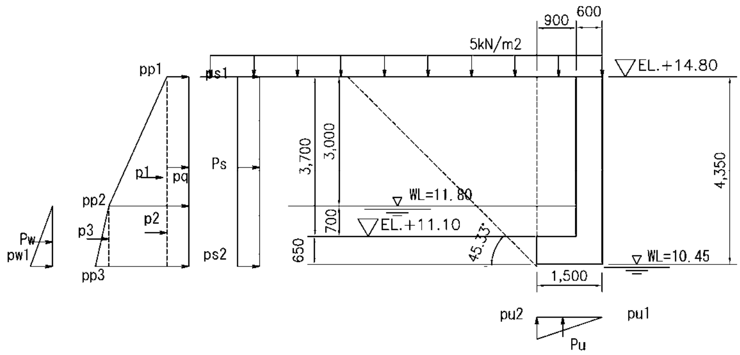
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	47.70	0.136	6.49
Total	0.00		0.00	47.70		6.49

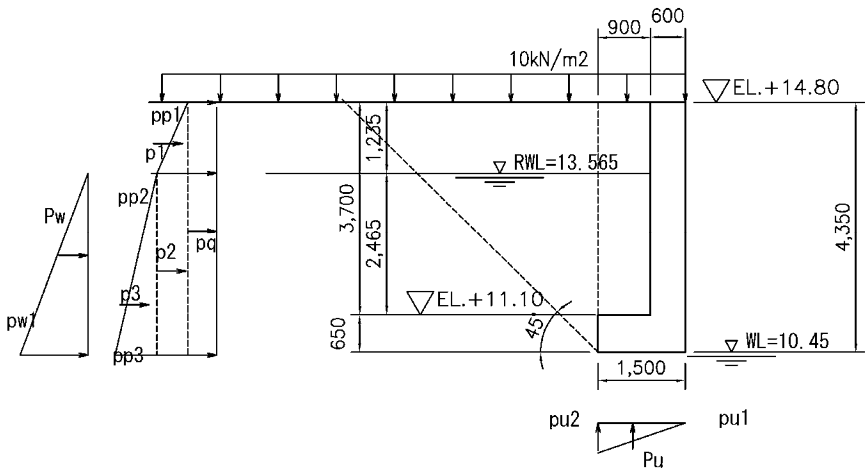
(2) Seismic Condition



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	8.93	0.450	4.02
Total	0.00					

3.5 Uplift

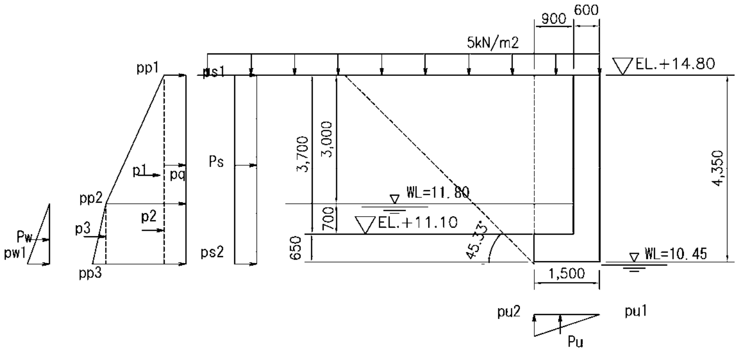
(1) Normal Conditon



	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	30.58	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	22.94	1.000	22.94	0.00	0.000	0.00
Total	22.94		22.94	0.00		0.00

(2) Seismic Conditon

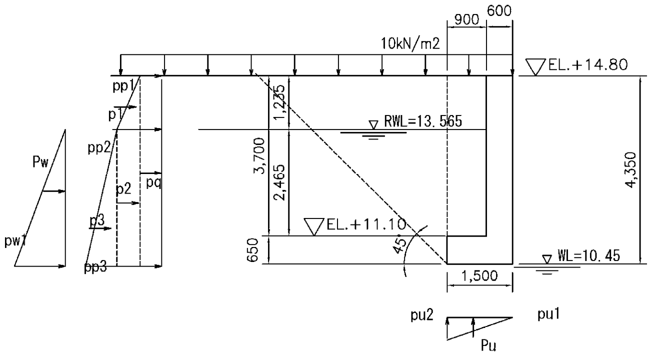


	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	13.23	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	9.92	1.000	9.92	0.00	0.000	0.00
Total	9.92		9.92	0.00		0.00

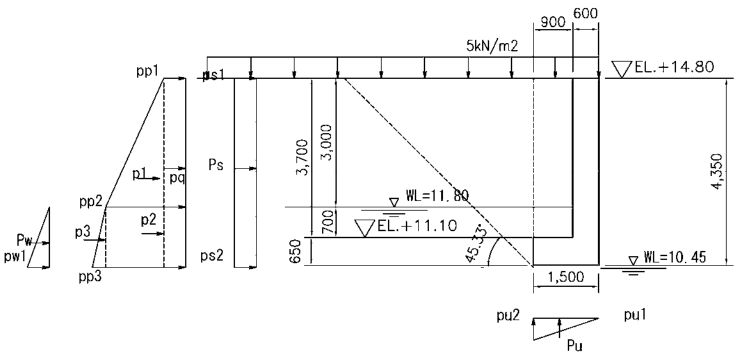
3.6 Surcharge

(1) Normal Condition



	q(kN/m^2)	B(m)	X(m)	V(kN)	V・X(kNm)
1	10.0	0.90	1.050	9.00	9.45
Total				9.00	9.45

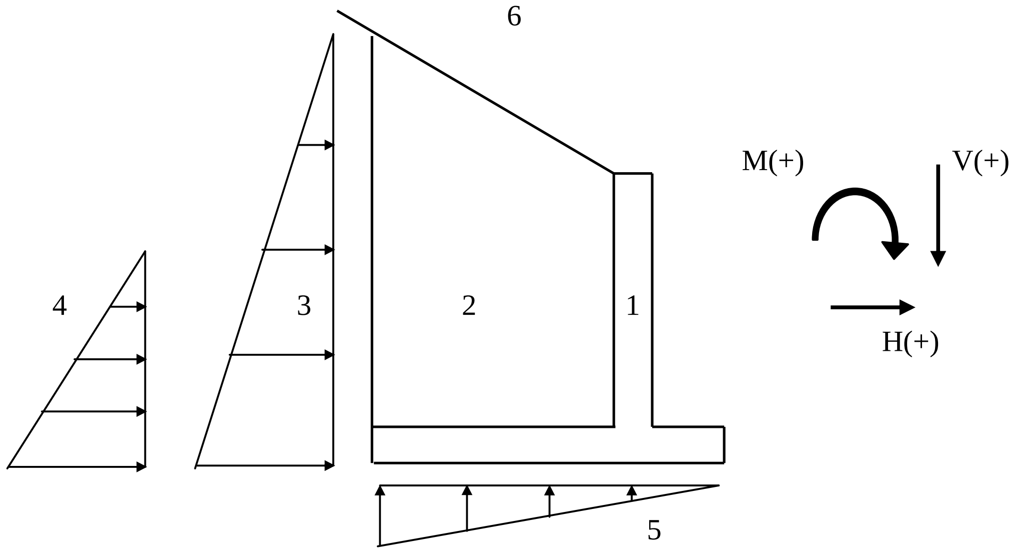
(2) Seismic Conditon



	q(kN/m^2)	B(m)	X(m)	V(kN)	V・X(kNm)
1	5.0	0.90	1.050	4.50	4.73
Total				4.50	4.73

3.7 Summary of Load

(1) Normal Condition



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	76.68	33.53	0.00	0.00
2	Weight of Back Fill	64.38	67.60	0.00	0.00
3	Earth Pressure	0.00	0.00	85.75	69.13
4	Water Pressure	0.00	0.00	47.70	6.49
5	Uplift	-22.94	-22.94	0.00	0.00
6	Srcharge	9.00	9.45	0.00	0.00
	Total	127.12	87.64	133.45	75.62

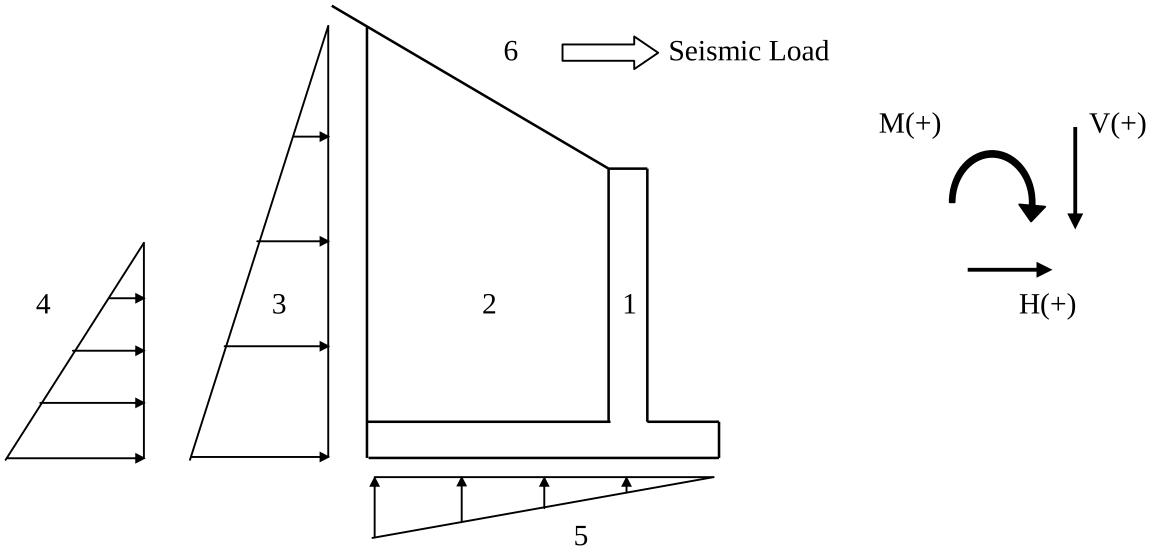
Vertical Force V= 127.12 kN

Horizontal Force H= 133.45 kN

Moment M= -12.02 kNm

position X = V・X/V= 87.64 / 127.12 = 0.69 m

(2) Seismic Condition (→)

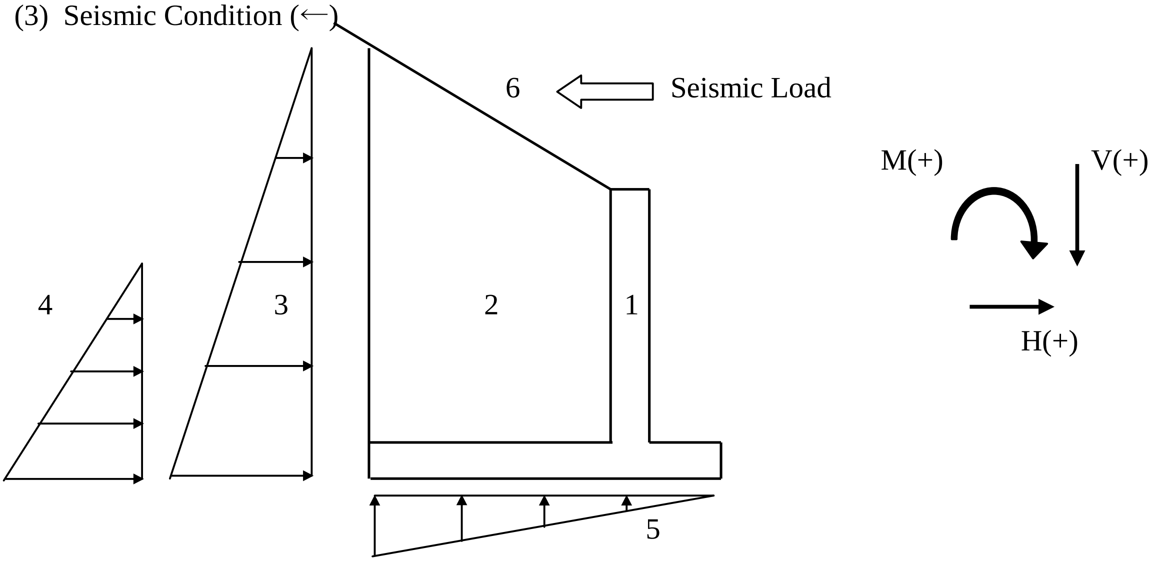


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	76.68	33.53	15.34	14.30
2	Weight of Back Fill	61.20	64.26	12.24	19.16
3	Earth Pressure	21.59	32.39	81.07	53.92
4	Water Pressure	0.00	0.00	8.93	4.02
5	Uplift	-9.92	-9.92	0.00	0.00
6	Srcharge	4.50	4.73	0.00	0.00
	Total	154.05	124.99	117.58	91.40

Vertical Force	V =	154.05	kN
Horizontal Force	H =	117.58	kN
Moment	M =	-33.59	kNm

position X = V・X/V= 124.99 / 154.05 = 0.81 m

(3) Seismic Condition (←)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	76.68	33.53	-15.34	-14.30
2	Weight of Back Fill	61.20	64.26	-12.24	-19.16
3	Earth Pressure	21.59	32.39	81.07	53.92
4	Water Pressure	0.00	0.00	8.93	4.02
5	Uplift	-9.92	-9.92	0.00	0.00
6	Srcharge	4.50	4.73	0.00	0.00
	Total	154.05	124.99	62.42	24.48

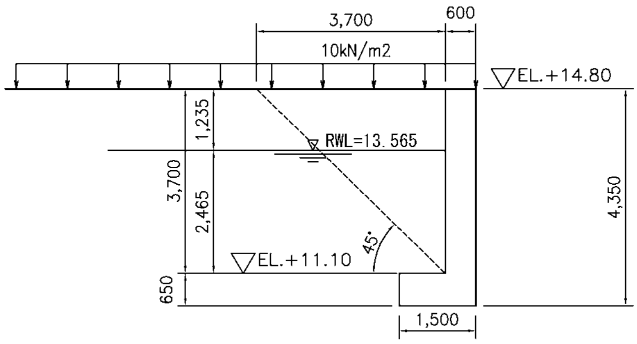
Vertical Force	V =	154.05	kN
Horizontal Force	H =	62.42	kN
Moment	M =	-100.51	kNm

position X = V・X/V= 124.99 / 154.05 = 0.81 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



A= 1/2×(0 + 3.7)× 0 = 0.00 m^2

Converted Load of Embankment

	A		γ		L		qo		
q=	(0.00	×	18.00	+	3.7	×	10.0	)／	3.700

= 10.00 kN/m^2

Earth Pressure at Rest

Po ＝ ko･(q＋γ･h＋γ'･h)

= 0.5 × (10.00 + 18.00 × 1.23 + 11.00 × 2.47)

= 29.66 kN/m^2

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	10.00	kN/m^2

Water Pressure

Pw＝ γw･h

＝ 9.80 × 2.47

＝ 24.21 kN/m^2

Bendin Moment and Shear Force

Effective Load

W ＝ 29.66 + 24.21 ＝ 53.87 kN/m^2

Bending Moment

M ＝ 1/2･W･L^2

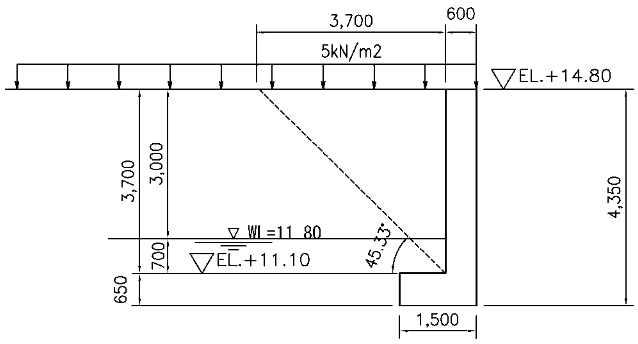
＝ 1/2 × 53.87 × 1.0^2 ＝ 26.94 kN/m^2

Shear Force

S ＝ W･L

= 53.87 ×1.0 ＝ 53.87 kN/m^2

(2) Seismic Condition



A= 1/2×(0 + 3.7)× 0 = 0.00 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	0.00	×	18.00	+	3.7	×	5.0	)／	3.700

= 5.00 kN/m^2

Active Earth Pressure

Pea= kea・(q＋γ・h)＋kea'・γ'・h

= 0.452 × (5.00 + 18.00 × 3.00 + 11.00 × 0.70)

= 30.15 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	5.00	kN/m^2

Water Pressure

Pw ＝ γw ・ h

＝ 9.80 × 0.70

＝ 6.86 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 3.70 × 0.200 ＝ 17.76 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 37.01 kN/m^2

Effective Load

W ＝ 30.15 + 6.86 + 17.76 ＝ 54.77 kN/m^2

Bending Moment

M ＝ 1/2・W・L^2

＝ 1/2 × 54.77 × 1.0^2 ＝ 27.39 kN/m^2

Shear Force

S ＝ W・L

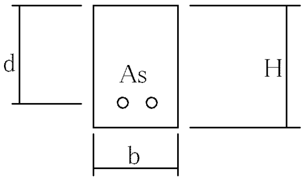
= 54.77 ×1.0 ＝ 54.77 kN/m^2

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	26.94	kN・m
Shear Force	S =	53.87	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	60.0	cm
Equivalent Height（Tensile Side）	d =	51.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.818	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	121.565	N/mm^2	○
Shearing Stress of Concrete	τ =	0.110	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0008871　　　　＝ 6.050

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1186318

j ＝ 1-(k/3)

＝ 0.9604561

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 181.8061607

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 12156.46043

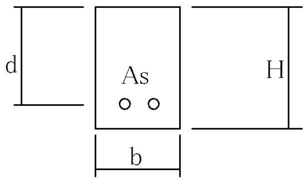
τ＝ (S*10^3)/(b*j*d)

＝ 10.9976345

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	27.39	kN・m
Shear Force	S =	54.77	kN

Width of Member	b =	100.0	cm
Hight of Member	H =	60.0	cm
Equivalent Height（Tensile Side）	d =	51.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.848	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	123.595	N/mm^2	○
Shearing Stress of Concrete	τ =	0.112	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0008871

x ＝ k*d

＝ 6.050

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1186318

j ＝ 1-(k/3)

＝ 0.9604561

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 184.8430119

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 12359.51935

τ＝ (S*10^3)/(b*j*d)

＝ 11.1813707

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.65					=	15.60	kN/m^2
Surcharge	10.00							=	10.00	kN/m^2
Soil	64.38							=	64.38	kN/m^2
Uplift	-9.80	×	3.12	×	0.60	/	1.50	=	-12.23	kN/m^2
Subgrade Reaction	-71.15							=	-71.15	kN/m^2
									6.60	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	161.47	×	1.50					=	242.21	kN
(Vertical wall)	0.60	×	2.00	×	4.05	×	24.0	=	116.64	kN
Soil	2.00	×	4.05	×	1.50	×	18.0	=	218.70	kN
Uplift	-9.80	×	3.12	×	4.05	×	1.50	=	-185.75	kN
Load of Breast Wall	127.12	×	2.00					=	254.24	kN
									646.04	kN

Area of Base	4.05	×	1.50	+	1.0	×	1.5	×	2	=	9.08	m^2
Subgrade Reaction : 245.08 / 8.55	646.04	/	9.08							=	71.15	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 6.60 × 1.0^2 = 3.30 kN·m

Shear Force

$S = W \cdot L =$ 71.15 × 1.00 = 71.15 kN

(2) Seismic Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.65					=	15.60	kN/m^2
Surcharge	5.00							=	5.00	kN/m^2
Soil	64.38							=	64.38	kN/m^2
Uplift	-9.80	×	1.35	×	0.60	/	1.50	=	-5.29	kN/m^2
Subgrade Reaction	-88.69							=	-88.69	kN/m^2
									-9.00	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	161.47	×	1.50					=	242.21	kN
(Vertical wall)	0.60	×	2.00	×	4.05	×	24.0	=	116.64	kN
Soil	2.00	×	4.05	×	1.50	×	18.0	=	218.70	kN
Uplift	-9.80	×	1.35	×	4.05	×	1.50	=	-80.37	kN
Load of Breast Wall	154.05	×	2.00					=	308.10	kN
									805.28	kN

Subgrade Reaction : 245.08 / 8.55 805.28 / 9.08 = 88.69 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × -9.00 × 1.0^2 = -4.50 kN·m

Shear Force

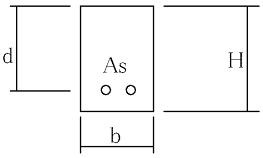
$S = W \cdot L =$ 88.69 × 1.00 = 88.69 kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	3.30	kN・m
Shear Force	S =	71.15	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	70.0	cm
Equivalent Height（Tensile Side）	d =	61.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.169	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	12.410	N/mm^2	○
Shearing Stress of Concrete	τ =	0.121	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0007416

x ＝ k*d

＝ 6.652

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1090552

j ＝ 1-(k/3)

＝ 0.9636483

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 16.8779357

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 1240.98345

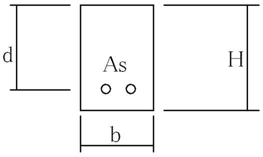
τ＝ (S*10^3)/(b*j*d)

＝ 12.1039330

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	11.5	cm



Bending Moment	M =	4.50	kN・m
Shear Force	S =	88.69	kN

Width of Member	b =	100.0	cm
Hight of Member	H =	70.0	cm
Equivalent Height（Tensile Side）	d =	58.500	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.246	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	17.659	N/mm^2	○
Shearing Stress of Concrete	τ =	0.157	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0007733

x ＝ k*d

＝ 6.507

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1112259

j ＝ 1-(k/3)

＝ 0.9629247

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 24.5545807

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 1765.87698

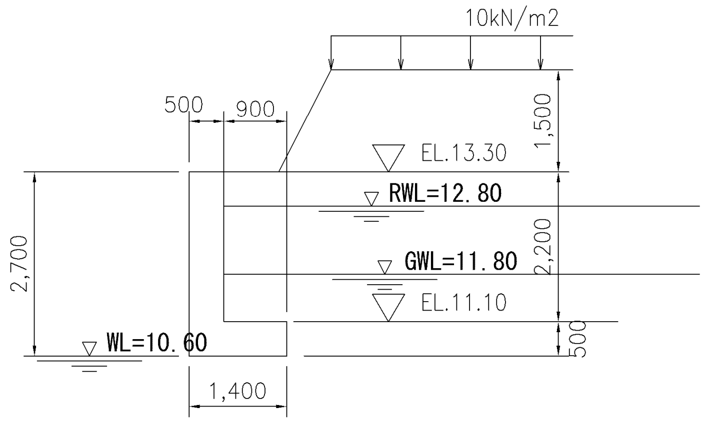
τ＝ (S*10^3)/(b*j*d)

＝ 15.7444126

Structural Culculation of Breast Wall

MSL3 Land Side

1. Calculation Model



・Dimension

Height of Vertical wall	2.20	m
Tickness of Vertical Wall	0.50	m
Width of Bottom Slab	1.40	m
Tickness of Bottom Slab	0.50	m
Height of Embankment	1.50	m

・Elevation

Top of the wall	EL+	13.30	m
Upperside of Bottom Slab	EL+	11.10	m
Underside of Bottom Slab	EL+	10.60	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	10.60	12.80	—	○	○	○	○	○	○
2	Seismic	10.60	11.80	←	◎	◎	○	○	○	○
3	Seismic	10.60	11.80	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

－ : Without Considering

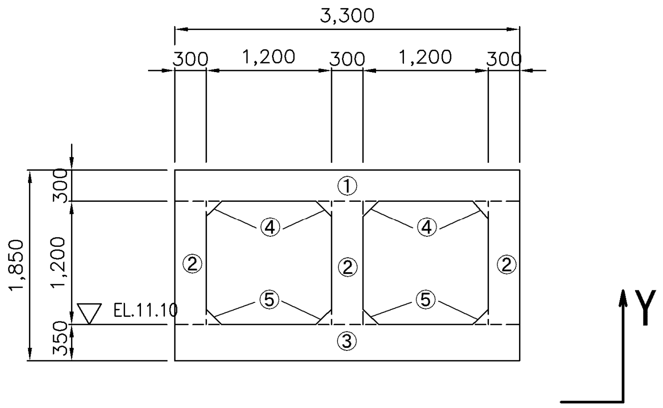
2. Design Condition

Unit Weight of Concrete		————	24.0	kN/m^3
Unit Weight of Water		————	9.8	kN/m^3
Unit Weight of Soil				
Saturated		————	20.0	kN/m^3
Wet		————	18.0	kN/m^3
Submerged		————	11.0	kN/m^3
Backfill Material		————	Sandy Soil	
internal Friction Angle of Soil		————	30.00	°
Seismic Lad		————	0.20	
Water Level				
Forth	Normal	————	10.60	m
	Seismic	————	10.60	m
Back	Normal	————	12.80	m
	Seismic	————	11.80	m
Difference	Normal	————	2.20	m
	Seismic	————	1.20	m
Earth Pressure	Normal	————	Earth Pressure at Rest	
	Seismic	————	Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane		————	0.00	°
Surchage	Normal	————	10.0	kN/m^2
	Seismic	————	5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height（h）	α	n	A（m^2）	Y（m）	A・Y（m^3）
①	3.300	0.300	1.0	1	0.990	1.700	1.683
②	0.300	1.200	1.0	3	1.080	0.950	1.026
③	3.300	0.350	1.0	1	1.155	0.175	0.202
④	0.150	0.150	0.5	4	0.045	1.500	0.068
⑤	0.150	0.150	0.5	4	0.045	0.400	0.018
計					3.315		2.997

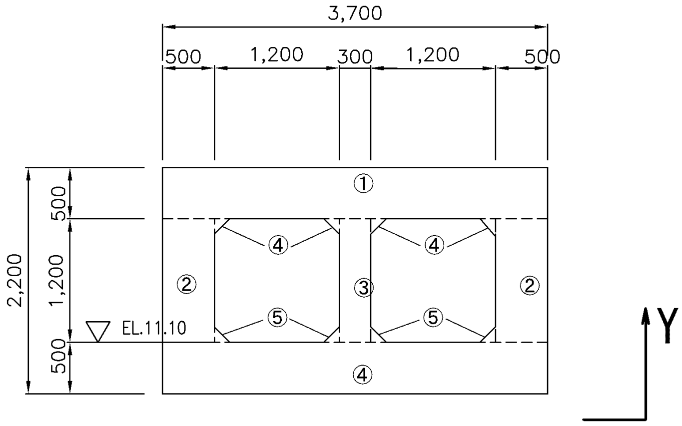
α；Triangle 0.5, Rectangle 1.0

ｎ； Number

Centroid of Box Culvet

Y＝ ΣA・Y／ΣA ＝ 0.904 m

Breast Wal Section



	Width (B)	Height（h）	α	n	A（m^2）
①	3.700	0.500	1.0	1	1.850
②	0.500	1.200	1.0	2	1.200
③	0.300	1.200	1.0	1	0.360
④	3.850	0.500	1.0	1	1.925
⑤	0.150	0.150	0.5	4	0.045
⑥	0.150	0.150	0.5	4	0.045
計					5.425

α;Triangle 0.5, Rectangle 1.0

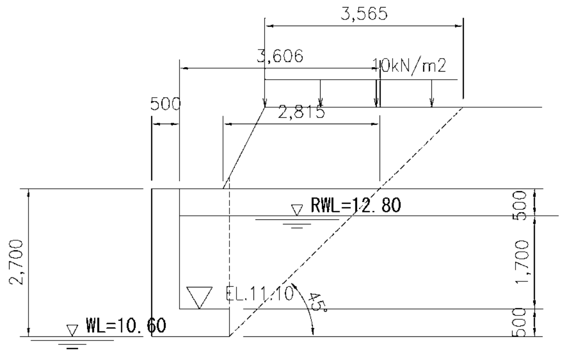
ｎ;　Number

Weight of Body

W = γ×A×1.0　　＝　130.200　kN

3.2 Converted Load of Embankment

(1) Normal Condition



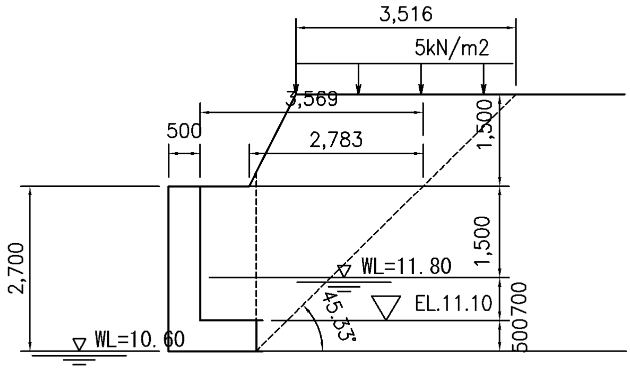
A= 1/2×(2.815 + 3.565)× 1.5 = 4.79 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	4.79	×	18.00	+	3.565	×	10.0	)／	3.600

= 33.85 kN/m^2

(2) Seismic Condition



A= 1/2×(2.783 + 3.516)× 1.5 = 4.72 m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	4.72	×	18.00	+	3.516	×	5.0	)／	3.569

= 28.73 kN/m^2

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

φ = 30 ° θ = 0.000 ° $\delta = \Phi / 2$ = 15.0 ° α = 0 °

kh = 0.20 $\theta o = \tan^{-1} kh$ = 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta o - \theta)}{\cos\theta o \times \cos^2\theta \times \cos(\theta + \theta o + \delta) \times \left\{1 + \sqrt{\dfrac{\sin(\varphi+\delta)\times\sin(\varphi-\alpha-\theta o)}{\cos(\theta+\theta o+\delta)\times\cos(\theta-\alpha)}}\right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{1 + \sqrt{\dfrac{0.2266}{0.8964}}\right\}^2} = \mathbf{0.452}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °

$\delta =$ $\varphi/2 =$ 15 °(Between soil and soil)

$\theta =$ 0

$\alpha =$ 0

$\theta_O =$ $\tan^{-1} Kh =$ 11.310 °

kh = 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi + \delta + \theta - \alpha) + \sec(\varphi + \delta + \theta - \alpha) \cdot X$$

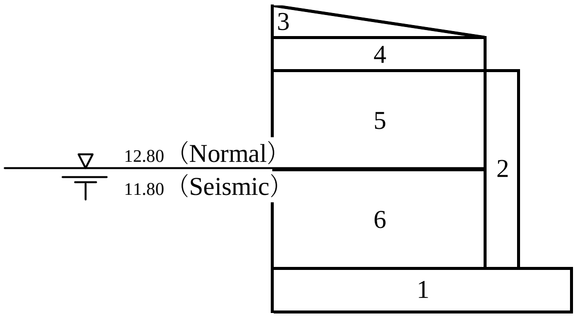
$$X = \sqrt{\frac{\cos(\theta + \delta + \theta_O) \cdot \sin(\varphi + \delta)}{\cos(\theta - \alpha) \cdot \sin(\varphi - \alpha - \theta_O)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1} 1 / 0.9892 = \boxed{45.311 °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.400	0.500	1.00		24.0	16.80
2	0.500	2.200	1.00		24.0	26.40

No	V (kN)	X (m)	V・X (kNm)	H (kN)	Y (m)	H・Y (kNm)
1	16.80	0.700	11.76	3.36	-0.654	-2.20
2	26.40	0.250	6.60	5.28	0.696	3.67
Total	43.20		18.36	8.64		1.47

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.120	0.230	1.00		18.0	0.25
4	0.900	0.000	1.00		18.0	0.00
5	0.900	0.500	1.00		18.0	8.10
6	0.900	1.700	1.00		20.0	30.60

No	V (kN)	X (m)	V・X (kNm)
3	0.25	1.340	0.34
4	0.00	0.950	0.00
5	8.10	0.950	7.70
6	30.60	0.950	29.07
Total	38.95		37.11

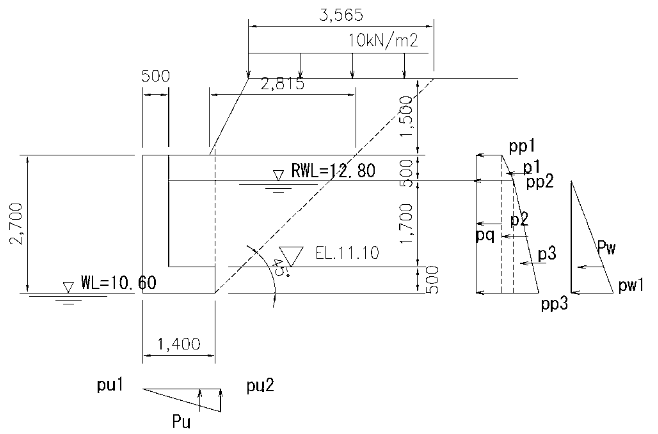
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.120	0.230	1.00		18.0	0.25
4	0.900	0.000	1.00		18.0	0.00
5	0.900	1.500	1.00		18.0	24.30
6	0.900	0.700	1.00		20.0	12.60

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
3	0.25	1.340	0.34	0.05	1.911	0.10
4	0.00	0.950	0.00	0.00	1.796	0.00
5	24.30	0.950	23.09	4.86	1.046	5.08
6	12.60	0.950	11.97	2.52	-0.054	-0.14
Total	37.15		35.40	7.43		5.04

(5) Earth Pressure

1) Normal Condition

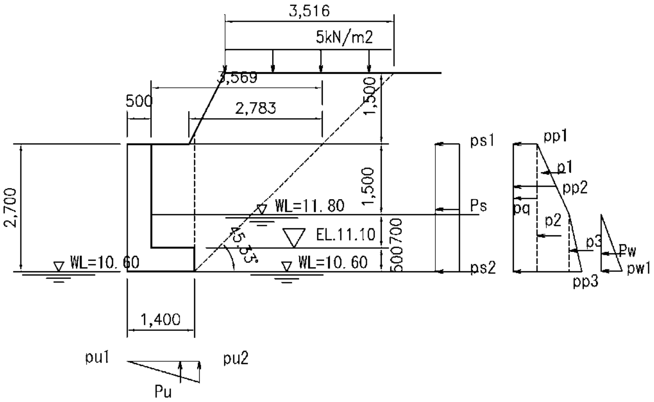


φ	:	internal Friction Angle of Soil	30
Ko	:	Earth Pressure at Rest	0.5
q	:	Converted Load of Embankment	33.85 kN/m^2

	V(kN)	H(kN/m)
Pp1	0.00	16.93
Pp2	0.00	21.43
Pp3	0.00	33.53

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	1.13	1.639	1.85
Earth Pressure (p2)	0.00	0.000	0.00	9.90	0.196	1.94
Earth Pressure (p3)	0.00	0.000	0.00	13.31	-0.171	-2.28
Earth Pressure (pq)	0.00	0.000	0.00	45.70	0.446	20.38
Total	0.00		0.00	70.04		21.89

2) Seismic Condition



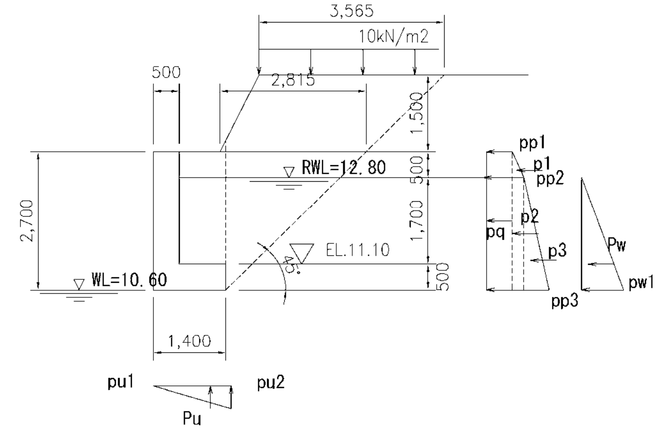
φ	:	internal Friction Angle of Soil	30	°
δ	:	Angle of Wall Friction	15	°
		δ=φ/2（Seismic Condition）		
α	:	Angle Between Ground Surface and Horizontal Plane	0	°
θ	:	Angle Between Back side Surface of Wall and Vertical Plane	0	°
Ka	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	28.73	kN/m^2

	V(kN)	H(kN/m)
Earth Pressure (pp1)		12.99
Earth Pressure (pp2)		25.19
Earth Pressure (pp3)		31.16

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Earth Pressure (p1)	2.37	1.400	3.32	8.84	0.796	7.04
Earth Pressure (p2)	3.79	1.400	5.31	7.07	-0.304	-2.15
Earth Pressure (p3)	0.93	1.400	1.30	3.46	-0.504	-1.74
Earth Pressure (pq)	9.07	1.400	12.70	33.87	0.446	15.11
Total	16.16		22.63	53.24		18.26

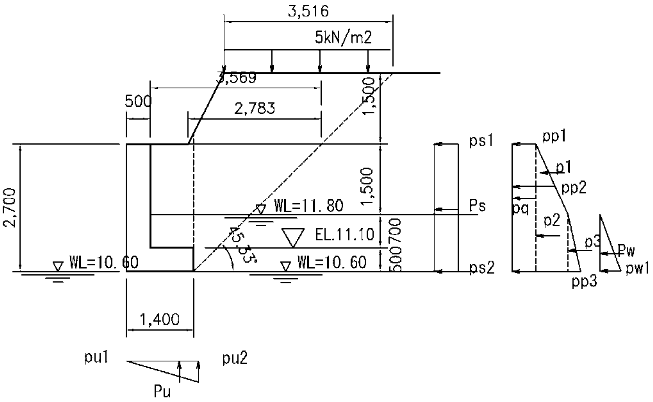
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	23.72	-0.171	-4.06
Total	0.00		0.00	23.72		-4.06

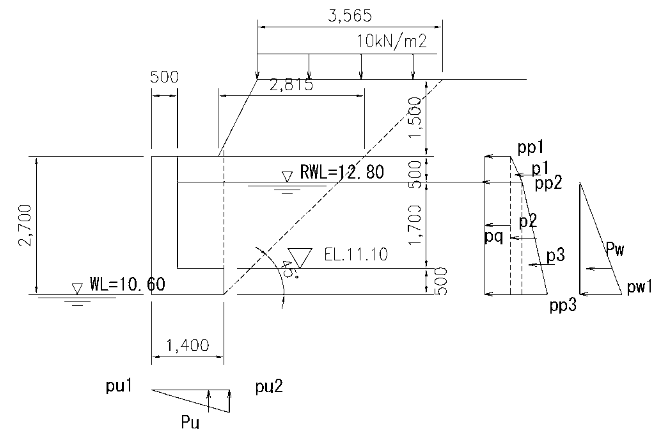
(2) Seismic Condition



	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	7.06	0.400	2.82
Total	0.00					

3.5 Uplift

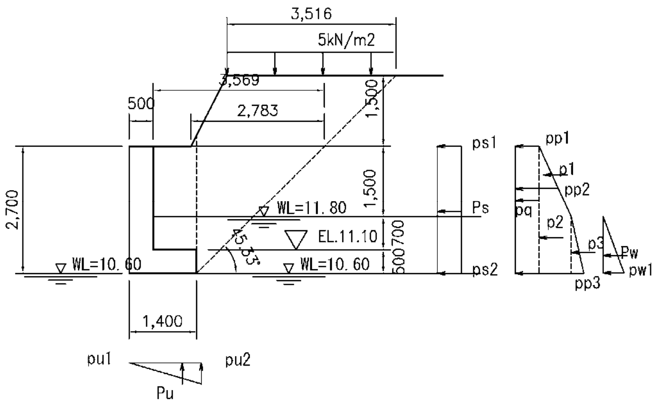
(1) Normal Conditon



	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	21.56	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	15.09	0.933	14.08	0.00	0.000	0.00
Total	15.09		14.08	0.00		0.00

(2) Seismic Conditon

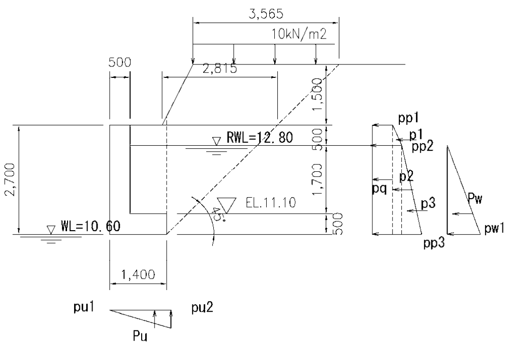


	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	11.76	0.00

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Uplift Pu	8.23	0.933	7.68	0.00	0.000	0.00
Total	8.23		7.68	0.00		0.00

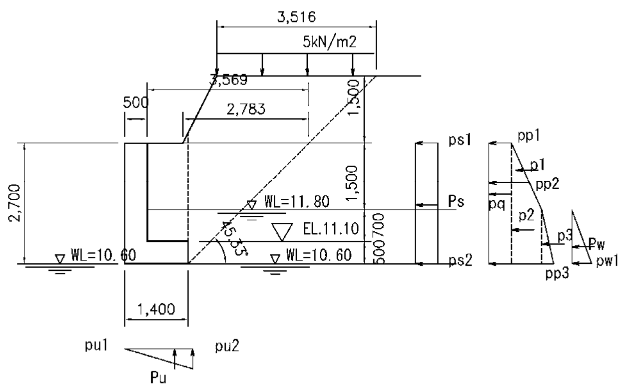
3.6 Surcharge

(1) Normal Condition



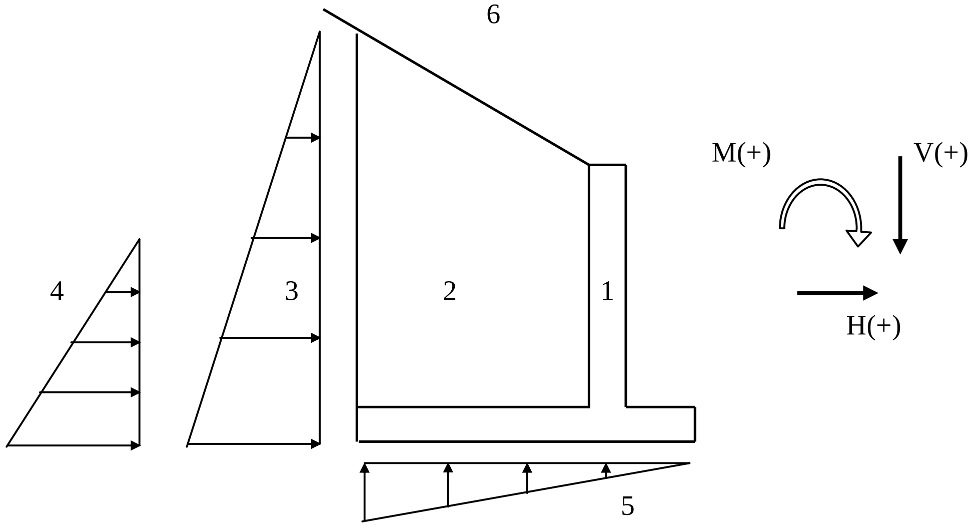
	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	10.0	0.00		0.00	0.00
Total				0.00	0.00

(2) Seismic Conditon



	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	5.0	0.00		0.00	0.00
Total				0.00	0.00

3.7 Summary of Load
(1) Normal Condition

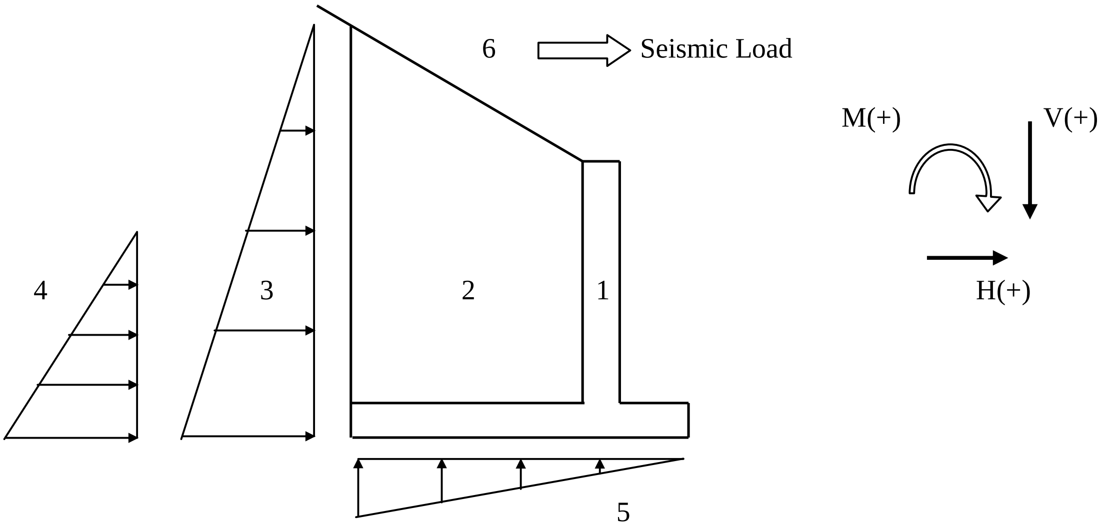


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	43.20	18.36	0.00	0.00
2	Weight of Back Fill	38.95	37.11	0.00	0.00
3	Earth Pressure	0.00	0.00	70.04	21.89
4	Water Pressure	0.00	0.00	23.72	-4.06
5	Uplift	-15.09	-14.08	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	Total	67.06	41.39	93.76	17.83

Vertical Force	V=	67.06	kN
Horizontal Force	H=	93.76	kN
Moment	M=	-23.56	kNm

position X = V・X/V= 41.39 / 67.06 = 0.62 m

(2) Seismic Condition (→)

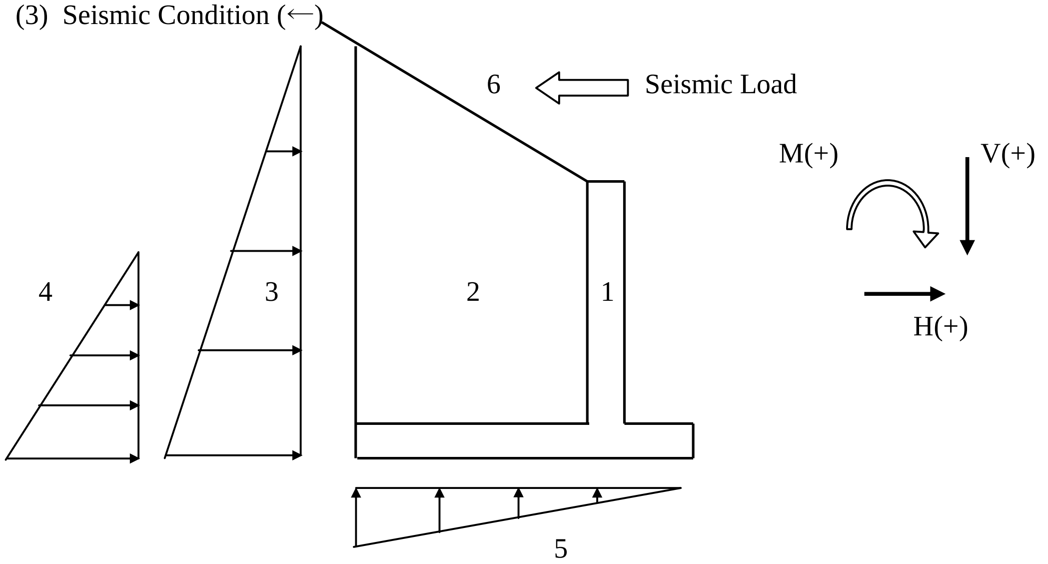


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	43.20	18.36	8.64	1.47
2	Weight of Back Fill	37.15	35.40	7.43	5.04
3	Earth Pressure	16.16	22.63	53.24	18.26
4	Water Pressure	0.00	0.00	7.06	2.82
5	Uplift	-8.23	-7.68	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	88.28	68.71	76.37	27.59

Vertical Force	V =	88.28	kN
Horizontal Force	H =	76.37	kN
Moment	M =	-41.12	kNm

position X = V・X/V= 68.71 / 88.28 = 0.78 m

(3) Seismic Condition (←)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	43.20	18.36	-8.64	-1.47
2	Weight of Back Fill	37.15	35.40	-7.43	-5.04
3	Earth Pressure	16.16	22.63	53.24	18.26
4	Water Pressure	0.00	0.00	7.06	2.82
5	Uplift	-8.23	-7.68	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	88.28	68.71	44.23	14.57

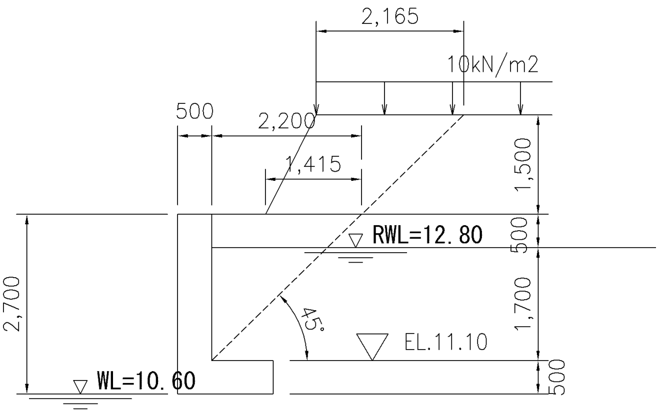
Vertical Force	V =	88.28	kN
Horizontal Force	H =	44.23	kN
Moment	M =	-54.14	kNm

position X = V・X/V= 68.71 / 88.28 = 0.78 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



$A = 1/2 \times (1.415 + 2.165) \times 1.5 = 2.69 \ m^2$

Converted Load of Embankment

	A		γ		L		qo		
q=	(2.69	×	18.00	+	2.165	×	10.0	)／	2.200

$= 31.85 \ kN/m^2$

Earth Pressure at Rest

$Po = ko \cdot (q + \gamma \cdot h + \gamma' \cdot h)$

$= 0.5 \times (31.85 + 18.00 \times 0.50 + 11.00 \times 1.70)$

$= 29.78 \ kN/m^2$

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	31.85	kN/m^2

Water Pressure

$Pw = \gamma w \cdot h$

$= 9.80 \times 1.70$

$= 16.66 \ kN/m^2$

Bendin Moment and Shear Force

Effective Load

$W = 29.78 + 16.66 = 46.44 \ kN/m^2$

Bending Moment

$M = 1/2 \cdot W \cdot L^2$

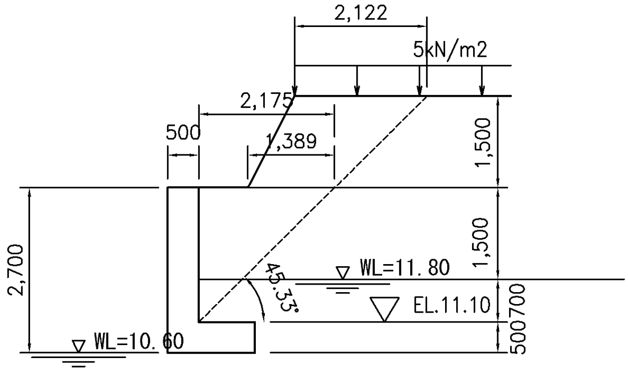
$= 1/2 \times 46.44 \times 1.0^2 = 23.22 \ kN/m^2$

Shear Force

$S = W \cdot L$

$= 46.44 \times 1.0 = 46.44 \ kN/m^2$

(2) Seismic Condition



A= 1/2×(1.389 + 2.122)× 1.5 = 2.63 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	2.63	×	18.00	+	2.122	×	5.0	)／	2.175

= 26.64 kN/m^2

Active Earth Pressure

Pea= kea・(q＋γ・h)＋kea'・γ'・h

= 0.452 × (26.64 + 18.00 × 1.50 + 11.00 × 0.70)

= 27.73 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	26.64	kN/m^2

Water Pressure

Pw ＝ γw・h

= 9.80 × 0.70

= 6.86 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 2.20 × 0.200 ＝ 10.56 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 34.59 kN/m^2

Effective Load

W ＝ 27.73 + 6.86 + 10.56 ＝ 45.15 kN/m^2

Bending Moment

M ＝ 1/2・W・L^2

＝ 1/2 × 45.15 × 1.0^2 ＝ 22.58 kN/m^2

Shear Force

S ＝ W・L

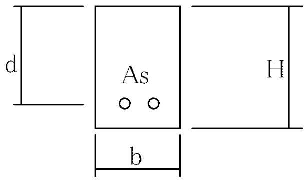
= 45.15 ×1.0 ＝ 45.15 kN/m^2

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	23.22	kN・m
Shear Force	S =	46.44	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete σc=	2.200	N/mm^2	○
Tensile Stress of Rainforcing Bar σs=	130.920	N/mm^2	○
Shearing Stress of Concrete τ =	0.118	N/mm^2	○

Allowable Compressive Stress of Concrete σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 219.9594087

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 13091.97045

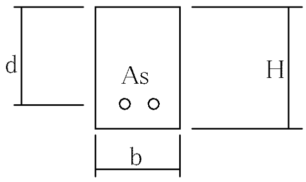
τ＝ (S*10^3)/(b*j*d)

＝ 11.8454578

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	22.58	kN・m
Shear Force	S =	45.15	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	2.139	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	127.311	N/mm^2	○
Shearing Stress of Concrete	τ =	0.115	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0011034

x ＝ k*d

＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 213.8967894

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 12731.12372

τ＝ (S*10^3)/(b*j*d)

＝ 11.5164173

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50					=	12.00	kN/m^2
Surcharge	10.00							=	10.00	kN/m^2
Soil	38.95							=	38.95	kN/m^2
Uplift	-9.80	×	2.20	×	0.50	/	1.40	=	-7.70	kN/m^2
Subgrade Reaction	-34.28							=	-34.28	kN/m^2
									18.97	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	130.20	×	1.40					=	182.28	kN
(Vertical wall)	0.50	×	0.50	×	3.70	×	24.0	=	22.20	kN
Soil	0.50	×	3.70	×	1.40	×	18.0	=	46.62	kN
Uplift	-9.80	×	2.20	×	3.70	×	1.40	=	-111.68	kN
Load of Breast Wall	67.06	×	2.00					=	134.12	kN
									273.54	kN

Area of Base	3.70	×	1.40	+	1.0	×	1.4	×	2	=	7.98	m^2
Subgrade Reaction : 245.08 / 8.55	273.54	/	7.98							=	34.28	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 18.97 × 1.0^2 = 9.49 kN·m

Shear Force

$S = W \cdot L =$ 34.28 × 1.00 = 34.28 kN

(2) Seismic Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50					=	12.00	kN/m^2
Surcharge	5.00							=	5.00	kN/m^2
Soil	38.95							=	38.95	kN/m^2
Uplift	-9.80	×	1.20	×	0.50	/	1.40	=	-4.20	kN/m^2
Subgrade Reaction	-45.96							=	-45.96	kN/m^2
									5.79	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	130.20	×	1.40					=	182.28	kN
(Vertical wall)	0.50	×	0.50	×	3.70	×	24.0	=	22.20	kN
Soil	0.50	×	3.70	×	1.40	×	18.0	=	46.62	kN
Uplift	-9.80	×	1.20	×	3.70	×	1.40	=	-60.92	kN
Load of Breast Wall	88.28	×	2.00					=	176.56	kN
									366.74	kN

Subgrade Reaction : 245.08 / 8.55 366.74 / 7.98 = 45.96 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 5.79 × 1.0^2 = 2.90 kN·m

Shear Force

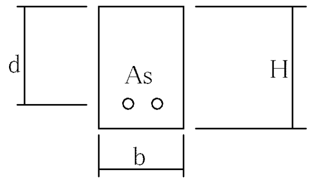
$S = W \cdot L =$ 45.96 × 1.00 = 45.96 kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	9.49	kN・m
Shear Force	S =	34.28	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.899	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	53.507	N/mm^2	○
Shearing Stress of Concrete	τ =	0.087	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 89.8972777

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 5350.68043

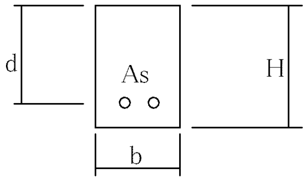
τ＝ (S*10^3)/(b*j*d)

＝ 8.7438047

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	2.90	kN・m
Shear Force	S =	45.96	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.275	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	16.351	N/mm^2	○
Shearing Stress of Concrete	τ =	0.117	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 27.4712440

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 1635.08675

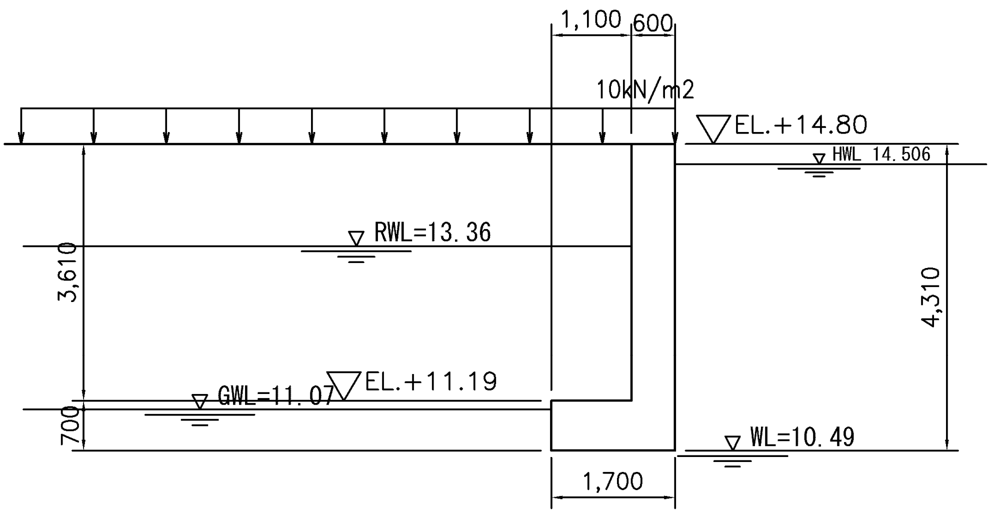
τ＝ (S*10^3)/(b*j*d)

＝ 11.7230241

Structural Culculation of Breast Wall

MSL4 River Side

1. Calculation Model



・Dimension

Height of Vertical wall	3.61	m
Tickness of Vertical Wall	0.50	m
Width of Bottom Slab	1.70	m
Tickness of Bottom Slab	0.70	m
Height of Embankment	0	m

・Elevation

Top of the wall	EL+	14.80	m
Upperside of Bottom Slab	EL+	11.19	m
Underside of Bottom Slab	EL+	10.49	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	10.49	13.36	―	○	○	○	○	○	○
2	Seismic	10.49	11.07	←	◎	◎	○	○	○	○
3	Seismic	10.49	11.07	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

― : Without Considering

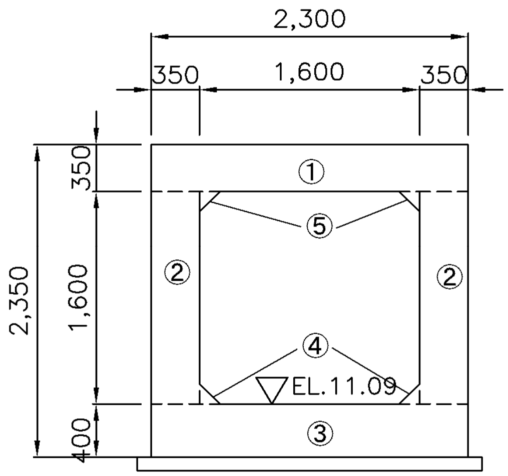
2. Design Condition

Unit Weight of Concrete		————	24.0	kN/m^3
Unit Weight of Water		————	9.8	kN/m^3
Unit Weight of Soil				
Saturated		————	20.0	kN/m^3
Wet		————	18.0	kN/m^3
Submerged		————	11.0	kN/m^3
Backfill Material		————	Sandy Soil	
internal Friction Angle of Soil		————	30.00	°
Seismic Lad		————	0.20	
Water Level				
Forth	Normal	————	10.49	m
	Seismic	————	10.49	m
Back	Normal	————	13.36	m
	Seismic	————	11.07	m
Difference	Normal	————	2.87	m
	Seismic	————	0.58	m
Earth Pressure	Normal	————	Earth Pressure at Rest	
	Seismic	————	Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane		————	0.00	°
Surchage	Normal	————	10.0	kN/m^2
	Seismic	————	5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height (h)	α	n	A (m²)	Y (m)	A・Y (m³)
①	2.300	0.350	1.0	1	0.805	2.175	1.751
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.300	0.400	1.0	1	0.920	0.200	0.184
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
計					2.891		3.334

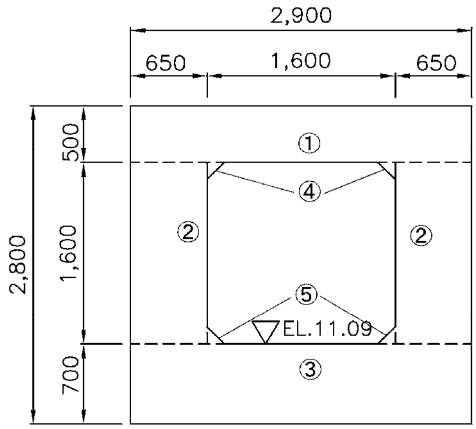
α ;Triangle 0.5, Rectangle 1.0

n; Number

Centroid of Box Culvet

$Y = \Sigma A \cdot Y / \Sigma A$ = 1.153 m

Breast Wal Section



	Width (B)	Height (h)	α	n	A (m^2)
①	2.900	0.500	1.0	1	1.450
②	0.650	1.600	1.0	2	2.080
③	2.900	0.700	1.0	1	2.030
④	0.150	0.150	0.5	2	0.023
⑤	0.150	0.150	0.5	2	0.023
計					5.606

α ;Triangle 0.5, Rectangle 1.0

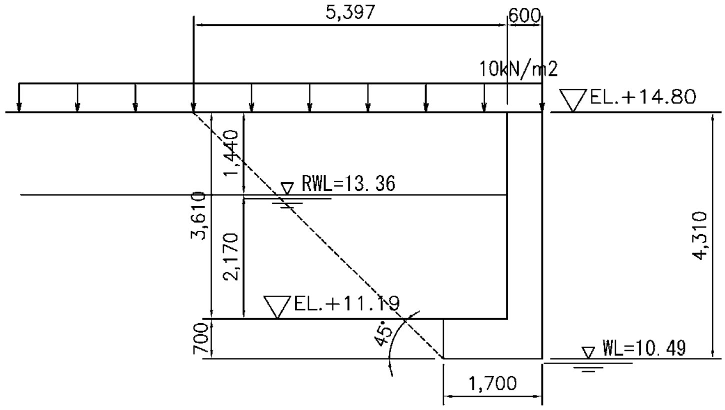
n; Number

Weight of Body

W = $\gamma \times A \times 1.0$ = 134.544 kN

3.2 Converted Load of Embankment

(1) Normal Condition



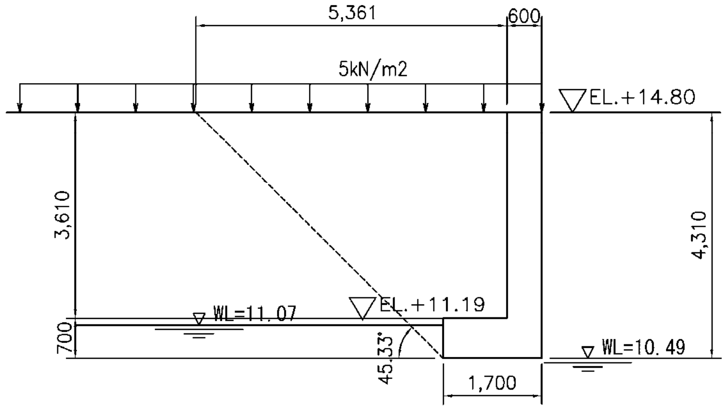
A=	1/2×(	0	+	5.397	)×	0	=	0.00	m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.397	×	10.0	)／	5.397

= 10.00 kN/m^2

(2) Seismic Condition



A=	1/2×(	0.000	+	5.361	)×	0	=	0.00	m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.361	×	5.0	)／	5.361

= 5.00 kN/m^2

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

φ＝ 30 ° θ＝ 0.000 ° δ=Φ／2＝ 15.0 ° α＝ 0 °

kh＝ 0.20 θo＝tan⁻¹ kh＝ 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta_o - \theta)}{\cos\theta_o \times \cos^2\theta \times \cos(\theta + \theta_o + \delta) \times \left\{1 + \sqrt{\dfrac{\sin(\varphi + \delta) \times \sin(\varphi - \alpha - \theta_o)}{\cos(\theta + \theta_o + \delta) \times \cos(\theta - \alpha)}}\right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{1 + \sqrt{\dfrac{0.2266}{0.8964}}\right\}^2} = \mathbf{0.452}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °

$\delta =$ $\varphi/2 =$ 15 °(Between soil and soil)

$\theta =$ 0

$\alpha =$ 0

$\theta_O =$ $\tan^{-1} Kh =$ 11.310 °

$kh =$ 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi + \delta + \theta - \alpha) + \sec(\varphi + \delta + \theta - \alpha) \cdot X$$

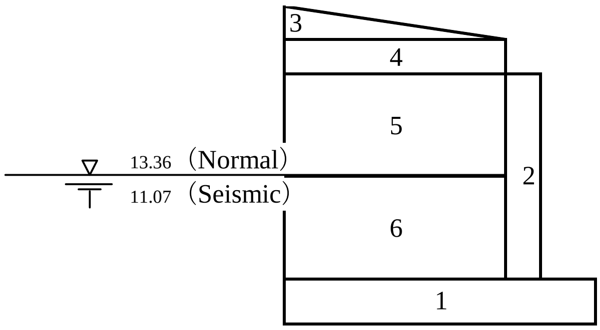
$$X = \sqrt{\frac{\cos(\theta + \delta + \theta_O) \cdot \sin(\varphi + \delta)}{\cos(\theta - \alpha) \cdot \sin(\varphi - \alpha - \theta_O)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1} 1 / 0.9892 = \boxed{45.311 °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.700	0.700	1.00		24.0	28.56
2	0.600	3.610	1.00		24.0	51.98

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
1	28.56	0.850	24.28	5.71	-0.803	-4.59
2	51.98	0.300	15.59	10.40	1.352	14.06
Total	80.54		39.87	16.11		9.47

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	1.100	0.000	1.00		18.0	0.00
4	1.100	0.000	1.00		18.0	0.00
5	1.100	1.440	1.00		18.0	28.51
6	1.100	2.170	1.00		20.0	47.74

No	V (kN)	X (m)	V･X (kNm)
3	0.00	1.150	0.00
4	0.00	1.150	0.00
5	28.51	1.150	32.79
6	47.74	1.150	54.90
Total	76.25		87.69

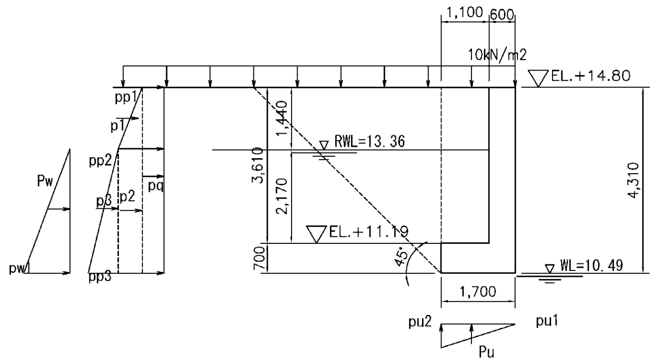
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	1.100	0.000	1.00		18.0	0.00
4	1.100	0.000	1.00		18.0	0.00
5	1.100	3.610	1.00		18.0	71.48
6	1.100	0.000	1.00		20.0	0.00

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
3	0.00	1.150	0.00	0.00	3.157	0.00
4	0.00	1.150	0.00	0.00	3.157	0.00
5	71.48	1.150	82.20	14.30	1.352	19.33
6	0.00	1.150	0.00	0.00	-0.453	0.00
Total	71.48		82.20	14.30		19.33

(5) Earth Pressure

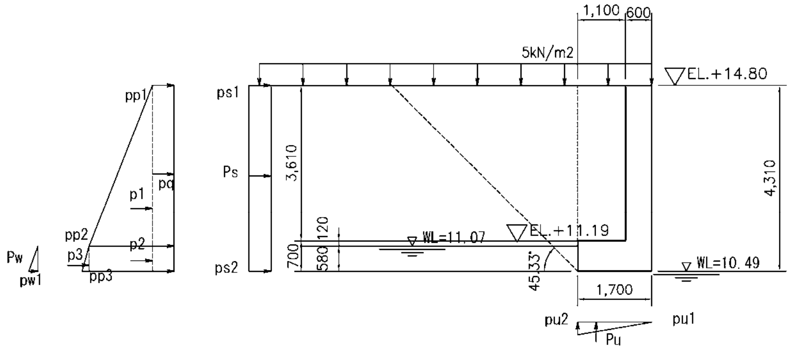
1) Normal Condition



φ	:	internal Friction Angle of Soil	30	
Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	10.00	kN/m^2

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	9.33	2.197	20.50
Earth Pressure (p2)	0.00	0.000	0.00	37.20	0.282	10.49
Earth Pressure (p3)	0.00	0.000	0.00	22.65	-0.196	-4.44
Earth Pressure (pq)	0.00	0.000	0.00	21.55	1.002	21.59
Total	0.00		0.00	90.73		48.14

2) Seismic Condition

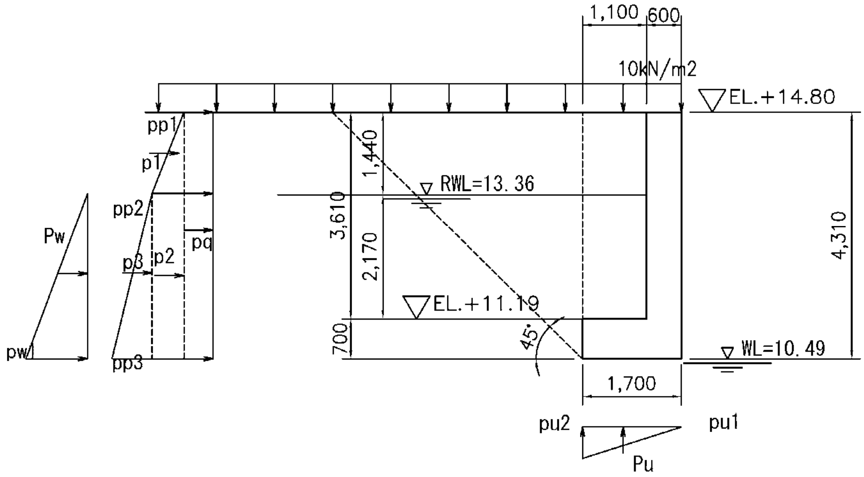


φ	:	internal Friction Angle of Soil	30 °
δ	:	Angle of Wall Friction δ=φ/2（Seismic Condition）	15 °
α	:	Angle Between Ground Surface and Horizontal Plane	0 °
θ	:	Angle Between Back side Surface of Wall and Vertical Plane	0 °
Ka	:	Coefficient of Active Earth Pressure	0.452
q	:	Converted Load of Embankment	5.0 kN/m^2

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	14.65	1.700	24.91	54.67	0.670	36.63
Earth Pressure (p2)	2.28	1.700	3.88	17.00	-0.863	-14.67
Earth Pressure (p3)	0.22	1.700	0.37	8.94	-0.960	-8.58
Earth Pressure (pq)	2.52	1.700	4.28	9.41	1.002	9.43
Total	19.67		29.16	90.02		22.81

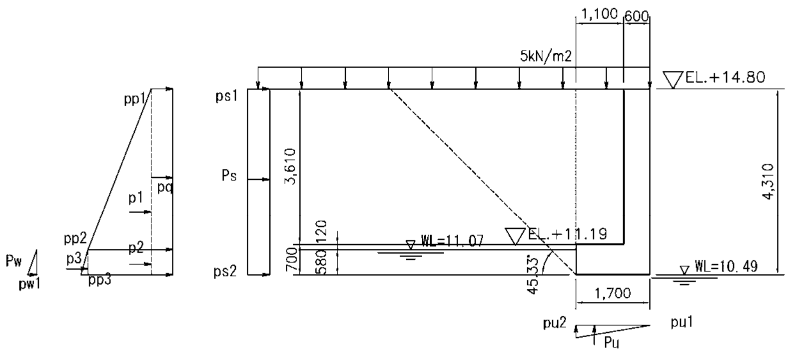
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	40.36	-0.196	-7.91
Total	0.00		0.00	40.36		-7.91

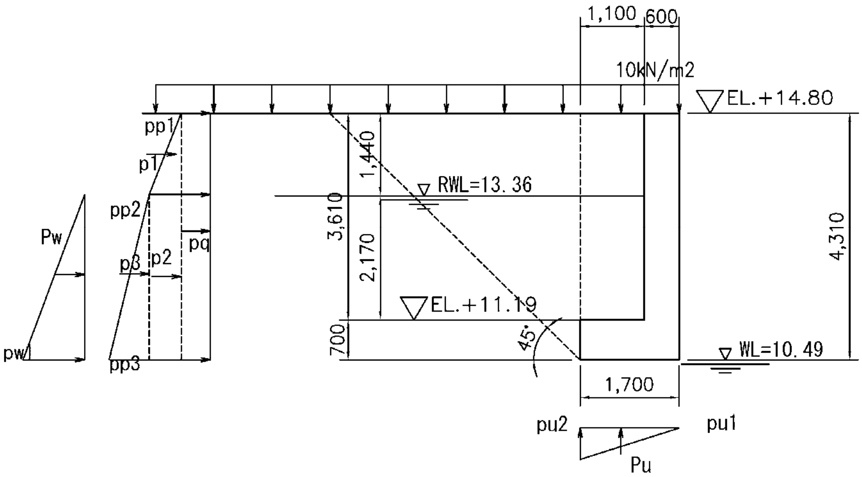
(2) Seismic Condition



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	1.65	-0.960	-1.58
Total	0.00		0.00	1.65		-1.58

3.5 Uplift

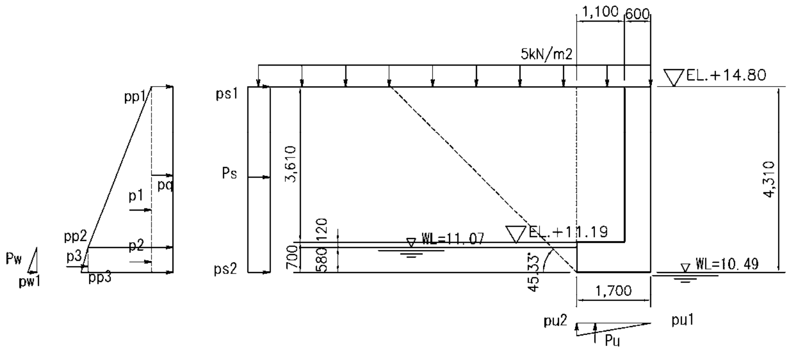
(1) Normal Conditon



	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	28.13	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	23.91	1.133	27.09	0.00	0.000	0.00
Total	23.91		27.09	0.00		0.00

(2) Seismic Conditon

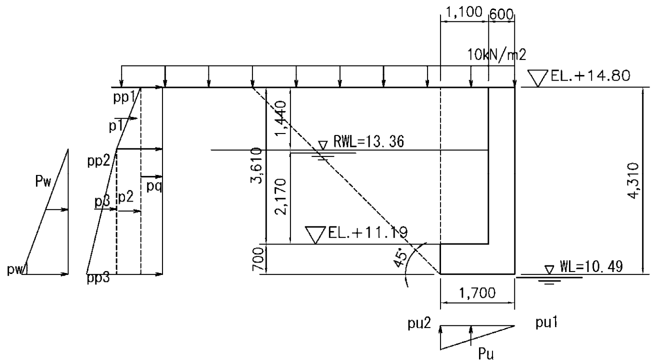


	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	5.68	0.00

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Uplift Pu	4.83	1.133	5.47	0.00	0.000	0.00
Total	4.83		5.47	0.00		0.00

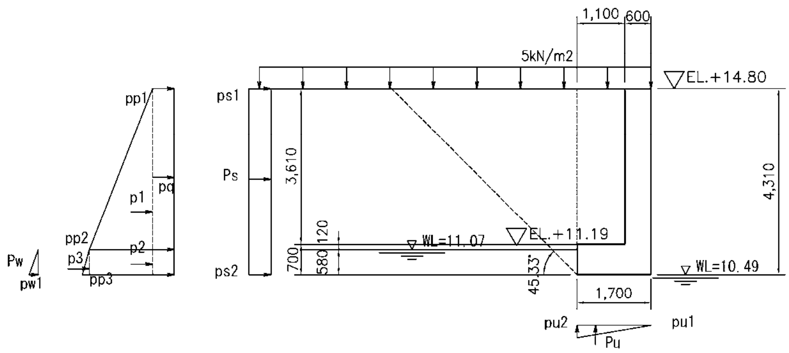
3.6 Surcharge

(1) Normal Condition



	q(kN/m^2)	B(m)	X(m)	V(kN)	V・X(kNm)
1	10.0	1.10	1.150	11.00	12.65
Total				11.00	12.65

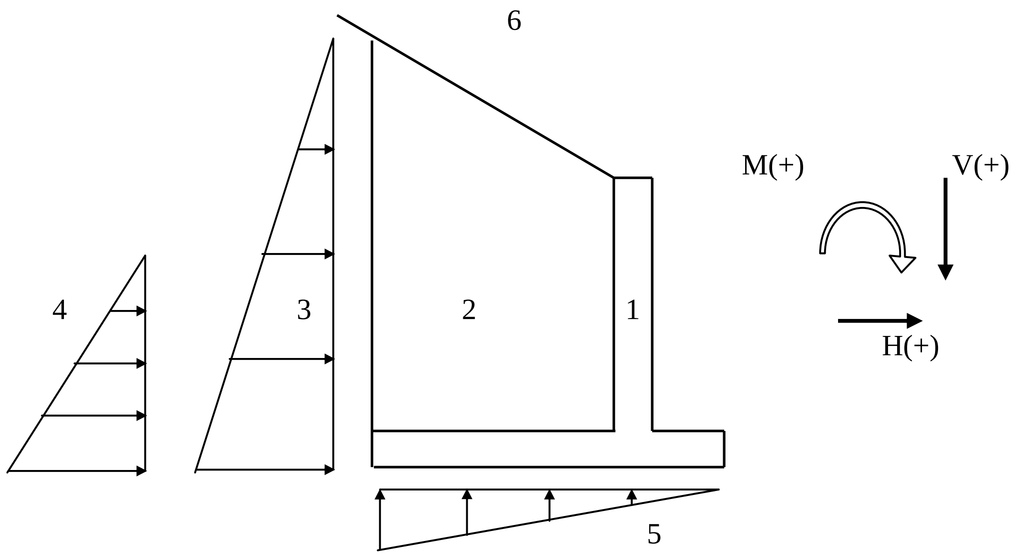
(2) Seismic Conditon



	q(kN/m^2)	B(m)	X(m)	V(kN)	V・X(kNm)
1	5.0	1.10	1.150	5.50	6.33
Total				5.50	6.33

3.7 Summary of Load

(1) Normal Condition

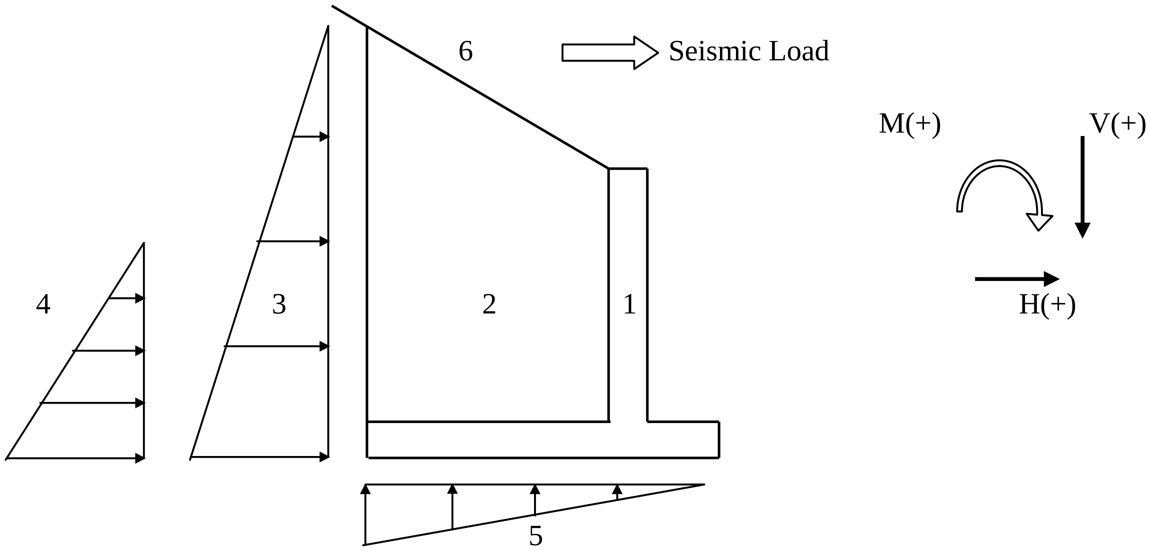


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.54	39.87	0.00	0.00
2	Weight of Back Fill	76.25	87.69	0.00	0.00
3	Earth Pressure	0.00	0.00	90.73	48.14
4	Water Pressure	0.00	0.00	40.36	-7.91
5	Uplift	-23.91	-27.09	0.00	0.00
6	Srcharge	11.00	12.65	0.00	0.00
	Total	143.88	113.12	131.09	40.23

Vertical Force	V=	143.88	kN
Horizontal Force	H=	131.09	kN
Moment	M=	-72.89	kNm

position X = V・X/V= 113.12 / 143.88 = 0.79 m

(2) Seismic Condition (→)

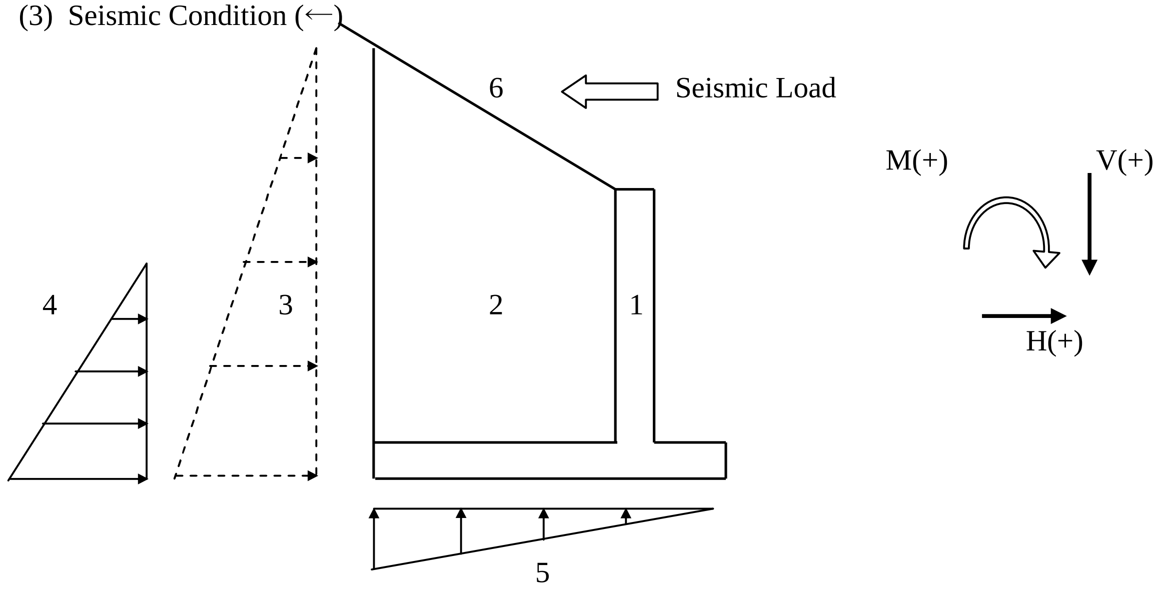


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.54	39.87	16.11	9.47
2	Weight of Back Fill	71.48	82.20	14.30	19.33
3	Earth Pressure	19.67	29.16	90.02	22.81
4	Water Pressure	0.00	0.00	1.65	-1.58
5	Uplift	-4.83	-1.13	0.00	0.00
6	Srcharge	5.50	6.33	0.00	0.00
	Total	172.36	156.43	122.08	50.03

Vertical Force V = 172.36 kN
Horizontal Force H = 122.08 kN
Moment M = -106.40 kNm

position X = V・X/V= 156.43 / 172.36 = 0.91 m

(3) Seismic Condition (←)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.54	39.87	-16.11	-9.47
2	Weight of Back Fill	71.48	82.20	-14.30	-19.33
3	Earth Pressure	19.67	29.16	90.02	22.81
4	Water Pressure	0.00	0.00	1.65	-1.58
5	Uplift	-4.83	-1.13	0.00	0.00
6	Srcharge	5.50	6.33	0.00	0.00
	Total	172.36	156.43	61.26	-7.57

Vertical Force V = 172.36 kN

Horizontal Force H = 61.26 kN

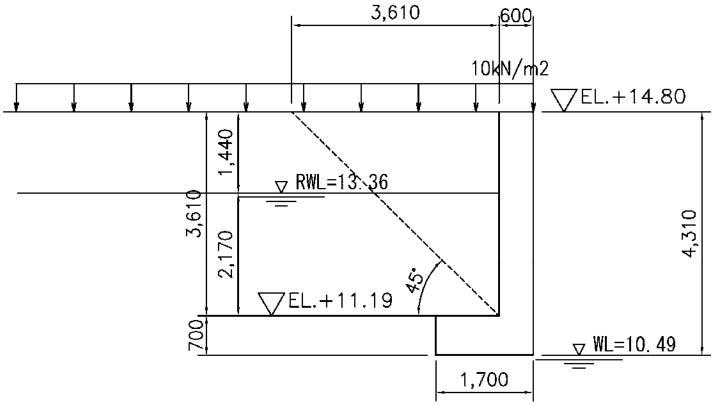
Moment M = -164.00 kNm

position X = V・X/V= 156.43 / 172.36 = 0.91 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



$A = 1/2\times(\quad 0 \quad + \quad 3.61 \quad)\times \quad 0 \quad = \quad 0.00 \quad m^2$

Converted Load of Embankment

$$q = \underset{A}{(\;0.00} \times \underset{\gamma}{18.00} + \underset{L}{3.61} \times \underset{qo}{10.0}\;) / 3.610$$

$= \quad 10.00 \quad kN/m^2$

Earth Pressure at Rest

$$Po = ko\cdot(q+\gamma\cdot h+\gamma'\cdot h)$$

$= \quad 0.5 \quad \times (\quad 10.00 \quad + \quad 18.00 \quad \times \quad 1.44 \quad + \quad 11.00 \quad \times \quad 2.17 \quad)$

$= \quad 29.90 \quad kN/m^2$

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	10.00	kN/m^2

Water Pressure

$$Pw = \gamma w\cdot h$$

$= \quad 9.80 \quad \times \quad 2.17$

$= \quad 21.27 \quad kN/m^2$

Bendin Moment and Shear Force

Effective Load

$W = \quad 29.90 \quad + \quad 21.27 \quad = \quad 51.17 \quad kN/m^2$

Bending Moment

$M = 1/2\cdot W\cdot L^2$

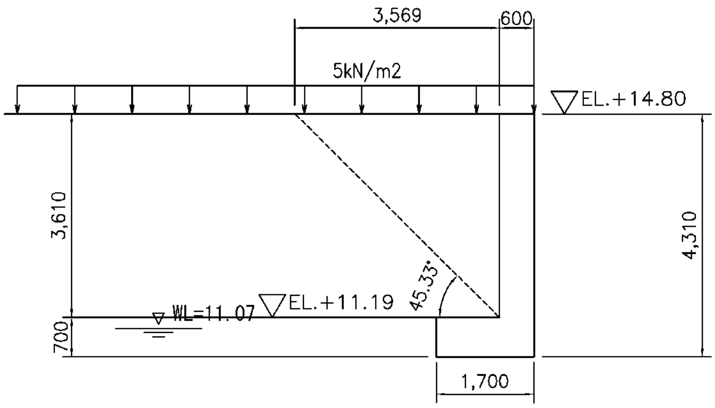
$= \quad 1/2 \times \quad 51.17 \quad \times 1.0^2 \quad = \quad 25.59 \quad kN/m^2$

Shear Force

$S = W\cdot L$

$= \quad 51.17 \quad \times 1.0 \quad = \quad 51.17 \quad kN/m^2$

(2) Seismic Condition



A= 1/2×(0 + 3.569)× 0 = 0.00 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	0.00	×	18.00	+	3.569	×	5.0	)／	3.569

= 5.00 kN/m^2

Active Earth Pressure

Pea= kea･(q＋γ･h)＋kea'･γ'･h

＝ 0.452 × (5.00 + 18.00 × 3.61 + 11.00 × 0.00 ）

= 31.63 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	5.00	kN/m^2

Water Pressure

Pw ＝ γw ･ h

＝ 9.80 × 0.00

= 0.00 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 3.61 × 0.200 ＝ 17.33 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 31.63 kN/m^2

Effective Load

W ＝ 31.63 + 0.00 + 17.33 ＝ 48.96 kN/m^2

Bending Moment

M ＝ 1/2･W･L^2

＝ 1/2 × 48.96 × 1.0^2 ＝ 24.48 kN/m^2

Shear Force

S ＝ W･L

= 48.96 ×1.0 ＝ 48.96 kN/m^2

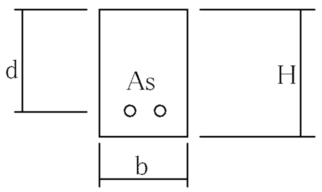
11.240

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	25.59	kN・m
Shear Force	S =	51.17	kN

Width of Member	b =	100.0	cm
Hight of Member	H =	60.0	cm
Equivalent Height（Tensile Side）	d =	51.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.727	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	115.473	N/mm^2	○
Shearing Stress of Concrete	τ =	0.104	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　x ＝ k*d

＝ 0.0008871　　　＝ 6.050

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1186318

j ＝ 1-(k/3)

＝ 0.9604561

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 172.6956070

σs＝ (M*10^5)/(b*d^2*p*j)

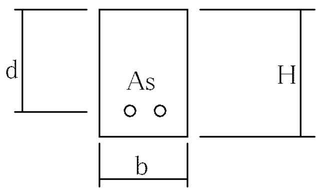
＝ 11547.28368

τ＝ (S*10^3)/(b*j*d)

＝ 10.4464257

(2) Seisimic Condition

Position of Rainforcing Bar



Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm

Bending Moment	M =	24.48	kN・m
Shear Force	S =	48.96	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	60.0	cm
Equivalent Height（Tensile Side）	d =	51.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.652	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	110.464	N/mm^2	○
Shearing Stress of Concrete	τ =	0.100	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0008871　　　　＝ 6.050

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1186318

j ＝ 1-(k/3)

＝ 0.9604561

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 165.2047073

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 11046.40502

τ＝ (S*10^3)/(b*j*d)

＝ 9.9952512

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.70				=	16.80	kN/m^2
Surcharge	10.00						=	10.00	kN/m^2
Soil	76.25						=	76.25	kN/m^2
Uplift	-9.80	×	2.87	×	0.60	/ 1.70	=	-9.93	kN/m^2
Subgrade Reaction	-68.54						=	-68.54	kN/m^2
								24.58	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	134.54	×	1.70					=	228.72	kN
(Vertical wall)	0.60	×	1.48	×	2.90	×	24.0	=	61.80	kN
Soil	1.48	×	2.90	×	1.70	×	18.0	=	131.34	kN
Uplift	-9.80	×	2.87	×	2.90	×	1.70	=	-138.66	kN
Load of Breast Wall	143.88	×	2.00					=	287.76	kN
									570.96	kN

Area of Base	2.90	×	1.70	+	1.0	×	1.7	×	2	=	8.33	m^2
Subgrade Reaction : 245.08 / 8.55	570.96	/	8.33							=	68.54	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 24.58 × 1.0^2 = 12.29 kN·m

Shear Force

$S = W \cdot L =$ 68.54 × 1.00 = 68.54 kN

(2) Seismic Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.70				=	16.80	kN/m^2
Surcharge	5.00						=	5.00	kN/m^2
Soil	76.25						=	76.25	kN/m^2
Uplift	-9.80	×	0.58	×	0.60	/ 1.70	=	-2.01	kN/m^2
Subgrade Reaction	-88.66						=	-88.66	kN/m^2
								7.38	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	134.54	×	1.70					=	228.72	kN
(Vertical wall)	0.60	×	1.48	×	2.90	×	24.0	=	61.80	kN
Soil	1.48	×	2.90	×	1.70	×	18.0	=	131.34	kN
Uplift	-9.80	×	0.58	×	2.90	×	1.70	=	-28.02	kN
Load of Breast Wall	172.36	×	2.00					=	344.72	kN
									738.56	kN

Subgrade Reaction : 245.08 / 8.55 738.56 / 8.33 = 88.66 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 7.38 × 1.0^2 = 3.69 kN·m

Shear Force

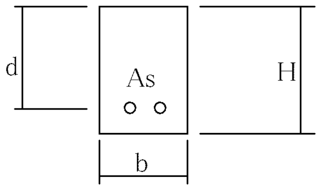
$S = W \cdot L =$ 88.66 × 1.00 = 88.66 kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	12.29	kN・m
Shear Force	S =	68.54	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.164	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	69.294	N/mm^2	○
Shearing Stress of Concrete	τ =	0.175	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 116.4212374

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 6929.38488

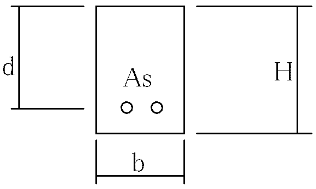
τ＝ (S*10^3)/(b*j*d)

＝ 17.4825081

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	3.69	kN・m	
Shear Force	S =	88.66	kN	
Width of Member	b =	100.0	cm	
Hight of Member	H =	50.0	cm	
Equivalent Height（Tensile Side）	d =	41.000	cm	
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2	
Modulous of Elasticity	n =	9.000		
Compressive Stress of Concrete	σc=	0.350	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	20.805	N/mm^2	○
Shearing Stress of Concrete	τ =	0.226	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 34.9547898

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 2080.50693

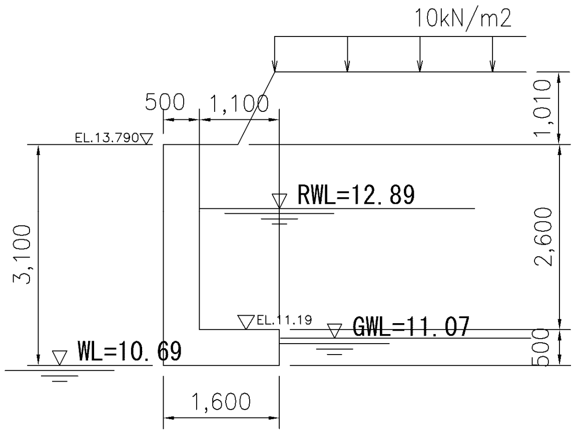
τ＝ (S*10^3)/(b*j*d)

＝ 22.6145195

Structural Culculation of Breast Wall

MSL4 Land Side

1. Calculation Model



・Dimension

Height of Vertical wall	2.60	m
Tickness of Vertical Wall	0.50	m
Width of Bottom Slab	1.60	m
Tickness of Bottom Slab	0.50	m
Height of Embankment	1.01	m

・Elevation

Top of the wall	EL+	13.79	m
Upperside of Bottom Slab	EL+	11.19	m
Underside of Bottom Slab	EL+	10.69	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	10.69	12.89	―	○	○	○	○	○	○
2	Seismic	10.69	11.07	←	◎	◎	○	○	○	○
3	Seismic	10.69	11.07	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

― : Without Considering

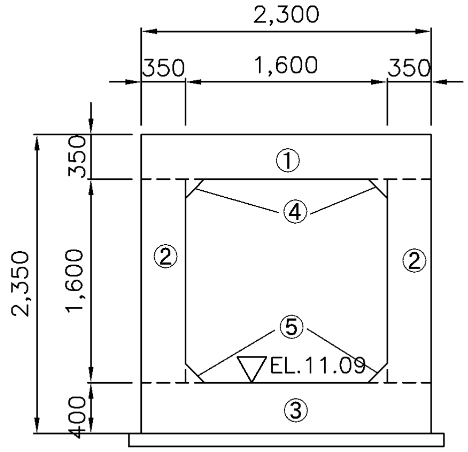
2. Design Condition

Unit Weight of Concrete			24.0	kN/m^3
Unit Weight of Water			9.8	kN/m^3
Unit Weight of Soil				
Saturated			20.0	kN/m^3
Wet			18.0	kN/m^3
Submerged			11.0	kN/m^3
Backfill Material			Sandy Soil	
internal Friction Angle of Soil			30.00	°
Seismic Lad			0.20	
Water Level				
Forth	Normal		10.69	m
	Seismic		10.69	m
Back	Normal		12.89	m
	Seismic		11.07	m
Difference	Normal		2.20	m
	Seismic		0.38	m
Earth Pressure	Normal		Earth Pressure at Rest	
	Seismic		Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane			0.00	°
Surchage	Normal		10.0	kN/m^2
	Seismic		5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height（h）	α	n	A（m^2）	Y（m）	A・Y（m^3）
①	2.300	0.350	1.0	1	0.805	2.175	1.751
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.300	0.400	1.0	1	0.920	0.200	0.184
④	0.150	0.150	0.5	2	0.023	1.950	0.045
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
計					2.891		3.334

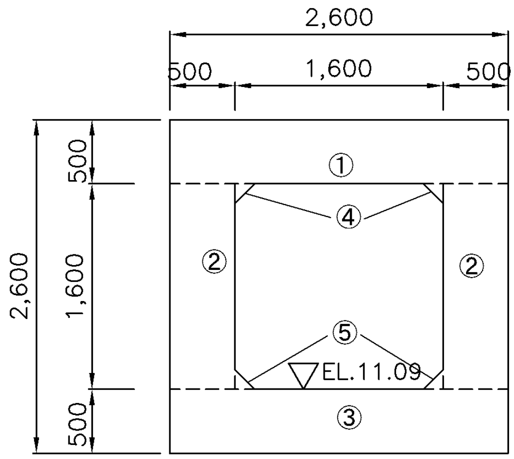
α；Triangle 0.5, Rectangle 1.0

ｎ；　Number

Centroid of Box Culvet

Y＝ ΣA・Y／ΣA　　＝　1.153　m

Breast Wal Section



	Width (B)	Height（h）	α	n	A（m^2）
①	2.600	0.500	1.0	1	1.300
②	0.500	1.600	1.0	2	1.600
③	2.600	0.500	1.0	1	1.300
④	0.150	0.150	0.5	2	0.023
⑤	0.150	0.150	0.5	2	0.023
計					4.246

α；Triangle 0.5, Rectangle 1.0

ｎ；　Number

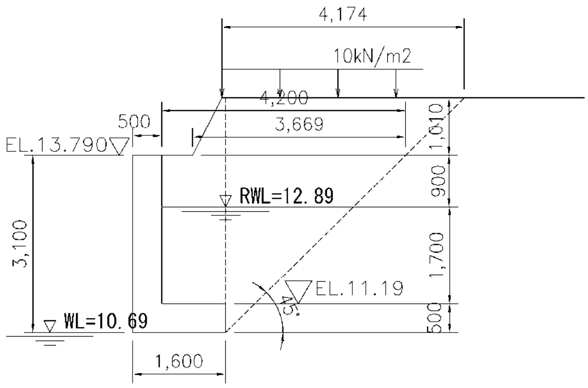
Weight of Body

W = $\gamma \times A \times 1.0$　　＝　101.904　kN

11.250

3.2 Converted Load of Embankment

(1) Normal Condition



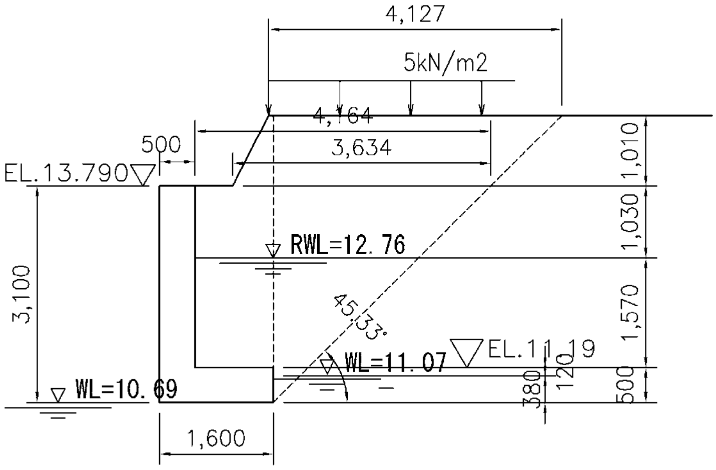
A= 1/2×(3.669 + 4.174)× 1.01 = 3.96 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	3.96	×	18.00	+	4.174	×	10.0	)／	4.200

= 26.91 kN/m^2

(2) Seismic Condition



A=	1/2×(	3.634	+	4.127	)×	1.01	=	3.92	m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	3.92	×	18.00	+	4.127	×	5.0	)／	4.164

= 21.90 kN/m^2

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

$\varphi =$ 30 ° $\theta =$ 0.000 ° $\delta = \Phi / 2 =$ 15.0 ° $\alpha =$ 0 °

$kh =$ 0.20 $\theta o = \tan^{-1} kh =$ 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta o - \theta)}{\cos\theta o \times \cos^2\theta \times \cos(\theta + \theta o + \delta) \times \left\{1 + \sqrt{\dfrac{\sin(\varphi + \delta) \times \sin(\varphi - \alpha - \theta o)}{\cos(\theta + \theta o + \delta) \times \cos(\theta - \alpha)}}\right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{1 + \sqrt{\dfrac{0.2266}{0.8964}}\right\}^2} = \underline{\underline{\mathbf{0.452}}}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °

$\delta =$ $\varphi/2 =$ 15 °(Between soil and soil)

$\theta =$ 0

$\alpha =$ 0

$\theta_O =$ $\tan^{-1} Kh =$ 11.310 °

$kh =$ 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi + \delta + \theta - \alpha) + \sec(\varphi + \delta + \theta - \alpha) \cdot X$$

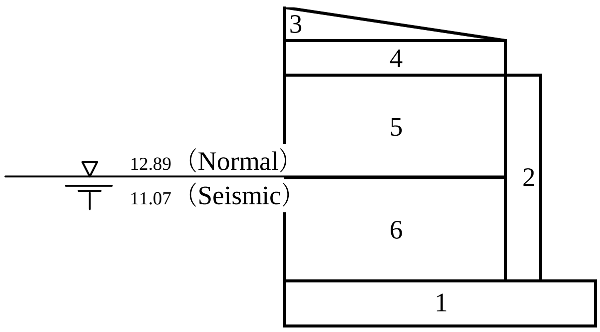
$$X = \sqrt{\frac{\cos(\theta + \delta + \theta_O) \cdot \sin(\varphi + \delta)}{\cos(\theta - \alpha) \cdot \sin(\varphi - \alpha - \theta_O)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1} 1 / 0.9892 = \boxed{45.311 °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.600	0.500	1.00		24.0	19.20
2	0.500	2.600	1.00		24.0	31.20

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
1	19.20	0.800	15.36	3.84	-0.903	-3.47
2	31.20	0.250	7.80	6.24	0.647	4.04
Total	50.40		23.16	10.08		0.57

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.569	1.010	1.00		18.0	5.17
4	1.100	0.000	1.00		18.0	0.00
5	1.100	0.900	1.00		18.0	17.82
6	1.100	1.700	1.00		20.0	37.40

No	V (kN)	X (m)	V･X (kNm)
3	5.17	1.316	6.80
4	0.00	1.050	0.00
5	17.82	1.050	18.71
6	37.40	1.050	39.27
Total	60.39		64.78

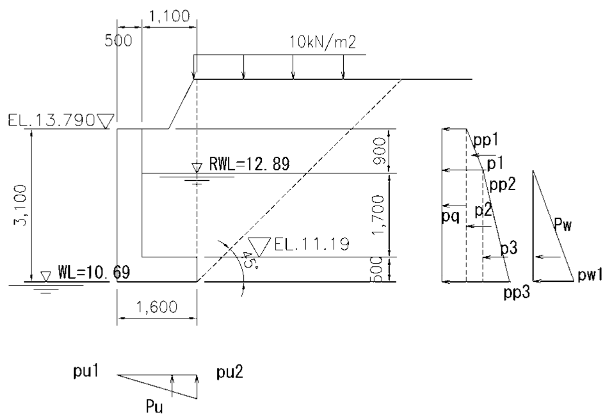
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.569	1.010	1.00		18.0	5.17
4	1.100	0.000	1.00		18.0	0.00
5	1.100	2.600	1.00		18.0	51.48
6	1.100	0.000	1.00		20.0	0.00

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
3	5.17	1.316	6.80	1.03	2.452	2.53
4	0.00	1.050	0.00	0.00	1.947	0.00
5	51.48	1.050	54.05	10.30	0.647	6.66
6	0.00	1.050	0.00	0.00	-0.653	0.00
Total	56.65		60.85	11.33		9.19

(5) Earth Pressure

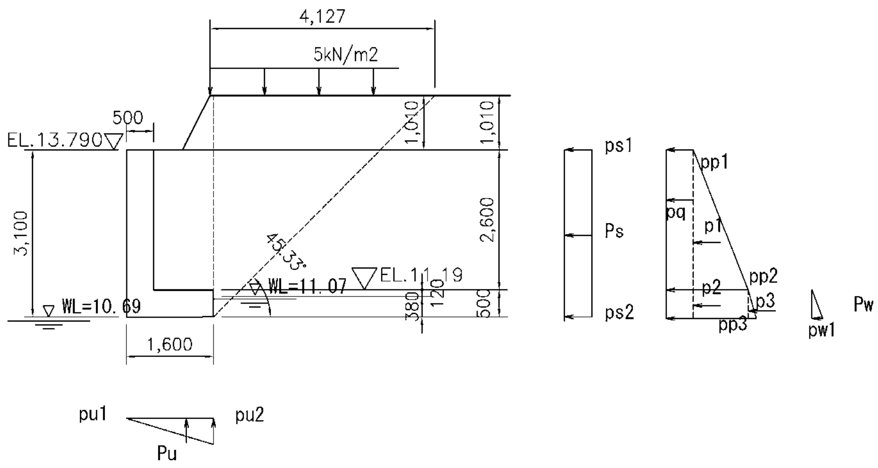
1) Normal Condition



φ	:	internal Friction Angle of Soil	30
Ko	:	Earth Pressure at Rest	0.5
q	:	Converted Load of Embankment	26.91 kN/m^2

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	3.65	1.947	7.11
Earth Pressure (p2)	0.00	0.000	0.00	17.82	-0.053	-0.94
Earth Pressure (p3)	0.00	0.000	0.00	13.31	-0.420	-5.59
Earth Pressure (pq)	0.00	0.000	0.00	41.71	0.397	16.56
Total	0.00		0.00	76.49		17.14

2) Seismic Condition

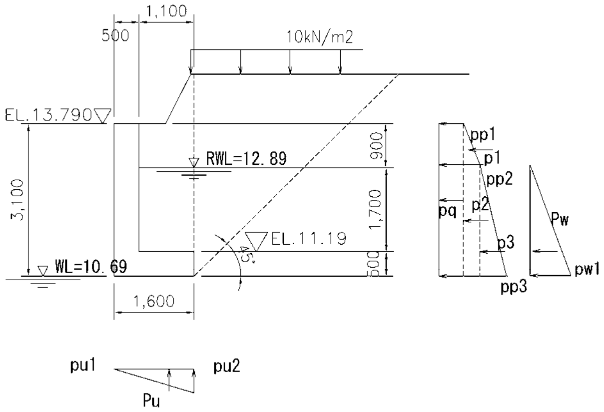


φ	:	internal Friction Angle of Soil	30	°
δ	:	Angle of Wall Friction	15	°
		δ=φ/2（Seismic Condition）		
α	:	Angle Between Ground Surface and Horizontal Plane	0	°
θ	:	Angle Between Back side Surface of Wall and Vertical Plane	0	°
Ka	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	21.9	kN/m^2

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Earth Pressure (p1)	7.79	1.600	12.46	29.07	0.134	3.90
Earth Pressure (p2)	1.09	1.600	1.74	8.12	-0.963	-7.82
Earth Pressure (p3)	0.09	1.600	0.14	0.35	-1.026	-0.36
Earth Pressure (pq)	0.16	1.600	0.26	0.61	0.397	0.24
Total	9.13		14.60	38.15		-4.04

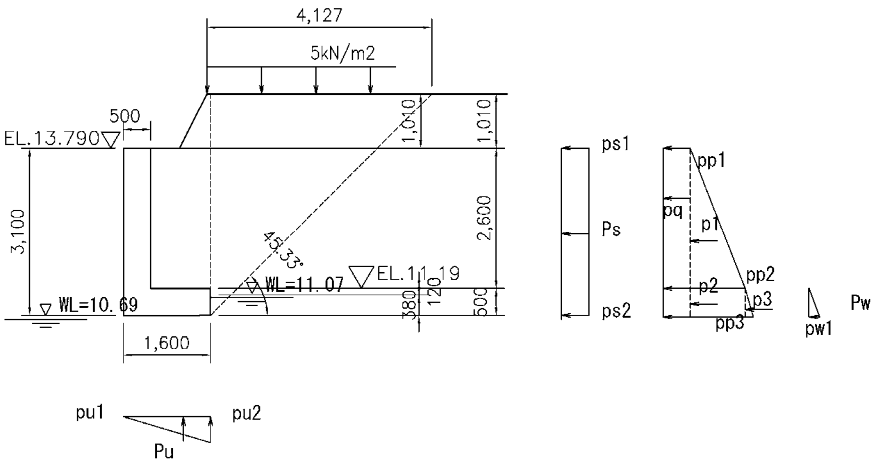
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	23.72	-0.420	-9.96
Total	0.00		0.00	23.72		-9.96

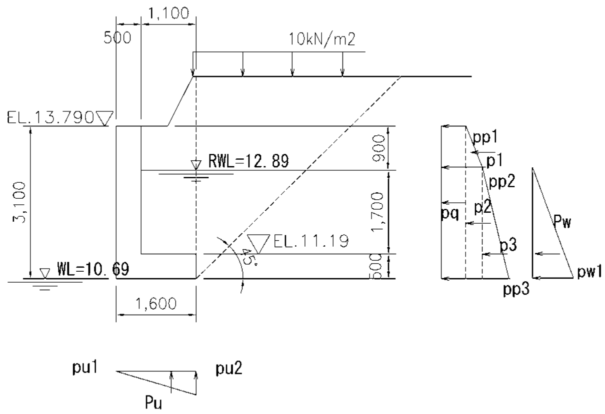
(2) Seismic Condition



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	0.71	-1.026	-0.73
Total	0.00		0.00	0.71		-0.73

3.5 Uplift

(1) Normal Conditon

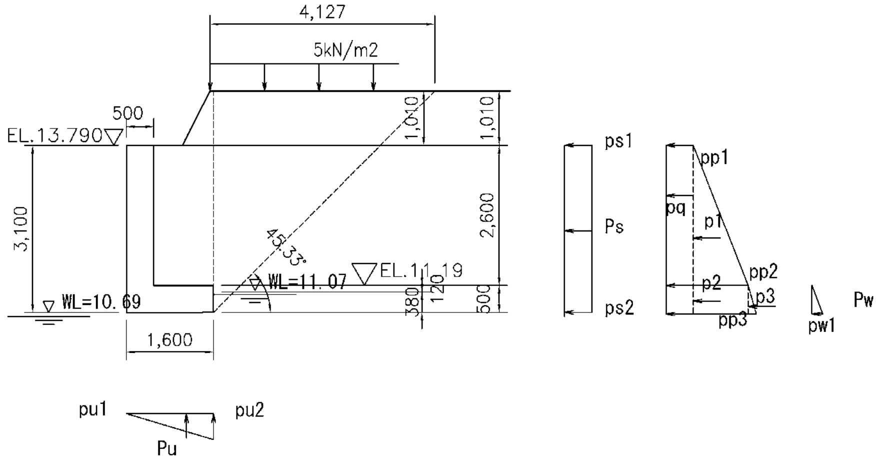


Width of Bottom Slab 1.6 m

	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	21.56	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	17.25	1.067	18.41	0.00	0.000	0.00
Total	17.25		18.41	0.00		0.00

(2) Seismic Conditon



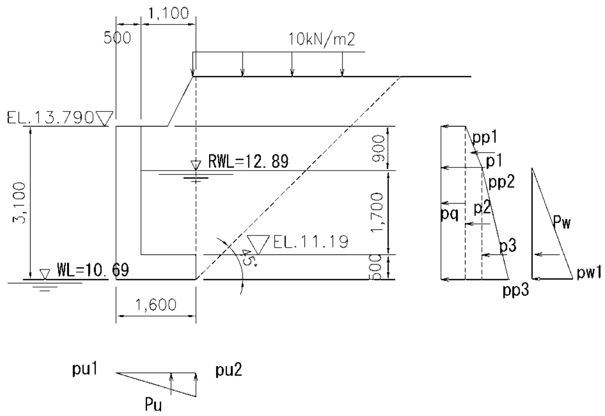
	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	3.72	0.00

Width of Bottom Slab 1.6 m

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	2.98	1.067	3.18	0.00	0.000	0.00
Total	2.98		3.18	0.00		0.00

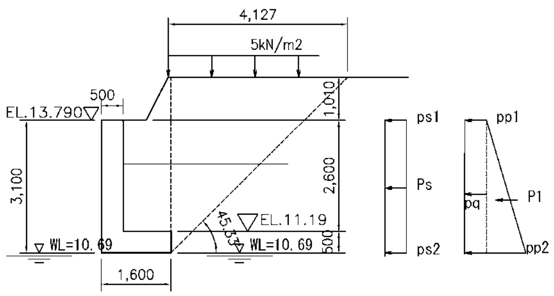
3.6 Surcharge

(1) Normal Condition



	$q(kN/m^2)$	B(m)	X(m)	V(kN)	V・X(kNm)
1	10.0	0.00	0.500	0.00	0.00
Total				0.00	0.00

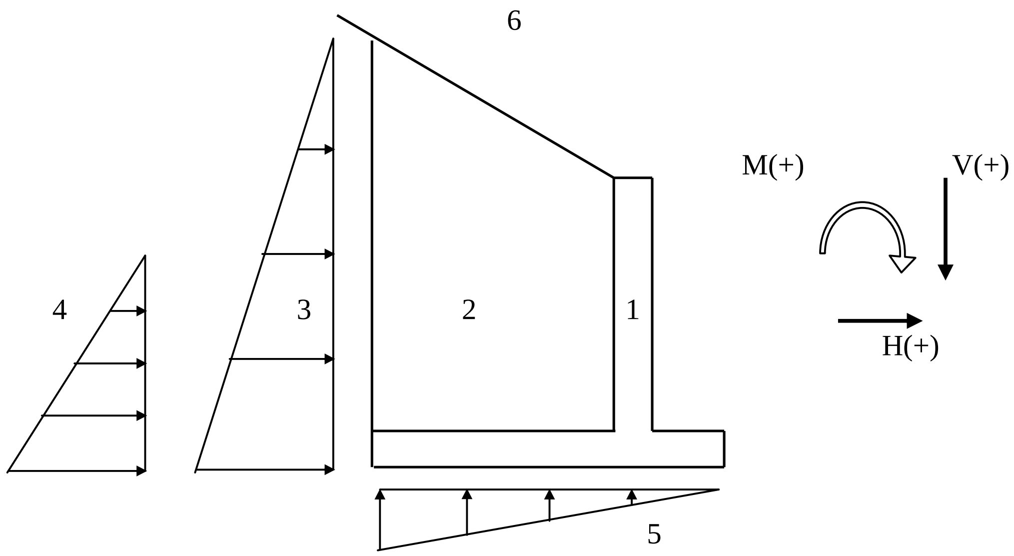
(2) Seismic Conditon



	$q(kN/m^2)$	B(m)	X(m)	V(kN)	V・X(kNm)
1	5.0	0.00	0.000	0.00	0.00
Total				0.00	0.00

3.7 Summary of Load

(1) Normal Condition

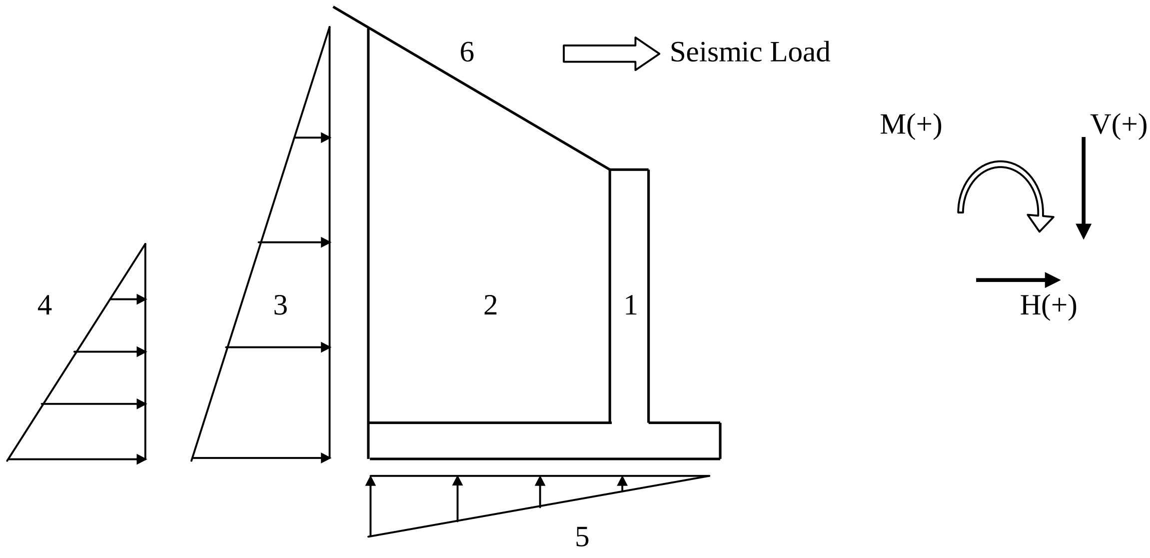


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	0.00	0.00
2	Weight of Back Fill	60.39	64.78	0.00	0.00
3	Earth Pressure	0.00	0.00	76.49	17.14
4	Water Pressure	0.00	0.00	23.72	-9.96
5	Uplift	-17.25	-18.41	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	Total	93.54	69.53	100.21	7.18

Vertical Force	V=	93.54	kN
Horizontal Force	H=	100.21	kN
Moment	M=	-62.35	kNm

position X = V・X/V= 69.53 / 93.54 = 0.74 m

(2) Seismic Condition (→)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	10.08	0.57
2	Weight of Back Fill	56.65	60.85	11.33	9.19
3	Earth Pressure	9.13	14.60	38.15	-4.04
4	Water Pressure	0.00	0.00	0.71	-0.73
5	Uplift	-2.98	-3.18	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	113.20	95.43	60.27	4.99

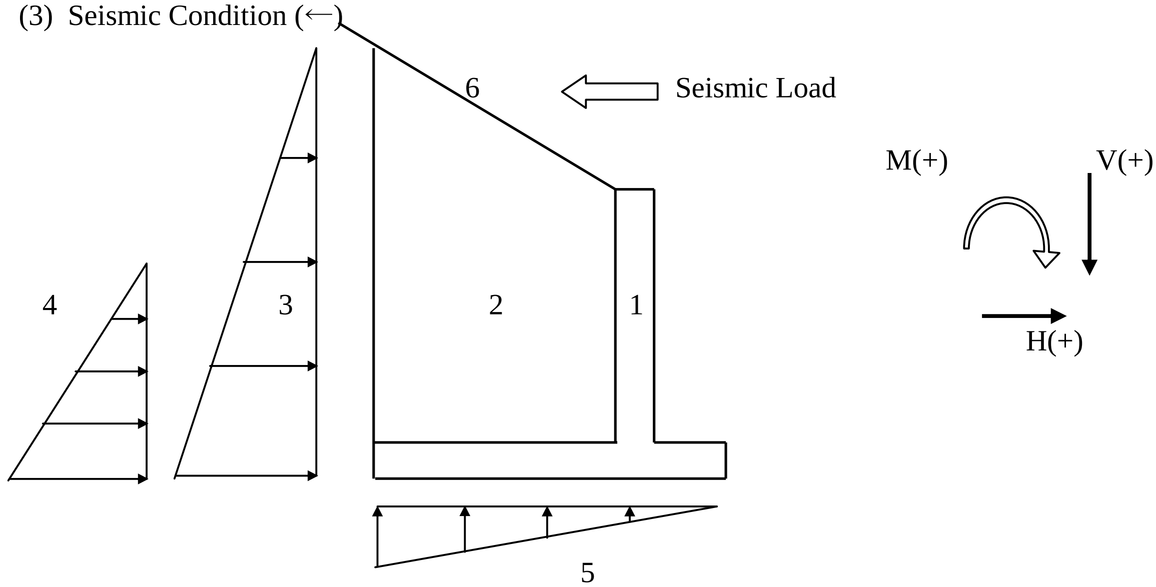
Vertical Force V = 113.20 kN

Horizontal Force H = 60.27 kN

Moment M = -90.44 kNm

position X = V・X/V= 95.43 / 113.20 = 0.84 m

(3) Seismic Condition (←)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	-10.08	-0.57
2	Weight of Back Fill	56.65	60.85	-11.33	-9.19
3	Earth Pressure	9.13	14.60	38.15	-4.04
4	Water Pressure	0.00	0.00	0.71	-0.73
5	Uplift	-2.98	-3.18	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	113.20	95.43	17.45	-14.53

Vertical Force V = 113.20 kN

Horizontal Force H = 17.45 kN

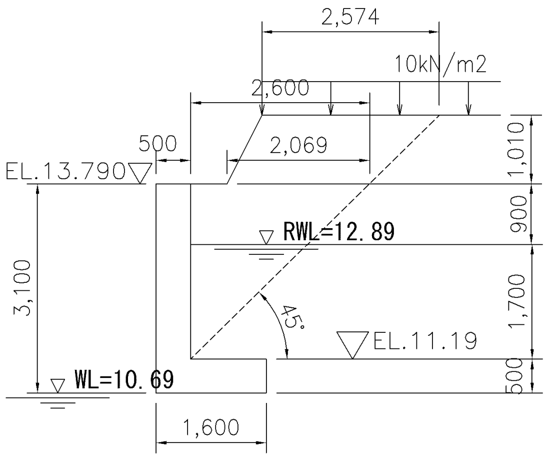
Moment M = -109.96 kNm

position X = V・X/V= 95.43 / 113.20 = 0.84 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



A= 1/2×(2.069 + 2.574)× 1.01 = 2.34 m^2

Converted Load of Embankment

	A		γ		L		qo		
q=	(2.34	×	18.00	+	2.574	×	10.0	)／	2.600

= 26.10 kN/m^2

Earth Pressure at Rest

Po ＝ ko･(q＋γ･h＋γ'･h)

= 0.5 × (26.10 + 18.00 × 0.90 + 11.00 × 1.70)

= 30.50 kN/m^2

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	26.10	kN/m^2

Water Pressure

Pw＝ γw･h

= 9.80 × 1.70

= 16.66 kN/m^2

Bendin Moment and Shear Force

Effective Load

W ＝ 30.50 + 16.66 ＝ 47.16 kN/m^2

Bending Moment

M ＝ 1/2･W･L^2

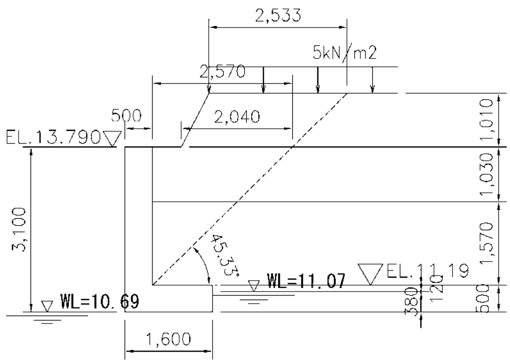
＝ 1/2 × 47.16 × 1.0^2 ＝ 23.58 kN/m^2

Shear Force

S ＝ W･L

= 47.16 ×1.0 ＝ 47.16 kN/m^2

(2) Seismic Condition



A= 1/2×(2.04 + 2.533)× 1.01 = 2.31 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	2.31	×	18.00	+	2.533	×	5.0	)／	2.570

= 21.11 kN/m^2

Active Earth Pressure

Pea= kea･(q＋γ･h)＋kea'･γ'･h

= 0.452 × (21.11 + 18.00 × 2.60 + 11.00 × 0.00)

= 30.70 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	21.11	kN/m^2

Water Pressure

Pw ＝ γw・h

= 9.80 × 0.00

= 0.00 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 2.60 × 0.200 ＝ 12.48 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 30.70 kN/m^2

Effective Load

W＝ 30.70 + 0.00 + 12.48 ＝ 43.18 kN/m^2

Bending Moment

M＝ 1/2･W･L^2

＝ 1/2 × 43.18 × 1.0^2 ＝ 21.59 kN/m^2

Shear Force

S＝ W･L

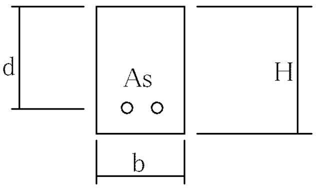
= 43.18 ×1.0 ＝ 43.18 kN/m^2

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	23.58	kN・m
Shear Force	S =	47.16	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	2.234	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	132.949	N/mm^2	○
Shearing Stress of Concrete	τ =	0.120	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)
＝ 0.0011034

x ＝ k*d
＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p
＝ 0.1313486

j ＝ 1-(k/3)
＝ 0.9562171

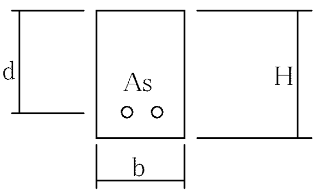
σc＝ (2*M*10^5)/(b*d^2*k*j)
＝ 223.3696321

σs＝ (M*10^5)/(b*d^2*p*j)
＝ 13294.94674

τ＝ (S*10^3)/(b*j*d)
＝ 12.0291083

(2) Seisimic Condition

Position o Diameter	12.0	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	21.59	kN・m
Shear Force	S =	43.18	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	2.045	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	121.729	N/mm^2	○
Shearing Stress of Concrete	τ =	0.110	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　x ＝ k*d

＝ 0.0011034　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 204.5186750

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 12172.93893

τ＝ (S*10^3)/(b*j*d)

＝ 11.0139291

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50						=	12.00	kN/m^2
Surcharge	10.00								=	10.00	kN/m^2
Soil	60.39								=	60.39	kN/m^2
Uplift	-9.80	×	2.20	×	0.50	/	1.60		=	-6.74	kN/m^2
Subgrade Reaction	-42.59								=	-42.59	kN/m^2
										33.06	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	101.90	×	1.60					=	163.04	kN
(Vertical wall)	0.50	×	0.50	×	2.60	×	24.0	=	15.60	kN
Soil	0.50	×	2.60	×	1.60	×	18.0	=	37.44	kN
Uplift	-9.80	×	2.20	×	2.60	×	1.60	=	-89.69	kN
Load of Breast Wall	93.54	×	2.00					=	187.08	kN
									313.47	kN

Area of Base	2.60	×	1.60	+	1.0	×	1.6	×	2	=	7.36	m^2
Subgrade Reaction : 245.08 / 8.55	313.47	/	7.36							=	42.59	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$	1/2	×	33.06	×	1.0^2	=	16.53	kN·m

Shear Force

$S = W \cdot L =$	42.59	×	1.00	=	42.59	kN

(2) Seismic Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50						=	12.00	kN/m^2
Surcharge	5.00								=	5.00	kN/m^2
Soil	60.39								=	60.39	kN/m^2
Uplift	-9.80	×	0.38	×	0.50	/	1.60		=	-1.16	kN/m^2
Subgrade Reaction	-58.01								=	-58.01	kN/m^2
										18.22	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	101.90	×	1.60					=	163.04	kN
(Vertical wall)	0.50	×	0.50	×	2.60	×	24.0	=	15.60	kN
Soil	0.50	×	2.60	×	1.60	×	18.0	=	37.44	kN
Uplift	-9.80	×	0.38	×	2.60	×	1.60	=	-15.49	kN
Load of Breast Wall	113.20	×	2.00					=	226.40	kN
									426.99	kN

Subgrade Reaction : 245.08 / 8.55	426.99	/	7.36	=	58.01	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$	1/2	×	18.22	×	1.0^2	=	9.11	kN·m

Shear Force

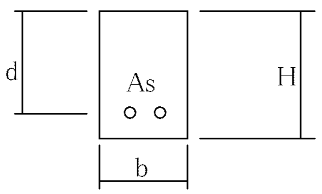
$S = W \cdot L =$	58.01	×	1.00	=	58.01	kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	16.53	kN・m
Shear Force	S =	42.59	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.566	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	93.200	N/mm^2	○
Shearing Stress of Concrete	τ =	0.109	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)
＝ 0.0011034

x ＝ k*d
＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p
＝ 0.1313486

j ＝ 1-(k/3)
＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)
＝ 156.5860907

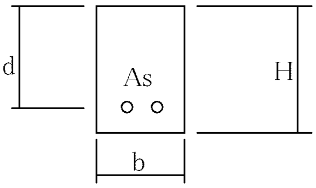
σs＝ (M*10^5)/(b*d^2*p*j)
＝ 9319.99447

τ＝ (S*10^3)/(b*j*d)
＝ 10.8634377

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12.0	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	9.11	kN・m
Shear Force	S =	58.01	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.863	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	51.364	N/mm^2	○
Shearing Stress of Concrete	τ =	0.148	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 86.2975975

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 5136.42768

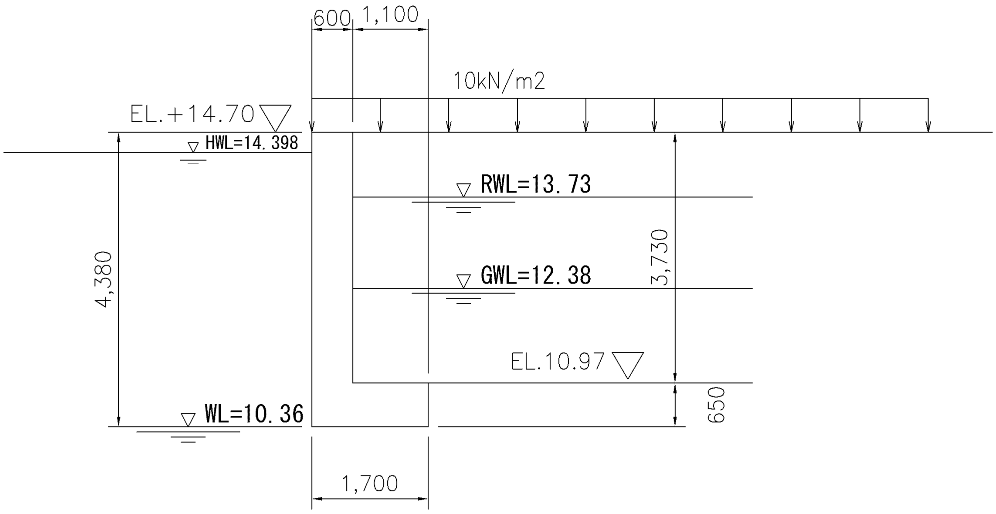
τ＝ (S*10^3)/(b*j*d)

＝ 14.7966194

Structural Calculation of Breast Wall

MSR3 River Side

1. Calculation Model



・Dimension

Height of Vertical wall	3.73	m
Tickness of Vertical Wall	0.50	m
Width of Bottom Slab	1.70	m
Tickness of Bottom Slab	0.65	m
Height of Embankment	0.00	m

・Elevation

Top of the wall	EL+	14.70	m
Upperside of Bottom Slab	EL+	10.97	m
Underside of Bottom Slab	EL+	10.32	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	10.32	13.73	―	○	○	○	○	○	○
2	Seismic	10.32	12.38	←	◎	◎	○	○	○	○
3	Seismic	10.32	12.38	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

― : Without Considering

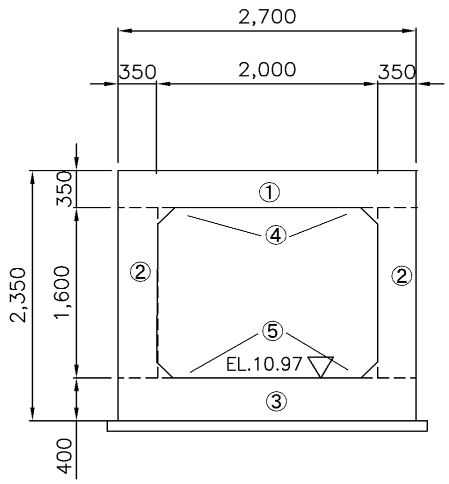
2. Design Condition

Unit Weight of Concrete		————	24.0	kN/m^3
Unit Weight of Water		————	9.8	kN/m^3
Unit Weight of Soil				
Saturated		————	20.0	kN/m^3
Wet		————	18.0	kN/m^3
Submerged		————	11.0	kN/m^3
Backfill Material		————	Sandy Soil	
internal Friction Angle of Soil		————	30.00	°
Seismic Lad		————	0.20	
Water Level				
Forth	Normal	————	10.32	m
	Seismic	————	10.32	m
Back	Normal	————	13.73	m
	Seismic	————	12.38	m
Difference	Normal	————	3.41	m
	Seismic	————	2.06	m
Earth Pressure	Normal	————	Earth Pressure at Rest	
	Seismic	————	Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane		————	0.00	°
Surchage	Normal	————	10.0	kN/m^2
	Seismic	————	5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height（h）	α	n	A（m^2）	Y（m）	A・Y（m^3）
①	2.700	0.350	1.0	1	0.945	2.175	2.055
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.700	0.400	1.0	1	1.080	0.200	0.216
④	0.150	0.150	0.5	2	0.023	1.550	0.036
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
計					3.191		3.661

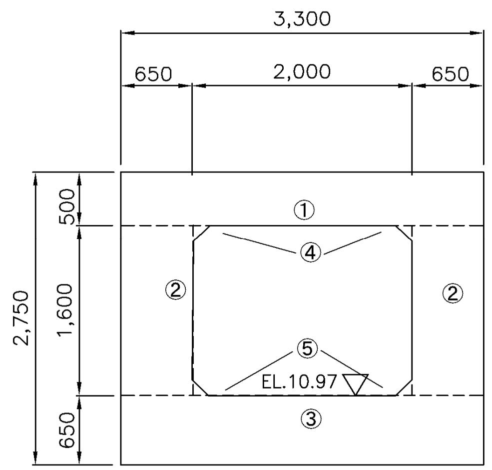
α；Triangle 0.5, Rectangle 1.0

ｎ； Number

Centroid of Box Culvet

Y＝ ΣA・Y／ΣA ＝ 1.147 m

Breast Wal Section



	Width (B)	Height（h）	α	n	A（m^2）
①	3.300	0.500	1.0	1	1.650
②	0.650	1.600	1.0	2	2.080
③	3.300	0.650	1.0	1	2.145
④	0.150	0.150	0.5	2	0.023
⑤	0.150	0.150	0.5	2	0.023
計					5.921

α；Triangle 0.5, Rectangle 1.0

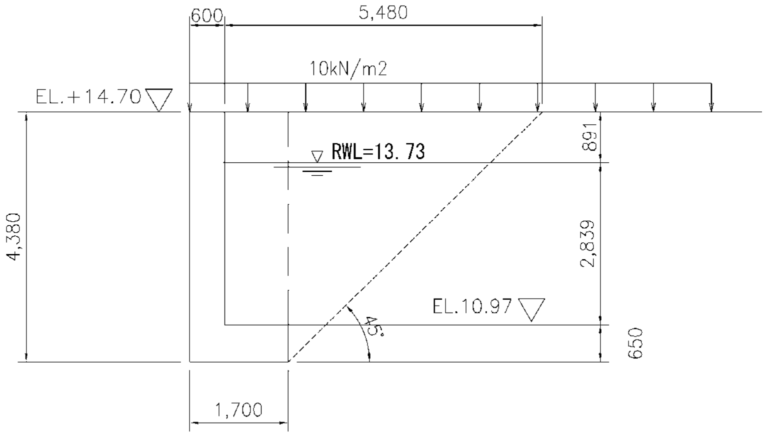
ｎ；　Number

Weight of Body

$W = \gamma \times A \times 1.0$ ＝ 142.104 kN

3.2 Converted Load of Embankment

(1) Normal Condition



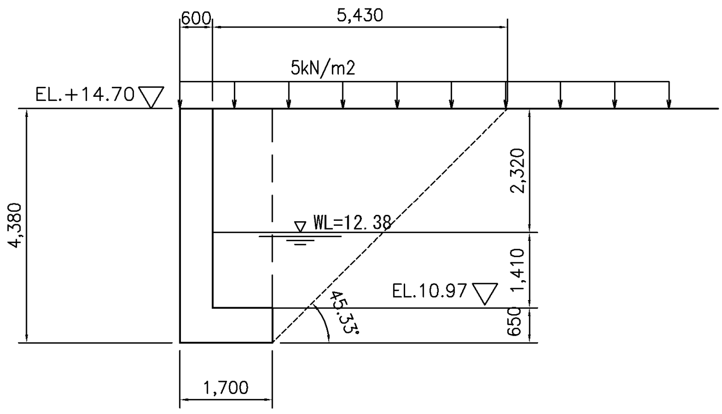
A= 1/2×(0 + 5.48)× 0 = 0.00 m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.48	×	10.0	)／	5.480

= 10.00 kN/m^2

(2) Seismic Condition



A=	1/2×(	0.000	+	5.43	)×	0	=	0.00 m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	0.00	×	18.00	+	5.43	×	5.0	)／	5.43

= 5.00 kN/m^2

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

$\varphi =$ 30 ° $\theta =$ 0.000 ° $\delta = \Phi / 2 =$ 15.0 ° $\alpha =$ 0 °

$kh =$ 0.20 $\theta o = \tan^{-1} kh =$ 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta o - \theta)}{\cos\theta o \times \cos^2\theta \times \cos(\theta + \theta o + \delta) \times \left\{ 1 + \sqrt{\dfrac{\sin(\varphi + \delta) \times \sin(\varphi - \alpha - \theta o)}{\cos(\theta + \theta o + \delta) \times \cos(\theta - \alpha)}} \right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{ 1 + \sqrt{\dfrac{0.2266}{0.8964}} \right\}^2} = \underline{\underline{\mathbf{0.452}}}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °
$\delta =$ $\varphi/2 =$ 15 °(Between soil and soil)
$\theta =$ 0
$\alpha =$ 0
$\theta_O =$ $\tan^{-1} Kh =$ 11.310 °
$kh =$ 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi + \delta + \theta - \alpha) + \sec(\varphi + \delta + \theta - \alpha) \cdot X$$

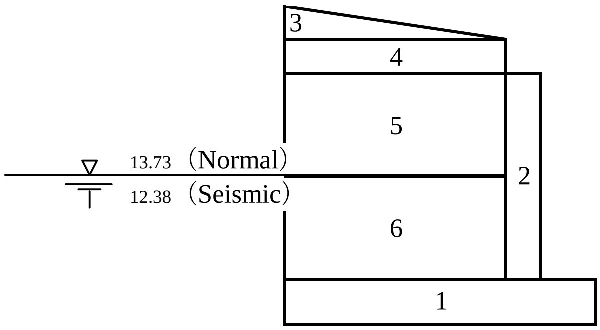
$$X = \sqrt{\frac{\cos(\theta + \delta + \theta_O) \cdot \sin(\varphi + \delta)}{\cos(\theta - \alpha) \cdot \sin(\varphi - \alpha - \theta_O)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1} 1 / 0.9892 = \boxed{45.311 °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.700	0.650	1.00		24.0	26.52
2	0.600	3.730	1.00		24.0	53.71

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
1	26.52	0.850	22.54	5.30	-0.822	-4.36
2	53.71	0.300	16.11	10.74	1.368	14.69
Total	80.23		38.65	16.04		10.33

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	1.100	0.000	1.00		18.0	0.00
4	1.100	0.000	1.00		18.0	0.00
5	1.100	0.970	1.00		18.0	19.21
6	1.100	2.760	1.00		20.0	60.72

No	V (kN)	X (m)	V･X (kNm)
3	0.00	1.150	0.00
4	0.00	1.150	0.00
5	19.21	1.150	22.09
6	60.72	1.150	69.83
Total	79.93		91.92

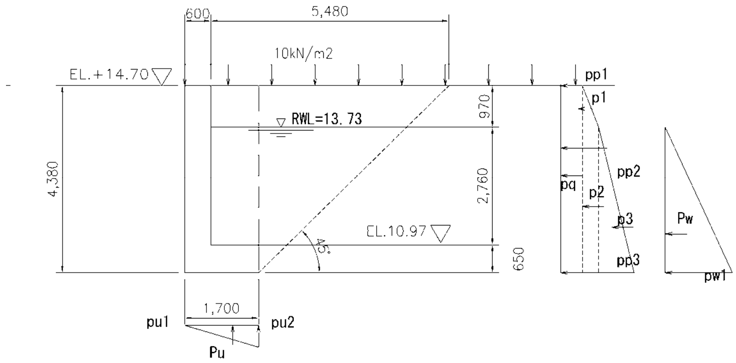
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	1.100	0.000	1.00		18.0	0.00
4	1.100	0.000	1.00		18.0	0.00
5	1.100	2.320	1.00		18.0	45.94
6	1.100	1.410	1.00		20.0	31.02

No	V (kN)	X (m)	V･X (kNm)	H (kN)	Y (m)	H･Y (kNm)
3	0.00	1.150	0.00	0.00	3.233	0.00
4	0.00	1.150	0.00	0.00	3.233	0.00
5	45.94	1.150	52.83	9.19	2.073	19.05
6	31.02	1.150	35.67	6.20	0.208	1.29
Total	76.96		88.50	15.39		20.34

(5) Earth Pressure

1) Normal Condition

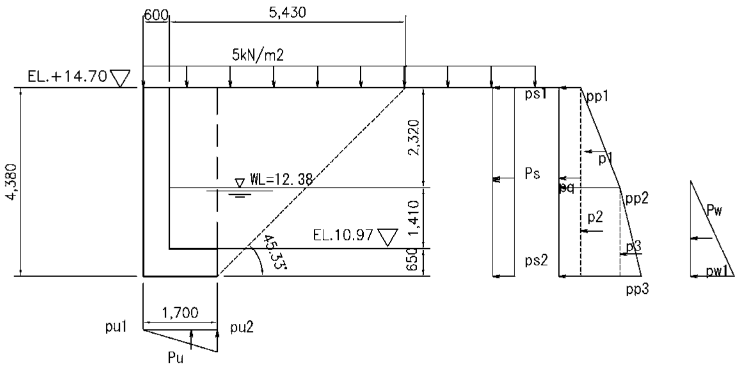


φ	:	internal Friction Angle of Soil	30
Ko	:	Earth Pressure at Rest	0.5
q	:	Converted Load of Embankment	10.00 kN/m^2

	V(kN)	H(kN/m)
Pp1	0.00	5.00
Pp2	0.00	13.73
Pp3	0.00	32.49

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	4.23	2.586	10.94
Earth Pressure (p2)	0.00	0.000	0.00	29.77	0.558	16.61
Earth Pressure (p3)	0.00	0.000	0.00	31.98	-0.010	-0.32
Earth Pressure (pq)	0.00	0.000	0.00	21.90	1.033	22.62
Total	0.00		0.00	87.88		49.85

2) Seismic Condition



φ : internal Friction Angle of Soil 30 °

δ : Angle of Wall Friction 15 °

δ=φ/2（Seismic Condition）

α : Angle Between Ground Surface and Horizontal Plane 0 °

θ : Angle Between Back side Surface of Wall and Vertical Plane 0 °

Ka : Coefficient of Active Earth Pressure 0.452

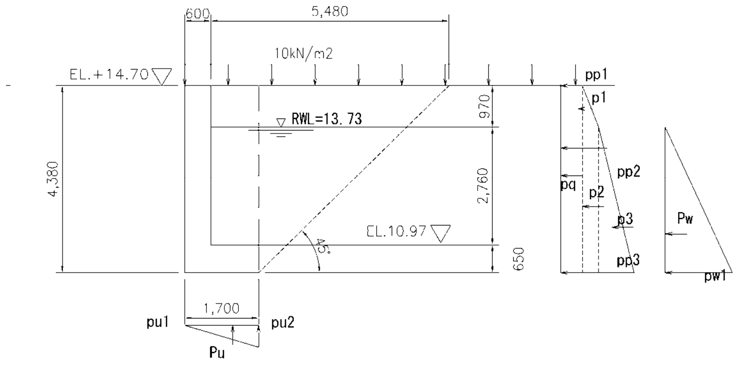
q : Converted Load of Embankment 5.0 kN/m^2

	V(kN)	H(kN/m)
Earth Pressure (pp1)		2.26
Earth Pressure (pp2)		21.14
Earth Pressure (pp3)		31.38

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	5.67	1.700	9.64	21.15	1.686	35.66
Earth Pressure (p2)	10.06	1.700	17.10	18.78	2.808	52.73
Earth Pressure (p3)	2.53	1.700	4.30	10.19	-0.460	-4.69
Earth Pressure (pq)	2.47	1.700	4.20	9.56	1.043	9.97
Total	20.73		35.24	59.68		93.67

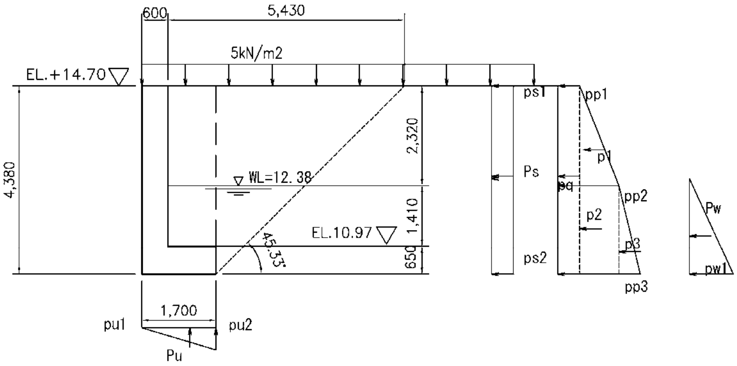
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	56.98	-0.010	-0.57
Total	0.00		0.00	56.98		-0.57

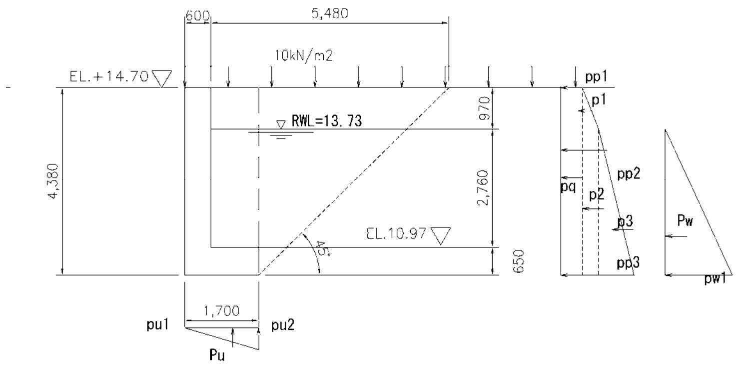
(2) Seismic Condition



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	20.79	0.687	14.28
Total	0.00					

3.5 Uplift

(1) Normal Conditon



	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	33.42	0.00

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Uplift Pu	28.41	1.133	32.19	0.00	0.000	0.00
Total	28.41		32.19	0.00		0.00

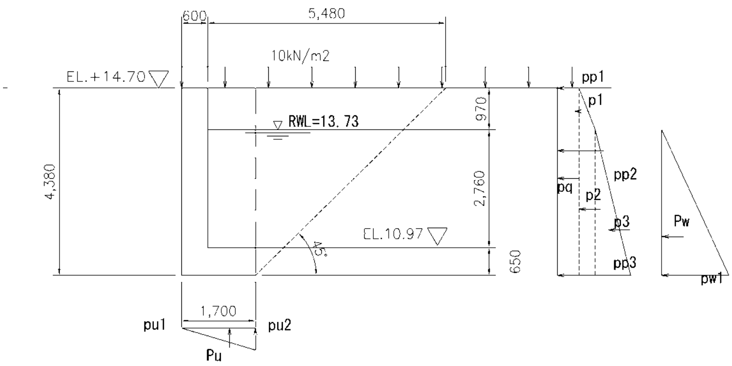
(2) Seismic Conditon

	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	20.19	0.00

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Uplift Pu	17.16	1.133	19.44	0.00	0.000	0.00
Total	17.16		19.44	0.00		0.00

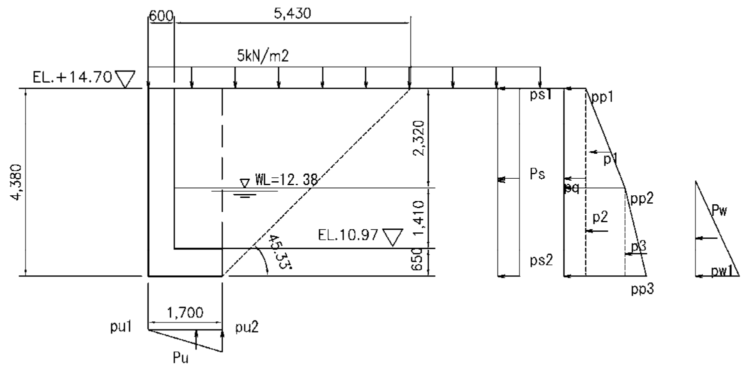
3.6 Surcharge

(1) Normal Condition



	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	10.0	1.10	1.15	11.00	12.65
Total				11.00	12.65

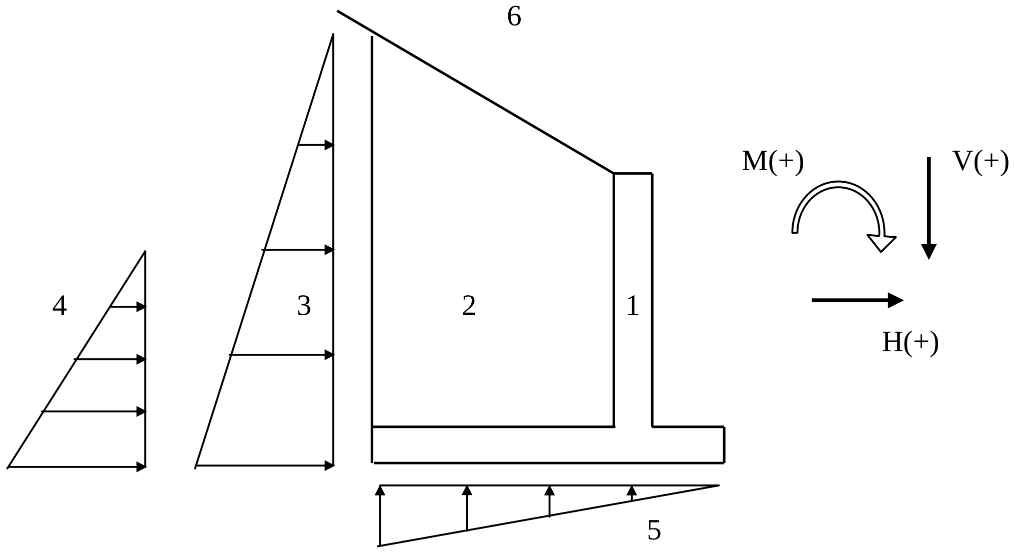
(2) Seismic Conditon



	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	5.0	1.10	1.15	5.50	6.33
Total				5.50	6.33

3.7 Summary of Load

(1) Normal Condition

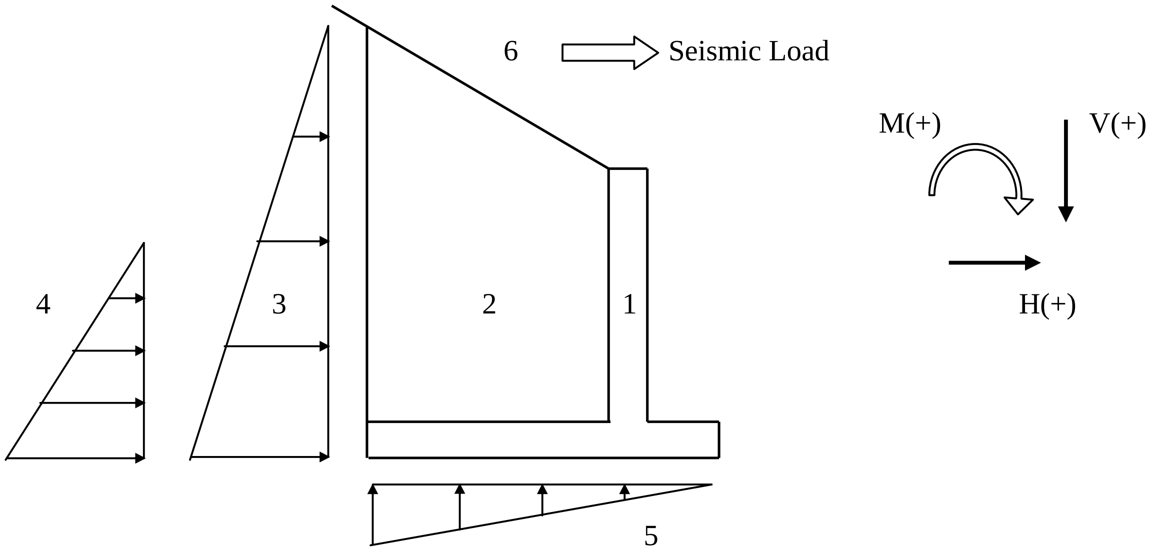


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.23	38.65	0.00	0.00
2	Weight of Back Fill	79.93	91.92	0.00	0.00
3	Earth Pressure	0.00	0.00	87.88	49.85
4	Water Pressure	0.00	0.00	56.98	-0.57
5	Uplift	-28.41	-32.19	0.00	0.00
6	Srcharge	11.00	12.65	0.00	0.00
	Total	142.75	111.03	144.86	49.28

Vertical Force	V=	142.75	kN
Horizontal Force	H=	144.86	kN
Moment	M=	-61.75	kNm

position X = V・X/V= 111.03 / 142.75 = 0.78 m

(2) Seismic Condition (→)

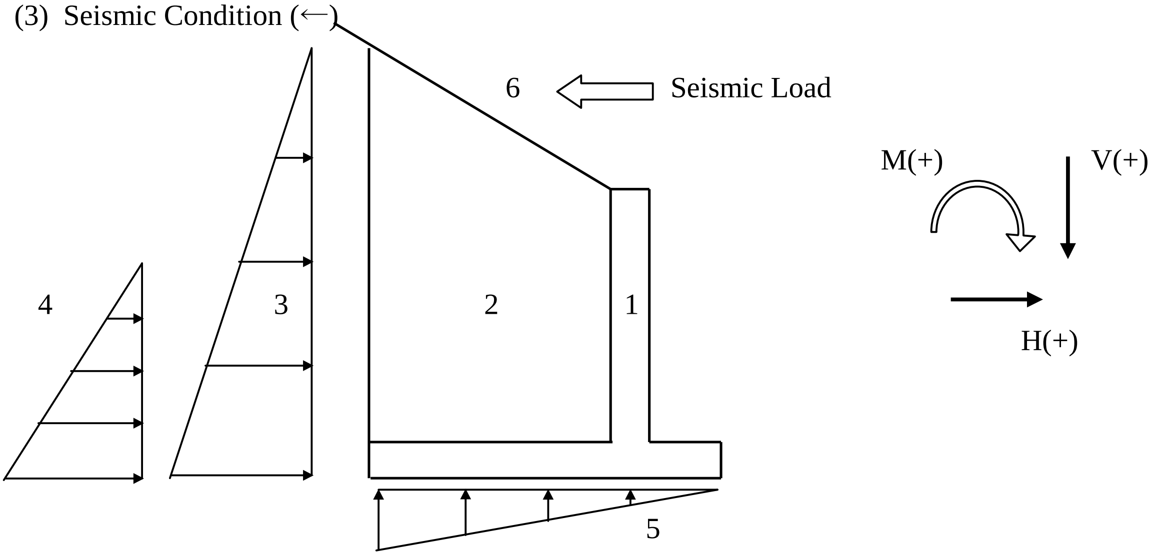


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.23	38.65	16.04	10.33
2	Weight of Back Fill	76.96	88.50	15.39	20.34
3	Earth Pressure	20.73	35.24	59.68	93.67
4	Water Pressure	0.00	0.00	20.79	14.28
5	Uplift	-17.16	-19.44	0.00	0.00
6	Srcharge	5.50	6.33	0.00	0.00
	合計	166.26	149.28	111.90	138.62

Vertical Force	V =	166.26	kN
Horizontal Force	H =	111.90	kN
Moment	M =	-10.66	kNm

position X = V・X/V= 149.28 / 166.26 = 0.90 m

(3) Seismic Condition (←)



		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	80.23	38.65	-16.04	-10.33
2	Weight of Back Fill	76.96	88.50	-15.39	-20.34
3	Earth Pressure	20.73	35.24	59.68	93.67
4	Water Pressure	0.00	0.00	20.79	14.28
5	Uplift	-17.16	-19.44	0.00	0.00
6	Srcharge	5.50	6.33	0.00	0.00
	合計	166.26	149.28	49.04	77.28

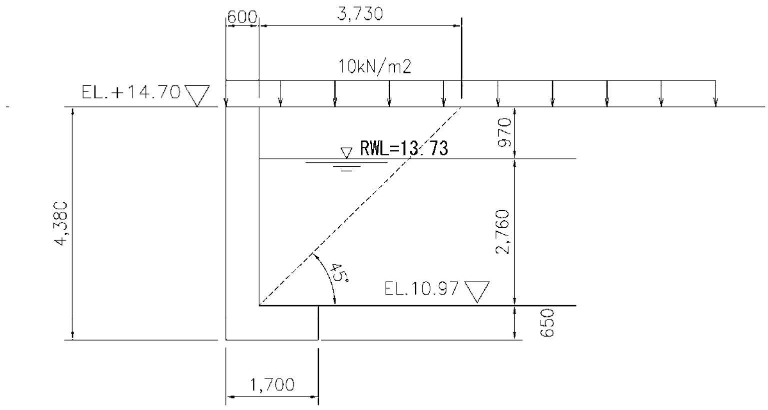
Vertical Force	V =	166.26	kN
Horizontal Force	H =	49.04	kN
Moment	M =	-72.00	kNm

position X = V・X/V= 149.28 / 166.26 = 0.90 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



$A = 1/2 \times (\quad 0 \quad + \quad 3.73 \quad) \times \quad 0 \quad = \quad 0.00 \quad m^2$

Converted Load of Embankment

	A		γ		L		qo		
q=	(0.00	×	18.00	+	3.73	×	10.0	)／	3.730

= 10.00 kN/m^2

Earth Pressure at Rest

$Po = ko \cdot (q + \gamma \cdot h + \gamma' \cdot h)$

$= 0.5 \times (10.00 + 18.00 \times 0.97 + 11.00 \times 2.76)$

$= 28.91 \quad kN/m^2$

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	10.00	kN/m^2

Water Pressure

$Pw = \gamma w \cdot h$

$= 9.80 \times 2.76$

$= 27.05 \quad kN/m^2$

Bendin Moment and Shear Force

Effective Load

$W = 28.91 + 27.05 = 55.96 \quad kN/m^2$

Bending Moment

$M = 1/2 \cdot W \cdot L^2$

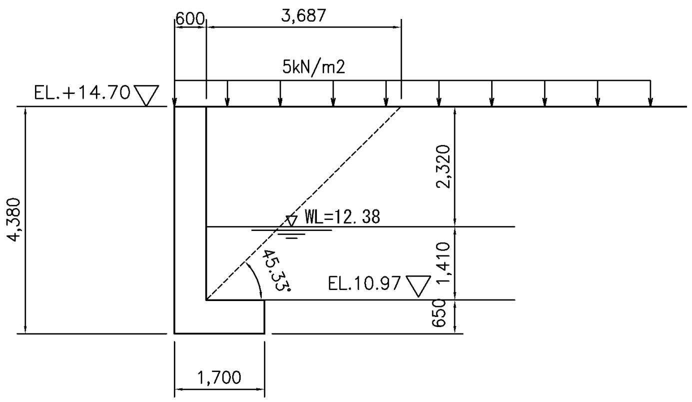
$= 1/2 \times 55.96 \times 1.0^2 = 27.98 \quad kN/m^2$

Shear Force

$S = W \cdot L$

$= 55.96 \times 1.0 = 55.96 \quad kN/m^2$

(2) Seismic Condition



A= 1/2×(0 + 3.687)× 0 = 0.00 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	0.00	×	18.00	+	3.687	×	5.0	)／	3.687

= 5.00 kN/m^2

Active Earth Pressure

Pea= kea・(q＋γ・h)＋kea'・γ'・h

= 0.452 × (5.00 + 18.00 × 2.32 + 11.00 × 1.41)

= 28.15 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	5.00	kN/m^2

Water Pressure

Pw ＝ γw・h

= 9.80 × 1.41

= 13.82 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 3.73 × 0.200 ＝ 17.90 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 41.97 kN/m^2

Effective Load

W ＝ 28.15 + 13.82 + 17.90 ＝ 59.87 kN/m^2

Bending Moment

M ＝ 1/2・W・L^2

＝ 1/2 × 59.87 × 1.0^2 ＝ 29.94 kN/m^2

Shear Force

S ＝ W・L

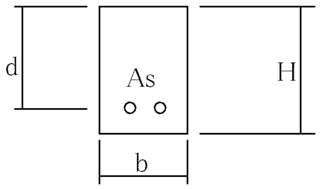
= 59.87 ×1.0 ＝ 59.87 kN/m^2

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	16	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	27.98	kN・m
Shear Force	S =	55.96	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	8.044	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	2.063	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	89.967	N/mm^2	○
Shearing Stress of Concrete	τ =	0.145	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0019620　　　　＝ 7.015

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1710953

j ＝ 1-(k/3)

＝ 0.9429682

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 206.3359328

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 8996.71466

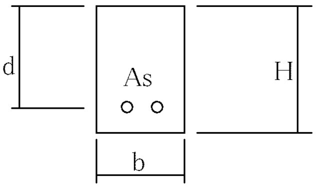
τ＝ (S*10^3)/(b*j*d)

＝ 14.4742744

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	16	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	29.94	kN・m
Shear Force	S =	59.87	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	8.044	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	2.208	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	96.269	N/mm^2	○
Shearing Stress of Concrete	τ =	0.155	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	12.30	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.00	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0019620

x ＝ k*d

＝ 7.015

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1710953

j ＝ 1-(k/3)

＝ 0.9429682

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 220.7897722

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 9626.93484

τ＝ (S*10^3)/(b*j*d)

＝ 15.4856113

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertival Wall

Dead load	24.00	×	0.65					=	15.60	kN/m^2
Surcharge	10.00							=	10.00	kN/m^2
Soil	79.93							=	79.93	kN/m^2
Uplift	-9.80	×	3.41	×	0.60	/	1.70	=	-11.79	kN/m^2
Subgrade Reaction	-65.05							=	-65.05	kN/m^2
									28.69	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	142.10	×	1.70					=	241.57	kN
(Vertical wall)	0.60	×	1.66	×	3.30	×	24.0	=	78.88	kN
Soil	1.66	×	3.30	×	1.70	×	18.0	=	167.63	kN
Uplift	-9.80	×	3.41	×	3.30	×	1.70	=	-187.47	kN
Load of Breast Wall	142.75		2.00					=	285.50	kN
									586.11	kN

Area of Base:	3.30	×	1.70	+	1.0	×	1.7	×	2 =	9.01	m^2
Subgrade Reaction :	586.11	/	9.01						=	65.05	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 28.69 × 1.0^2 = 14.35 kN·m

Shear Force

$S = W \cdot L =$ 65.05 × 1.00 = 65.05 kN

(2) Seismic Condition

Load at Base of Vertival Wall

Dead load	24.00	×	0.65					=	15.60	kN/m^2
Surcharge	5.00							=	5.00	kN/m^2
Soil	79.93							=	79.93	kN/m^2
Uplift	-9.80	×	2.06	×	0.60	/	1.70	=	-7.13	kN/m^2
Subgrade Reaction	-78.51							=	-78.51	kN/m^2
									14.89	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	142.10	×	1.70					=	241.57	kN
(Vertical wall)	0.60	×	1.66	×	3.30	×	24.0	=	78.88	kN
Soil	1.66	×	3.30	×	1.70	×	18.0	=	167.63	kN
Uplift	-9.80	×	2.06	×	3.30	×	1.70	=	-113.25	kN
Load of Breast Wall	166.26	×	2.00					=	332.52	kN
									707.35	kN

Subgrade Reaction : 707.35 / 9.01 = 78.51 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 14.89 × 1.0^2 = 7.45 kN·m

Shear Force

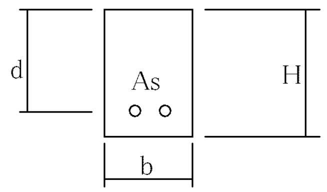
$S = W \cdot L =$ 78.51 × 1.00 = 78.51 kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	14.35	kN・m
Shear Force	S =	65.05	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.359	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	80.909	N/mm^2	○
Shearing Stress of Concrete	τ =	0.166	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0011034

x ＝ k*d

＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 135.9352935

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 8090.86029

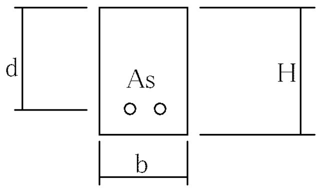
τ＝ (S*10^3)/(b*j*d)

＝ 16.5923133

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	7.45	kN・m
Shear Force	S =	78.51	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	0.706	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	42.005	N/mm^2	○
Shearing Stress of Concrete	τ =	0.200	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)
＝ 0.0011034

x ＝ k*d
＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p
＝ 0.1313486

j ＝ 1-(k/3)
＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)
＝ 70.5726785

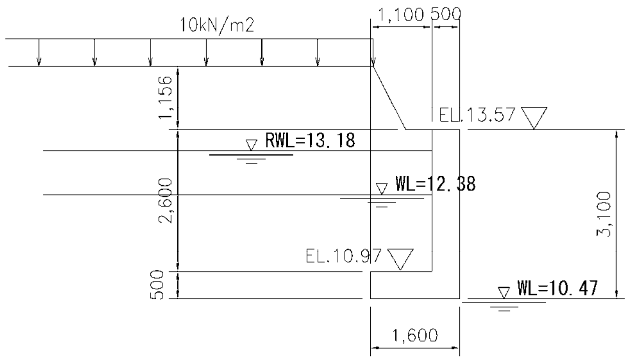
σs＝ (M*10^5)/(b*d^2*p*j)
＝ 4200.48148

τ＝ (S*10^3)/(b*j*d)
＝ 20.0255575

Structural Calculation of Breast Wall

MSR3 Land Side

1. Calculation Model



・Dimension

Height of Vertical wall	2.60	m
Tickness of Vertical Wall	0.50	m
Width of Bottom Slab	1.40	m
Tickness of Bottom Slab	0.50	m
Height of Embankment	1.16	m

・Elevation

Top of the wall	EL+	13.57	m
Upperside of Bottom Slab	EL+	10.97	m
Underside of Bottom Slab	EL+	10.47	m

Loading Condition

	State	Water Level		Loading Condition						
		Forth	Back	Direction of Seismic Force	Dead Load	Backfill	Earth Pressure	Water Pressure	Surcahrge	Buoyancy Force
1	Normal	13.18	10.47	—	○	○	○	○	○	○
2	Seismic	10.47	12.38	←	◎	◎	○	○	○	○
3	Seismic	10.47	12.38	→	◎	◎	○	○	○	○

○ : Considering

◎ : Considering Seismic Force

－ : Without Considering

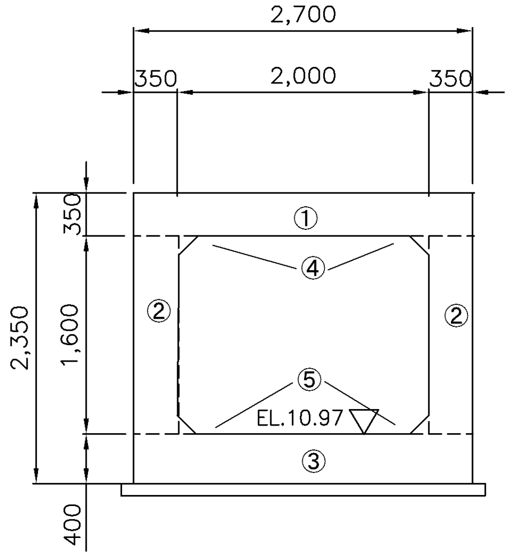
2. Design Condition

Unit Weight of Concrete			24.0	kN/m^3
Unit Weight of Water			9.8	kN/m^3
Unit Weight of Soil				
Saturated			20.0	kN/m^3
Wet			18.0	kN/m^3
Submerged			11.0	kN/m^3
Backfill Material			Sandy Soil	
internal Friction Angle of Soil			30.00	°
Seismic Lad			0.20	
Water Level				
Forth	Normal		10.47	m
	Seismic		10.47	m
Back	Normal		13.18	m
	Seismic		12.38	m
Difference	Normal		2.71	m
	Seismic		1.91	m
Earth Pressure	Normal		Earth Pressure at Rest	
	Seismic		Active Earth Pressure	
Angle between Back Side Srface of Wall and Vertical Plane			0.00	°
Surchage	Normal		10.0	kN/m^2
	Seismic		5.0	kN/m^2

3. Calculation of Load

3.1 Weight of Culvert

・Culvert Section



	Width (B)	Height（h）	α	n	A（m^2）	Y（m）	A・Y（m^3）
①	2.700	0.350	1.0	1	0.945	2.175	2.055
②	0.350	1.600	1.0	2	1.120	1.200	1.344
③	2.700	0.400	1.0	1	1.080	0.200	0.216
④	0.150	0.150	0.5	2	0.023	1.550	0.036
⑤	0.150	0.150	0.5	2	0.023	0.450	0.010
計					3.191		3.661

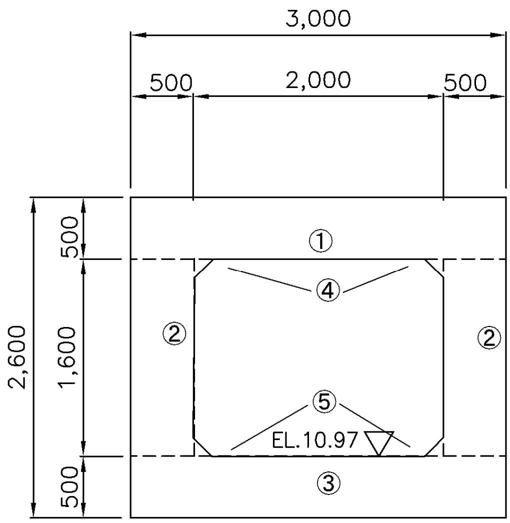
α；Triangle 0.5, Rectangle 1.0

ｎ； Number

Centroid of Box Culvet

Y＝ ΣA・Y／ΣA ＝ 1.147 m

Breast Wal Section



	Width (B)	Height（h）	α	n	A（m^2）
①	3.000	0.500	1.0	1	1.500
②	0.500	1.600	1.0	2	1.600
③	3.000	0.500	1.0	1	1.500
④	0.150	0.150	0.5	2	0.023
⑤	0.150	0.150	0.5	2	0.023
計					4.646

α；Triangle 0.5, Rectangle 1.0

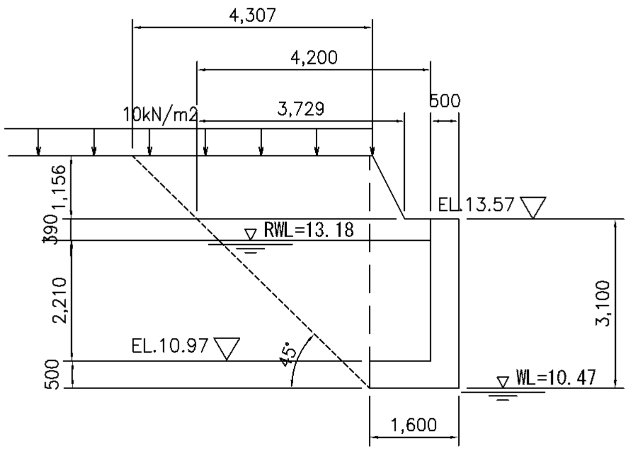
ｎ； Number

Weight of Body

W = γ×A×1.0 ＝ 111.504 kN

3.2 Converted Load of Embankment

(1) Normal Condition



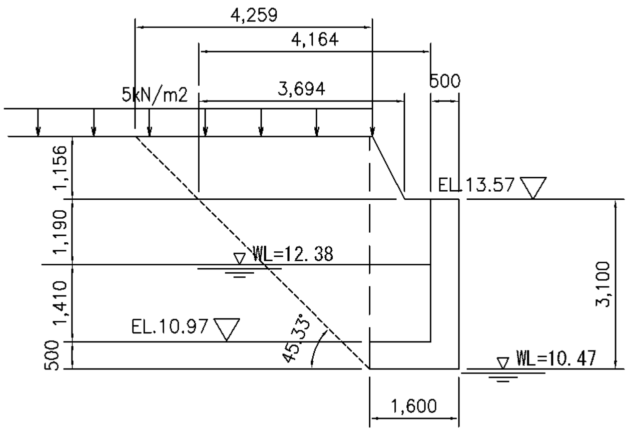
A= 1/2×(3.729 + 4.307)× 1.16 = 4.66 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	4.66	×	18.00	+	4.307	×	10.0	)／	4.200

= 30.23 kN/m^2

(2) Seismic Condition



A=	1/2×(	3.694	+	4.259	)×	1.16	=	4.61	m^2

Converted Load of Embankment

		A		γ		L		qo		
q=	(	4.61	×	18.00	+	4.259	×	5.0	)／	4.164

= 25.04 kN/m^2

Coefficient of Earth Pressure

Normal Condition

Earth Pressure at Rest

Seismic Condition

Active Earth Pressure (Coulomb)

internal Friction Angle of Soil	$\Phi =$	30.00	°
Angle Between Ground Surface and Horizontal Plane	$\alpha =$	0.00	°
Angle Between Back side Surface of Wall and Vertical Plane	$\theta =$	0.00	°
Angle of Wall Friction	$\delta = \Phi / 2 =$	15.00	°
Seismic Load	$kh =$	0.20	

$$\theta_0 = \tan^{-1} kh = 11.310°$$

3.3 Earth Pressure

(1) Coefficient of Earth Pressure

1) Normal Condition

Earth Pressure at Rest, K_0 = **0.5**

2) Seismic Condition

Active Earth Pressure (Coulomb)

φ = 30 ° θ = 0.000 ° $\delta = \Phi/2$ = 15.0 ° α = 0 °

kh = 0.20 $\theta o = \tan^{-1} kh$ = 11.310 °

$$Kea = \frac{\cos^2(\varphi - \theta o - \theta)}{\cos\theta o \times \cos^2\theta \times \cos(\theta + \theta o + \delta) \times \left\{1 + \sqrt{\dfrac{\sin(\varphi + \delta) \times \sin(\varphi - \alpha - \theta o)}{\cos(\theta + \theta o + \delta) \times \cos(\theta - \alpha)}}\right\}^2}$$

$$= \frac{0.8973}{0.8790 \times \left\{1 + \sqrt{\dfrac{0.2266}{0.8964}}\right\}^2} = \mathbf{0.452}$$

(2) Angle of Active Rupture in Seismic Condition

$\varphi =$ 30 °

$\delta = \varphi/2 =$ 15 °(Between soil and soil)

$\theta =$ 0

$\alpha =$ 0

$\theta_O = \tan^{-1}Kh =$ 11.310 °

kh = 0.2

$$\cot(\zeta_{EA} - \alpha) = -\tan(\varphi+\delta+\theta-\alpha) + \sec(\varphi+\delta+\theta-\alpha)\cdot X$$

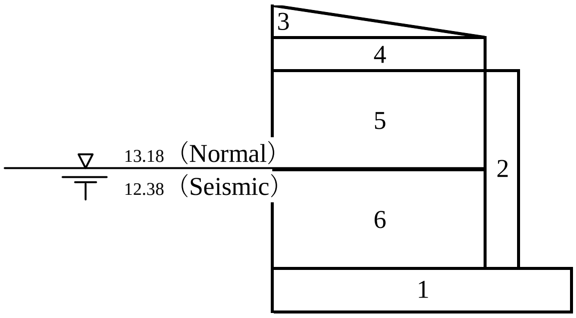
$$X = \sqrt{\frac{\cos(\theta+\delta+\theta_O)\cdot\sin(\varphi+\delta)}{\cos(\theta-\alpha)\cdot\sin(\varphi-\alpha-\theta_O)}}$$

$$= \sqrt{\frac{0.6339}{0.3204}} = 1.4066$$

$$\cot(\zeta_{EA} - \alpha) = -1.0000 + 1.4142 \times 1.4066 = 0.9892$$

$$\zeta_{EA} = \tan^{-1}\ 1 \ / \ 0.9892 = \boxed{45.311\ °}$$

(3) Weight of Concrete and Backfill



Weight of Body

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
1	1.600	0.500	1.00		24.0	19.20
2	0.500	2.600	1.00		24.0	31.20

No	V (kN)	X (m)	V・X (kNm)	H (kN)	Y (m)	H・Y (kNm)
1	19.20	0.800	15.36	3.84	-0.897	-3.44
2	31.20	0.250	7.80	6.24	0.653	4.07
Total	50.40		23.16	10.08		0.63

Weight of Soil (Normal)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.703	1.156	1.00		18.0	7.31
4	1.100	0.000	1.00		18.0	0.00
5	1.100	0.390	1.00		18.0	7.72
6	1.100	2.210	1.00		20.0	48.62

No	V (kN)	X (m)	V・X (kNm)
3	7.31	1.249	9.13
4	0.00	1.050	0.00
5	7.72	1.050	8.11
6	48.62	1.050	51.05
Total	63.65		68.29

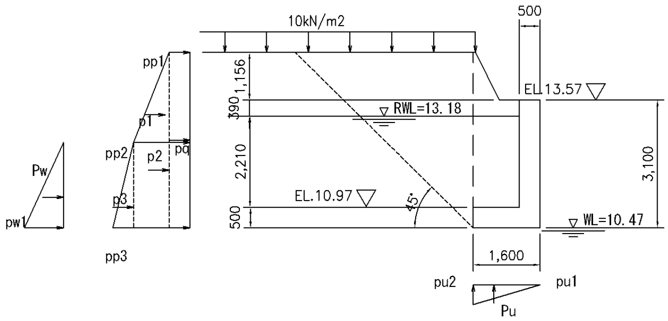
Weight of Soil (Seismic)

No	Width (m)	Height (m)	Length (m)		Unit Weight (kN/m^3)	V (kN)
3	0.703	1.156	1.00		18.0	7.31
4	1.100	0.000	1.00		18.0	0.00
5	1.100	1.190	1.00		18.0	23.56
6	1.100	1.410	1.00		20.0	31.02

No	V (kN)	X (m)	V・X (kNm)	H (kN)	Y (m)	H・Y (kNm)
3	7.31	1.249	9.13	1.46	2.531	3.70
4	0.00	1.050	0.00	0.00	1.953	0.00
5	23.56	1.050	24.74	4.71	1.358	6.40
6	31.02	1.050	32.57	6.20	0.058	0.36
Total	61.89		66.44	12.37		10.46

(5) Earth Pressure

1) Normal Condition

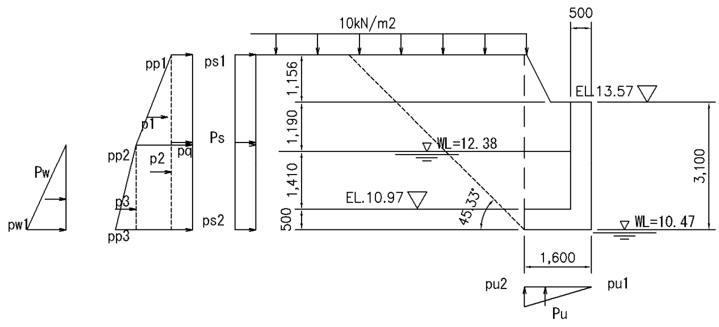


φ	:	internal Friction Angle of Soil	30
Ko	:	Earth Pressure at Rest	0.5
q	:	Converted Load of Embankment	30.23 kN/m^2

	V(kN)	H(kN/m)
Pp1	0.00	15.12
Pp2	0.00	18.63
Pp3	0.00	33.54

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Earth Pressure (p1)	0.00	0.000	0.00	0.68	1.693	1.15
Earth Pressure (p2)	0.00	0.000	0.00	9.51	0.208	1.98
Earth Pressure (p3)	0.00	0.000	0.00	20.20	-0.244	-4.93
Earth Pressure (pq)	0.00	0.000	0.00	46.86	0.403	18.88
Total	0.00		0.00	77.25		17.08

2) Seismic Condition



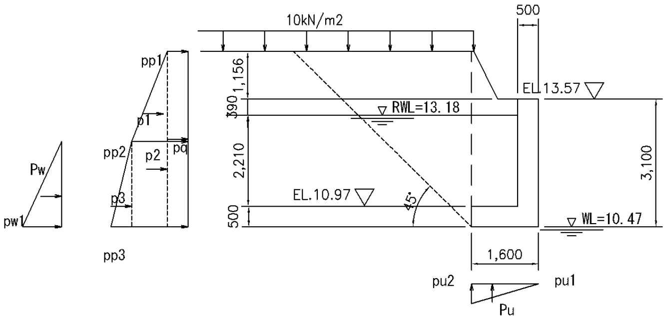
φ	:	internal Friction Angle of Soil	30	°
δ	:	Angle of Wall Friction	15	°
		δ=φ/2（Seismic Condition）		
α	:	Angle Between Ground Surface and Horizontal Plane	0	°
θ	:	Angle Between Back side Surface of Wall and Vertical Plane	0	°
Ka	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	25.04	kN/m^2

	V(kN)	H(kN/m)
Earth Pressure (pp1)		11.32
Earth Pressure (pp2)		21.00
Earth Pressure (pp3)		30.50

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Earth Pressure (p1)	1.49	1.600	2.38	5.56	1.160	6.45
Earth Pressure (p2)	4.79	1.600	7.66	8.93	-0.192	-1.71
Earth Pressure (p3)	2.35	1.600	3.76	8.76	-0.510	-4.47
Earth Pressure (pq)	9.08	1.600	14.53	33.89	0.403	13.66
Total	17.71		28.33	57.14		13.93

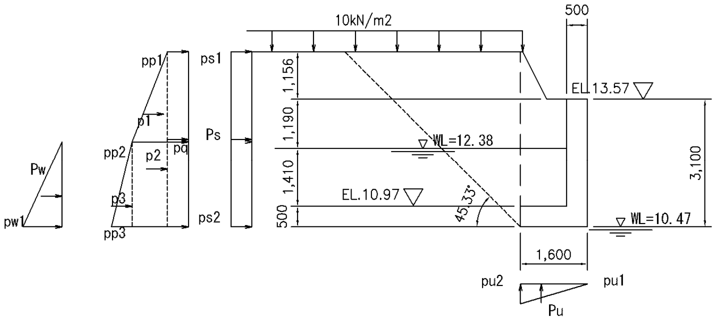
3.4 Water Pressure

(1) Normal



	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	35.99	-0.244	-8.78
Total	0.00		0.00	35.99		-8.78

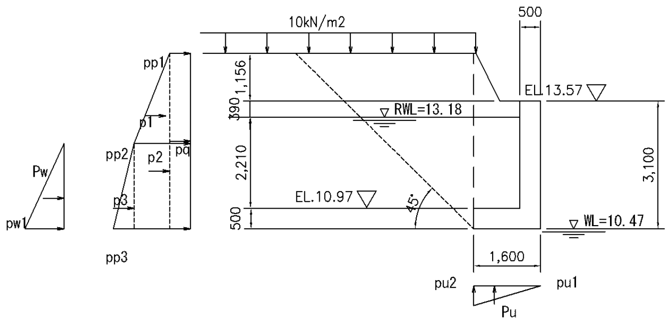
(2) Seismic Condition



	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Water Pressure pw3	0.00	0.000	0.00	17.88	0.637	11.39
Total	0.00					

3.5 Uplift

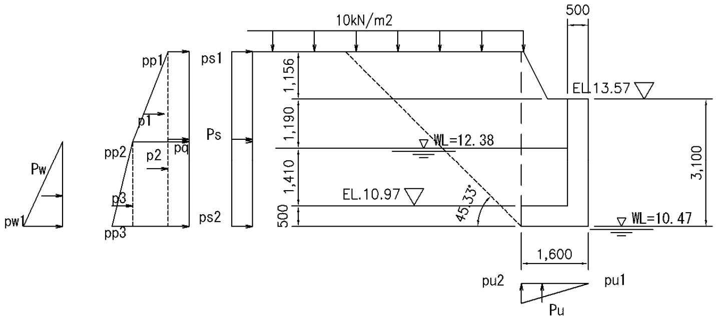
(1) Normal Conditon



	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	26.56	0.00

	V(kN)	X(m)	V・X(kNm)	H(kN)	Y(m)	H・Y(kNm)
Uplift Pu	21.25	1.067	22.67	0.00	0.000	0.00
Total	21.25		22.67	0.00		0.00

(2) Seismic Conditon

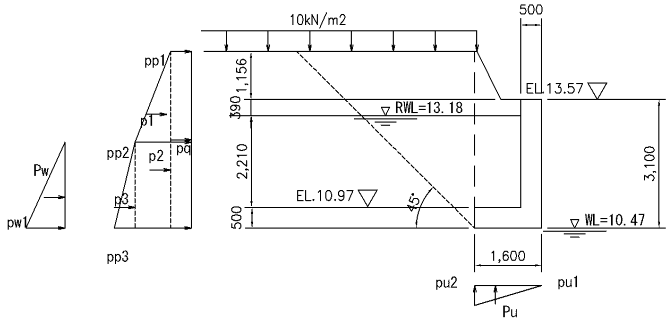


	V(kN)	H(kN/m)
Pu1	0.00	0.00
Pu2	18.72	0.00

	V(kN)	X(m)	V･X(kNm)	H(kN)	Y(m)	H･Y(kNm)
Uplift Pu	14.98	1.067	15.98	0.00	0.000	0.00
Total	14.98		15.98	0.00		0.00

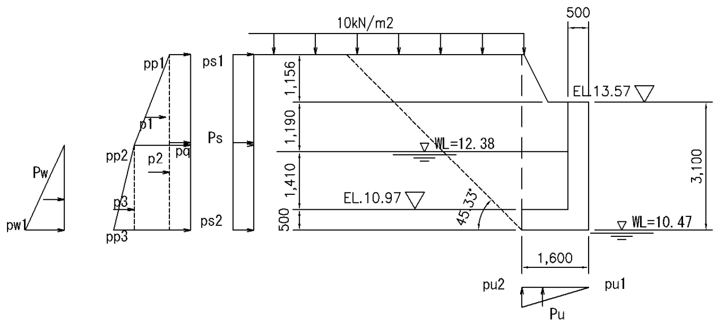
3.6 Surcharge

(1) Normal Condition



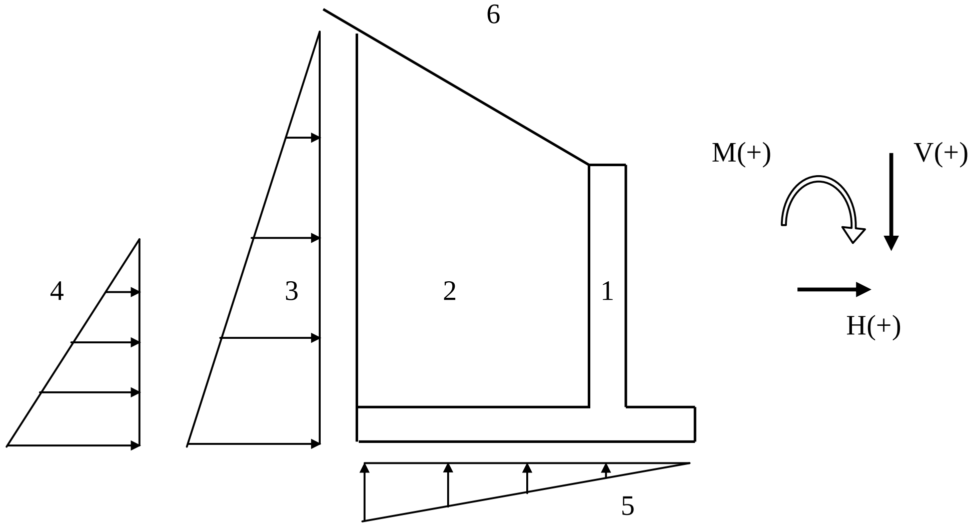
	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	10.0	0.00		0.00	0.00
Total				0.00	0.00

(2) Seismic Conditon



	q(kN/m^2)	B(m)	X(m)	V(kN)	V･X(kNm)
1	5.0	0.00		0.00	0.00
Total				0.00	0.00

3.7 Summary of Load
(1) Normal Condition

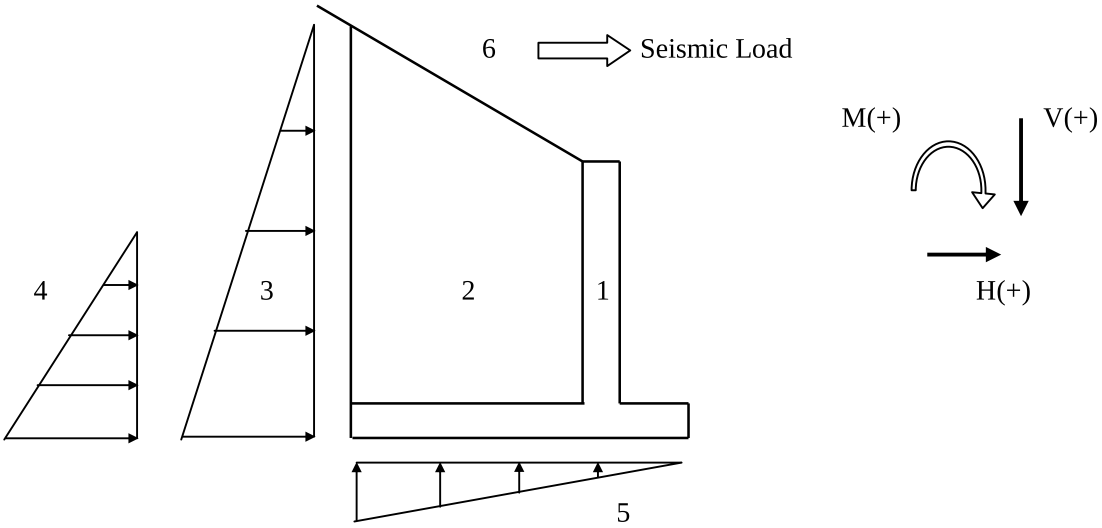


		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	0.00	0.00
2	Weight of Back Fill	63.65	68.29	0.00	0.00
3	Earth Pressure	0.00	0.00	77.25	17.08
4	Water Pressure	0.00	0.00	35.99	-8.78
5	Uplift	-21.25	-22.67	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	Total	92.80	68.78	113.24	8.30

Vertical Force	V=	92.80	kN
Horizontal Force	H=	113.24	kN
Moment	M=	-60.48	kNm

position X = V・X/V= 68.78 / 92.80 = 0.74 m

(2) Seismic Condition (→)

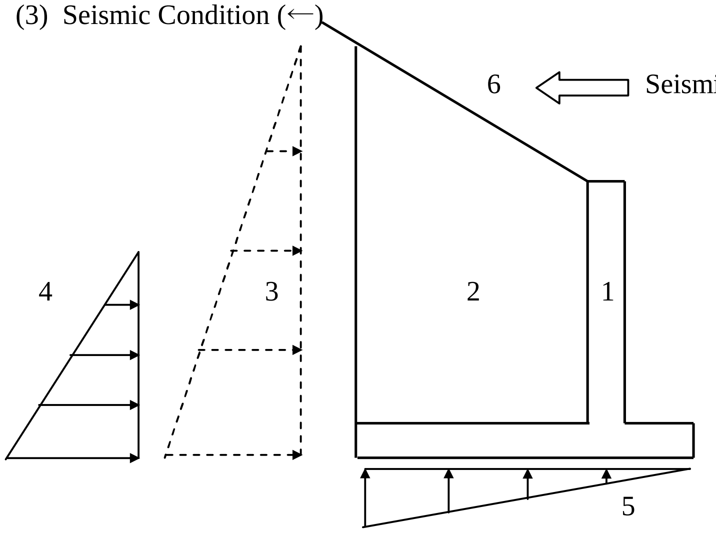


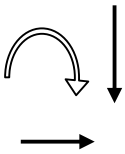
		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	10.08	0.63
2	Weight of Back Fill	61.89	66.44	12.37	10.46
3	Earth Pressure	17.71	28.33	57.14	13.93
4	Water Pressure	0.00	0.00	17.88	11.39
5	Uplift	-14.98	-15.98	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	115.02	101.95	97.47	36.41

Vertical Force	V =	115.02	kN
Horizontal Force	H =	97.47	kN
Moment	M =	-65.54	kNm

position X = V・X/V= 101.95 / 115.02 = 0.89 m

(3) Seismic Condition (←)





		V(kN)	V・X(kNm)	H(kN)	H・Y(kNm)
1	Weight of Concrete	50.40	23.16	-10.08	-0.63
2	Weight of Back Fill	61.89	66.44	-12.37	-10.46
3	Earth Pressure	17.71	28.33	57.14	13.93
4	Water Pressure	0.00	0.00	17.88	11.39
5	Uplift	-14.98	-15.98	0.00	0.00
6	Srcharge	0.00	0.00	0.00	0.00
	合計	115.02	101.95	52.57	14.23

Vertical Force V = 115.02 kN

Horizontal Force H = 52.57 kN

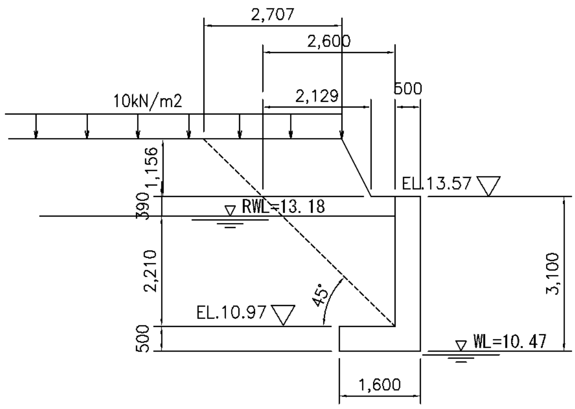
Moment M = -87.72 kNm

position X = V・X/V= 101.95 / 115.02 = 0.89 m

4. Calculation of Vertical wall

4.1 Calculation of Bending Moment and Sher Stress

(1) Normal Condition



A= 1/2×(2.129 + 2.707)× 1.16 = 2.80 m^2

Converted Load of Embankment

	A	γ		L		qo		
q=	(2.80 ×	18.00	+	2.707	×	10.0	)／	2.600

= 29.80 kN/m^2

Earth Pressure at Rest

$Po = ko \cdot (q + \gamma \cdot h + \gamma' \cdot h)$

= 0.5 × (29.80 + 18.00 × 0.39 + 11.00 × 2.21)

= 30.57 kN/m^2

Ko	:	Earth Pressure at Rest	0.5	
q	:	Converted Load of Embankment	29.80	kN/m^2

Water Pressure

$Pw = \gamma w \cdot h$

= 9.80 × 2.21

= 21.66 kN/m^2

Bendin Moment and Shear Force

Effective Load

W = 30.57 + 21.66 = 52.23 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2$

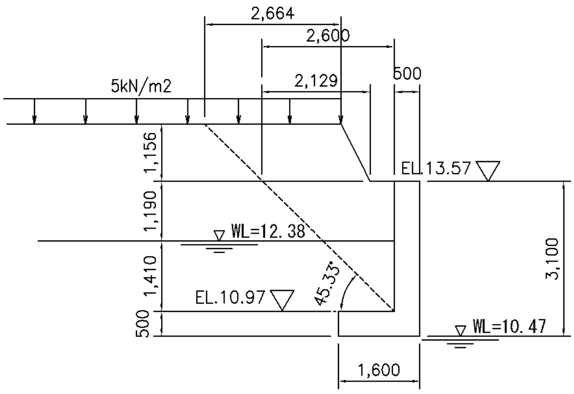
= 1/2 × 52.23 × 1.0^2 = 26.12 kN/m^2

Shear Force

$S = W \cdot L$

= 52.23 ×1.0 = 52.23 kN/m^2

(2) Seismic Condition



A= 1/2×(2.129 + 2.664)× 1.16 = 2.78 m^2

Converted Load of Embankment

	A		γ		L		qo		
q= (	2.78	×	18.00	+	2.664	×	5.0	)／	2.600

= 24.37 kN/m^2

Active Earth Pressure

Pea= kea･(q＋γ･h)＋kea'･γ'･h

= 0.452 × (24.37 + 18.00 × 1.19 + 11.00 × 1.41)

= 27.71 kN/m^2

Kea	:	Coefficient of Active Earth Pressure	0.452	
q	:	Converted Load of Embankment	24.37	kN/m^2

Water Pressure

Pw ＝ γw ･ h

= 9.80 × 1.41

= 13.82 kN/m^2

Seismic Load

Ph ＝ γc x h x Kh

= 24.00 × 2.60 × 0.200 ＝ 12.48 kN/m^2

Kh : Seismic Load 0.200

Bendin Moment and Shear Force ＝ 41.53 kN/m^2

Effective Load

W ＝ 27.71 + 13.82 + 12.48 ＝ 54.01 kN/m^2

Bending Moment

M ＝ 1/2･W･L^2

＝ 1/2 × 54.01 × 1.0^2 ＝ 27.01 kN/m^2

Shear Force

S ＝ W･L

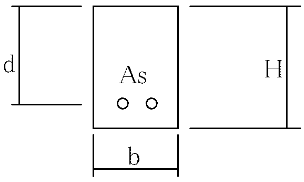
= 54.01 ×1.0 ＝ 54.01 kN/m^2

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter	16	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	26.12	kN・m
Shear Force	S =	52.23	kN

Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	8.044	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.926	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	83.986	N/mm^2	○
Shearing Stress of Concrete	τ =	0.135	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)

＝ 0.0019620

x ＝ k*d

＝ 7.015

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1710953

j ＝ 1-(k/3)

＝ 0.9429682

σc ＝ (2*M*10^5)/(b*d^2*k*j)

＝ 192.6195341

σs ＝ (M*10^5)/(b*d^2*p*j)

＝ 8398.64857

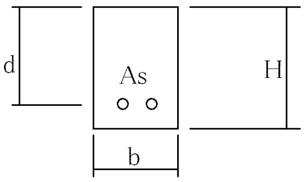
τ ＝ (S*10^3)/(b*j*d)

＝ 13.5094952

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	16	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	27.01	kN・m
Shear Force	S =	54.01	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	8.044	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.992	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	86.848	N/mm^2	○
Shearing Stress of Concrete	τ =	0.140	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2	

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0019620　　　　＝ 7.015

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1710953

j ＝ 1-(k/3)

＝ 0.9429682

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 199.1827571

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 8684.81998

τ＝ (S*10^3)/(b*j*d)

＝ 13.9698992

5. Calculation of Bottom Slab

5.1 Calculation of Bending Moment and Shear Force

(1) Normal Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50					=	12.00	kN/m^2
Surcharge	10.00							=	10.00	kN/m^2
Soil	63.65							=	63.65	kN/m^2
Uplift	-9.80	×	2.71	×	0.50	/	1.60	=	-8.30	kN/m^2
Subgrade Reaction	-37.22							=	-37.22	kN/m^2
									40.13	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	111.50	×	1.60					=	178.40	kN
(Vertical wall)	0.50	×	0.50	×	3.00	×	24.0	=	18.00	kN
Soil	0.50	×	3.00	×	1.60	×	18.0	=	43.20	kN
Uplift	-9.80	×	2.71	×	3.00	×	1.60	=	-127.48	kN
Load of Breast Wall	92.80	×	2.00					=	185.60	kN
									297.72	kN

Area of Base	3.00	×	1.60	+	1.0	×	1.6	×	2 =	8.00	m^2
Subgrade Reaction : 245.08 / 8.55	297.72	/	8.00						=	37.22	kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 40.13 × 1.0^2 = 20.07 kN·m

Shear Force

$S = W \cdot L =$ 37.22 × 1.00 = 37.22 kN

(2) Seismic Condition

Load at Base of Vertical Wall

Dead load	24.00	×	0.50					=	12.00	kN/m^2
Surcharge	5.00							=	5.00	kN/m^2
Soil	63.65							=	63.65	kN/m^2
Uplift	-9.80	×	1.91	×	0.50	/	1.60	=	-5.85	kN/m^2
Subgrade Reaction	-47.47							=	-47.47	kN/m^2
									27.33	kN/m^2

Calculation of Subgrade Reaction

Dead Load (Clvert)	111.50	×	1.60					=	178.40	kN
(Vertical wall)	0.50	×	0.50	×	3.00	×	24.0	=	18.00	kN
Soil	0.50	×	3.00	×	1.60	×	18.0	=	43.20	kN
Uplift	-9.80	×	1.91	×	3.00	×	1.60	=	-89.85	kN
Load of Breast Wall	115.02	×	2.00					=	230.04	kN
									379.79	kN

Subgrade Reaction : 245.08 / 8.55 379.79 / 8.00 = 47.47 kN/m^2

Bending Moment

$M = 1/2 \cdot W \cdot L^2 =$ 1/2 × 27.33 × 1.0^2 = 13.67 kN·m

Shear Force

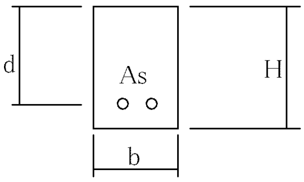
$S = W \cdot L =$ 47.47 × 1.00 = 47.47 kN

4.2 Rainfocement Calcularion

(1) Normal Condition

Position of Rainforcing Bar

Diameter 12 mm
Number of Rainforcing Bar 4.0
Cover Concrete 9.0 cm



Bending Moment	M =	20.07	kN・m
Shear Force	S =	37.22	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.901	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	113.159	N/mm^2	○
Shearing Stress of Concrete	τ =	0.095	N/mm^2	○
Allowable Compressive Stress of Concrete	σca=	8.2	N/mm^2	
Allowable Tensile Stress of Rainforcing Bar	σsa=	140.0	N/mm^2	
Allowable Shear Stress of Concrete	τa =	0.36	N/mm^2	

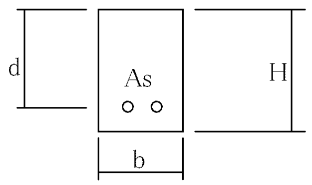
Single Rainforcing Bar

p ＝ As/(b*d)　　　x ＝ k*d
＝ 0.0011034　　　＝ 5.385
k ＝ √(2*n*p+(n*p)^2)-n*p
＝ 0.1313486
j ＝ 1-(k/3)
＝ 0.9562171
σc＝ (2*M*10^5)/(b*d^2*k*j)
＝ 190.1199541
σs＝ (M*10^5)/(b*d^2*p*j)
＝ 11315.92795
τ＝ (S*10^3)/(b*j*d)
＝ 9.4937110

(2) Seisimic Condition

Position of Rainforcing Bar

Diameter	12	mm
Number of Rainforcing Bar	4.0	
Cover Concrete	9.0	cm



Bending Moment	M =	13.67	kN・m
Shear Force	S =	47.47	kN
Width of Member	b =	100.0	cm
Hight of Member	H =	50.0	cm
Equivalent Height（Tensile Side）	d =	41.000	cm
Total Cross-Sectional Area（Tensile Si	A_s =	4.524	cm^2
Modulous of Elasticity	n =	9.000	

Compressive Stress of Concrete	σc=	1.295	N/mm^2	○
Tensile Stress of Rainforcing Bar	σs=	77.075	N/mm^2	○
Shearing Stress of Concrete	τ =	0.121	N/mm^2	○

Allowable Compressive Stress of Concrete	σca=	12.3	N/mm^2
Allowable Tensile Stress of Rainforcing Bar	σsa=	210.0	N/mm^2
Allowable Shear Stress of Concrete	τa =	0.54	N/mm^2

Single Rainforcing Bar

p ＝ As/(b*d)　　　　x ＝ k*d

＝ 0.0011034　　　　＝ 5.385

k ＝ √(2*n*p+(n*p)^2)-n*p

＝ 0.1313486

j ＝ 1-(k/3)

＝ 0.9562171

σc＝ (2*M*10^5)/(b*d^2*k*j)

＝ 129.4937604

σs＝ (M*10^5)/(b*d^2*p*j)

＝ 7707.46064

τ＝ (S*10^3)/(b*j*d)

＝ 12.1081800

11.6 Wing Wall

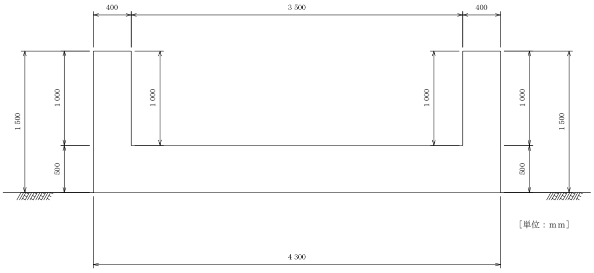
(1) Design Condition

Design Condition is same as Breast wall.

(2) Structural Calculation

The detail of structural calculation of breast wall in longitudinal direction is indicated from the following page.

1 MSL3 Wing Wall (U-shaped)



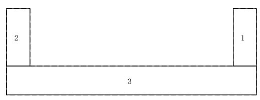
1.1 Water Level

[1]Normal Condition Left Fl = 1.250 m Inside Fi = 0.000 m, Right Fr = 1.250 m

[2]Seismic Left Fl = 0.730 m Inside Fi = 0.000 m, Right Fr = 0.730 m

1.2 Stability Analysis

(1) Center of Gravity



	Width × Height × Depth	$Vi(m^3)$	Center of gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.400× 1.000× 1.000	0.400	4.100	1.000	1.640	0.400	
2	0.400× 1.000× 1.000	0.400	0.200	1.000	0.080	0.400	
3	4.300× 0.500× 1.000	2.150	2.150	0.250	4.623	0.538	
Σ		2.950	——	——	6.343	1.338	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 6.343／ 2.950 = 2.150 (m)
YG = Σ（Vi・Yi）／ΣVi = 1.338／ 2.950 = 0.453 (m)

(2) Weight of Body and Uplift

(a) Weight of Body

[1]Normal Condition

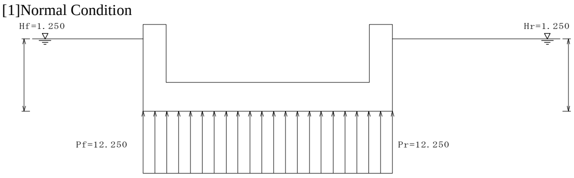
	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 0.800 ＝ 19.200	2.150
Bottom Slab	24.000 × 2.150 ＝ 51.600	2.150

[2]Seismic Condition

	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 0.800 ＝ 19.200	2.150
Bottom Slab	24.000 × 2.150 ＝ 51.600	2.150

(b) Uplift

[1]Normal Condition



Water level Hf = 1.250 (m)
Strength of Water Pressure Pf = 12.250 (kN/m^2)
Buoyancy Effecting on Body

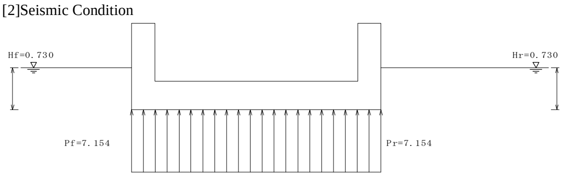
$$U = \frac{Pf+Pr}{2}\cdot Bj\cdot Bc\cdot \lambda = 52.675\ (kN)$$

Position (From the Left Edge)

$$X = \frac{Pf+2\cdot Pr}{3\cdot(Pf+Pr)}\cdot Bj = 2.150\ (m)$$

Where,
Bj : Width of Footing Bj= = 4.300 (m)
Bc: Length of Footing Bc = 1.000 (m)
λ : Reduction Coefficient of buoyancy λ = 1.000

[2]Seismic Condition



Water level Hf = 0.730 (m)
Strength of Water Pressure Pf = 7.154 (kN/m^2)
Buoyancy Effecting on Body

$$U = \frac{Pf+Pr}{2}\cdot Bj\cdot Bc\cdot \lambda = 30.762\ (kN)$$

Position (From the Left Edge)

$$X = \frac{Pf+2\cdot Pr}{3\cdot(Pf+Pr)}\cdot Bj = 2.150\ (m)$$

Where,
Bj : Width of Footing Bj= = 4.300 (m)
Bc: Length of Footing Bc = 1.000 (m)
λ : Reduction Coefficient of buoyancy λ = 1.000

(3) Stability against Buoyancy Force

$$F_s = \frac{\Sigma Vu + \alpha \cdot Pv}{U}$$

Where,
ΣVu : Total Vertical Force (kN
α : Efficiency Rate of Earth Pressure α＝0.000
Pv : Vertical Element of Earth Pressure (kN)
U : Buoyancy (kN)

	ΣVu (kN)	Pv (kN)	U (kN)	Safty Factor f_s	Required FS f_{sa}
Normal Condition	70.800	0.000	52.675	1.344	1.330
Seismic Condition	70.800	0.000	30.762	2.302	1.330

1.3 Structural Calculation

1.3.1 Calculation of Earth Pressure

[1]Normal Condition

■Earth Pressure at Rest

Height of Imaginary Back Side H = 1.250 m
Height above Water Surface H1 = 0.250 m
Height under Water Surface H2 = 1.000 m
Angle Between Back side Surface of Wall and Vertical Plane α = 0.000 °
Unit Weight of Back fill Ma γs = 18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	5.000	5.000	0.000
p2	7.250	7.250	0.000
p3	12.350	12.350	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 5.000+\ 7.250)\times 0.250 = 1.531\ kN$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2}\cdot(p2+p3)\cdot H2 = \frac{1}{2}\times(\ 7.250+\ 12.350)\times 1.000 = 9.800\ kN$$

Total Earth Pressure
P_0 ＝ P ＝ P1＋P2 ＝ 1.531＋9.800 ＝ 11.331 kN

[1]Seismic Condition

■Earth Pressure at Rest

Height of Imaginary Back Side	H	=	1.250 m
Height above Water Surface	H1	=	0.770 m
Height under Water Surface	H2	=	0.480 m
Angle Between Back side Surface of Wall and Vertical Plane	α	=	0.000 °
Unit Weight of Back fill Ma	γs	=	18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.500	2.500	0.000
p2	9.430	9.430	0.000
p3	11.878	11.878	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 2.500+\ \ 9.430)\times\ 0.770 =\ \ \ 4.593\ \mathrm{kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2}\cdot(p3+p4)\cdot H2 = \frac{1}{2}\times(\ 9.430+\ 11.878)\times\ 0.480 =\ \ \ 5.114\ \mathrm{kN}$$

Total Earth Pressure

$P_0 = P = P1 + P2 = 4.593 + 5.114 = 9.707$ kN

■Active Earth Pressure in Normal Condition（Coulomb）

Internal Friction Angle of Soil	φ =	30.000 °
Angle Between Ground Surface and Horizontal Plane	β =	0.000 °
Angle of Wall Friction	δ =	10.000 °

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ )}{\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +10.000^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +10.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ )}{\cos(0.000^\circ\ +10.000^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.3085$

Strength of Earth Pressure

Horizontal　$ph = p\cdot\cos(\alpha+\delta)$

Vertical　$pv = p\cdot\sin(\alpha+\delta)$

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	1.542	1.519	0.268
p2	5.818	5.729	1.010
p3	7.328	7.217	1.272

Earth Pressure above Water Surface

$$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 1.542+\ \ 5.818)\times\ 0.770 =\ \ \ 2.834\ \mathrm{kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2}\cdot(p2+p3)\cdot H2 = \frac{1}{2}\times(\ 5.818+\ \ 7.328)\times\ 0.480 =\ \ \ 3.155\ \mathrm{kN}$$

Total Earth Pressure

$P = P1 + P2 = 2.834 + 3.155 = 5.989$ kN

Horizontal Elements of Earth Pressure is as follows.

$P_H = P\cdot\cos(\alpha+\delta) = 5.989\times\cos(\ 0.000^\circ + 10.000^\circ) = 5.898$ kN

■Active Earth Pressure in Seismic Condition　(Mononobe-Okabe)

Angle of Wall Friction　　$\delta = 0.000°$

Seismic Compound Angle above Water Surface

$\theta = \tan^{-1}kh = \tan^{-1}0.20 = 11.310°$

Active Earth Pressure above Water Surface

$$K = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos 11.310^\circ\ \cdot\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +0.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.4733$

In case $\varphi-\beta-\theta<0$, $\sin(\varphi-\beta-\theta) = 0$

Strength of Earth Pressure

Horizontal　$ph = p\cdot\cos(\alpha+\delta)$

Vertical　$pv = p\cdot\sin(\alpha+\delta)$

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.366	2.366	0.000
p2	8.926	8.926	0.000
p3	8.926	8.926	0.000
p4	11.243	11.243	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 2.366+\ \ 8.926)\times\ 0.770 =\ \ \ 4.347\ \mathrm{kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2}\cdot(p3+p4)\cdot H2 = \frac{1}{2}\times(\ 8.926+\ 11.243)\times\ 0.480 =\ \ \ 4.840\ \mathrm{kN}$$

Total Earth Pressure

$P = P1 + P2 = 4.347 + 4.840 = 9.188$ kN

Horizontal Elements of Earth Pressure is as follows.

$P_{HE} = P\cdot\cos(\alpha+\delta) = \ \ 9.188\times\cos(\ 0.000^\circ + \ 0.000^\circ) = 9.188$ kN

■Summary of Earth Pressure

Strength of Earth Pressure

$p_{0E} = p_0 + (\ p_{HE} - p_H\ )$

Where,

p_{0E}：Strength of Earth Pressure in Seismic Condition

p_0：Strength of Earth Pressure in Normal Condition (Horizontal)

p_{HE}：Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition

p_H：Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition

	Normal Earth Pressure at Rest p_O (kN/m^2)	Seismic Active Earth Pressure p_{HE} (kN/m^2)	Normal Active Earth Pressure p_H (kN/m^2)	Seismic Pressure at Res p_{OE} (kN/m^2)
Top	2.500	2.366	1.519	3.347
Water Surface	9.430	8.926	5.729	12.626
Bottom	11.878	11.243	7.217	15.904

Total Earth Pressure

$P_{0E} = P_0 + (\ P_{HE} - P_H\ )$

$= 9.707 + (\ 9.188 - 5.898\ )$

$= 12.997$ kN

Where,

P_{0E}：Total Earth Pressure in Seismic Condition (kN)

P_0：Total Earth Pressure in Normal Condition Horizontal (kN)

P_{HE}：Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)

P_H：Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)

1.3.2 Side wall

(1) Water Pressure

$pi = hi\cdot Gw$

Where,

pi：Strength of Water Pressure at the Edge of Bottom Slab (kN/m^2)

hi：Water Depth (m)

Gw：Unit Weight of Water (kN/m^3),　Gw = 9.800

[1]Normal Condition

Water Depth hi (m)	Strength pi (kN/m^2)
1.000	9.800

[2]Seismic Condition

Water Depth hi (m)	Strength pi (kN/m^2)
0.480	4.704

(2) Subgrade Reaction

[1]Normal Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	1.000	9.600	9.600	9.600	0.000	0.000
Weight of Right Side wall	Right Side Wall	1.000	9.600	9.600	9.600	3.900	37.440
Weight of Bottom Slab	Bottom Slab	3.900	12.000	12.000	46.800	1.950	91.260
Buoyancy	Bottom Slab	3.900	-12.250	-12.250	-47.775	1.950	-93.161
Total	———	———	———	———	18.225	———	35.539

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Left Side Wall	0.250	5.000	7.250	1.531	1.117	1.711
Earth Pressure (Left)	Left Side Wall	1.000	7.250	12.350	9.800	0.457	4.475
Earth Pressure (Right)	Right Side Wall	0.250	-5.000	-7.250	-1.531	1.117	-1.711
Earth Pressure (Right)	Right Side Wall	1.000	-7.250	-12.350	-9.800	0.457	-4.475
Water Pressure (Left)	Left Side Wall	1.000	0.000	9.800	4.900	0.333	1.633
Water Pressure (Right)	Right Side Wall	1.000	0.000	-9.800	-4.900	0.333	-1.633
Total	———	———	———	———	0.000	———	0.000

Vertical Force $\quad V = \quad \Sigma Vi = 18.225(kN)$

Horizontal Force $\quad H = \quad \Sigma Hi = 0.000(kN)$

Moment Mo $\quad = \quad \Sigma Mxi + \Sigma Myi = 35.539 + 0.000 = 35.539(kN.m)$

[2]Seismic Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	1.000	9.600	9.600	9.600	0.000	0.000
Weight of Right Side wall	Right Side Wall	1.000	9.600	9.600	9.600	3.900	37.440
Weight of Bottom Slab	Bottom Slab	3.900	12.000	12.000	46.800	1.950	91.260
Buoyancy	Bottom Slab	3.900	-7.154	-7.154	-27.901	1.950	-54.406
Total	———	———	———	———	38.099	———	74.294

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Right Side Wall	0.770	-3.347	-12.626	-6.150	0.790	-4.861
Earth Pressure (Left)	Right Side Wall	0.480	-12.626	-15.904	-6.847	0.231	-1.580
Earth Pressure (Right)	Left Side Wall	0.770	-3.347	-12.626	-6.150	0.790	-4.861
Earth Pressure (Right)	Right Side Wall	0.480	-12.626	-15.904	-6.847	0.231	-1.580
Water Pressure (Left)	Left Side Wall	0.480	0.000	4.704	1.129	0.160	0.181
Water Pressure (Right)	Left Side Wall	0.480	0.000	-4.704	-1.129	0.160	-0.181
Total	———	———	———	———	-12.997	———	-6.442

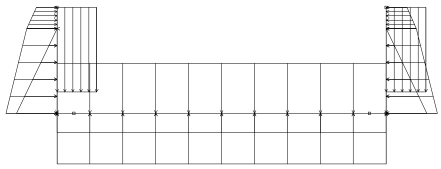
Vertical Force $\quad V = \quad \Sigma Vi = 38.099(kN)$

Horizontal Force $\quad H = \quad \Sigma Hi = -12.997(kN)$

Moment Mo $\quad = \quad \Sigma Mxi + \Sigma Myi = 74.294 + -6.442 = 67.852(kN.m)$

(3) Summary of Load

[1]Normal Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	1.000	9.600	9.600
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	1.000	9.600	9.600
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	3.900	12.000	12.000

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.250	5.000	7.250
Earth Pressure (Left)	Left Side Wall	Horizontal	0.250	1.000	7.250	12.350
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.250	-5.000	-7.250
Earth Pressure (Right)	Right Side Wall	Horizontal	0.250	1.000	-7.250	-12.350
Water Pressure (Left)	Left Side Wall	Horizontal	0.250	1.000	0.000	9.800
Water Pressure (Right)	Right Side Wall	Horizontal	0.250	1.000	0.000	-9.800

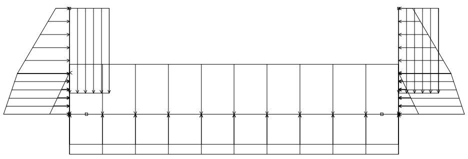
■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	3.900	-12.250	-12.250

■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	3.900	-4.673	-4.673

[2]Seismic Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	1.000	9.600	9.600
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	1.000	9.600	9.600
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	3.900	12.000	12.000

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Water Pressure (Left)	Left Side Wall	Horizontal	0.760	0.490	0.000	4.802
Water Pressure Right)	Right Side Wall	Horizontal	0.760	0.490	0.000	-4.802
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.770	3.347	12.626
Earth Pressure (Left)	Left Side Wall	Horizontal	0.770	0.480	12.626	15.904
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.770	-3.347	-12.626
Earth Pressure (Right)	Right Side Wall	Horizontal	0.770	0.480	-12.626	-15.904

■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	3.900	-7.252	-7.252

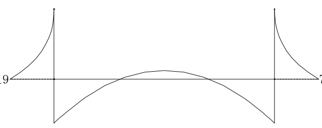
■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	3.900	-9.671	-9.671

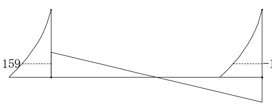
(4) Cross Section Force

[1]Normal Condition

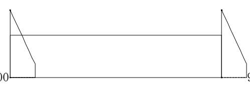
(a) Bending Moment



(b) Shear Force

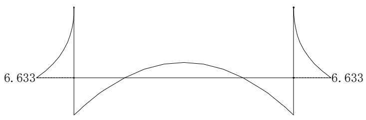


(c) Axial Force

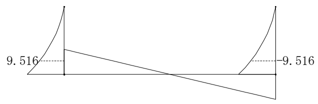


[2]Seismic Condition

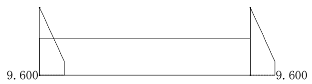
(a) Bending Moment



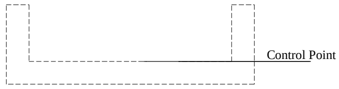
(b) Shear Force



(c) Axial Force



(5) Stress Calculation



1) Bending Stress

(a) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	9.0	D13	3.57	4.525
	2	——	—	——	———

Required Area of Reinforcement 1.549 (cm^2)

(b) Stress Calculation

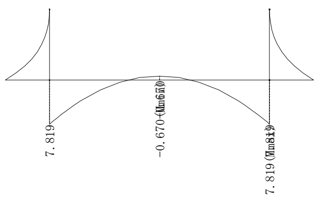
			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	7.819	6.633
Axial Force	N	kN	——	——
Shear Force	S	kN	11.159	9.516
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	400.0	400.0
Effective Width	d	mm	310.0	310.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	46.338	46.338
Stress Intensity (Concrete)	σ_c	N/mm^2	1.146	0.972
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	58.666	49.770
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm^2	0.036	0.031
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

1.3.3 Bottom Slab

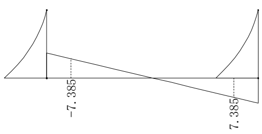
(1) Cross Section Force

[1]Normal Condition

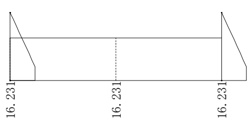
(a) Bending Moment



(b) Shear Force

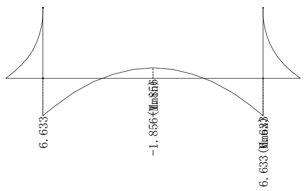


(c) Axial Force

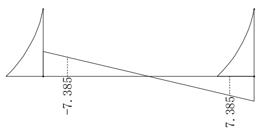


[2]Seismic Condition

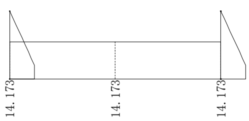
(a) Bending Moment



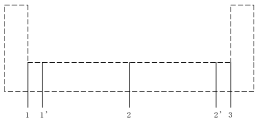
(b) Shear Force



(c) Axial Force



(2) Stress Calculation



1) Control point of Bending Stress

	1	2	3
Control point	0.200	1.950	3.700

2) Control Point of Shear Stress

	1'	2'
Control point	0.450	3.450

(3) Calculation of Bending Stress

(a) Control point 1

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	―	―	―	―
	2'	―	―	―	―
Outside	1	11.5	D13	3.57	4.525
	2	―	―	―	―

Required Area of Reinforcement 1.240 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	7.819	6.633
Axial Force	N	kN	―	―
Shear Force	S	kN	―	―
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	500.0	500.0
Effective Width	d	mm	385.0	385.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	52.063	52.063
Stress Intensity (Concrete)	σ_c	N/mm^2	0.817	0.693
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	47.004	39.876
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○

(b) Control point 2

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	9.0	D13	3.57	4.525
	2'	―	―	―	―
Outside	1	―	―	―	―
	2	―	―	―	―

Required Area of Reinforcement 0.267 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	-1.541	-2.727
Axial Force	N	kN	―	―
Shear Force	S	kN	―	―
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	500.0	500.0
Effective Width	d	mm	410.0	410.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	53.864	53.864
Stress Intensity (Concrete)	σ_c	N/mm^2	0.146	0.258
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	8.685	15.369
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○

(4) Calculation of Shear Stress

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	―	―
Axial Force	N	kN	―	―
Shear Force	S	kN	7.385	7.385
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	500.0	500.0
Bending Moment	d	mm	385.0	385.0
Stress Intensity by Shear Force	τ	N/mm^2	0.019	0.019
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

1.4. Stability Against Bearing

Stability against bearing of wing wall is evaluated by yielding displacement of ground. The yeilding displacement of ground is defined as allowable displacement for the ground acting as elastic body.

・Yeilding Displacement of Ground is determinded as follows;

Yeilding Displacement of Grou ≦ 5.0 cm

≦ 1.0% of Width of Foundation

	① Wdth of Foundation (cm)	② 1 % of ① (cm)				Allowable Relative Displacement (c m)
MSL3	430.0	4.3	<	5.0	cm	4.3

2) Vertical Displacement of Foundation UnderSide

Vertical displacement of foudation underside is calculated as follows;

$$\delta_V = \frac{1}{K_V} \cdot \frac{V}{A}$$

where;

δ_V: Vertilcal Displacement (m)

V: Vertical Load (kN)

A: Area of Foudation UnderSide 23.22 (m^2)

K_V: Vertical Coefficient of Subgrade Reactic Normal 5674.45 (kN/m^2)

Seismic 11348.90 (kN/m^2)

・Calculation of Vertical Load

<Normal Condition> Full Flow

Main Body	70.8 *x	5.4	＝	382.32	(kN)
Backfill	0		＝	0.00	(kN)
Water	34.79 *x	5.4	＝	187.87	(kN)
				570.19	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

<Seismic Condition> No Water Flow

Main Body	70.8 *x	5.4	＝	382.32	(kN)
Backfill	0		＝	0.00	(kN)
Water	0 *x	5.4	＝	0.00	(kN)
				382.32	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

Case	1	2
	Normal	Seismic
K_v (kN/m^2)	5674.45	11348.90
δ_V (cm)	0.433	0.145
δ_a (cm)	4.300	4.300
Evaluation	OK	OK

・Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvo : Coefficient of Vertical Reaction of soil (kN/m3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E0m obtained by various soil tests a

$$Kvo = \frac{1}{0.3} \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eom : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{ (Kv \cdot D) / (4 \cdot E \cdot I) \}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert $(kN \cdot m^2)$

E= 2.45E+07 kN/m^2

I= 0.4402 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3414.62 = 45528.3 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.15422

β・L= 0.833 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body $Bv = \sqrt{L \cdot D} = \sqrt{5.40 \times 4.30} = \boxed{4.819}$ m

Case2 Elastic body $Bv = \sqrt{D / \beta} = \sqrt{4.30 / 0.15422} = 5.280$ m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 45528.3 \times (4.819 / 0.3)^{-3/4} = 5674 \quad (kN/m^3)$

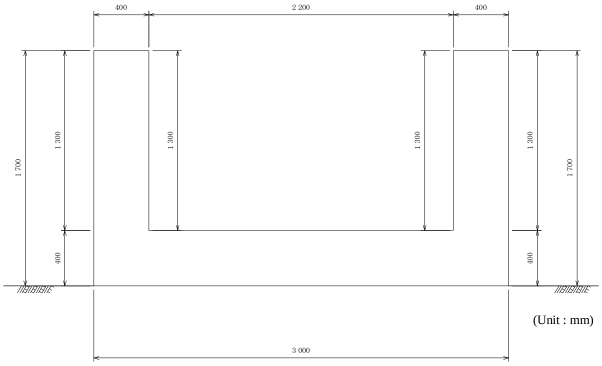
Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L			(kN/m²)
B ;	Loading Width = Width of Wing Wall	B=	4.300	m
L ;	Loading Length = Length of a span	L=	5.400	m
h_n ;	Depth for Calculation	hn=	12.900	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each Layer (kN/m²)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m²)	(m)	(m)	(m)	(m)	(m)	(kN/m²)	(°)			
1	3414.62	4.300	5.400	12.900	4.070	4.070	2830	30	0.1721	0.112	0.0000397
2					2.000	6.070	14700			0.022	0.0000015
3					3.500	9.570	2840			0.024	0.0000083
4					3.330	12.900	16800			0.013	0.0000008
5											
6											
7											
8											
9											
10											
				Total	12.900					Total	0.0000504

2 MSL4 Wing Wall (U-shaped)



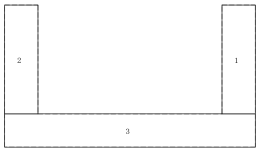
2.1 Water Level

[1]Normal Condition Left Fl = 1.330 m, Inside Fi = 0.000 m, Right Fr = 1.330 m

[2]Seismic Left Fl = 0.580 m, Inside Fi = 0.000 m, Right Fr = 0.580 m

2.2 Stability Analysis

(1) Center of Gravity



	Width × Height × Depth	Vi(m^3)	Center of gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.400× 1.300× 1.000	0.520	2.800	1.050	1.456	0.546	
2	0.400× 1.300× 1.000	0.520	0.200	1.050	0.104	0.546	
3	3.000× 0.400× 1.000	1.200	1.500	0.200	1.800	0.240	
Σ		2.240	──	──	3.360	1.332	

Center of Gravity XG ＝ Σ (Vi・Xi) ／ΣVi ＝ 3.360／ 2.240 ＝ 1.500 (m)

YG ＝ Σ(Vi・Yi)／ΣVi ＝ 1.332／ 2.240 ＝ 0.595 (m)

(2) Weight of Body and Uplift

(a) Weight of Body

[1]Normal Condition

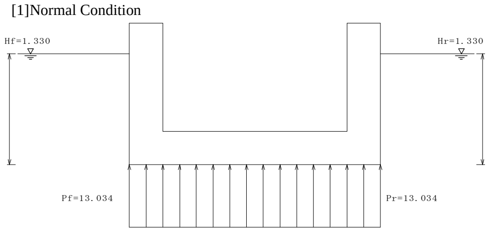
	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 1.040 ＝ 24.960	1.500
Bottom Slab	24.000 × 1.200 ＝ 28.800	1.500

[2]Seismic Condition

	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 1.040 ＝ 24.960	1.500
Bottom Slab	24.000 × 1.200 ＝ 28.800	1.500

(b) Uplift

[1]Normal Condition



Water level Hf ＝ 1.330 (m)

Strength of Water Pressure Pf ＝ 13.034 (kN/m^2)

Buoyancy Effecting on Body

$$U = \frac{Pf+Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 39.102 \ (kN)$$

Position (From the Left Edge)

$$X = \frac{Pf+2 \cdot Pr}{3 \cdot (Pf+Pr)} \cdot Bj = 1.500 \ (m)$$

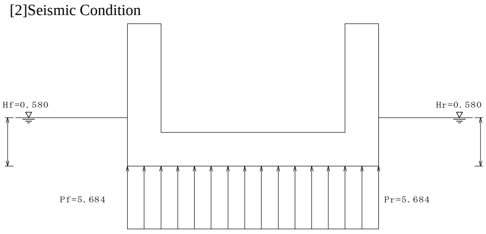
Where,

Bj : Width of Footing Bj ＝ 3.000 (m)

Bc : Length of Footing Bc ＝ 1.000 (m)

λ : Reduction Coefficient of buoyancy λ ＝ 1.000

[2]Seismic Condition



Water level Hf ＝ 0.580 (m)

Strength of Water Pressure Pf ＝ 5.684 (kN/m^2)

Buoyancy Effecting on Body

$$U = \frac{Pf+Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 17.052 \ (kN)$$

Position (From the Left Edge)

$$X = \frac{Pf+2 \cdot Pr}{3 \cdot (Pf+Pr)} \cdot Bj = 1.500 \ (m)$$

Where,

Bj : Width of Footing Bj ＝ 3.000 (m)

Bc : Length of Footing Bc ＝ 1.000 (m)

λ : Reduction Coefficient of buoyancy λ ＝ 1.000

(3) Stability against Buoyancy Force

$$F_s = \frac{\Sigma Vu + \alpha \cdot Pv}{U}$$

Where,
ΣVu : Total Vertical Force (kN)
α : Efficiency Rate of Earth Pressure α＝0.000
Pv : Vertical Element of Earth Pressure (kN)
U : Buoyancy (kN)

	ΣVu (kN)	Pv (kN)	U (kN)	Safty Factor f_s		Required FS f_{sa}
Normal Condition	53.760	0.000	39.102	1.375	≧	1.330
Seismic Condition	53.760	0.000	17.052	3.153	≧	1.330

2.3 Structural Calculation

2.3.1 Calculation of Earth Pressure

[1]Normal Condition

■Earth Pressure at Rest

Height of Imaginary Back Side H ＝ 1.500 m
Height above Water Surface H1 ＝ 0.370 m
Height under Water Surface H2 ＝ 1.130 m
Angle Between Back side Surface of Wall and Vertical Plane α ＝ 0.000 °
Unit Weight of Back fill Ma γs ＝ 18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	5.000	5.000	0.000
p2	8.330	8.330	0.000
p3	14.093	14.093	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1+p2) \cdot H1 = \frac{1}{2} \times (5.000 + 8.330) \times 0.370 = 2.466 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p2+p3) \cdot H2 = \frac{1}{2} \times (8.330 + 14.093) \times 1.130 = 12.669 \text{ kN}$$

Total Earth Pressure
P_0 ＝ P ＝ P1＋P2 ＝ 2.466＋12.669 ＝ 15.135 kN

[2]Seismic Condition

■Earth Pressure at Rest

Height of Imaginary Back Side H ＝ 1.500 m
Height above Water Surface H1 ＝ 1.120 m
Height under Water Surface H2 ＝ 0.380 m
Angle Between Back side Surface of Wall and Vertical Plane α ＝ 0.000 °
Unit Weight of Back fill γs ＝ 18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.500	2.500	0.000
p2	12.580	12.580	0.000
p3	14.518	14.518	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1+p2) \cdot H1 = \frac{1}{2} \times (2.500 + 12.580) \times 1.120 = 8.445 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p3+p4) \cdot H2 = \frac{1}{2} \times (12.580 + 14.518) \times 0.380 = 5.149 \text{ kN}$$

Total Earth Pressure
P_0 ＝ P ＝ P1＋P2 ＝ 8.445＋5.149 ＝ 13.593 kN

■Active Earth Pressure in Normal Condition（Coulomb）

internal Friction Angle of Soil φ ＝ 30.000 °
Angle Between Ground Surface and Horizontal Plane β ＝ 0.000 °
Angle of Wall Friction δ ＝ 10.000 °

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha \cdot \cos(\alpha + \delta) \cdot \left[1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\alpha + \delta) \cdot \cos(\alpha - \beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00° - 0.000°)}{\cos^2 0.000° \cdot \cos(0.000° + 10.000°)} \times \frac{1}{\left[1 + \sqrt{\frac{\sin(30.00° + 10.000°) \cdot \sin(30.00° - 0.000°)}{\cos(0.000° + 10.000°) \cdot \cos(0.000° - 0.000°)}}\right]^2}$$

＝ 0.3085

Strength of Earth Pressure
Horizontal ph ＝ p・cos(α＋δ)
Vertical pv ＝ p・sin(α＋δ)

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	1.542	1.519	0.268
p2	7.761	7.643	1.348
p3	8.957	8.821	1.555

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1+p2) \cdot H1 = \frac{1}{2} \times (1.542 + 7.761) \times 1.120 = 5.210 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p2+p3) \cdot H2 = \frac{1}{2} \times (7.761 + 8.957) \times 0.380 = 3.176 \text{ kN}$$

Total Earth Pressure
P ＝ P1＋P2 ＝ 5.210＋3.176 ＝ 8.386 kN

Horizontal Elements of Earth Pressure is as follows.
P_H ＝ P・cos(α＋δ) ＝ 8.386×cos(0.000°＋10.000°) ＝ 8.259 kN

■Active Earth Pressure in Seismic Condition （Mononobe-Okabe）

Angle of Wall Friction δ ＝ 0.000 °

Seismic Compound Angle above Water Surface
θ ＝ $\tan^{-1}$kh ＝ $\tan^{-1}$0.20 ＝ 11.310 °

Active Earth Pressure above Water Surface

$$K = \frac{\cos^2(\phi - \alpha - \theta)}{\cos\theta \cdot \cos^2\alpha \cdot \cos(\alpha + \delta + \theta) \cdot \left[1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta - \theta)}{\cos(\alpha + \delta + \theta) \cdot \cos(\alpha - \beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00° - 0.000° - 11.310°)}{\cos 11.310° \cdot \cos^2 0.000° \cdot \cos(0.000° + 0.000° + 11.310°)} \times \frac{1}{\left[1 + \sqrt{\frac{\sin(30.00° + 0.000°) \cdot \sin(30.00° - 0.000° - 11.310°)}{\cos(0.000° + 0.000° + 11.310°) \cdot \cos(0.000° - 0.000°)}}\right]^2}$$

＝ 0.4733

In case φ－β－θ＜0, sin(φ－β－θ) ＝ 0

11.342

Strength of Earth Pressure
Horizontal ph ＝ p・cos(α＋δ)
Vertical pv ＝ p・sin(α＋δ)

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.366	2.366	0.000
p2	11.907	11.907	0.000
p3	11.907	11.907	0.000
p4	13.742	13.742	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1+p2) \cdot H1 = \frac{1}{2} \times (2.366 + 11.907) \times 1.120 = 7.993 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p3+p4) \cdot H2 = \frac{1}{2} \times (11.907 + 13.742) \times 0.380 = 4.873 \text{ kN}$$

Total Earth Pressure
P ＝ P1＋P2 ＝ 7.993＋4.873 ＝ 12.867 kN

Horizontal Elements of Earth Pressure is as follows.
P_{HE} ＝ P・cos(α＋δ) ＝ 12.867×cos(0.000°＋ 0.000°) ＝ 12.867 kN

■Summary of Earth Pressure

Strength of Earth Pressure
$p_{0E} = p_0 + (p_{HE} - p_H)$
Where,
p_{0E} : Strength of Earth Pressure in Seismic Condition
p_0 : Strength of Earth Pressure in Normal Condition (Horizontal)
p_{HE} : Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition
p_H : Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition

	Normal Earth Pressure at Rest p_O (kN/m^2)	Seismic Active Earth Pressure p_{HE} (kN/m^2)	Normal Active Earth Pressure p_H (kN/m^2)	Seismic Pressure at Res p_{OE} (kN/m^2)
Top	2.500	2.366	1.519	3.347
Water Surface	12.580	11.907	7.643	16.844
Bottom	14.518	13.742	8.821	19.439

Total Earth Pressure
$P_{0E} = P_0 + (P_{HE} - P_H)$
= 13.593 + (12.867 - 8.259)
= 18.201 kN
Where,
P_{0E} : Total Earth Pressure in Seismic Condition (kN)
P_0 : Total Earth Pressure in Normal Condition（Horizontal）(kN)
P_{HE} : Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)
P_H : Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)

2.3.2 Side wall

(1) Water Pressure

pi ＝ hi・Gw

Where,

pi：Strength of Water Pressure at the Edge of Bottom Slab (kN/m^2)

hi：Water Depth (m)

Gw：Unit Weight of Water (kN/m^3), Gw ＝ 9.800

[1]Normal Condition

Water Depth hi (m)	Strength pi (kN/m^2)
1.130	11.074

[2]Seismic Condition

Water Depth hi (m)	Strength pi (kN/m^2)
0.380	3.724

(2) Subgrade Reaction

[1]Normal Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	1.300	9.600	9.600	12.480	0.000	0.000
Weight of Right Side wall	Right Side Wall	1.300	9.600	9.600	12.480	2.600	32.448
Weight of Bottom Slab	Bottom Slab	2.600	9.600	9.600	24.960	1.300	32.448
Buoyancy	Bottom Slab	2.600	-13.034	-13.034	-33.888	1.300	-44.055
Total	———	———	———	———	16.032	———	20.841

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Left Side Wall	0.370	5.000	8.330	2.466	1.300	3.205
Earth Pressure (Left)	Left Side Wall	1.130	8.330	14.093	12.669	0.517	6.545
Earth Pressure (Right)	Right Side Wall	0.370	-5.000	-8.330	-2.466	1.300	-3.205
Earth Pressure (Right)	Right Side Wall	1.130	-8.330	-14.093	-12.669	0.517	-6.545
Water Pressure (Left)	Left Side Wall	1.130	0.000	11.074	6.257	0.377	2.357
Water Pressure (Right)	Right Side Wall	1.130	0.000	-11.074	-6.257	0.377	-2.357
Total	———	———	———	———	0.000	———	0.000

Vertical Force V ＝ ΣVi ＝ 16.032(kN)

[2]Seismic Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	1.300	9.600	9.600	12.480	0.000	0.000
Weight of Right Side wall	Right Side Wall	1.300	9.600	9.600	12.480	2.600	32.448
Weight of Bottom Slab	Bottom Slab	2.600	9.600	9.600	24.960	1.300	32.448
Buoyancy	Bottom Slab	2.600	-5.684	-5.684	-14.778	1.300	-19.212
Total	———	———	———	———	35.142	———	45.684

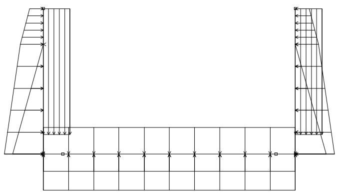
■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Right Side Wall	1.120	-3.347	-16.844	-11.307	0.815	-9.218
Earth Pressure (Left)	Right Side Wall	0.380	-16.844	-19.439	-6.894	0.185	-1.279
Earth Pressure (Right)	Left Side Wall	1.120	-3.347	-16.844	-11.307	0.815	-9.218
Earth Pressure (Right)	Right Side Wall	0.380	-16.844	-19.439	-6.894	0.185	-1.279
Water Pressure (Left)	Left Side Wall	0.380	0.000	3.724	0.708	0.127	0.090
Water Pressure (Right)	Left Side Wall	0.380	0.000	-3.724	-0.708	0.127	-0.090
Total	———	———	———	———	-16.636	———	-9.714

Vertical Force V ＝ ΣVi ＝ 35.142(kN)

(3) Summary of Load

[1]Normal Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	1.300	9.600	9.600
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	1.300	9.600	9.600
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	2.600	9.600	9.600

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.370	5.000	8.330
Earth Pressure (Left)	Left Side Wall	Horizontal	0.370	1.130	8.330	14.093
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.370	-5.000	-8.330
Earth Pressure (Right)	Right Side Wall	Horizontal	0.370	1.130	-8.330	-14.093
Water Pressure (Left)	Left Side Wall	Horizontal	0.370	1.130	0.000	11.074
Water Pressure (Right)	Right Side Wall	Horizontal	0.370	1.130	0.000	-11.074

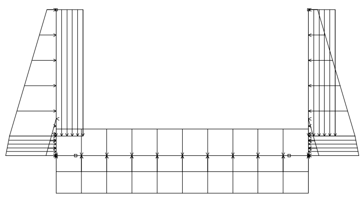
■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	2.600	-13.034	-13.034

■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	2.600	-6.166	-6.166

[2]Seismic Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	0.000	1.300	9.600	9.600	0.000
Weight of Right Side wall	Right Side Wall	0.000	1.300	9.600	9.600	0.000
Weight of Bottom Slab	Bottom Slab	0.000	2.600	9.600	9.600	0.000

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Water Pressure (Left)	Left Side Wall	Horizontal	1.120	0.380	0.000	3.724
Water Pressure (Right)	Right Side Wall	Horizontal	1.120	0.380	0.000	-3.724
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	1.300	3.347	16.844
Earth Pressure (Left)	Left Side Wall	Horizontal	1.300	0.200	16.844	18.210
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	1.300	-3.347	-16.844
Earth Pressure (Right)	Right Side Wall	Horizontal	1.300	0.200	-16.844	-18.210

■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	2.600	-5.684	-5.684

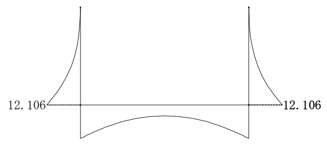
■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	2.600	-13.516	-13.516

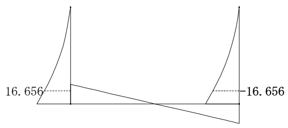
(4) Cross Section Force

[1]Normal Condition

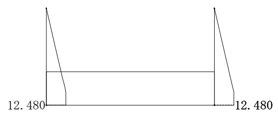
(a) Bending Moment



(b) Shear Force

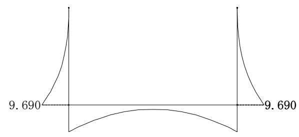


(c) Axial Force

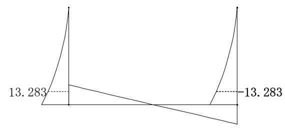


[2]Seismic Condition

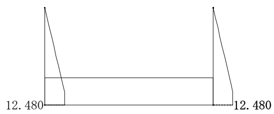
(a) Bending Moment



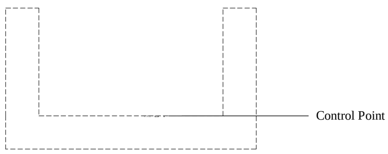
(b) Shear Force



(c) Axial Force



(5) Stress Calculation



1) Bending Stress

(a) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	9.0	D13	3.57	4.525
	2	——	—	——	———

Required Area of Reinforcement 2.415 (cm^2)

(b) Stress Calculation

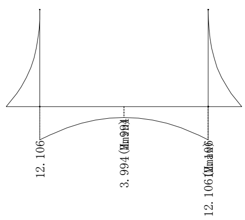
			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	12.106	9.690
Axial Force	N	kN	——	——
Shear Force	S	kN	16.656	13.283
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	400.0	400.0
Effective Width	d	mm	310.0	310.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	46.338	46.338
Stress Intensity (Concrete)	σ_c	N/mm^2	1.774	1.420
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	90.832	72.704
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm^2	0.054	0.043
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

2.3.3 Bottom Slab

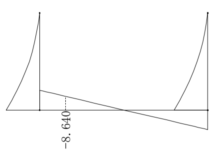
(1) Cross Section Force

[1]Normal Condition

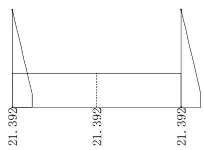
(a) Bending Moment



(b) Shear Force

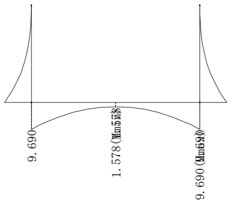


(c) Axial Force

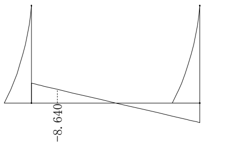


[2]Seismic Condition

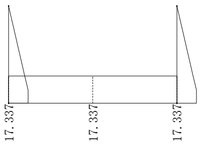
(a) Bending Moment



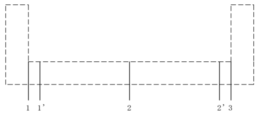
(b) Shear Force



(c) Axial Force



(2) Stress Calculation



1) Control point of Bending Stress

	1	2
Control point	0.200	1.300

2) Control Point of Shear Stress

	1'
Control point	0.400

(3) Calculation of Bending Stress

(a) Control point 1

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	11.5	D12	4.00	4.525
	2	——	—	——	———

Required Area of Reinforcement 2.635 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	12.106	9.690
Axial Force	N	kN	——	——
Shear Force	S	kN	——	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	400.0	400.0
Effective Width	d	mm	285.0	285.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	44.287	44.287
Stress Intensity (Concrete)	σ_c	N/mm^2	2.024	1.620
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	98.999	79.242
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	140.000	210.000
Evaluation of Compression			○	○

(b) Control point 2

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	11.5	D12	4.00	4.525
	2	——	—	——	———

Required Area of Reinforcement 0.855 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	3.994	1.578
Axial Force	N	kN	——	——
Shear Force	S	kN	——	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	400.0	400.0
Effective Width	d	mm	285.0	285.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	44.287	44.287
Stress Intensity (Concrete)	σ_c	N/mm^2	0.668	0.264
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	32.664	12.907
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	140.000	210.000
Evaluation of Compression			○	○

(4) Calculation of Shear Stress

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	——	——
Axial Force	N	kN	——	——
Shear Force	S	kN	8.640	8.640
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	400.0	400.0
Bending Moment	d	mm	285.0	285.0
Stress Intensity by Shear Force	τ	N/mm^2	0.030	0.030
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

2.4. Stability Against Bearing

Stability against bearing of wing wall is evaluated by yielding displacement of ground. The yeilding displacement of ground is defined as allowable displacement for the ground acting as elastic body.

・Yeilding Displacement of Ground is determinded as follows;

Yeilding Displacement of Grou ≦ 5.0 cm

≦ 1.0% of Width of Foundation

	① Wdth of Foundation (cm)	② 1 % of ① (cm)				Allowable Relative Displacement (c m)
MSL4	300.0	3.0	<	5.0	cm	3.0

2) Vertical Displacement of Foundation UnderSide

Vertical displacement of foudation underside is calculated as follows;

$$\delta_V = \frac{1}{K_V} \cdot \frac{V}{A}$$

where;

δ_V:	Vertilcal Displacement			(m)
V:	Vertical Load			(kN)
A:	Area of Foudation UnderSide		18	(m^2)
K_V:	Vertical Coefficient of Subgrade Reactio	Normal	4115.90	(kN/m^2)
		Seismic	8231.79	(kN/m^2)

・Calculation of Vertical Load

<Normal Condition> Full Flow

Main Body	53.76	*x	6	＝	322.56	(kN)
Backfill	0.00			＝	0.00	(kN)
Water	28.03	*x	6	＝	168.18	(kN)
					490.74	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

<Seismic Condition> No Water Flow

Main Body	53.76	*x	6	＝	322.56	(kN)
Backfill	0.00			＝	0.00	(kN)
Water	0.00	*x	6	＝	0.00	(kN)
					322.56	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

Case	1	2
	Normal	Seismic
K_v (kN/m^2)	4115.90	8231.79
δ_V (cm)	0.662	0.218
δ_a (cm)	3.000	3.000
Evaluation	OK	OK

・Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvc : Coefficient of Vertical Reaction of soil (kN/m3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E0m obtained by various soil tests a

$$Kvo = \frac{1}{0.3} \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eon : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{ (Kv \cdot D) / (4 \cdot E \cdot I) \}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 0.5589 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 2251.19 = 30015.9 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.12253

β・L= 0.735 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body	$Bv=\sqrt{L \cdot D}$	=	$\sqrt{6.00 \times 3.00}$			=	$\boxed{4.243}$	m
Case2 Elastic body	$Bv=\sqrt{D/\beta}$	=	$\sqrt{3.00 / 0.12253}$			=	4.948	m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 30015.9 \times (4.243 / 0.3)^{-3/4} = 4116$ (kN/m^3)

Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\,L}{(L+2h_n \cdot \tan\theta)\,B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L		(kN/m^2)	
B ;	Loading Width = Width of Wing Wall	B=	3.000	m
L ;	Loading Length = Length of a span	L=	6.000	m
h_n ;	Depth for Calculation	hn=	9.000	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each Layer (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom	B	L	h_n	Tickness of Layer	h i	Ei	θ	f n	f i	1 / E i × f i
	(kN/m^2)	(m)	(m)	(m)	(m)	(m)	(kN/m^2)	(°)			
1	2251.19	3.000	6.000	9.000	4.070	4.070	1880	30	0.4910	0.364	0.0001937
2					2.000	6.070	4200			0.067	0.0000159
3					2.930	9.000	7050			0.060	0.0000085
4											
5											
6											
7											
8											
9											
10											
				Total	9.000					Total	0.0002181

1 MSR3 Wing Wall (U-shaped)

1.1 Water Level

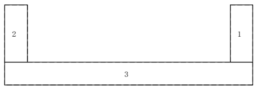
[1]Normal Condition Left Fl = 0.790 m, Inside Fi = 0.000 m, Right Fr = 0.790 m

(Unit : mm)

[2]Seismic Left Fl = 0.650 m, Inside Fi = 0.000 m, Right Fr = 0.650 m

1.2 Stability Analysis

(1) Center of Gravity



	Width × Height × Depth	Vi(m^3)	Center of gravity (m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1	0.300× 0.500× 1.000	0.150	3.050	0.600	0.458	0.090	
2	0.300× 0.500× 1.000	0.150	0.150	0.600	0.023	0.090	
3	3.200× 0.350× 1.000	1.120	1.600	0.175	1.792	0.196	
Σ		1.420	——	——	2.272	0.376	

Center of Gravity XG ＝ Σ (Vi・Xi) ／ΣVi ＝ 2.272／ 1.420 ＝ 1.600 (m)
YG ＝ Σ(Vi・Yi)／ΣVi ＝ 0.376／ 1.420 ＝ 0.265 (m)

(2) Weight of Body and Uplift

(a) Weight of Body

[1]Normal Condition

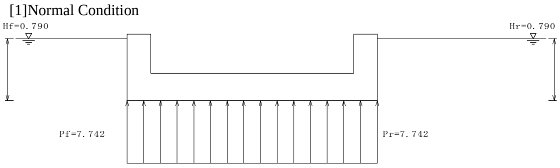
	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 0.300 ＝ 7.200	1.600
Bottom Slab	24.000 × 1.120 ＝ 26.880	1.600

[2]Seismic Condition

	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Side Wall	24.000 × 0.300 ＝ 7.200	1.600
Bottom Slab	24.000 × 1.120 ＝ 26.880	1.600

(b) Uplift

[1]Normal Condition



Water level Hf ＝ 0.790 (m)
Strength of Water Pressure Pf ＝ 7.742 (kN/m^2)
Buoyancy Effecting on Body

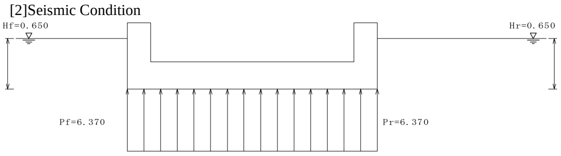
$$U = \frac{Pf + Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 24.774 \text{ (kN)}$$

Position (From the Left Edge)

$$X = \frac{Pf + 2 \cdot Pr}{3 \cdot (Pf + Pr)} \cdot Bj = 1.600 \text{ (m)}$$

Where,
Bj : Width of Footing Bj ＝ 3.200 (m)
Bc : Length of Footing Bc ＝ 1.000 (m)
λ : Reduction Coefficient of buoyancy λ ＝ 1.000

[2]Seismic Condition



Water level Hf ＝ 0.650 (m)
Strength of Water Pressure Pf ＝ 6.370 (kN/m^2)

Buoyancy Effecting on Body

$$U = \frac{Pf + Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 20.384 \text{ (kN)}$$

Position (From the Left Edge)

$$X = \frac{Pf + 2 \cdot Pr}{3 \cdot (Pf + Pr)} \cdot Bj = 1.600 \text{ (m)}$$

Where,
Bj : Width of Footing Bj ＝ 3.200 (m)
Bc : Length of Footing Bc ＝ 1.000 (m)
λ : Reduction Coefficient of buoyancy λ ＝ 1.000

(3) Stability against Buoyancy Force

$$F_s = \frac{\Sigma Vu + \alpha \cdot Pv}{U}$$

Where,
ΣVu : Total Vertical Force (kN)
α : Efficiency Rate of Earth Pressure α＝0.000
Pv : Vertical Element of Earth Pressure (kN)
U : Buoyancy (kN)

	ΣVu (kN)	Pv (kN)	U (kN)	Safty Factor f_s	Required FS f_{sa}
Normal Condition	34.080	0.000	24.774	1.376 ≧	1.330
Seismic Condition	34.080	0.000	20.384	1.672 ≧	1.330

1.3 Structural Calculation

1.3.1 Calculation of Earth Pressure

[1]Normal Condition

■Earth Pressure at Rest

Height of Imaginary Back Side H ＝ 0.675 m
Height above Water Surface H1 ＝ 0.060 m
Height under Water Surface H2 ＝ 0.615 m
Angle Between Back side Surface of Wall and Vertical Plane α ＝ 0.000 °
Unit Weight of Back fill Ma γs ＝ 18.000 kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	5.000	5.000	0.000
p2	5.540	5.540	0.000
p3	8.677	8.677	0.000

Earth Pressure above Water Surface

$$P1 = \frac{1}{2} \cdot (p1 + p2) \cdot H1 = \frac{1}{2} \times (5.000 + 5.540) \times 0.060 = 0.316 \text{ kN}$$

Earth Pressure under Water Surface

$$P2 = \frac{1}{2} \cdot (p2 + p3) \cdot H2 = \frac{1}{2} \times (5.540 + 8.677) \times 0.615 = 4.372 \text{ kN}$$

Total Earth Pressure
P_0 ＝ P ＝ P1＋P2 ＝0.316＋4.372 ＝ 4.688 kN

[2]Seismic Condition

■Earth Pressure at Rest

Height of Imaginary Back Side $H = 0.675$ m
Height above Water Surface $H1 = 0.200$ m
Height under Water Surface $H2 = 0.475$ m
Angle Between Back side Surface of Wall and Vertical Plane $\alpha = 0.000$ °
Unit Weight of Back fill $\gamma s = 18.000$ kN/m^3

Strength of Earth Pressure

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.500	2.500	0.000
p2	4.300	4.300	0.000
p3	6.722	6.722	0.000

Earth Pressure above Water Surface

$$P1= \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 2.500+\ 4.300)\times 0.200 = \ 0.680\ kN$$

Earth Pressure under Water Surface

$$P2= \frac{1}{2}\cdot(p3+p4)\cdot H2 = \frac{1}{2}\times(\ 4.300+\ 6.722)\times 0.475 = \ 2.618\ kN$$

Total Earth Pressure

$P_0 = P = P1+P2 = 0.680+2.618 = 3.298$ kN

■Active Earth Pressure in Normal Condition（Coulomb）

internal Friction Angle of Soil $\varphi = 30.000$ °
Angle Between Ground Surface and Horizontal Plane $\beta = 0.000$ °
Angle of Wall Friction $\delta = 10.000$ °

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ )}{\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +10.000^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +10.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ )}{\cos(0.000^\circ\ +10.000^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.3085$

Strength of Earth Pressure

Horizontal $ph = p\cdot\cos(\alpha+\delta)$
Vertical $pv = p\cdot\sin(\alpha+\delta)$

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	1.542	1.519	0.268
p2	2.653	2.613	0.461
p3	4.147	4.084	0.720

Earth Pressure above Water Surface

$$P1= \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 1.542+\ 2.653)\times 0.200 = \ 0.420\ kN$$

Earth Pressure under Water Surface

$$P2= \frac{1}{2}\cdot(p2+p3)\cdot H2 = \frac{1}{2}\times(\ 2.653+\ 4.147)\times 0.475 = \ 1.615\ kN$$

Total Earth Pressure

$P = P1+P2 = 0.420+1.615 = 2.035$ kN

Horizontal Elements of Earth Pressure is as follows.

$P_H = P\cdot\cos(\alpha+\delta) = 2.035\times\cos(0.000°+10.000°) = 2.004$ kN

■Active Earth Pressure in Seismic Condition （Mononobe-Okabe）

Angle of Wall Friction $\delta = 0.000$ °

Seismic Compound Angle above Water Surface

$\theta = \tan^{-1}kh = \tan^{-1}0.20 = 11.310$ °

Active Earth Pressure above Water Surface

$$K = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos 11.310^\circ\ \cdot\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +0.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.4733$

In case $\varphi-\beta-\theta<0$, $\sin(\varphi-\beta-\theta) = 0$

Strength of Earth Pressure

Horizontal $ph = p\cdot\cos(\alpha+\delta)$
Vertical $pv = p\cdot\sin(\alpha+\delta)$

	Strength of Earth Pressure (kN/m^2)	Horizontal Element (kN/m^2)	Vertical Element (kN/m^2)
p1	2.366	2.366	0.000
p2	4.070	4.070	0.000
p3	4.070	4.070	0.000
p4	6.363	6.363	0.000

Earth Pressure above Water Surface

$$P1= \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 2.366+\ 4.070)\times 0.200 = \ 0.644\ kN$$

Earth Pressure under Water Surface

$$P2= \frac{1}{2}\cdot(p3+p4)\cdot H2 = \frac{1}{2}\times(\ 4.070+\ 6.363)\times 0.475 = \ 2.478\ kN$$

Total Earth Pressure

$P = P1+P2 = 0.644+2.478 = 3.122$ kN

Horizontal Elements of Earth Pressure is as follows.

$P_{HE} = P\cdot\cos(\alpha+\delta) = 3.122\times\cos(0.000°+0.000°) = 3.122$ kN

■Summary of Earth Pressure

Strength of Earth Pressure

$p_{0E} = p_0 + (p_{HE} - p_H)$

Where,

p_{0E} : Strength of Earth Pressure in Seismic Condition
p_0 : Strength of Earth Pressure in Normal Condition (Horizontal)
p_{HE} : Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition
p_H : Horizontal Element of Strength of Earth Pressure in Seismic and Active Condition

	Normal Earth Pressure at Rest p_O (kN/m^2)	Seismic Active Earth Pressure p_{HE} (kN/m^2)	Normal Active Earth Pressure p_H (kN/m^2)	Seismic Pressure at Res p_{OE} (kN/m^2)
Top	2.500	2.366	1.519	3.347
Water Surface	4.300	4.070	2.613	5.758
Bottom	6.722	6.363	4.084	9.001

Total Earth Pressure

$P_{0E} = P_0 + (P_{HE} - P_H)$
$= 3.298 + (3.122 - 2.004)$
$= 4.416$ kN

Where,

P_{0E} : Total Earth Pressure in Seismic Condition (kN)
P_0 : Total Earth Pressure in Normal Condition（Horizontal）(kN)
P_{HE} : Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)
P_H : Horizontal Element of Total Earth Pressure in Seismic and Active Condition (kN)

1.3.2 Side wall

(1) Water Pressure

$pi = hi\cdot Gw$

Where,

pi : Strength of Water Pressure at the Edge of Bottom Slab (kN/m^2)
hi : Water Depth (m)
Gw : Unit Weight of Water (kN/m^3), $Gw = 9.800$

[1]Normal Condition

Water Depth hi (m)	Strength pi (kN/m^2)
0.615	6.027

[2]Seismic Condition

Water Depth hi (m)	Strength pi (kN/m^2)
0.475	4.655

(2) Subgrade Reaction

[1]Normal Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	0.500	7.200	7.200	3.600	0.000	0.000
Weight of Right Side wall	Right Side Wall	0.500	7.200	7.200	3.600	2.900	10.440
Weight of Bottom Slab	Bottom Slab	2.900	8.400	8.400	24.360	1.450	35.322
Buoyancy	Bottom Slab	2.900	-7.742	-7.742	-22.452	1.450	-32.555
Total	———	———	———	———	9.108	———	13.207

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Left Side Wall	0.060	5.000	5.540	0.316	0.644	0.204
Earth Pressure (Left)	Left Side Wall	0.615	5.540	8.677	4.372	0.285	1.245
Earth Pressure (Right)	Right Side Wall	0.060	-5.000	-5.540	-0.316	0.644	-0.204
Earth Pressure (Right)	Right Side Wall	0.615	-5.540	-8.677	-4.372	0.285	-1.245
Water Pressure (Left)	Left Side Wall	0.615	0.000	6.027	1.853	0.205	0.380
Water Pressure (Right)	Right Side Wall	0.615	0.000	-6.027	-1.853	0.205	-0.380
Total	———	———	———	———	0.000	———	0.000

Vertical Force $V = \Sigma Vi = 9.108$ (kN)
Horizontal Force $H = \Sigma Hi = 0.000$ (kN)

Moment Mo = ΣMxi＋ΣMyi ＝13.207 ＋0.000 ＝13.207(kN.m)

[2]Seismic Condition

■Summary of Vertical Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Weight of Left Side wall	Left Side Wall	0.500	7.200	7.200	3.600	0.000	0.000
Weight of Right Side wall	Right Side Wall	0.500	7.200	7.200	3.600	2.900	10.440
Weight of Bottom Slab	Bottom Slab	2.900	8.400	8.400	24.360	1.450	35.322
Buoyancy	Bottom Slab	2.900	-6.370	-6.370	-18.473	1.450	-26.786
Total	————	———	———	———	13.087	———	18.976

■Summary of Horizontal Force

Load	Member	Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)	Total Load Vi(kN)	Position (m)	Moment Mxi(kN.m)
Earth Pressure (Left)	Right Side Wall	0.200	-3.347	-5.758	-0.910	0.566	-0.516
Earth Pressure (Left)	Right Side Wall	0.475	-5.758	-9.001	-3.505	0.220	-0.772
Earth Pressure (Right)	Left Side Wall	0.200	-3.347	-5.758	-0.910	0.566	-0.516
Earth Pressure (Right)	Right Side Wall	0.475	-5.758	-9.001	-3.505	0.220	-0.772
Water Pressure (Left)	Left Side Wall	0.475	0.000	4.655	1.106	0.158	0.175
Water Pressure (Right)	Left Side Wall	0.475	0.000	-4.655	-1.106	0.158	-0.175
Total	————	———	———	———	-4.416	———	-1.287

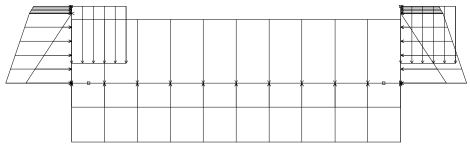
Vertical Force V ＝ ΣVi ＝13.087(kN)

Horizontal Force H ＝ ΣHi ＝-4.416(kN)

Moment Mo ＝ ΣMxi＋ΣMyi ＝ 18.976 ＋-1.287 ＝ 17.689(kN.m)

Summary of Load

[1]Normal Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	0.500	7.200	7.200
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	0.500	7.200	7.200
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	2.900	8.400	8.400

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.060	5.000	5.540
Earth Pressure (Left)	Left Side Wall	Horizontal	0.060	0.615	5.540	8.677
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.060	-5.000	-5.540
Earth Pressure (Right)	Right Side Wall	Horizontal	0.060	0.615	-5.540	-8.677
Water Pressure (Left)	Left Side Wall	Horizontal	0.060	0.615	0.000	6.027
Water Pressure (Right)	Right Side Wall	Horizontal	0.060	0.615	0.000	-6.027

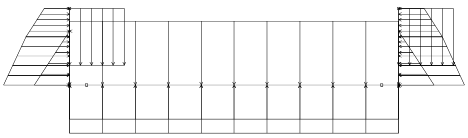
■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	2.900	-7.742	-7.742

■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	2.900	-3.141	-3.141

[2]Seismic Condition



■Weight of Body

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Weight of Left Side wall	Left Side Wall	Vertical	0.000	0.500	7.200	7.200
Weight of Right Side wall	Right Side Wall	Vertical l	0.000	0.500	7.200	7.200
Weight of Bottom Slab	Bottom Slab	Vertical	0.000	2.900	8.400	8.400

■Earth Pressure and Water Pressure

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Water Pressure (Left)	Left Side Wall	Horizontal	0.200	0.475	0.000	4.655
Water Pressure Right)	Right Side Wall	Horizontal	0.200	0.475	0.000	-4.655
Earth Pressure (Left)	Left Side Wall	Horizontal	0.000	0.250	3.347	5.758
Earth Pressure (Left)	Left Side Wall	Horizontal	0.250	0.425	5.758	8.660
Earth Pressure (Right)	Right Side Wall	Horizontal	0.000	0.250	-3.347	-5.758
Earth Pressure (Right)	Right Side Wall	Horizontal	0.250	0.425	-5.758	-8.660

■Buoyancy

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Buoyancy	Bottom Slab	Vertical	0.000	2.900	-6.370	-6.370

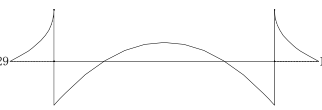
■Subgrade Reaction

Load	Member	Direction	Position (m)	Load Length (m)	Strength 1 (kN/m^2)	Strength 2 (kN/m^2)
Subgrade Reaction	Bottom Slab	Vertical	0.000	2.900	-4.513	-4.513

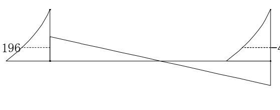
(3) Cross Section Force

[1]Normal Condition

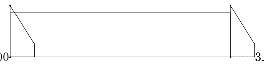
(a) Bending Moment



(b) Shear Force

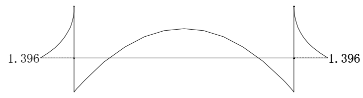


(c) Axial Force

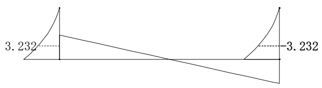


[2]Seismic Condition

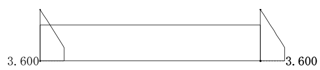
(a) Bending Moment



(b) Shear Force



(c) Axial Force

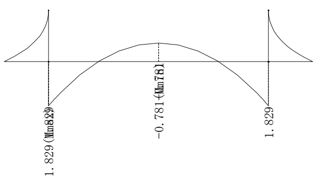


1.3.3 Bottom Slab

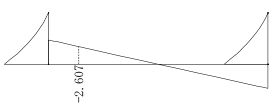
(1) Cross Section Force

[1]Normal Condition

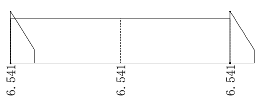
(a) Bending Moment



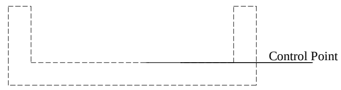
(b) Shear Force



(c) Axial Force



(4) Stress Calculation



1) Bending Stress

(a) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	—	–	—	—
	2'	—	–	—	—
Outside	1	9.0	D12	4	4.525
	2	—	–	—	—

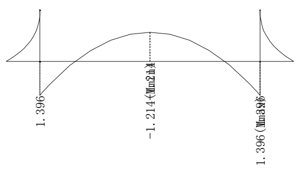
Required Area of Reinforcement 0.530 (cm^2)

(b) Stress Calculation

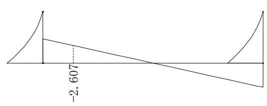
			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	1.829	1.396
Axial Force	N	kN	—	—
Shear Force	S	kN	4.196	3.232
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	300.0	300.0
Effective Width	d	mm	210.0	210.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	37.500	37.500
Stress Intensity (Concrete)	σ_c	N/mm^2	0.494	0.377
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	20.465	15.617
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○
Stress Intensity by Shear Force	τ	N/mm^2	0.020	0.015
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

[2]Seismic Condition

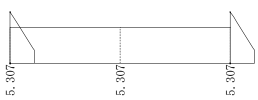
(a) Bending Moment



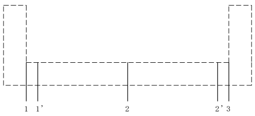
(b) Shear Force



(c) Axial Force



(2) Stress Calculation



1) Control point of Bending Stress

	1	2	3
Control point	0.150	1.450	2.750

2) Control Point of Shear Stress

	1'
Control point	0.400

(3) Calculation of Bending Stress

(a) Control point 1

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	——	—	——	———
	2'	——	—	——	———
Outside	1	11.5	D12	4	4.525
	2	——	—	——	———

Required Area of Reinforcement 0.473 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	1.829	1.396
Axial Force	N	kN	——	——
Shear Force	S	kN	——	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	350.0	350.0
Effective Width	d	mm	235.0	235.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	39.862	39.862
Stress Intensity (Concrete)	σ_c	N/mm^2	0.414	0.316
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	18.232	13.914
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○

(b) Control point 2

1) Bar Arrangement

		Cover (cm)	Diameter	Number	Cross Sectional Area (cm^2)
Inside	1'	9.0	D12	4	4.525
	2'	——	—	——	———
Outside	1	——	—	——	———
	2	——	—	——	———

Required Area of Reinforcement 0.188 (cm^2)

2) Stress Calculation

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	-0.781	-1.214
Axial Force	N	kN	——	——
Shear Force	S	kN	——	——
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	350.0	350.0
Effective Width	d	mm	260.0	260.0
Applied area of Reinforcement (Tension)	A_s	cm^2	4.525	4.525
(Compression)	A_s'	cm^2		
Neutral Axis	X	mm	42.126	42.126
Stress Intensity (Concrete)	σ_c	N/mm^2	0.151	0.234
Allowable Stress Intensity (Concrete)	σ _ca	N/mm^2	8.200	12.300
Evaluation of Compression			○	○
Stress Intensity (Reinforcing Bar)	σ_s	N/mm^2	7.016	10.909
Allowable Stress Intensity (Reinforcing Bar)	σ _sa	N/mm^2	168.000	252.000
Evaluation of Compression			○	○

(4) Calculation of Shear Stress

			Normal Condition	Seismic Condition
Bending Moment	M	kN.m	——	——
Axial Force	N	kN	——	——
Shear Force	S	kN	2.607	2.607
Width of Member	B	mm	1000.0	1000.0
Height of Member	H	mm	350.0	350.0
Bending Moment	d	mm	235.0	235.0
Stress Intensity by Shear Force	τ	N/mm^2	0.011	0.011
Allowable Stress Intensity by Shear Force	τ _a1	N/mm^2	0.360	0.540
Evaluation			○	○

3.4. Stability Against Bearing

Stability against bearing of wing wall is evaluated by yielding displacement of ground.
The yeilding displacement of ground is defined as allowable displacement for the ground acting as elastic body.

・Yeilding Displacement of Ground is determinded as follows;

Yeilding Displacement of Grou ≦ 5.0 cm
≦ 1.0% of Width of Foundation

	① Wdth of Foundation (cm)	② 1 % of ① (cm)				Allowable Relative Displacement (c m)
MSR3	320.0	3.2	<	5.0	cm	3.2

2) Vertical Displacement of Foundation UnderSide

Vertical displacement of foudation underside is calculated as follows;

$$\delta_V = \frac{1}{K_V} \cdot \frac{V}{A}$$

where;

δ_V:	Vertilcal Displacement			(m)
V:	Vertical Load			(kN)
A:	Area of Foudation UnderSide		18.88	(m^2)
K_V:	Vertical Coefficient of Subgrade Reactic	Normal	6780.68	(kN/m^2)
		Seismic	13561.36	(kN/m^2)

・Calculation of Vertical Load

<Normal Condition> Full Flow

Main Body	34.08	*x	5.9	=	201.07	(kN)
Backfill	0.00			=	0.00	(kN)
Water	10.78	*x	5.9	=	63.60	(kN)
					264.67	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

<Seismic Condition> No Water Flow

Main Body	34.08	*x	5.9	=	201.07	(kN)
Backfill	0.00			=	0.00	(kN)
Water	0.00	*x	5.9	=	0.00	(kN)
					201.07	(kN)

*Regarding Weight in Unit Length, Refer to longitudinal calculation of box culvert.

Case	1	2
	Normal	Seismic
K_v (kN/m^2)	6780.68	13561.36
δ_V (cm)	0.207	0.079
δ_a (cm)	3.200	3.200
Evaluation	OK	OK

・Modulus of Sbgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvo : Coefficient of Vertical Reaction of soil (kN/m3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E0m obtained by various soil tests a

$$Kvo = \frac{1}{0.3} \quad \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eom : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

$$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 0.0603 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 3775.67 = 50342.3 kN/m^3

β= ｛（Kv・D）/(4・E・I)｝ $^{1/4}$

= 0.24616

β・L= 1.452 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body Bv= $\sqrt{L \cdot D}$ = $\sqrt{5.90 \times 3.20}$ = $\boxed{4.345}$ m

Case2 Elastic body Bv= $\sqrt{D/\beta}$ = $\sqrt{3.20 / 0.24616}$ = 3.605 m

Kv = Kvo × $(Bv/0.3)^{-3/4}$

= 50342.3 × $(4.345 / 0.3)^{-3/4}$ = 6781 (kN/m^3)

Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\displaystyle\sum_{i=1}^{n} \frac{1}{Ei} \log \frac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\displaystyle\sum_{i=1}^{n} \frac{1}{Ei} \times f\,i}$$

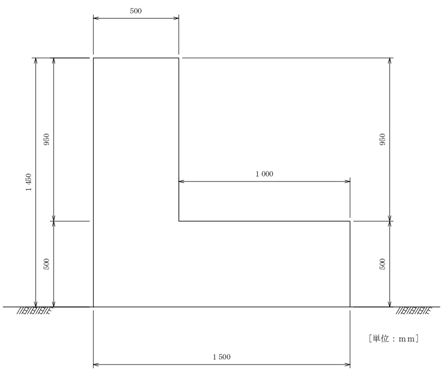
Eom ;	Converted Modulus of Deformation in B≠L		(kN/m^2)	
B ;	Loading Width = Width of Wing Wall	B=	3.200	m
L ;	Loading Length = Length of a span	L=	5.900	m
h_n ;	Depth for Calculation	hn=	9.600	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each Layer (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom (kN/m^2)	B (m)	L (m)	h_n (m)	Tickness of Layer (m)	h i (m)	Ei (kN/m^2)	θ (°)	f n	f i	1 / E i × f i
1	3775.67	3.200	5.900	9.600	3.590	3.590	4720	30	0.4387	0.299	0.0000633
2					1.500	5.090	1400			0.053	0.0000376
3					4.510	9.600	5710			0.087	0.0000153
4											
5											
6											
7											
8											
9											
10											
				Total	9.600					Total	0.0001162

4 MSR-4 Wing Wall (L-type)

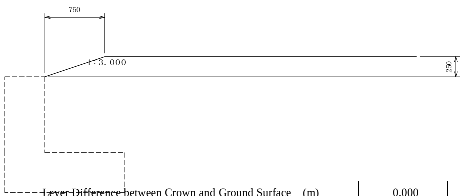
4.1 Design Conditions

4.1.1 Dimension



(Block Length) B ＝ 2700(mm)

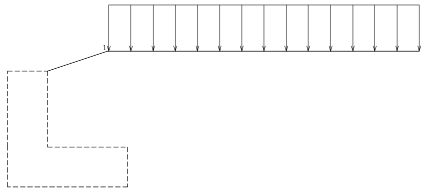
4.1.2 Shape of Backfill and Embankment



~~Lever Difference~~ between Crown and Ground Surface (m)	0.000
Gradient	3.000
Embankment Height (m)	0.250

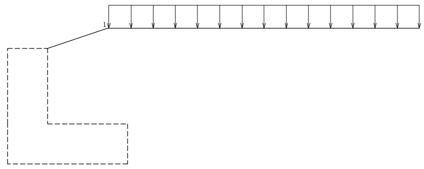
4.1.3 Load

[1] Normal Condition



NO.	Load Position (m)	Load width (m)	Load Intensity (kN/m^2)	
			Start	End
1	0.766	∞	10.000	10.000

[2] Seismic



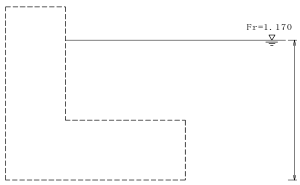
NO.	Load Position (m)	Load Width (m)	Load Intensity (kN/m^2)	
			Start	End
1	0.766	∞	5.000	5.000

4.1.4 Water Lever

[1] Normal Condition

■ Water Lever 1

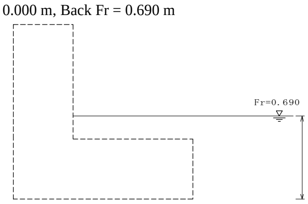
Front Ff = 0.000 m, Back Fr = 1.170 m



[2] Seismic

■ Water Lever 2

Front Ff = 0.000 m, Back Fr = 0.690 m

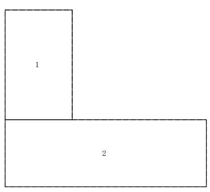


4.2 Stability Analysis

4.2.1 Weight without Considering Water Level

(1)Body Weight

1) Block



2) Center of Gravity

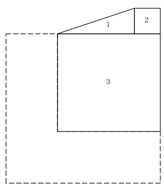
Part	With × Height × Depth	Volume Vi(m^3)	Center of Gravity (m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1 2	0.500× 0.950× 1.000 1.500× 0.500× 1.000	0.475 0.750	0.250 0.750	0.975 0.250	0.119 0.563	0.463 0.188	
Σ		1.225	——	——	0.681	0.651	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.681／ 1.225 ＝ 0.556 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.651／ 1.225 ＝ 0.531 (m)

(2) Backfill

1) Block



2) Volume・Center of gravity

Part	With × Height × Depth	Volume Vi(m^3)	Center of gravity(m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1 2 3	1/2× 0.750× 0.250× 1.000 0.250× 0.250× 1.000 1.000× 0.950× 1.000	0.094 0.063 0.950	1.000 1.375 1.000	1.533 1.575 0.975	0.094 0.086 0.950	0.144 0.098 0.926	
Σ		1.106	——	——	1.130	1.169	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 1.130／ 1.106 ＝ 1.021 (m)

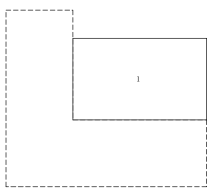
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 1.169／ 1.106 ＝ 1.056 (m)

4.2.2 Weight Considering Water Level

(1) Backfill

[1] Normal

1) Block



2) Volume・Center of gravity

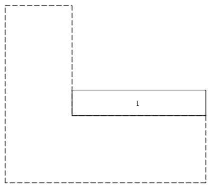
Block	With × Height × Depth	Vi(m^3)	Center of Gravity(m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1	1.000× 0.670× 1.000	0.670	1.000	0.835	0.670	0.559	
Σ		0.670	——	——	0.670	0.559	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.670／ 0.670 ＝ 1.000 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.559／ 0.670 ＝ 0.835 (m)

[2] Seismic

1) Block



2) Center of Gravity

Part	With × Height × Depth	Vi(m^3)	Center of Gravity(m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1	1.000× 0.190× 1.000	0.190	1.000	0.595	0.190	0.113	
Σ		0.190	——	——	0.190	0.113	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.190／ 0.190 ＝ 1.000 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.113／ 0.190 ＝ 0.595 (m)

4.2.3 Vertical and Horizontal Force

(1) Weight of Body

[1] Normal

	Weight W ＝ γ ・ V (kN)	Action Position X (m)
Body	24.000 × 1.225 ＝ 29.400	0.556

[2] Seismic

	Weight W ＝ γ ・ V (kN)	Action position X (m)
Body	24.000 × 1.225 ＝ 29.400	0.556

	Horizontal Force H ＝ W ・ kh (kN)	Action position Y (m)
Body	29.400 × 0.20 ＝ 5.880	0.531

(2) Weight of Soil, Uplift

[1] Normal

1) Weight of Soil

Parts Divided by Water Level

Parts	Total V(m^3)	Center of Gravity(m) X	Y	Below the Water Level Vl(m^3)	Center of Gravity(m) Xl	Yl
Siol(back)	1.106	1.021	1.056	0.670	1.000	0.835

Parts	Above than water level Vu(m^3)	Center of Gravity(m) Xu	Yu
Siol(back)	0.436	1.054	1.396

Volume above the water level

Vu ＝ V－Vl

Center of Gravity Higher than the Water Level

Xu ＝ (V・X－Vl・Xl)／Vu

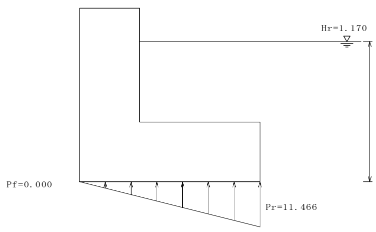
Yu ＝ (V・Y－Vl・Yl)／Vu

Force due to Soil

	Above the Water Level Wu ＝ Vu・（unit wet weight） (kN)	Below the Water Level Wl ＝ Vl・（unit saturated weight） (kN)
soil	0.436 × 18.000 ＝ 7.853	0.670 × 20.000 ＝ 13.400

	W Wu ＋ Wl (kN)	X (Wu・Xu＋Wl・Xl)/W (m)
soil	21.253	1.020

2) Uplift



The Water Level Front Hf ＝ 0.000 (m)

The Water Level Back Hr ＝ 1.170 (m)

Intensity of Water Pressure Pf ＝ 0.000 (kN/m^2)

Pr ＝ 11.466 (kN/m^2)

Buoyancy Effecting at Bottom

$$U = \frac{Pf+Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 8.599 \ (kN)$$

Position （From the front of footing）

$$X = \frac{Pf+2 \cdot Pr}{3 \cdot (Pf+Pr)} \cdot Bj = 1.000 \ (m)$$

Where、

Bj ：Width of Footing Bj ＝ 1.500 (m)

Bc ：Length of Foting Bc ＝ 1.000 (m)

λ ：Reduction Coefficient of Buoyancy λ ＝ 1.000

[2] Seismic

1) Gravity of Soil

Parts Divided by Water Level

	Total V(m^3)	Center of Gravity(m) X	Y	Below the Water Level Vl(m^3)	Center of Gravity(m) Xl	Yl
Siol	1.106	1.021	1.056	0.190	1.000	0.595

	Above the Water Level Vu(m^3)	Center of Gravity(m) Xu	Yu
Siol	0.916	1.026	1.152

Volume Above the Water Level

Vu ＝ V－Vl

Center of gravity of the soil above the water level

Xu ＝ (V・X－Vl・Xl)／Vu

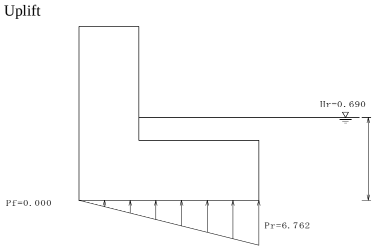
Yu ＝ (V・Y－Vl・Yl)／Vu

Vertical and Horizontal Force

	Weight of Soil Above Water Level Wu ＝ Vu・（unit wet weight） (kN)	Weight of Soil Below Water Leve Wl ＝ Vl・（unit saturated weight） (kN)
soil	0.916 × 18.000 ＝ 16.493	0.190 × 20.000 ＝ 3.800

	Total Weight:W Wu ＋ Wl (kN)	Acting Position:X (Wu・Xu＋Wl・Xl)/W (m)	Horizontal Force:H W ・ kh (kN)	Acting Position:Y (m)
soil	20.293	1.021	20.293 × 0.20 ＝ 4.059	1.048

2) Uplift



Front Water Level Hf ＝ 0.000 (m)

Back Water Level Hr ＝ 0.690 (m)

Intensity of Water Pressure at the Front of Footing Pf ＝ 0.000 (kN/m^2)

Intensity of Water Pressure at the Back of Footing Pr ＝ 6.762 (kN/m^2)

Buoyancy Acting on Body

$$U = \frac{Pf+Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 5.071 \ (kN)$$

Acting Position（From the front of the footing）

$$X = \frac{Pf+2 \cdot Pr}{3 \cdot (Pf+Pr)} \cdot Bj = 1.000 \ (m)$$

Where、

Bj ：Width of Footing Bj ＝ 1.500 (m)

Bc ：Length of Footing Bc ＝ 1.000 (m)

λ ：Reduction Coefficient of Buoyancy λ ＝ 1.000

(3) Summary of Body Weight

[1] Normal (Water Level 1)

	Weight Ni (kN)	Horizontal Hi (kN)	Position(m)		Moment(kN.m)	
			Xi	Yi	Ni・Xi	Hi・Yi
Body	29.400	0.000	0.556	0.000	16.350	0.000
Soil	21.253	0.000	1.020	0.000	21.677	0.000
Total	50.653	0.000	——	——	38.027	0.000

[2] Seismic (Water Level 2)

	Weight Ni (kN)	Horizontal Hi (kN)	Position(m)		Moment(kN.m)	
			Xi	Yi	Ni・Xi	Hi・Yi
Body	29.400	5.880	0.556	0.531	16.350	3.123
Soil	20.293	4.059	1.021	1.048	20.722	4.252
Total	49.693	9.939	——	——	37.072	7.375

(4) Earth and Water Pressure

(a) Earth Pressure

[1] Normal Condition

Earth Pressure

Position of Imaginary Back Side xp = 1.500 m

yp = 0.000 m

Height of Imaginary Back Side H = 1.450 m

Height Above Water Surface H1 = 0.280 m

Height Below Water Surface H2 = 1.170 m

Angel between Back Side Surface of Wall and Vertical Plane α = 0.000 °

Unit Weight of Backfill γs = 18.000 kN/m^3

Internal Friction Angel of Soil φ = 30.000 °

Angle between Ground Surface and Horizontal Plane β = 0.000 °

Wall Friction Angel δ = 25.000 °

Acting load by embankment

$$I_w = 1+\left(\frac{X}{H_w}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{X}{H_w}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_w}\right)-\frac{2}{\pi}\left(\frac{X}{H_w}\right)$$

$$= 1+\left(\frac{1.208}{1.450}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{1.208}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{1.208}{1.450}\right)-\frac{2}{\pi}\left(\frac{1.208}{1.450}\right)$$

$$= 0.41450$$

$$q_w = \gamma \cdot H_s \cdot I_w$$

$$= 18.000\times 0.806\times 0.41450$$

$$= 6.010 \text{ kN/m}^2$$

Converted Load

$q = q_w$

Where、

X : Distance of Imaginary Back Side(m), X1＋X2／2

X1 : Horizontal Length of Embankment(m), X1 = 0.000

X2 : Length of Slop (m), X2 = 2.417

H_w : Height of Body(m)

H_s : $H_0 + H_1$(m)

H_0 : Height of Embankment(m), H_0 = 0.250

H_1 : Height of Conversion Embankment(m), H_1 = Q／γ = 0.556

Q : Surchage (kN/m^2), Q = 10.000

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\cdot\left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00°-0.000°)}{\cos^2 0.000°\cdot\cos(0.000°+30.000°)}$$

$$\times\frac{1}{\left[1+\sqrt{\frac{\sin(30.00°+30.000°)\cdot\sin(30.00°-0.000°)}{\cos(0.000°+30.000°)\cdot\cos(0.000°-0.000°)}}\right]^2}$$

$$= 0.2972$$

Earth Pressure at Top

p1 = q・K = 6.010×0.2972 = 1.786 kN/m^2

Earth Pressure at Water Level

$$p2 = K\cdot\gamma s\cdot H1+p1$$

$$= 0.2972\times 18.000\times 0.280+1.786$$

$$= 3.284 \text{ kN/m}^2$$

Earth Pressure at Bottom

$$p3 = K\cdot(\gamma sat-\gamma w)\cdot H2+p2$$

$$= 0.2972\times(20.000-9.800)\times 1.170+3.284$$

$$= 6.830 \text{ kN/m}^2$$

Earth Pressure Force above Water Level

$$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(1.786+3.284)\times 0.280 = 0.710 \text{ kN}$$

Earth Pressure Force Below Water Level

$$P2 = \frac{1}{2}\cdot(p2+p3)\cdot H2 = \frac{1}{2}\times(3.284+6.830)\times 1.170 = 5.917 \text{ kN}$$

Total Earth Pressure Force

P = P1＋P2 = 0.710＋5.917 = 6.627 kN

Horizontal Component and Vertical Element of Earth Pressure Force

Ph = P・cos(α＋δ) = 5.500×cos(0.000°＋25.000°) = 5.739 kN

Pv = P・sin(α＋δ) = 5.500×sin(0.000°＋25.000°) = 3.313 kN

Acting Position

$$M1 = P1\cdot\left(\frac{2\cdot p1+p2}{p1+p2}\cdot\frac{H1}{3}+H2\right)$$

$$= 0.710\times\left(\frac{2\times 1.786+3.284}{1.786+3.284}\times\frac{0.280}{3}+1.170\right)$$

$$= 0.920 \text{ kN.m}$$

$$M2 = P2\cdot\left(\frac{2\cdot p2+p3}{p2+p3}\cdot\frac{H2}{3}\right)$$

$$= 5.917\times\left(\frac{2\times 3.284+6.830}{3.284+6.830}\times\frac{1.170}{3}\right)$$

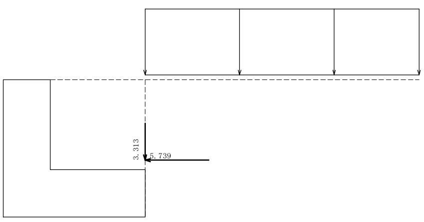
$$= 3.059 \text{ kN.m}$$

$$Ho = \frac{M1+M2}{P1+P2} = \frac{0.920+3.059}{0.710+5.917} = 0.600 \text{ m}$$

x = xp－Ho・tanα = 1.500－0.600×tan0.000°= 1.500 m

y = yp＋Ho = 0.000＋0.600 = 0.600 m

・Earth Pressure



[2] Seismic

Distance of Imaginary Back Side xp = 1.500 m

yp = 0.000 m

Height of Imaginary Back Side H = 1.450 m

Height Above Water Surface H1 = 0.760 m

Height Below Water Surface H2 = 0.690 m

Angle Between Back Side and Vertical Plane α = 0.000 °

Unit Weight of Soil γs = 18.000 kN/m^3

Internal Friction Angle of Soil φ = 30.000 °

Angle Between Ground Surface and Horizontal Plane β = 0.000 °

Angle of Wall Friction δ = 10.000 °

Acting Load by Embankment

$$I_w = 1+\left(\frac{X}{H_w}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{X}{H_w}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_w}\right)-\frac{2}{\pi}\left(\frac{X}{H_w}\right)$$

$$= 1+\left(\frac{0.792}{1.450}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{0.792}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{0.792}{1.450}\right)-\frac{2}{\pi}\left(\frac{0.792}{1.450}\right)$$

$$= 0.53752$$

$$q_w = \gamma\cdot H_s\cdot I_w$$

$$= 18.000\times 0.528\times 0.53752$$

$$= 5.106 \text{ kN/m}^2$$

$q = q_w$

Where、

X : Imaginary Distance (m), X1＋X2／2

X1 : Horizontal Length of Embankment(m), X1 = 0.000

X2 : Length (m), X2 = 1.583

H_w : Height of Body(m),

H_s : $H_0 + H_1$(m)

H_0 : Height of Embankment(m), H_0 = 0.250

H_1 : Height of Conservation Embankment(m) H_1 = Q／γ = 0.278

Q : Surcharge(kN/m^2), Q = 5.000

Compound angle of seismic

θ = tan^{-1}kh = tan^{-1}0.20 = 11.310 °

Coefficient of active earth pressure

$$K = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\cdot\left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00°-0.000°-11.310°)}{\cos 11.310°\cdot\cos^2 0.000°\cdot\cos(0.000°+10.000°+11.310°)}$$

$$\times\frac{1}{\left[1+\sqrt{\frac{\sin(30.00°+10.000°)\cdot\sin(30.00°-0.000°-11.310°)}{\cos(0.000°+10.000°+11.310°)\cdot\cos(0.000°-0.000°)}}\right]^2}$$

$$= 0.4544$$

If, $\varphi - \beta - \theta < 0$, then $\sin(\varphi - \beta - \theta) = 0$

Earth Pressure of Top

$p1 = q \cdot K = 5.106 \times 0.4544 = 2.320\ kN/m^2$

Earth Pressure of Water Level

$p2 = K \cdot \gamma s \cdot H1 + p1$

$= 0.4544 \times 18.000 \times 0.760 + 2.320$

$= 8.537\ kN/m^2$

Earth Pressure Below Water Level

$p3 = p2 = 8.537\ kN/m^2$

Earth Pressure at Bottom

$p4 = K' \cdot (\gamma sat - \gamma w) \cdot H2 + p3$

$= 0.4544 \times (20.000 - 9.800) \times 0.690 + 8.537$

$= 11.735\ kN/m^2$

Earth Pressure Force Above Water Surface

$$P1 = \frac{1}{2} \cdot (p1 + p2) \cdot H1 = \frac{1}{2} \times (2.320 + 8.537) \times 0.760 = 4.126\ kN$$

Earth Pressure Force Below Water Surface

$$P2 = \frac{1}{2} \cdot (p3 + p4) \cdot H2 = \frac{1}{2} \times (8.537 + 11.735) \times 0.690 = 6.994\ kN$$

Total Pressure Earth Pressure

$P = P1 + P2 = 4.126 + 6.994 = 11.120\ kN$

Horizontal Component and Vertical Element of Earth Pressure Force is as Follows.

$$Ph = P \cdot \cos(\alpha + \delta + \theta) = 11.120 \times \cos(0.000° + 10.000° + 11.310°) = 10.360\ kN$$

$$Pv = P \cdot \sin(\alpha + \delta + \theta) = 11.120 \times \sin(0.000° + 10.000° + 11.310°) = 4.041\ kN$$

Acting Position

$$M1 = P1 \cdot \left(\frac{2 \cdot p1 + p2}{p1 + p2} \cdot \frac{H1}{3} + H2\right)$$

$$= 4.126 \times \left(\frac{2 \times 2.320 + 8.537}{2.320 + 8.537} \times \frac{0.760}{3} + 0.690\right)$$

$= 4.114\ kN.m$

$$M2 = P2 \cdot \left(\frac{2 \cdot p3 + p4}{p3 + p4} \cdot \frac{H2}{3}\right)$$

$$= 6.994 \times \left(\frac{2 \times 8.537 + 11.735}{8.537 + 11.735} \times \frac{0.690}{3}\right)$$

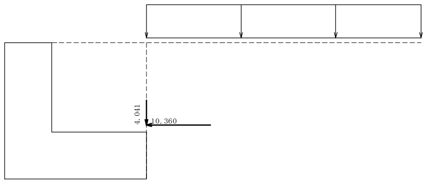
$= 2.287\ kN.m$

$$Ho = \frac{M1 + M2}{P1 + P2} = \frac{4.114 + 2.287}{4.126 + 6.994} = 0.576\ m$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.576 \times \tan 0.000° = 1.500\ m$

$y = yp + Ho = 0.000 + 0.576 = 0.576\ m$

・Earth Pressure Force



(b) Water Pressure

$$P = \frac{1}{2} \cdot \gamma w \cdot h^2$$

$$Y = \frac{h}{3}$$

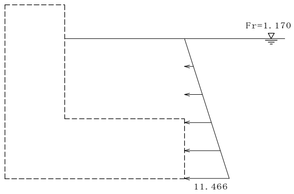
Where、

γw : Unit Weight of Water (kN/m^3), $\gamma w = 9.800$

h : Water Depth (m)

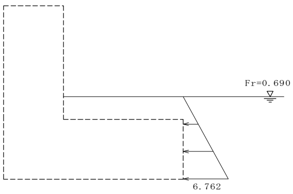
Y : Acting Position (m)

[1] Normal Condition



	h (m)	P (kN)	Position Y (m)
Back	1.170	6.708	0.390

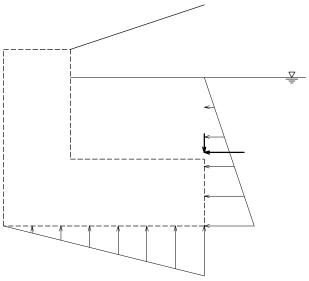
[2] Seismic



	h (m)	P (kN)	Position Y (m)
Back	0.690	2.333	0.230

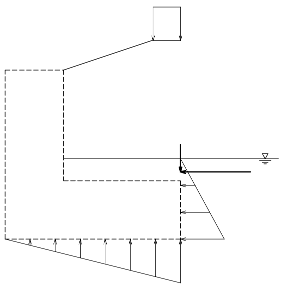
4.2.4 Summary of Acting Force

[1] Normal



Item	Vertical N_i (kN)	Horizontal H_i (kN)	Arm Length		Rotation Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Body Weight	50.653	0.000	0.751	0.000	38.027	0.000
Uplift	-8.599	0.000	1.000	0.000	-8.599	0.000
Load	2.340	0.000	1.383	0.000	3.236	0.000
Water pressure	0.000	6.708	0.000	0.390	0.000	2.616
Earth Pressure	3.313	5.739	1.500	0.600	4.970	3.446
Total	47.707	12.447	——	——	37.634	6.062

[2] Seismic



Item	Vertical N_i (kN)	Horizontal H_i (kN)	Arm Length		Rotation Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Body Weight	49.693	9.939	0.746	0.742	37.072	7.375
Uplift	-5.071	0.000	1.000	0.000	-5.071	0.000
Load	1.170	0.000	1.383	0.000	1.618	0.000
Water pressure	0.000	2.333	0.000	0.230	0.000	0.537
Earth Pressure	4.041	10.360	1.500	0.576	6.062	5.963
Total	49.833	22.632	——	——	39.680	13.875

Item	Vertical N_i (kN)	Horizontal H_i (kN)	Arm Length		Rotation Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Earth Pressure	4.041	10.360	1.500	0.576	6.062	5.963
Total	49.833	22.632	——	——	39.680	13.875

Load condition	N_o (kN)	H_o (kN)	M_o (kN.m)
Normal	47.707	12.447	31.572
Seismic	49.833	22.632	25.806

(2) Summary of Acting Force at center of Footing

Vertical Force: $N_c = N_o$ (kN)

Horizontal Force: $H_c = H_o$ (kN)

Rotation Moment: $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where、

Width of Footing: $B_j = 1.500$ (m)

■ Unit of Width

Condition	N_c (kN)	H_c (kN)	M_c (kN.m)
Normal	47.707	12.447	4.209
Seismic	49.833	22.632	11.569

■ Full Width(2.70m)

Condition	N_c (kN)	H_c (kN)	M_c (kN.m)
Normal	128.809	33.606	11.363
Seismic	134.548	61.105	31.236

4.2.5 Results of Stability Analysis

(1) Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where、

d : Distance of Resultant (m)
ΣMr : Resistance Moment (kN.m)
ΣMt : Overturning Moment (kN.m)
ΣV : Total Vertical Element Resultant (kN)

$$e = \frac{B}{2} - d$$

Where、

e : Eccentric Distance on bottom (m)
B : Width(m), B ＝ 1.500

e_a＝ B／n

Where、

e_a : Allowable Eccentric Distance (m)
n : factor of Safe

Condition	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)
Normal	37.634	6.062	47.707	0.662	0.088	≦	0.250
Seismic	39.680	13.875	49.833	0.518	0.232	≦	0.500

(2) Sliding

$$F_s = \frac{\Sigma V \cdot \mu + C_B \cdot B}{\Sigma H}$$

Where、

ΣV : Total Vertical Force (kN)
ΣH : Total Horizontal Force (kN)
μ : Coefficient of Friction, μ＝0.000
C_B : Cohesion (kN/m^2), C_B ＝ 6.000
B : Width(m), B ＝ 1.500

Condition	Vertical ΣV(kN)	Horizontal ΣH(kN)	Factor of Safe F_s		F_{sa}
Normal	47.707	12.447	0.723	＜	1.500
Seismic	49.833	22.632	0.398	＜	1.200

Enough stability is not ensured. Hence, stability against sliding is calculated as follows;

1) Considering the weight of U-shape Section

Normal: Horizontal Load = 33.69 kN
Horizontal Resistance Force = 74.70 kN
Fs (74.7−0) / 33.69 = **2.22 >1.5 OK**

Seismic: Horizontal Load = 89.33 kN
Horizontal Resistance Force = 74.70 kN
Fs (74.7−0) / 89.33 = **0.84 <1.2 NG**

Though weight of U-shaped section is considered, enough stability cannot be ensured.

2) Considering the effect of SSP against Sliding

Horizontal Load Shared by SSP during Seismic:
(61.11+28.224)*1.2-(24.3+50.4)= 32.50 kN
Length of Wall is 2.7m
Per Unit Length: 32.5008/ 2.7 = 12.04 kN

Coefficient of Lateral Modulus of Subgrade Reaction
Average N Value is Considered as 1:

aE_0= 1*2800*2= 5600 kN/m^2
Sheet Pile StypeⅢw is Used: I= 32400 cm^4/m = 0.000324 m^4/m
Z= 1800cm^3/m = 0.0018 m^3/m

kho = 1/0.3x aEo Assume kh= 5250 kN/m^3
= 1/0.3x5600 β= ((kh･D)/4･EI))$^{1/4}$ D=1m
= 18667 kN/m^3 = 0.37725 m^{-1}

Bv= SQRT(D/b) = 1.628
Therefore kh= kho x (Bv/0.3)$^{-(3/4)}$
= 18667x(1.63/0.3)$^{-(3/4)}$
= 5250 Assumed Value is Correct

･Caculation of Lengh of Sheet Pile
Embeded length l= 3 / β = 3 / 0.37725 = 7.952 m
= 8.0 m
Total Length of SSP L= 8+0.3 = 8.3 = **8.5 m**

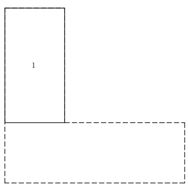
･Maxmum Bending Moment
Chang's form Considered As Pile Embedded in the Ground
Horizontal Load Acting at SSP = 12.04 kN
Mmax = -H･e$^{-\pi/4}$(sin(π/4))/b = 0.3225Hp/b
= 10.29 kN･m
Bending Stress Intesity s = Mmax/Z
= **5.7 N/mm^2 < 180 N/mm^2**

By considering the effect of SSP, enough stability can be ensured.

4.3 Design of Vertical Wall

4.3.1 Wight without Considering Water Level

(1)Block



(2)Volume, Center of Gravity

Part	With × Height × Depth	Volume Vi(m^3)	Center of Gravity (m) Xi	Yi	Vi・Xi	Vi・Yi	Remarks
1	0.500× 0.950× 1.000	0.475	0.250	0.475	0.119	0.226	
Σ		0.475	—	—	0.119	0.226	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.119／ 0.475 ＝ 0.250 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.226／ 0.475 ＝ 0.475 (m)

(1) Body Weight

[1]Normal

Part	W ＝ γ ・ V (kN)	Location X (m)
Body	24.000 × 0.475 ＝ 11.400	0.000

Actiong Location

X ＝ Xc－XG ＝ 0.250－0.250
＝ 0.000 m

w、

Xc : Distance Form Front Side of Vertical Wall to Control Point (m)

[2]Seismic

Part	W ＝ γ ・ V (kN)	Location X (m)
Body	24.000 × 0.475 ＝ 11.400	0.000

Part	W ＝ γ ・ V (kN)	Location X (m)
Body	11.400 × 0.200 ＝ 2.280	0.475

Acting Location

X ＝ Xc－XG ＝ 0.250－0.250
＝ 0.000 m

Where,

Xc : Distance Form Front Side of Vertical Wall to Control Point (m)

4.3.2 Earth Pressure

[1] Normal Condition

[1] Normal Condition
Earth Pressure

Position of Imaginary Back Side	xp ＝	0.250 m
	yp ＝	0.000 m
Height of Imaginary Back Side	H ＝	0.950 m
Height Above Water Surface	H1 ＝	0.280 m
Height Below Water Surface	H2 ＝	0.670 m
Angel between Back Side Surface of Wall and Vertical Plane	α ＝	0.000 °
Unit Weight of Backfill	γs ＝	18.000 kN/m^3
Internal Friction Angel of Soil	φ ＝	30.000 °
Angle between Ground Surface and Horizontal Plane	β ＝	0.000 °
Wall Friction Angel	δ ＝	15.000 °

Acting load by embankment

$$I_s = 1 + \left(\frac{X}{H_s}\right)^2 - \frac{2}{\pi}\left\{1 + \left(\frac{X}{H_s}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_s}\right) - \frac{2}{\pi}\left(\frac{X}{H_s}\right)$$

$$= 1 + \left(\frac{1.208}{1.450}\right)^2 - \frac{2}{\pi}\left\{1 + \left(\frac{1.208}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{1.208}{1.450}\right) - \frac{2}{\pi}\left(\frac{1.208}{1.450}\right)$$

$$= 0.41450$$

$$q_v = \gamma \cdot H_s \cdot I_s$$

$$= 18.000 \times 0.806 \times 0.41450$$

＝ 6.010 kN/m^2

Converted Load

q ＝ q_w

Where、

X : Distance of Imaginary Back Side(m), X1＋X2／2
X1 : Horizontal Length of Embankment(m), X1 ＝ 0.000
X2 : Length of Slop (m), X2 ＝ 2.417
H_w : Height of Body(m)
H_s : H_0＋ H_1(m)
H_0 : Height of Embankment(m), H_0 ＝ 0.250
H_1 : Height of Conversion Embankment(m), H_1 ＝ Q／γ ＝ 0.556
Q : Surchage (kN/m^2), Q ＝ 10.000

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ )}{\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +15.000^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +15.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ )}{\cos(0.000^\circ\ +15.000^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.3014$

Pressure at Top

$p1 = q\cdot K = 6.010\times0.3014 = 1.812\ kN/m^2$

Pressure at Water Level

$p2 = K\cdot\gamma s\cdot H1+p1$

$= 0.3014\times18.000\times0.280+1.812$

$= 3.331\ kN/m^2$

Pressure at Bottom

$p3 = K\cdot(\gamma sat-\gamma w)\cdot H2+p2$

$= 0.3014\times(20.000-9.800)\times0.670+3.331$

$= 5.391\ kN/m^2$

Earth Pressure Force

$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 1.812+\ 3.331)\times\ 0.280 =\ \ 0.720\ kN$

$P2 = \frac{1}{2}\cdot(p2+p3)\cdot H2 = \frac{1}{2}\times(\ 3.331+\ 5.391)\times\ 0.670 =\ \ 2.922\ kN$

Total

$P = P1+P2 = 0.720+2.922 = 3.642\ kN$

Horizontal Component of Earth Pressure Force

$Ph = P\cdot\cos(\alpha+\delta) = 3.642\times\cos(\ 0.000^\circ+15.000^\circ) = 3.517\ kN$

Vertical Element of Earth Pressure Force

$Pv = P\cdot\sin(\alpha+\delta) = 3.642\times\sin(\ 0.000^\circ+15.000^\circ) = 0.943\ kN$

Acting Position

$$M1 = P1\cdot\left(\frac{2\cdot p1+p2}{p1+p2}\cdot\frac{H1}{3}+H2\right)$$

$$= 0.720\times\left(\frac{2\times1.812+3.331}{1.812+3.331}\times\frac{0.280}{3}+0.670\right)$$

$= 0.573\ kN.m$

$$M2 = P2\cdot\left(\frac{2\cdot p2+p3}{p2+p3}\cdot\frac{H2}{3}\right)$$

$$= 2.922\times\left(\frac{2\times3.331+5.391}{3.331+5.391}\times\frac{0.670}{3}\right)$$

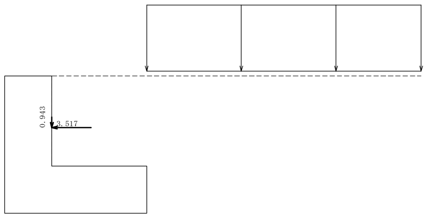
$= 0.902\ kN.m$

$$Ho = \frac{M1+M2}{P1+P2} = \frac{0.573+0.902}{0.720+2.922} = 0.405\ m$$

$x = Ho\cdot\tan\alpha-xp = 0.405\times\tan0.000^\circ-0.250 = -0.250\ m$

$y = yp+Ho = 0.000+0.405 = 0.405\ m$

・Earth Pressure



[2] Seismic

Distance of Imaginary Back Side	xp =	0.250 m
	yp =	0.000 m
Height of Imaginary Back Side	H =	0.950 m
Height Above Water Surface	H1 =	0.760 m
Height Below Water Surface	H2 =	0.190 m
Angle Between Back Side and Vertical Plane	α =	0.000 °
Unit Weight of Soil	γs =	18.000 kN/m^3
Internal Friction Angle of Soil	φ =	30.000 °
Angle Between Ground Surface and Horizontal Plane	β =	0.000 °
Angle of Wall Friction	δ =	0.000 °

Acting Load by Embankment

$$I_w = 1+\left(\frac{X}{H_w}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{X}{H_w}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_w}\right)-\frac{2}{\pi}\left(\frac{X}{H_w}\right)$$

$$= 1+\left(\frac{0.792}{1.450}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{0.792}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{0.792}{1.450}\right)-\frac{2}{\pi}\left(\frac{0.792}{1.450}\right)$$

$= 0.53752$

$q_w = \gamma\cdot H_s\cdot I_w$

$= 18.000\times0.528\times0.53752$

$= 5.106\ kN/m^2$

$q = q_w$

Where、

X : Imaginary Distance (m),	X1+X2／2
X1 : Horizontal Length of Embankment(m),	X1 = 0.000
X2 : Length (m),	X2 = 1.583
H_w : Height of Body(m),	
H_s : H_0+ H_1(m)	
H_0 : Height of Embankment(m),	H_0 = 0.250
H_1 : Height of Conservation Embankment(m),	H_1 = Q／γ = 0.278
Q : Surcharge(kN/m^2),	Q = 5.000

Seismic Compound Angle

$\theta = \tan^{-1}kh = \tan^{-1}0.20 = 11.310\ ^\circ$

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos 11.310^\circ\ \cdot\cos^2 0.000^\circ\ \cdot\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )}$$

$$\times\frac{1}{\left[1+\sqrt{\dfrac{\sin(30.00^\circ\ +0.000^\circ\ )\cdot\sin(30.00^\circ\ -0.000^\circ\ -11.310^\circ\ )}{\cos(0.000^\circ\ +0.000^\circ\ +11.310^\circ\ )\cdot\cos(0.000^\circ\ -0.000^\circ\ )}}\right]^2}$$

$= 0.4733$

Earth Pressure at Top

$p1 = q\cdot K = 5.106\times0.4733 = 2.417\ kN/m^2$

Earth Pressure at Water Level

$p2 = K\cdot\gamma s\cdot H1+p1$

$= 0.4733\times18.000\times0.760+2.417$

$= 8.891\ kN/m^2$

Earth Pressure Below Water Level

$p3 = p2 = 8.891\ kN/m^2$

Earth Pressure at Bottom

$p4 = K'\cdot(\gamma sat-\gamma w)\cdot H2+p3$

$= 0.4733\times(20.000-9.800)\times0.190+8.891$

$= 9.808\ kN/m^2$

Earth Pressure Force

$P1 = \frac{1}{2}\cdot(p1+p2)\cdot H1 = \frac{1}{2}\times(\ 2.417+\ 8.891)\times\ 0.760 =\ \ 4.297\ kN$

$P2 = \frac{1}{2}\cdot(p3+p4)\cdot H2 = \frac{1}{2}\times(\ 8.891+\ 9.808)\times\ 0.190 =\ \ 1.776\ kN$

Total Earth Pressure Force

$P = P1+P2 = 4.297+1.776 = 6.073\ kN$

Horizontal Component of Earth Pressure Force

$Ph = P\cdot\cos(\alpha+\delta+\theta) = 6.073\times\cos(\ 0.000^\circ+\ 0.000^\circ+11.310^\circ) = 5.955\ kN$

Vetical component of Earth Pressure

$Pv = P\cdot\sin(\alpha+\delta+\theta) = 6.073\times\sin(\ 0.000^\circ+\ 0.000^\circ+11.310^\circ) = 1.191\ kN$

Acting Position

$$M1 = P1\cdot\left(\frac{2\cdot p1+p2}{p1+p2}\cdot\frac{H1}{3}+H2\right)$$

$$= 4.297\times\left(\frac{2\times2.417+8.891}{2.417+8.891}\times\frac{0.760}{3}+0.190\right)$$

$= 2.138\ kN.m$

$$M2 = P2\cdot\left(\frac{2\cdot p3+p4}{p3+p4}\cdot\frac{H2}{3}\right)$$

$$= 1.776\times\left(\frac{2\times8.891+9.808}{8.891+9.808}\times\frac{0.190}{3}\right)$$

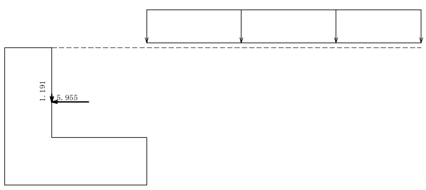
$= 0.166\ kN.m$

$$Ho = \frac{M1+M2}{P1+P2} = \frac{2.138+0.166}{4.297+1.776} = 0.379\ m$$

$x = Ho\cdot\tan\alpha-xp = 0.379\times\tan0.000^\circ-0.250 = -0.250\ m$

$y = yp+Ho = 0.000+0.379 = 0.379\ m$

・Earth Pressure



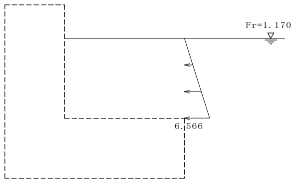
4.3.3 Water Pressure

$$P = \frac{1}{2} \cdot \gamma w \cdot h^2$$

$$Y = \frac{h}{3}$$

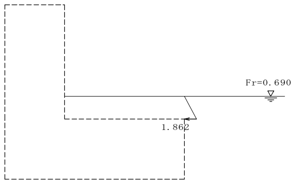
Where、
  γw：Unit Weight of Water (kN/m^3), γw＝9.800
  h：Water Level(m)
  Y：Acting Position (m)

[1] Normal



	h (m)	P (kN)	Y (m)
back	0.670	2.200	0.223

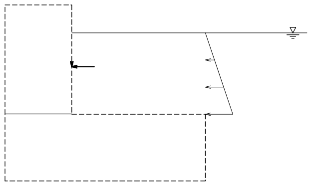
[2] Seismic



	h (m)	P (kN)	Y (m)
back	0.190	0.177	0.063

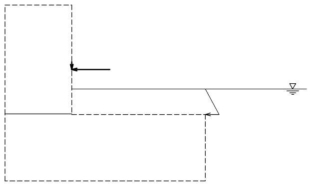
4.3.4 Summary of Section Force

[1] Normal



Load	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M ＝M_{xi}+M_{yi} (kN.m)
Gravity	11.400	0.000	0.000	0.000	0.000
Water force	0.000	2.200	0.000	0.223	0.491
Earth Pressure	0.943	3.517	-0.250	0.405	1.425
Total	0.000	5.717	——	——	1.916

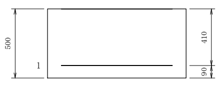
[2] Seismic



Load	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M ＝M_{xi}+M_{yi} (kN.m)
Gravity	11.400	2.280	0.000	0.475	1.083
Water force	0.000	0.177	0.000	0.063	0.011
Earth Pressure	1.191	5.955	-0.250	0.379	2.259
Total	0.000	8.412	——	——	3.353

4.3.5 Stress Calculation

(1) Bar Arrangement



		Cover (cm)	Diameter	Area (cm^2)	Number	Total Reinforcement Area (cm^2)
Inside	1'	——	–	——	——	——
	2'	——	–	——	——	——
Outside	1	9.0	D12	1.267	4.00	4.525
	2	——	–	——	——	——

Required Area of Reinforcement 0.339 (cm^2)

(2) Bending Stress Intensity

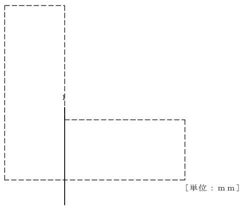
Condition	M (kN.m)	N (kN)	x (cm)	Stress Intensity (N/mm^2)(concrete)	Stress Intensity (N/mm^2)(bar)
Normal	1.916	0.000	5.386	0.181 ≦ 8.200	10.800 ≦ 140.000
Seismic	3.353	0.000	5.386	0.318 ≦ 12.300	18.901 ≦ 252.000

(3) Shear Stress Intensity

Condition	Shear Force S_h(kN)	Shear Stress Intensity (N/mm^2)
Normal	5.717	0.014 ≦ 0.360
Seismic	8.412	0.021 ≦ 0.540

4.4 Design of Bottom Slab

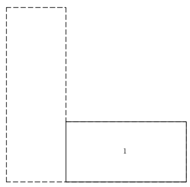
Position ＝ 0.000 m



4.4.1 Weight without Considering Water Level

(a) Body Weight

1)Block



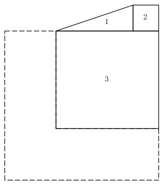
2)Body Weight, Center of Gravity

Part	With × Height × Depth	Volume Vi(m^3)	Center of Gravity (m)	Vi・Xi	Remarks
1	1.000× 0.500× 1.000	0.500	0.500	0.250	
Σ		0.500	——	0.250	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.250／ 0.500 ＝ 0.500 (m)

(b) Backfill

1)Block



2)Volume, Center of Gravity

Part	With × Height × Depth	Volume $V_i(m^3)$	Center of Gravity (m)	$V_i \cdot X_i$	Remarks
1 2 3	1/2× 0.750× 0.250× 1.000 0.250× 0.250× 1.000 1.000× 0.950× 1.000	0.094 0.063 0.950	0.500 0.875 0.500	0.047 0.055 0.475	
Σ		1.106	——	0.577	

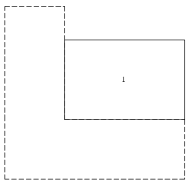
Center of Gravity $XG = \Sigma(Vi \cdot Xi) / \Sigma Vi =$ 0.577／ 1.106 ＝ 0.521 (m)

4.4.2 Weight Considering Water Level

(a) Backfill

[1]Normal

1) Block

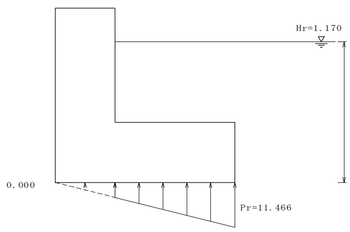


2) Volume, Center of Gravity

Part	With × Height × Depth	Volume $V_i(m^3)$	Center of Gravity (m)	$V_i \cdot X_i$	Remarks
1	1.000× 0.670× 1.000	0.670	0.500	0.335	
Σ		0.670	——	0.335	

Center of Gravity $XG = \Sigma(Vi \cdot Xi) / \Sigma Vi =$ 0.335／ 0.670 ＝ 0.500 (m)

2))Uplift



Water Level Front Hf ＝ 0.000 (m)
Water Level Back Hr ＝ 1.170 (m)

Water Pressure at Front Footing Pf ＝ 3.822 (kN/m^2)
Water Pressure at Back Footing Pr ＝ 11.466 (kN/m^2)

Uplift

$$U = \frac{Pf + Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 7.644 \quad (kN)$$

Acting Position

$$X = \frac{Pf + 2 \cdot Pr}{3 \cdot (Pf + Pr)} \cdot Bj = 0.583 \quad (m)$$

Where、

Bj : Width of Footing Bj ＝ 1.000 (m)
Bc : Length of Footing Bc ＝ 1.000 (m)
λ : Reduction Coefficient of Buoyancy λ ＝ 1.000

[2] Seimic

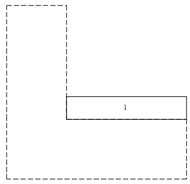
1) Weight of Soil

	Gravity Above Water Level (kN)	Gravity Below Water Level (kN)
Soil	0.916 × 18.000 ＝ 16.493	0.190 × 20.000 ＝ 3.800

	W Wu ＋ Wl (kN)	X (Wu・Xu＋Wl・Xl)/W (m)
Soil	20.293	0.521

[2]Seismic

1) Block



2) Volume, Center of Gravity

Part	With × Height × Depth	Volume $V_i(m^3)$	Center of Gravity (m)	$V_i \cdot X_i$	Remarks
1	1.000× 0.190× 1.000	0.190	0.500	0.095	
Σ		0.190	——	0.095	

Center of Gravity $XG = \Sigma(Vi \cdot Xi) / \Sigma Vi =$ 0.095／ 0.190 ＝ 0.500 (m)

4.4.3 Vertical Force

(a) Force by body

[1] Normal and Seismic Conditions

	Vertical Force W ＝ γ ・ V (kN)	Position X (m)
Body	24.000 × 0.500 ＝ 12.000	0.500

(b) Weight of soil, Uplift

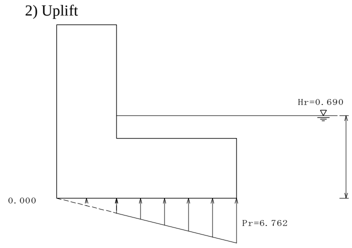
[1] Normal

1) Weight of Soil

	Gravity Above Water Surface (kN)	Gravity Below Water Surface (kN)
Soil	0.436 × 18.000 ＝ 7.853	0.670 × 20.000 ＝ 13.400

	W Wu ＋ Wl (kN)	X (Wu・Xu＋Wl・Xl)/W (m)
Soil	21.253	0.521

2) Uplift



Water Level Front Hf ＝ 0.000 (m)
Water Level Back Hr ＝ 0.690 (m)

Water Pressure at Front Footing Pf ＝ 2.254 (kN/m^2)
Water Pressure at Back Footing Pr ＝ 6.762 (kN/m^2)

Uplift

$$U = \frac{Pf + Pr}{2} \cdot Bj \cdot Bc \cdot \lambda = 4.508 \quad (kN)$$

$$X = \frac{Pf + 2 \cdot Pr}{3 \cdot (Pf + Pr)} \cdot Bj = 0.583 \quad (m)$$

Bj : Width of Footing Bj ＝ 1.000 (m)
Bc : Length of Footing Bc ＝ 1.000 (m)
λ : Reduction Coefficient of Buoyancy λ ＝ 1.000

(3) Summary of Weight

[1] Normal Condition

	Gravity Ni (kN)	Acting Position Xi (m)	Moment Ni・Xi (kN.m)
Body	12.000	0.500	6.000
Soil	21.253	0.520	11.051
Total	33.253	——	17.051

[2] Seismic

	Gravity Ni (kN)	Acting Position Xi (m)	Moment Ni・Xi (kN.m)
Body	12.000	0.500	6.000
Soil	20.293	0.521	10.575
Total	32.293	——	16.575

4.4.4 Earth Pressure

[1] Normal Condition

[1] Normal Condition

Earth Pressure

Position of Imaginary Back Side xp = 1.500 m

yp = 0.000 m

Height of Imaginary Back Side H = 1.450 m

Height Above Water Surface H1 = 0.280 m

Height Below Water Surface H2 = 1.170 m

Angel between Back Side Surface of Wall and Vertical Plane α = 0.000 °

Unit Weight of Backfill γs = 18.000 kN/m^3

Internal Friction Angel of Soil φ = 30.000 °

Angle between Ground Surface and Horizontal Plane β = 0.000 °

Wall Friction Angel δ = 30.000 °

Acting load by embankment

$$I_w = 1+\left(\frac{X}{H_w}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{X}{H_w}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_w}\right)-\frac{2}{\pi}\left(\frac{X}{H_w}\right)$$

$$= 1+\left(\frac{1.208}{1.450}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{1.208}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{1.208}{1.450}\right)-\frac{2}{\pi}\left(\frac{1.208}{1.450}\right)$$

$$= 0.41450$$

$$q_w = \gamma \cdot H_s \cdot I_w$$

$$= 18.000\times 0.806\times 0.41450$$

$$= 6.010 \text{ kN/m}^2$$

Converted Load

$q = q_w$

Where、

X : Distance of Imaginary Back Side(m), X1＋X2／2

X1 : Horizontal Length of Embankment(m), X1 = 0.000

X2 : Length of Slop (m), X2 = 2.417

H_w : Height of Body(m)

H_s : H_0＋ H_1(m)

H_0 : Height of Embankment(m), H_0 = 0.250

H_1 : Height of Conversion Embankment(m), H_1 = Q／γ = 0.556

Q : Surchage (kN/m^2), Q = 10.000

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cdot\cos(\alpha+\delta)\cdot\left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\cos(\alpha+\delta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ-0.000^\circ)}{\cos^2 0.000^\circ\cdot\cos(0.000^\circ+30.000^\circ)}$$

$$\times\frac{1}{\left[1+\sqrt{\frac{\sin(30.00^\circ+30.000^\circ)\cdot\sin(30.00^\circ-0.000^\circ)}{\cos(0.000^\circ+30.000^\circ)\cdot\cos(0.000^\circ-0.000^\circ)}}\right]^2}$$

$$= 0.2972$$

Earth Pressure at Top

p1= q・K =6.010×0.2972 = 1.786 kN/m^2

Earth Pressure on the Water Level

p2= K・γs・H1＋p1

= 0.2972×18.000×0.280＋1.786

= 3.284 kN/m^2

Earth Pressure at Bottom

p3= K・(γsat－γw)・H2＋p2

= 0.2972×(20.000－9.800)×1.170＋3.284

= 6.830 kN/m^2

Earth Pressure Force

$$P1=\frac{1}{2}\cdot(p1+p2)\cdot H1=\frac{1}{2}\times(1.786+3.284)\times 0.280 = 0.710 \text{ kN}$$

$$P2=\frac{1}{2}\cdot(p2+p3)\cdot H2=\frac{1}{2}\times(3.284+6.830)\times 1.170 = 5.917 \text{ kN}$$

Total Earth Pressure Force

P = P1＋P2 = 0.710＋5.917 = 6.627 kN

Vertical Element of Earth Pressure Force

Pv = P・sin(α＋δ) = 6.627×sin(0.000°＋30.000°) = 3.313 kN

Vertical Element of Earth Pressure

$$pv=\frac{2\cdot Pv}{L}=\frac{2\times 3.313}{1.000} = 6.627 \text{ kN/m}$$

Where、

pv : Equivalent Triangle Distributed Load

Pv : Vertical Element of Earth Pressure Force

L : Length of the Heel

Distance from Base to Control Point L1 = 0.000 m

Distributed Loading Width behind Control Point L2 = 1.000 m

Intensity of Distributed Load at Control Point

$$pd=\frac{pv}{L}\cdot L1=\frac{6.627}{1.000}\times 0.000 = 0.000 \text{ kN/m}$$

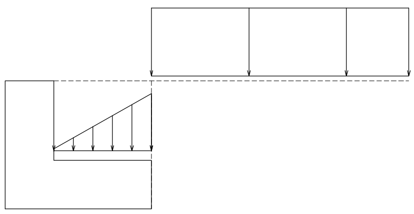
Vertical Force

$$N=\frac{1}{2}\cdot(pd+pv)\cdot L2=\frac{1}{2}\times(0.000+6.627)\times 1.000 = 3.313 \text{ kN}$$

Acting Position

$$x=\frac{pd+2\cdot pv}{pd+pv}\cdot\frac{L2}{3}=\frac{0.000+2\times 6.627}{0.000+6.627}\times\frac{1.000}{3} = 0.667 \text{ m}$$

Earth Pressure



[2] Seismic

Distance of Imaginary Back Side xp = 1.500 m

yp = 0.000 m

Height of Imaginary Back Side H = 1.450 m

Height Above Water Surface H1 = 0.760 m

Height Below Water Surface H2 = 0.690 m

Angle Between Back Side and Vertical Plane α = 0.000 °

Unit Weight of Soil γs = 18.000 kN/m^3

Internal Friction Angle of Soil φ = 30.000 °

Angle Between Ground Surface and Horizontal Plane β = 0.000 °

Angle of Wall Friction δ = 10.000 °

Acting Load by Embankment

$$I_w = 1+\left(\frac{X}{H_w}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{X}{H_w}\right)^2\right\}\tan^{-1}\left(\frac{X}{H_w}\right)-\frac{2}{\pi}\left(\frac{X}{H_w}\right)$$

$$= 1+\left(\frac{0.792}{1.450}\right)^2-\frac{2}{\pi}\left\{1+\left(\frac{0.792}{1.450}\right)^2\right\}\tan^{-1}\left(\frac{0.792}{1.450}\right)-\frac{2}{\pi}\left(\frac{0.792}{1.450}\right)$$

$$= 0.53752$$

$$q_w = \gamma \cdot H_s \cdot I_w$$

$$= 18.000\times 0.528\times 0.53752$$

$$= 5.106 \text{ kN/m}^2$$

$q = q_w$

Where、

X : Imaginary Distance (m), X1＋X2／2

X1 : Horizontal Length of Embankment(m), X1 = 0.000

X2 : Length (m), X2 = 1.583

H_w : Height of Body(m),

H_s : H_0＋ H_1(m)

H_0 : Height of Embankment(m), H_0 = 0.250

H_1 : Height of Conservation Embankment(m), H_1 = Q／γ = 0.278

Q : Surcharge(kN/m^2), Q = 5.000

Seismic Compound Angle

θ = tan^{-1}kh = tan^{-1}0.20 = 11.310 °

Coefficient of Active Earth Pressure

$$K = \frac{\cos^2(\phi-\alpha-\theta)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\alpha+\delta+\theta)\cdot\left[1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta-\theta)}{\cos(\alpha+\delta+\theta)\cdot\cos(\alpha-\beta)}}\right]^2}$$

$$= \frac{\cos^2(30.00^\circ-0.000^\circ-11.310^\circ)}{\cos 11.310^\circ\cdot\cos^2 0.000^\circ\cdot\cos(0.000^\circ+10.000^\circ+11.310^\circ)}$$

$$\times\frac{1}{\left[1+\sqrt{\frac{\sin(30.00^\circ+10.000^\circ)\cdot\sin(30.00^\circ-0.000^\circ-11.310^\circ)}{\cos(0.000^\circ+10.000^\circ+11.310^\circ)\cdot\cos(0.000^\circ-0.000^\circ)}}\right]^2}$$

$$= 0.4544$$

Earth Pressure at Top

p1= q・K = 5.106×0.4544 = 2.320 kN/m^2

Earth Pressure at Water Surface

p2= K・γs・H1＋p1

= 0.4544×18.000×0.760＋2.320

= 8.537 kN/m^2

Earth Pressure below Water Level

p3= p2 = 8.537 kN/m^2

Earth Pressure at Bottom

p4= K'・(γsat－γw)・H2＋p3

= 0.4544×(20.000－9.800)×0.690＋8.537

= 11.735 kN/m^2

Earth Pressure above Water Surface

$$P1=\frac{1}{2}\cdot(p1+p2)\cdot H1=\frac{1}{2}\times(2.320+8.537)\times 0.760 = 4.126 \text{ kN}$$

Earth Pressure below Water Surface

$$P2=\frac{1}{2}\cdot(p3+p4)\cdot H2=\frac{1}{2}\times(8.537+11.735)\times 0.690 = 6.994 \text{ kN}$$

Total Earth Pressure

P = P1＋P2 = 4.126＋6.994 = 11.120 kN

Horizontal Component of Earth Pressure Force

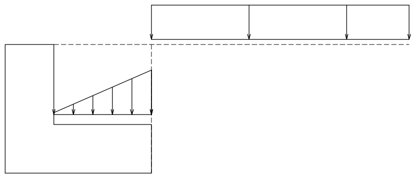
Pv = P・sin(α＋δ＋θ) = 11.120×sin(0.000°＋10.000°＋11.310°) = 4.041 kN

$$pv=\frac{2\cdot Pv}{L}=\frac{2\times 4.041}{1.000} = 8.082 \text{ kN/m}$$

Acting Position

$$x = \frac{pd+2\cdot pv}{pd+pv}\cdot\frac{L2}{3} = \frac{0.000+2\times 8.082}{0.000+8.082}\times\frac{1.000}{3} = 0.667\ \mathrm{m}$$

・Earth Pressure



4.4.5 Subgrade Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2)\cdot L$$

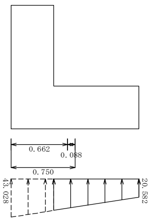
Acting Position

$$X = \frac{2\cdot q1+q2}{3\cdot(q1+q2)}\cdot L$$

Where、

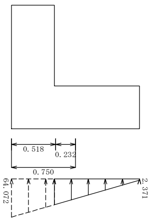
q1 : Intensity of Subgrade Reaction at front side of bottom slab(kN/m^2)
q2 : Intensity of Subgrade Reaction at control point (kN/m^2)
L : Acting Width(m)

[1]Normal



地盤反力度(kN/㎡)		作用幅 L (m)	鉛直力 N (kN)	作用位置 X (m)
q1	q2			
20.582	35.546	1.000	28.064	0.456

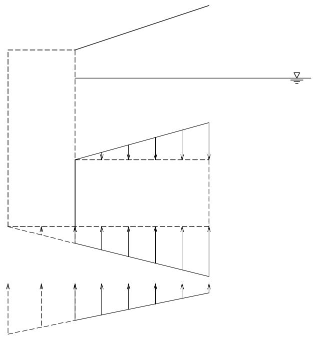
[2]地震時(水位2)



地盤反力度(kN/㎡)		作用幅 L (m)	鉛直力 N (kN)	作用位置 X (m)
q1	q2			
2.371	43.505	1.000	22.938	0.351

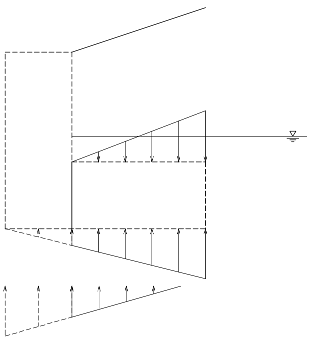
4.4.6 Summary Cross Section Force

[1] Normal



Load	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Body Weight	33.253	0.513	17.051
Uplift	-7.644	0.583	-4.459
Load	2.340	0.883	2.066
Earth Pressure Force	3.313	0.667	2.209
Resultant of Subgrade Reaction	-28.064	0.456	-12.785
Total	3.198	――	4.082

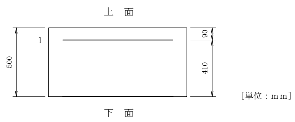
[2] Seismic



Load	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Body Weight	32.293	0.513	16.575
Uplift	-4.508	0.583	-2.630
Load	1.170	0.883	1.033
Earth Pressure Force	4.041	0.667	2.694
Resultant of Subgrade Reaction	-22.938	0.351	-8.041
Total	10.058	――	9.631

4.4.7 Stress Calculation

(1) Bar Arrangement



		Cover (cm)	Diameter	Area (cm^2/本)	Number	Cross Sectional Area (cm^2)
Upside	1	9.0	D12	1.267	4.00	4.525
	2	――	―	――	――	――
Downside	1'	――	―	――	――	――
	2'	――	―	――	――	――

Required Area of Reinforcement　0.339 (cm^2)

(2) Bending Stress Intensity

Conditions	M (kN.m)	x (cm)	Allowable Stress Intensity(concrete) (N/mm^2)	Allowable Stress Intensity (reinforcing bar) (N/mm^2)
Normal	1.916	5.386	0.181 ≦ 8.200	10.800 ≦ 140.000
Seismic	3.353	5.386	0.318 ≦ 12.300	18.901 ≦ 252.000

(3) Shear Stress Intensity

Conditions	Shear Force S_h(kN)	Effective Depth d(mm)	Shear Stress Intensity(N/mm^2)
Normal	3.198	410.000	0.008 ≦ 0.360
Seismic	10.058	410.000	0.025 ≦ 0.540

4.5 . Stability Against Bearing

Stability against bearing of wing wall is evaluated by yielding displacement of ground.
The yeilding displacement of ground is defined as allowable displacement for the ground acting as elastic body.

・Yeilding Displacement of Ground is determinded as follows;

Yeilding Displacement of Grou ≦ 5.0 cm
≦ 1.0% of Width of Foundation

	① Wdth of Foundation (cm)	② 1 % of ① (cm)				Allowable Relative Displacement (cm)
MSR4	150.0	1.5	<	5.0	cm	1.5

1) Intensity of Subgrade Reaction

(a) In case the center of affecting load is located in the core

Distributed in a Trapezoidal Shape

$$e<\frac{B}{6} \qquad q_{max}, q_{min} = \frac{V}{L \cdot B} \pm \frac{6M_B}{L \cdot B^2}$$

(b) In case the center of affecting load is located in the core

Distributed in a Triangle Shape

$$X = 3 \cdot \left(\frac{B}{2} - e \right)$$

where;

V: Vertilcal Load (kN)
M_B: Bending Moment Acting on centroid of foundation underside (kN/m)
e: Eccentric Distance (m)
X: Acting Width of Subgrade Reaction (m)

X < B, Distributed in Triangle Shape
X ≧ B, Distributed in Trapizoidal Shape

q_{max}: Maximum Intensity of Subgrade Reaction (kN/m^2)
q_{min}: Minimum Intensity of Subgrade Reaction (kN/m^2)
B: Foundation Width 1.5 (m)
L: Foudation Length 2.7 (m)

Case	1	2
	Normal	Seismic
L	2.7	2.7
B	1.5	1.5
V	128.809	134.548
M_B	11.363	31.236
e	0.088	0.232
X	1.986	1.554
q_{max}	43.027	64.072
q_{min}:	20.582	2.371

*Regarding Weight in Unit Length, Refer to calculation of L-type Wing Wall.

2) Vertical Displacement of Foundation UnderSide

Vertical displacement of foudation underside is calculated as follows;

$$\delta_V = \frac{1}{K_V} \cdot \frac{V}{A}$$

where;

δ_V: Vertilcal Displacement (m)
V: Vertical Load (kN)
A: Area of Foudation UnderSide 4.05 (m^2)
K_V: Vertical Coefficient of Subgrade Reactio Normal 7155.62 (kN/m^2)
Seismic 14311.24 (kN/m^2)

Case	1	2
	Normal	Seismic
K_v (kN/m^2)	7155.62	14311.24
δ_V (cm)	0.444	0.232
δ_a (cm)	1.500	1.500
Evaluation	OK	OK

4.5 . Stability Against Bearing

・Modulus of Subgrade Reaction

Modulus of subgrade reaction for calculation of longtitudial direction can be indicated by following equation.

$$Kv = Kvo \left(\frac{Bv}{0.3} \right)^{-3/4}$$

Where,

Kv : Modulus of Subgrade Reaction (kN/m^3)

Kvc : Coefficient of Vertical Reaction of soil (kN/m3)
Equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E0m obtained by various soil tests a

$$Kvo= \frac{1}{0.3} \alpha \cdot Eom$$

Bv : Conveted Loading Width of Foundation in a Direction Perpendicular to Load Action Direction

Eon : Modulus of Deformation of Soil for Designobtained by Soil Test or Equation as Shown in Table Below (kN/m^2)

Eom and α

Modulus of Deformation Eom (kN/m2)	α	
	Normal	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eom=2800N with N-value in Standard Penetration Test	1	2

α : Coefficient Given by the Table Above

Av : Vertical Load Action Area (m^2)

D : Width of Bottom Slab (m)

β : Characteristic Value of Culvert (m^{-1})

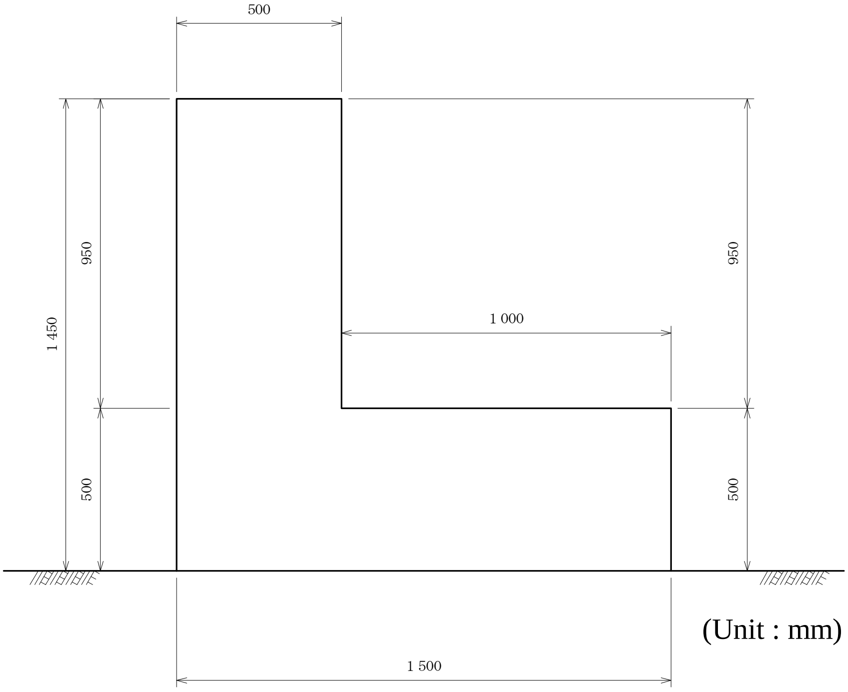
$$\beta= \{ (Kv \cdot D) / (4 \cdot E \cdot I) \}^{1/4}$$

Conveted Loading Width of Foundation Bv(m)

Evaluatioion of Culvert	Bv	Remarks
Rigid Body	$\sqrt{Av}$	$\beta \cdot L \leqq 1.5$
Elastic Body	$\sqrt{D/\beta}$	$\beta \cdot L > 1.5$

In Clculation of Bv of Clvert, Kv in Nomal Condition is Applied.

4.5 . Calculation of Scond Moment of Area



(Unit : mm)

	Dimension				$A(m^2)$	Y(m)	$A \cdot Y(m^3)$
	Width（B）	Height（h）	α	n			
①	0.500	0.950	1.0	1	0.475	0.975	0.463
②	1.500	0.500	1.0	1	0.750	0.250	0.188
計					1.225		0.651

α；Triangle: 0.5, Square: 1.0

ｎ； Number of Element

$Y = \Sigma A \cdot Y / \Sigma A = 0.531$ m

4.5 . Stability Against Bearing

Second Moment of Area

$$I_x = i_x + A \cdot Y^2$$

Where,

I_x ; Second Moment of Area at Centroid (m^4)

i_x ; Second Moment of Area Regarding Each Member (m^4)

A ; Area of Each Member (m^2)

Y ; Distance between Each Centroid and Combined Centroid (m)

	ix (m^4)	A (m^2)	Y (m)	Y^2 (m^2)	$A \cdot Y^2$ (m^4)	$ix + A \cdot Y^2$ (m^4)
①	0.03572	0.475	−0.444	0.197	0.0936	0.1293
②	0.01563	0.750	0.281	0.079	0.0593	0.0749
Total		1.225				0.2042

Area	**A=**	1.225	m^2
Second Moment	**Ix=**	0.2042	m^4

4.5 . Stability Against Bearing

a. Coefficient of Vertical Reaction of soil

E・I: Bending Rigidity of Culvert ($kN \cdot m^2$)

E= 2.45E+07 kN/m^2

I= 0.2042 m^4

◇ Nomal Conditon

Kvo= 1/0.3×α×Eom

= 1 / 0.3 × 4 × 2236.99 = 29826.5 kN/m^3

$\beta = \{(Kv \cdot D)/(4 \cdot E \cdot I)\}^{1/4}$

= 0.15218

β・L= 0.411 ≦ 1.5 Threrefore, this should be assumed as rigid body.

Calculation of Bv

Case1 Rigid Body	$Bv = \sqrt{L \cdot D}$	$= \sqrt{2.70 \times 1.50}$	=	$\boxed{2.012}$	m
Case2 Elastic body	$Bv = \sqrt{D / \beta}$	$= \sqrt{1.50 / 0.15218}$	=	3.140	m

$Kv = Kvo \times (Bv/0.3)^{-3/4}$

$= 29826.5 \times (2.012 / 0.3)^{-3/4} = 7156 \quad (kN/m^3)$

4.5 . Stability Against Bearing

Calculation Results of Converted Modulus of Deformation

$$Eom = \frac{\log \dfrac{(B+2h_n \cdot \tan\theta)\, L}{(L+2h_n \cdot \tan\theta)\, B}}{\sum_{i=1}^{n} \dfrac{1}{Ei} \log \dfrac{(B+2h_i \cdot \tan\theta)\,(L+2\,h_{i-1} \cdot \tan\theta)}{(L+2h_i \cdot \tan\theta)\,(B+2\,h_{i-1} \cdot \tan\theta)}} = \frac{f\,n}{\sum_{i=1}^{n} \dfrac{1}{Ei} \times f\,i}$$

Eom ;	Converted Modulus of Deformation in B≠L			(kN/m^2)
B ;	Loading Width = Width of Wing Wall	B=	1.500	m
L ;	Loading Length = Length of a span	L=	2.700	m
h_n ;	Depth for Calculation	hn=	4.500	m
h_i ;	Depth from the ground to the bottom Elevation of Each Layer (m)			
Ei ;	onverted Modulus of Deformation of Each Layer (kN/m^2)			
θ ;	Angle of Load Dispersion (θ=30°)			

	Eom (kN/m^2)	B (m)	L (m)	h_n (m)	Tickness of Layer (m)	h i (m)	Ei (kN/m^2)	θ (°)	f n	f i	1 / E i × f i
1	2236.99	1.500	2.700	4.500	4.450	4.450	2230	30	0.4229	0.422	0.0001891
2											
3											
4											
5											
6											
7											
8											
9											
10											
				Total	4.450					Total	0.0001891