REPUBLIC OF THE UNION OF MYANMAR
MINISTRY OF CONSTRUCTION
DEPARTMENT OF BRIDGE

DETAILED DESIGN STUDY ON THE BAGO RIVER BRIDGE CONSTRUCTION PROJECT

FINAL REPORT ATTACHMENTS

VOLUME II DESIGN REPORT

Part I Road Design

DECEMBER 2017

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

NIPPON KOEI CO., LTD.
ORIENTAL CONSULTANTS GLOBAL CO., LTD.
METROPOLITAN EXPRESSWAY COMPANY LIMITED.
CHODAI CO., LTD.
NIPPON ENGINEERING CONSULTANTS CO., LTD.

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ROAD DESIGN

ALIGNMENT CALCULATION REPORT

Definitive Alignment Calculation of Bago River Bridge Construction Project
Main Highway

POINT NAME	STATION	NORTHING X-COORDINATE	EASTING Y-COORDINATE	ELEMENT	AZIMUTH ANGLE	ELEMENT LENGTH	ACCUMULATED DISTANCE
BP	0+000.000000	1857219.291051	205789.549518				0.000000
KA 1-1	0+024.969805	1857233.508737	205769.022741	STRAIGHT LINE	304° 42' 29.009669"	24.969805	24.969805
KE 1-1	0+076.169805	1857263.372323	205727.441550	CLOTHOID A=160	307° 38' 29.767749"	51.200000	76.169805
KE 1-2	0+161.512727	1857320.993624	205664.628061	CIRCLE R=500	317° 25' 16.250510"	85.342923	161.512727
KA 1-2	0+212.712727	1857359.850350	205631.296633	CLOTHOID A=160	320° 21' 17.008590"	51.200000	212.712727
BC 2	0+521.900231	1857597.927606	205434.024909	STRAIGHT LINE	320° 21' 17.008590"	309.187504	521.900231
EC 2	0+857.521703	1857873.073202	205242.524037	CIRCLE R=2000	329° 58' 10.457547"	335.621472	857.521703
KA 3-1	2+627.420376	1859405.380223	204356.760802	STRAIGHT LINE	329° 58' 10.457547"	1769.898673	2627.420376
KE 3-1	2+680.991804	1859452.311131	204330.947038	CLOTHOID A=150	333° 37' 25.100803"	53.571429	2680.991804
KE 3-2	2+724.079800	1859491.826837	204313.816465	CIRCLE R=420	339° 30' 5.903241"	43.087995	2724.079800
KA 3-2	2+777.651228	1859542.749064	204297.209619	CLOTHOID A=150	343° 9' 20.546495"	53.571429	2777.651228
KA 4-1	2+782.485673	1859547.376091	204295.808734	STRAIGHT LINE	343° 9' 20.546495"	4.834445	2782.485673
KE 4-1	2+835.298173	1859597.467560	204279.125895	CLOTHOID A=130	338° 25' 39.671372"	52.812500	2835.298173
KE 4-2	2+961.570619	1859702.829467	204211.024695	CIRCLE R=320	315° 49' 7.291643"	126.272446	2961.570619
KA 4-2	3+014.383119	1859738.611303	204172.202890	CLOTHOID A=130	311° 5' 26.416517"	52.812500	3014.383119
EP	3+575.000000	1860107.078174	203749.682533	STRAIGHT LINE	311° 5' 26.416517"	560.616881	3575.000000

IP NO.	NORTHING X-COORDINATE	EASTING Y-COORDINATE
BP	1857219.291051	205789.549518
IP-1	1857287.217907	205691.480153
IP-2	1857727.447522	205326.704212
IP-3	1859470.623141	204319.046573
IP-4	1859660.753366	204261.482458
EP	1860107.078174	203749.682533

Definitive Alignment Calculation of Bago River Bridge Construction Project
Star City Approach Road to Bago River Bridge

POINT NAME	STATION	NORTHING X-COORDINATE	EASTING Y-COORDINATE	ELEMENT		AZIMUTH ANGLE	ELEMENT LENGTH	ACCUMULATED DISTANCE
BP	0+000.000000	1857586.250773	205393.281977					0.000000
BC-1	0+004.471511	1857589.735828	205396.083549	STRAIGHT LINE		38° 47' 42.593542"	4.471511	4.471511
EC-1	0+058.044963	1857624.134584	205436.728193	CIRCLE	R=140	60° 43' 13.433109"	53.573451	58.044963
KA 2-1	0+105.007058	1857647.102428	205477.690573	STRAIGHT LINE		60° 43' 13.433109"	46.962095	105.007058
KE 2-1	0+148.110506	1857663.282883	205517.356898	CLOTHOID	A=50	82° 0' 37.609033"	43.103448	148.110506
KE 2-2	0+367.483423	1857554.981013	205497.547926	CIRCLE	R=58	298° 43' 11.268296"	219.372917	367.483423
KA 2-2	0+410.586871	1857584.154535	205466.177078	CLOTHOID	A=50	320° 0' 35.444221"	43.103448	410.586871
BC-3	0+535.778322	1857680.070576	205385.722045	STRAIGHT LINE		320° 0' 35.444221"	125.191450	535.778322
EP	0+643.083345	1857765.821505	205321.300759	CIRCLE	R=1000	326° 9' 28.675974"	107.305023	643.083345

IP NO.	NORTHING X-COORDINATE	EASTING Y-COORDINATE
BP	1857586.250773	205393.281977
IP-1	1857610.871707	205413.074295
IP-2	1857622.576145	205433.948768
IP-3	1857721.216198	205351.208816
EP	1857765.821505	205321.300759

Definitive Alignment Calculation of Bago River Bridge Construction Project
Approach Road from the Toll Plaza to Shukhinthar Mayopat Road

POINT NAME	STATION	NORTHING X-COORDINATE	EASTING Y-COORDINATE	ELEMENT		AZIMUTH ANGLE	ELEMENT LENGTH	ACCUMULATED DISTANCE
BP	0+000.000000	1859387.083266	204379.898737					0.000000
KA 1-1	0+027.420376	1859410.822724	204366.175940	STRAIGHT LINE		329° 58' 10.457547"	27.420376	27.420376
KE 1-1	0+080.298246	1859457.142519	204340.689898	CLOTHOID	A=147.083849	333° 40' 19.933366"	52.877870	80.298246
KE 1-2	0+122.270570	1859495.635051	204324.002884	CIRCLE	R=409.125000	339° 33' 0.735807"	41.972324	122.270570
KA 1-2	0+175.148440	1859545.900332	204307.618036	CLOTHOID	A=147.083849	343° 9' 20.546495"	52.877870	175.148440
KA 2-1	0+179.982885	1859550.527359	204306.217150	STRAIGHT LINE		343° 9' 20.546495"	4.834445	179.982885
EP	0+228.545623	1859596.665312	204291.095464	CLOTHOID	A=132.996909	341° 1' 10.050264"	48.562738	228.545623

IP NO.	NORTHING X-COORDINATE	EASTING Y-COORDINATE
BP	1859387.083266	204379.898737
IP-1	1859474.977428	204329.090762
IP-2	1859581.403548	204296.869029
EP	1859596.665312	204291.095464

Definitive Alignment Calculation of Bago River Bridge Construction Project
Approach Road from Shukhinthar Mayopat Road to the Toll Plaza

POINT NAME	STATION	NORTHING X-COORDINATE	EASTING Y-COORDINATE	ELEMENT		AZIMUTH ANGLE	ELEMENT LENGTH	ACCUMULATED DISTANCE
BP	0+000.000000	1859376.198265	204361.068463					0.000000
KA 1-1	0+027.420376	1859399.937722	204347.345665	STRAIGHT LINE		329° 58' 10.457547"	27.420376	27.420376
KE 1-1	0+081.685363	1859447.479743	204321.204179	CLOTHOID	A=152.909864	333° 34' 39.093555"	54.264987	81.685363
KE 1-2	0+125.889030	1859488.018623	204303.630046	CIRCLE	R=430.875000	339° 27' 19.895993"	44.203667	125.889030
KA 1-2	0+180.154017	1859539.597796	204286.801203	CLOTHOID	A=152.909864	343° 9' 20.546495"	54.264987	180.154017
KA 2-1	0+184.988462	1859544.224823	204285.400317	STRAIGHT LINE		343° 9' 20.546495"	4.834445	184.988462
EP	0+219.973050	1859577.579400	204274.853056	CLOTHOID	A=127.777631	341° 0' 29.487429"	34.984588	219.973050

IP NO.	NORTHING X-COORDINATE	EASTING Y-COORDINATE
BP	1859376.198265	204361.068463
IP-1	1859466.268854	204309.002385
IP-2	1859566.613097	204278.622010
EP	1859577.579400	204274.853056

Definitive Alignment Calculation of Bago River Bridge Construction Project
Relocation Road between Shukhinthar Mayopat Road and Yangon - Thanlyin Bridge

POINT NAME	STATION	NORTHING X-COORDINATE	EASTING Y-COORDINATE	ELEMENT		AZIMUTH ANGLE	ELEMENT LENGTH	ACCUMULATED DISTANCE
BP	0+000.000000	1859592.452945	204287.866125					0.000000
BC 1	0+024.306092	1859591.810075	204312.163713	STRAIGHT LINE		91° 30' 56.114217"	24.306092	24.306092
EC-1	0+063.975512	1859568.426887	204340.633817	CIRCLE	R=30	167° 16' 42.957166"	39.669420	63.975512
to Yangon-Thanlyin Bridge								
BC-2	0+115.859871	1859517.816166	204352.059301	STRAIGHT LINE		167° 16' 42.957166"	51.884359	115.859871
EP	0+168.655834	1859474.391353	204380.139144	CIRCLE	R=75	126° 56' 43.638429"	52.795963	168.655834
from Yangon-Thanlyin Bridge								
BC-2	0+102.399065	1859530.946541	204349.095089	STRAIGHT LINE		167° 16' 42.957166"	38.423553	102.399065
EP	0+164.083776	1859492.287742	204392.126807	CIRCLE	R=50	96° 35' 35.259250"	61.684711	164.083776

IP NO.	NORTHING X-COORDINATE	EASTING Y-COORDINATE
BP	1859592.452945	204287.866125
IP-1	1859591.192789	204335.494363
to Yangon-Thanlyin Bridge		
IP-2	1859490.947372	204358.124992
EP	1859474.391353	204380.139144
from Yangon-Thanlyin Bridge		
IP-2	1859496.358958	204356.903314
EP	1859492.287742	204392.126807

Bago River Bridge Construction Project
Vertical Alignment Calculation Report

1 Main Highway

1.1 Point of Vertical Intersection

PVI	Station	Elevation (m)	Tangent (%)	Vertical Curve Length (m)	Vertical Curve Radius (m)
1	0+ 0.000000	5.695000	-	-	-
2	0+228.000000	5.467000	-0.100000 %	240	9,200
3	0+700.000000	17.267000	2.500000 %	296	13,500
4	1+088.000000	18.431000	0.300000 %	480	80,000
5	2+140.000000	15.275000	-0.300000 %	200	9,100
6	2+517.727273	5.831818	-2.500000 %	300	5,500
7	2+830.000000	15.200000	3.000000 %	130	5,200
8	2+960.000000	15.850000	0.500000 %	60	4,900
9	3+160.000000	14.420000	-0.715000 %	100	4,400
10	3+475.000000	4.970000	-3.000000 %	50	1,900
11	3+500.000000	4.895000	-0.300000 %	-	-

1.2 Station Increment Report

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 0.000	5.695000	-0.1000	0.0000	0.000000	
0 + 20.000	5.675000	-0.1000	20.0000	20.000000	
0 + 24.970	5.670030	-0.1000	4.9698	24.969805	KA 1-1
0 + 40.000	5.655000	-0.1000	15.0302	40.000000	
0 + 60.000	5.635000	-0.1000	20.0000	60.000000	
0 + 76.170	5.618830	-0.1000	16.1698	76.169805	KE 1-1
0 + 80.000	5.615000	-0.1000	3.8302	80.000000	
0 + 100.000	5.595000	-0.1000	20.0000	100.000000	

VIP 1 Acc.Dis.= 228.0000 EL.= 5.4670 VCL = 240.0000
I1 = -0.1000% I2 = 2.5000% R = 9230 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 108.000	5.587000	-0.1000	8.0000	108.000000	
0 + 120.000	5.582800	0.0300	12.0000	120.000000	
0 + 140.000	5.610467	0.2467	20.0000	140.000000	
0 + 160.000	5.681467	0.4633	20.0000	160.000000	
0 + 161.513	5.688600	0.4797	1.5127	161.512727	KE1-2
0 + 180.000	5.795800	0.6800	18.4873	180.000000	
0 + 200.000	5.953467	0.8967	20.0000	200.000000	
0 + 212.713	6.076212	1.0344	12.7127	212.712727	KA 1-2
0 + 220.000	6.154467	1.1133	7.2873	220.000000	
0 + 228.000	6.247000	1.2000	8.0000	228.000000	
0 + 240.000	6.398800	1.3300	12.0000	240.000000	

0 + 260.000	6.686467	1.5467	20.0000	260.000000	
0 + 280.000	7.017467	1.7633	20.0000	280.000000	
0 + 300.000	7.391800	1.9800	20.0000	300.000000	
0 + 320.000	7.809467	2.1967	20.0000	320.000000	
0 + 340.000	8.270467	2.4133	20.0000	340.000000	
0 + 348.000	8.467000	2.5000	8.0000	348.000000	
0 + 360.000	8.767000	2.5000	12.0000	360.000000	
0 + 380.000	9.267000	2.5000	20.0000	380.000000	
0 + 400.000	9.767000	2.5000	20.0000	400.000000	
0 + 420.000	10.267000	2.5000	20.0000	420.000000	
0 + 440.000	10.767000	2.5000	20.0000	440.000000	
0 + 460.000	11.267000	2.5000	20.0000	460.000000	
0 + 480.000	11.767000	2.5000	20.0000	480.000000	
0 + 500.000	12.267000	2.5000	20.0000	500.000000	
0 + 520.000	12.767000	2.5000	20.0000	520.000000	
0 + 521.900	12.814506	2.5000	1.9002	521.900231	BC 2
0 + 540.000	13.267000	2.5000	18.0998	540.000000	

VIP 2 Acc.Dis.= 700.0000 EL = 17.2670 VCL = 296.0000
I1 = 2.5000% I2 = 0.3000% R =13454 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 552.000	13.567000	2.5000	12.0000	552.000000
0 + 560.000	13.764622	2.4405	8.0000	560.000000
0 + 580.000	14.237865	2.2919	20.0000	580.000000
0 + 600.000	14.681378	2.1432	20.0000	600.000000
0 + 620.000	15.095162	1.9946	20.0000	620.000000
0 + 640.000	15.479216	1.8459	20.0000	640.000000
0 + 660.000	15.833541	1.6973	20.0000	660.000000
0 + 680.000	16.158135	1.5486	20.0000	680.000000
0 + 700.000	16.453000	1.4000	20.0000	700.000000
0 + 720.000	16.718135	1.2514	20.0000	720.000000
0 + 740.000	16.953541	1.1027	20.0000	740.000000
0 + 760.000	17.159216	0.9541	20.0000	760.000000
0 + 780.000	17.335162	0.8054	20.0000	780.000000
0 + 800.000	17.481378	0.6568	20.0000	800.000000
0 + 820.000	17.597865	0.5081	20.0000	820.000000
0 + 840.000	17.684622	0.3595	20.0000	840.000000
0 + 848.000	17.711000	0.3000	8.0000	848.000000

VIP 3 Acc.Dis.= 1088.0000 EL.= 18.4310 VCL = 480.0000
I1 = 0.3000% I2 = -0.3000% R =80000 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 848.000	17.711000	0.3000	0.0000	848.000000	
0 + 857.522	17.738998	0.2881	9.5217	857.521703	EC 2
0 + 860.000	17.746100	0.2850	2.4783	860.000000	
0 + 880.000	17.800600	0.2600	20.0000	880.000000	
0 + 900.000	17.850100	0.2350	20.0000	900.000000	
0 + 920.000	17.894600	0.2100	20.0000	920.000000	
0 + 940.000	17.934100	0.1850	20.0000	940.000000	
0 + 960.000	17.968600	0.1600	20.0000	960.000000	
0 + 980.000	17.998100	0.1350	20.0000	980.000000	
1 + 0.000	18.022600	0.1100	20.0000	1000.000000	
1 + 20.000	18.042100	0.0850	20.0000	1020.000000	
1 + 40.000	18.056600	0.0600	20.0000	1040.000000	
1 + 60.000	18.066100	0.0350	20.0000	1060.000000	
1 + 80.000	18.070600	0.0100	20.0000	1080.000000	
1 + 88.000	18.071000	0.0000	8.0000	1088.000000	
1 + 100.000	18.070100	-0.0150	12.0000	1100.000000	
1 + 120.000	18.064600	-0.0400	20.0000	1120.000000	
1 + 140.000	18.054100	-0.0650	20.0000	1140.000000	
1 + 160.000	18.038600	-0.0900	20.0000	1160.000000	
1 + 180.000	18.018100	-0.1150	20.0000	1180.000000	
1 + 200.000	17.992600	-0.1400	20.0000	1200.000000	
1 + 220.000	17.962100	-0.1650	20.0000	1220.000000	
1 + 240.000	17.926600	-0.1900	20.0000	1240.000000	
1 + 260.000	17.886100	-0.2150	20.0000	1260.000000	
1 + 280.000	17.840600	-0.2400	20.0000	1280.000000	
1 + 300.000	17.790100	-0.2650	20.0000	1300.000000	
1 + 320.000	17.734600	-0.2900	20.0000	1320.000000	
1 + 328.000	17.711000	-0.3000	8.0000	1328.000000	

Station	Elevation	Tangent (%)	Distance	Acc.Distance
1 + 340.000	17.675000	-0.3000	12.0000	1340.000000
1 + 360.000	17.615000	-0.3000	20.0000	1360.000000
1 + 380.000	17.555000	-0.3000	20.0000	1380.000000
1 + 400.000	17.495000	-0.3000	20.0000	1400.000000
1 + 420.000	17.435000	-0.3000	20.0000	1420.000000
1 + 440.000	17.375000	-0.3000	20.0000	1440.000000
1 + 460.000	17.315000	-0.3000	20.0000	1460.000000
1 + 480.000	17.255000	-0.3000	20.0000	1480.000000
1 + 500.000	17.195000	-0.3000	20.0000	1500.000000
1 + 520.000	17.135000	-0.3000	20.0000	1520.000000

1 + 540.000	17.075000	-0.3000	20.0000	1540.000000
1 + 560.000	17.015000	-0.3000	20.0000	1560.000000
1 + 580.000	16.955000	-0.3000	20.0000	1580.000000
1 + 600.000	16.895000	-0.3000	20.0000	1600.000000
1 + 620.000	16.835000	-0.3000	20.0000	1620.000000
1 + 640.000	16.775000	-0.3000	20.0000	1640.000000
1 + 660.000	16.715000	-0.3000	20.0000	1660.000000
1 + 680.000	16.655000	-0.3000	20.0000	1680.000000
1 + 700.000	16.595000	-0.3000	20.0000	1700.000000
1 + 720.000	16.535000	-0.3000	20.0000	1720.000000
1 + 740.000	16.475000	-0.3000	20.0000	1740.000000
1 + 760.000	16.415000	-0.3000	20.0000	1760.000000
1 + 780.000	16.355000	-0.3000	20.0000	1780.000000
1 + 800.000	16.295000	-0.3000	20.0000	1800.000000
1 + 820.000	16.235000	-0.3000	20.0000	1820.000000
1 + 840.000	16.175000	-0.3000	20.0000	1840.000000
1 + 860.000	16.115000	-0.3000	20.0000	1860.000000
1 + 880.000	16.055000	-0.3000	20.0000	1880.000000
1 + 900.000	15.995000	-0.3000	20.0000	1900.000000
1 + 920.000	15.935000	-0.3000	20.0000	1920.000000
1 + 940.000	15.875000	-0.3000	20.0000	1940.000000
1 + 960.000	15.815000	-0.3000	20.0000	1960.000000
1 + 980.000	15.755000	-0.3000	20.0000	1980.000000
2 + 0.000	15.695000	-0.3000	20.0000	2000.000000
2 + 20.000	15.635000	-0.3000	20.0000	2020.000000

VIP 4 Acc.Dis.= 2140.0000 EL.= 15.2750 VCL = 200.0000
I1 = -0.3000% I2 = -2.5000% R = 9090 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
2 + 40.000	15.575000	-0.3000	20.0000	2040.000000
2 + 60.000	15.493000	-0.5200	20.0000	2060.000000
2 + 80.000	15.367000	-0.7400	20.0000	2080.000000
2 + 100.000	15.197000	-0.9600	20.0000	2100.000000
2 + 120.000	14.983000	-1.1800	20.0000	2120.000000
2 + 140.000	14.725000	-1.4000	20.0000	2140.000000
2 + 160.000	14.423000	-1.6200	20.0000	2160.000000
2 + 180.000	14.077000	-1.8400	20.0000	2180.000000
2 + 200.000	13.687000	-2.0600	20.0000	2200.000000
2 + 220.000	13.253000	-2.2800	20.0000	2220.000000
2 + 240.000	12.775000	-2.5000	20.0000	2240.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance
2 + 260.000	12.275000	-2.5000	20.0000	2260.000000
2 + 280.000	11.775000	-2.5000	20.0000	2280.000000
2 + 300.000	11.275000	-2.5000	20.0000	2300.000000
2 + 320.000	10.775000	-2.5000	20.0000	2320.000000
2 + 340.000	10.275000	-2.5000	20.0000	2340.000000
2 + 360.000	9.775000	-2.5000	20.0000	2360.000000

VIP 5 Acc.Dis.= 2517.7273 EL.= 5.8318 VCL = 300.0000
I1 = -2.5000% I2 = 3.0000% R = 5454 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
2 + 367.727	9.581818	-2.5000	7.7273	2367.727273	
2 + 380.000	9.288807	-2.2750	12.2727	2380.000000	
2 + 400.000	8.870473	-1.9083	20.0000	2400.000000	
2 + 420.000	8.525473	-1.5417	20.0000	2420.000000	
2 + 440.000	8.253807	-1.1750	20.0000	2440.000000	
2 + 460.000	8.055473	-0.8083	20.0000	2460.000000	
2 + 480.000	7.930473	-0.4417	20.0000	2480.000000	
2 + 500.000	7.878807	-0.0750	20.0000	2500.000000	
2 + 517.727	7.894318	0.2500	17.7273	2517.727273	
2 + 520.000	7.900473	0.2917	2.2727	2520.000000	
2 + 540.000	7.995473	0.6583	20.0000	2540.000000	
2 + 560.000	8.163807	1.0250	20.0000	2560.000000	
2 + 580.000	8.405473	1.3917	20.0000	2580.000000	
2 + 600.000	8.720473	1.7583	20.0000	2600.000000	
2 + 620.000	9.108807	2.1250	20.0000	2620.000000	
2 + 627.420	9.271537	2.2610	7.4204	2627.420376	KA 3-1
2 + 640.000	9.570473	2.4917	12.5796	2640.000000	
2 + 660.000	10.105473	2.8583	20.0000	2660.000000	
2 + 667.727	10.331818	3.0000	7.7273	2667.727273	

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
2 + 680.000	10.700000	3.0000	12.2727	2680.000000	
2 + 680.992	10.729754	3.0000	0.9918	2680.991804	KE 3-1
2 + 700.000	11.300000	3.0000	19.0082	2700.000000	
2 + 720.000	11.900000	3.0000	20.0000	2720.000000	
2 + 724.080	12.022394	3.0000	4.0798	2724.079800	KE 3-2
2 + 740.000	12.500000	3.0000	15.9202	2740.000000	
2 + 760.000	13.100000	3.0000	20.0000	2760.000000	

VIP 6 Acc.Dis.= 2830.0000 EL.= 15.2000 VCL = 130.0000
I1 = 3.0000% I2 = 0.5000% R = 5200 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
2 + 765.000	13.250000	3.0000	5.0000	2765.000000	
2 + 777.651	13.614147	2.7567	12.6512	2777.651228	KA 3-2
2 + 780.000	13.678365	2.7115	2.3488	2780.000000	
2 + 782.486	13.745171	2.6637	2.4857	2782.485673	KA 4-1
2 + 800.000	14.182212	2.3269	17.5143	2800.000000	
2 + 820.000	14.609135	1.9423	20.0000	2820.000000	
2 + 830.000	14.793750	1.7500	10.0000	2830.000000	
2 + 835.298	14.883769	1.6481	5.2982	2835.298173	KE 4-1
2 + 840.000	14.959135	1.5577	4.7018	2840.000000	
2 + 860.000	15.232212	1.1731	20.0000	2860.000000	
2 + 880.000	15.428365	0.7885	20.0000	2880.000000	
2 + 895.000	15.525000	0.5000	15.0000	2895.000000	

Station	Elevation	Tangent (%)	Distance	Acc.Distance
2 + 900.000	15.550000	0.5000	5.0000	2900.000000
2 + 920.000	15.650000	0.5000	20.0000	2920.000000

VIP 7 Acc.Dis.= 2960.0000 EL.= 15.8500 VCL = 60.0000
I1 = 0.5000% I2 = -0.7150% R = 4938 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
2 + 930.000	15.700000	0.5000	10.0000	2930.000000	
2 + 940.000	15.739875	0.2975	10.0000	2940.000000	
2 + 960.000	15.758875	-0.1075	20.0000	2960.000000	
2 + 961.571	15.756937	-0.1393	1.5706	2961.570619	KE 4-2
2 + 980.000	15.696875	-0.5125	18.4294	2980.000000	
2 + 990.000	15.635500	-0.7150	10.0000	2990.000000	

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
3 + 0.000	15.564000	-0.7150	10.0000	3000.000000	
3 + 14.383	15.461161	-0.7150	14.3831	3014.383119	KA 4-2
3 + 20.000	15.421000	-0.7150	5.6169	3020.000000	
3 + 40.000	15.278000	-0.7150	20.0000	3040.000000	
3 + 60.000	15.135000	-0.7150	20.0000	3060.000000	
3 + 80.000	14.992000	-0.7150	20.0000	3080.000000	
3 + 100.000	14.849000	-0.7150	20.0000	3100.000000	

VIP 8 Acc.Dis.= 3160.0000 EL.= 14.4200 VCL = 100.0000
I1 = -0.7150% I2 = -3.0000% R = 4376 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
3 + 110.000	14.777500	-0.7150	10.0000	3110.000000
3 + 120.000	14.694575	-0.9435	10.0000	3120.000000
3 + 140.000	14.460175	-1.4005	20.0000	3140.000000
3 + 160.000	14.134375	-1.8575	20.0000	3160.000000
3 + 180.000	13.717175	-2.3145	20.0000	3180.000000
3 + 200.000	13.208575	-2.7715	20.0000	3200.000000
3 + 210.000	12.920000	-3.0000	10.0000	3210.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance
3 + 220.000	12.620000	-3.0000	10.0000	3220.000000
3 + 240.000	12.020000	-3.0000	20.0000	3240.000000
3 + 260.000	11.420000	-3.0000	20.0000	3260.000000
3 + 280.000	10.820000	-3.0000	20.0000	3280.000000
3 + 300.000	10.220000	-3.0000	20.0000	3300.000000
3 + 320.000	9.620000	-3.0000	20.0000	3320.000000
3 + 340.000	9.020000	-3.0000	20.0000	3340.000000
3 + 360.000	8.420000	-3.0000	20.0000	3360.000000
3 + 380.000	7.820000	-3.0000	20.0000	3380.000000
3 + 400.000	7.220000	-3.0000	20.0000	3400.000000
3 + 420.000	6.620000	-3.0000	20.0000	3420.000000
3 + 440.000	6.020000	-3.0000	20.0000	3440.000000

VIP 9 Acc.Dis.= 3475.0000 EL.= 4.9700 VCL = 50.0000
I1 = -3.0000% I2 = -0.3000% R = 1851 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
3 + 450.000	5.720000	-3.0000	10.0000	3450.000000
3 + 460.000	5.447000	-2.4600	10.0000	3460.000000
3 + 475.000	5.138750	-1.6500	15.0000	3475.000000
3 + 480.000	5.063000	-1.3800	5.0000	3480.000000
3 + 500.000	4.895000	-0.3000	20.0000	3500.000000

2 Star City Approach Road to Bago River Bridge

2.1 Point of Vertical Intersection Report

PVI	Station	Elevation (m)	Tangent (%)	Vertical Curve Length (m)	Vertical Curve Radius (m)
1	0+ 0.000000	4.470000	-	-	-
2	0+150.000000	4.470000	0.000000 %	30	10,000
3	0+329.942096	5.009826	0.300000 %	60	1,158
4	0+490.000000	13.779667	5.479168 %	60	1,827
5	0+540.000000	14.878003	2.196672 %	-	-

2.2 Station Increment Report

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 0.000	4.470000	0.0000	0.0000	0.000000	
0 + 4.472	4.470000	0.0000	4.4715	4.471511	BC-1
0 + 20.000	4.470000	0.0000	15.5285	20.000000	
0 + 40.000	4.470000	0.0000	20.0000	40.000000	
0 + 58.045	4.470000	0.0000	18.0450	58.044963	EC-1
0 + 60.000	4.470000	0.0000	1.9550	60.000000	
0 + 80.000	4.470000	0.0000	20.0000	80.000000	
0 + 100.000	4.470000	0.0000	20.0000	100.000000	
0 + 105.007	4.470000	0.0000	5.0071	105.007058	KA 2-1
0 + 120.000	4.470000	0.0000	14.9929	120.000000	

VIP 1 Acc.Dis.= 150.0000 EL.= 4.4700 VCL = 30.0000
I1 = 0.0000% I2 = 0.3000% R =10000 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 135.000	4.470000	0.0000	15.0000	135.000000	
0 + 140.000	4.471250	0.0500	5.0000	140.000000	
0 + 148.111	4.478594	0.1311	8.1105	148.110506	KE 2-1
0 + 150.000	4.481250	0.1500	1.8895	150.000000	
0 + 160.000	4.501250	0.2500	10.0000	160.000000	
0 + 165.000	4.515000	0.3000	5.0000	165.000000	

Station	levation	Tangent (%)	Distance	Acc.Distance
0 + 180.000	4.560000	0.3000	15.0000	180.000000
0 + 200.000	4.620000	0.3000	20.0000	200.000000
0 + 220.000	4.680000	0.3000	20.0000	220.000000
0 + 240.000	4.740000	0.3000	20.0000	240.000000
0 + 260.000	4.800000	0.3000	20.0000	260.000000
0 + 280.000	4.860000	0.3000	20.0000	280.000000

VIP 2 Acc.Dis.= 329.9421 EL.= 5.0098 VCL = 60.0000
I1 = 0.3000% I2 = 5.4792% R = 1158 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 299.942	4.919826	0.3000	19.9421	299.942096
0 + 300.000	4.920001	0.3050	0.0579	300.000000
0 + 320.000	5.153640	2.0314	20.0000	320.000000
0 + 329.942	5.398264	2.8896	9.9421	329.942096
0 + 340.000	5.732556	3.7578	10.0579	340.000000
0 + 359.942	6.653576	5.4792	19.9421	359.942096

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 360.000	6.656749	5.4792	0.0579	360.000000	
0 + 367.483	7.066778	5.4792	7.4834	367.483423	KE 2-2

0 + 380.000	7.752583	5.4792	12.5166	380.000000	
0 + 400.000	8.848416	5.4792	20.0000	400.000000	
0 + 410.587	9.428489	5.4792	10.5869	410.586871	KA-2-2
0 + 420.000	9.944250	5.4792	9.4131	420.000000	
0 + 440.000	11.040083	5.4792	20.0000	440.000000	

VIP 3 Acc.Dis.= 490.0000 EL.= 13.7797 VCL = 60.0000
I1 = 5.4792% I2 = 2.1967% R = 1827 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 460.000	12.135917	5.4792	20.0000	460.000000
0 + 480.000	13.122334	4.3850	20.0000	480.000000
0 + 490.000	13.533480	3.8379	10.0000	490.000000
0 + 500.000	13.889918	3.2908	10.0000	500.000000
0 + 520.000	14.438669	2.1967	20.0000	520.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance	
0 + 535.778	14.785267	2.1967	15.7783	535.778322	BC-3
0 + 540.000	14.878003	2.1967	4.2217	540.000000	

3 Approach Road between Shukhinthar Mayopat Road and the Toll Plaza

3.1 Approach Road from the Toll Plaza to Shukhinthar Mayopat Road

3.1.1 Point of Vertical Intersection Report

PVI	Station	Elevation (m)	Tangent (%)	Vertical Curve Length (m)	Vertical Curve Radius (m)
1	0+ 0.000000	8.510473	-	-	-
2	0+ 30.000000	9.037973	1.758333%	60	1,041
3	0+130.949322	5.000000	-4.000000%	60	1.500
4	0+228.545623	5.000000	0.000000%	-	-

3.1.2 Station Increment Report

VIP 1 Acc.Dis.= 30.0000 EL.= 9.0380 VCL = 60.0000
I1 = 1.7583% I2 = -4.0000% R = 1041 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 0.000	8.510473	1.7583	0.0000	0.000000
0 + 0.000	8.510473	1.7583	0.0000	0.000000
0 + 20.000	8.670195	-0.1611	20.0000	20.000000
0 + 30.000	8.606098	-1.1208	10.0000	30.000000
0 + 40.000	8.446029	-2.0806	10.0000	40.000000
0 + 60.000	7.837973	-4.0000	20.0000	60.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 80.000	7.037973	-4.0000	20.0000	80.000000
0 + 100.000	6.237973	-4.0000	20.0000	100.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 100.949	6.200000	-4.0000	0.9493	100.949322
0 + 120.000	5.558949	-2.7300	19.0507	120.000000
0 + 130.949	5.300000	-2.0000	10.9493	130.949322
0 + 140.000	5.146291	-1.3966	9.0507	140.000000
0 + 160.000	5.000300	-0.0633	20.0000	160.000000
0 + 160.949	5.000000	0.0000	0.9493	160.949322

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 180.000	5.000000	0.0000	19.0507	180.000000
0 + 200.000	5.000000	0.0000	20.0000	200.000000
0 + 220.000	5.000000	0.0000	20.0000	220.000000
0 + 228.546	5.000000	0.0000	8.5456	228.545623

VIP 2 Acc.Dis.= 130.9493 EL.= 5.0000 VCL = 60.0000
I1 = -4.0000% I2 = 0.0000% R = 1500 m

3.2 Approach Road from Shukhinthar Mayopat Road to the Toll Plaza

3.2.1 Point of Vertical Intersection Report

PVI	Station	Elevation (m)	Tangent (%)	Vertical Curve Length (m)	Vertical Curve Radius (m)
1	0+ 0.000000	8.510473	-	-	-
2	0+ 30.000000	9.037973	1.758333%	60	1,041
3	0+130.949322	5.000000	-4.000000%	60	1.500
4	0+220.000000	5.000000	0.000000%	-	-

3.2.2 Station Increment Report

VIP 1 Acc.Dis.= 30.0000 EL.= 9.0380 VCL = 60.0000
I1 = 1.7583% I2 = -4.0000% R = 1041 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 0.000	8.510473	1.7583	0.0000	0.000000
0 + 0.000	8.510473	1.7583	0.0000	0.000000
0 + 20.000	8.670195	-0.1611	20.0000	20.000000
0 + 30.000	8.606098	-1.1208	10.0000	30.000000
0 + 40.000	8.446029	-2.0806	10.0000	40.000000
0 + 60.000	7.837973	-4.0000	20.0000	60.000000

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 80.000	7.037973	-4.0000	20.0000	80.000000
0 + 100.000	6.237973	-4.0000	20.0000	100.000000

VIP 2 Acc.Dis.= 130.9493 EL.= 5.0000 VCL = 60.0000
I1 = -4.0000 % I2 = 0.0000 % R = 1500 m

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 100.949	6.200000	-4.0000	0.9493	100.949322
0 + 120.000	5.558949	-2.7300	19.0507	120.000000
0 + 130.949	5.300000	-2.0000	10.9493	130.949322
0 + 140.000	5.146291	-1.3966	9.0507	140.000000
0 + 160.000	5.000300	-0.0633	20.0000	160.000000
0 + 160.949	5.000000	0.0000	0.9493	160.949322

Station	Elevation	Tangent (%)	Distance	Acc.Distance
0 + 180.000	5.000000	0.0000	19.0507	180.000000
0 + 200.000	5.000000	0.0000	20.0000	200.000000
0 + 220.000	5.000000	0.0000	20.0000	220.000000

ROAD DESIGN

MECHANICALLY-STABILIZED EARTH WALL AND RETAINING WALL

(PACKAGE 1)

Main Road

- Design Conditions
- Mechanically-Stabilized Earth Wall
- Concrete Barrier

Ramp Road

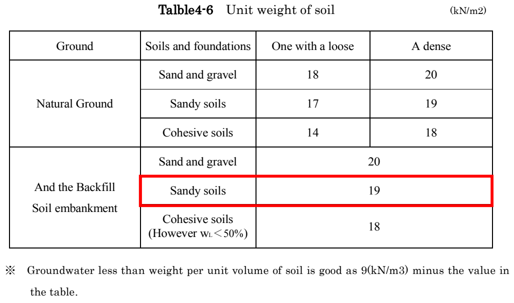
- Design Conditions
- Mechanically-Stabilized Earth Wall
- Mechanically-Stabilized Earth Wall Type-1
- Mechanically-Stabilized Earth Wall Type-2
- Concrete Barrier Type-1
- Concrete Barrier Type-2

Main Road

Design Conditions

THANLYIN MAIN DESIGN CONDITIONS

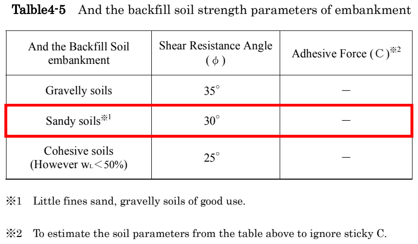
ITEM	DESIGN CRITERIA	(SUPPORTING DOCUMENTATION)	NOTE	ONSULTATION RESUL
1. Application criteria	1) Japan Road Association (2012). Road Earthwork: Retaining Wall Guidelines 2012. 2) Japan Road Association (2012). Specifications for Highway Bridges, the Commentary. 3) Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed. 4) American Association of State Highway and Transportation Officials (2012) , AASHTO LRFD Bridge Design Specifications, 6th Ed (US)	【Road Earthwork: Retaining Wall Guidelines　P.66】		
2. Soil Condition				
1). Embankment Material	γ=19kN/m^3, φ=30°, C=0kN/m^2			
3. Earthquake resistant design	Horizontal Seismic Coefficient **kh = Cz・kho = 1.0×0.18 = 0.18**			
4. Loading Conditions				
1). Main Load Loading	Road Department　11.6kN/m^2　(AASHTO LRFD 2012 Bridge)			
2). Special Loads				
Collision Load	Mechanically -stabilised earth wall　Handrail（A）　43kN/m (Only horizontal force taken into account)			
5. Study Model	MSE　=3750(Full Height)　H=3730(Finished Height)　＋　H4=1900　＝　56	Concrete Barrier Height h=1150		



Talble4-6　Unit weight of soil　(kN/m2)

Ground	Soils and foundations	One with a loose	A dense
Natural Ground	Sand and gravel	18	20
	Sandy soils	17	19
	Cohesive soils	14	18
And the Backfill Soil embankment	Sand and gravel	20	
	Sandy soils	19	
	Cohesive soils (However w_L＜50%)	18	

※　Groundwater less than weight per unit volume of soil is good as 9(kN/m3) minus the value in the table.

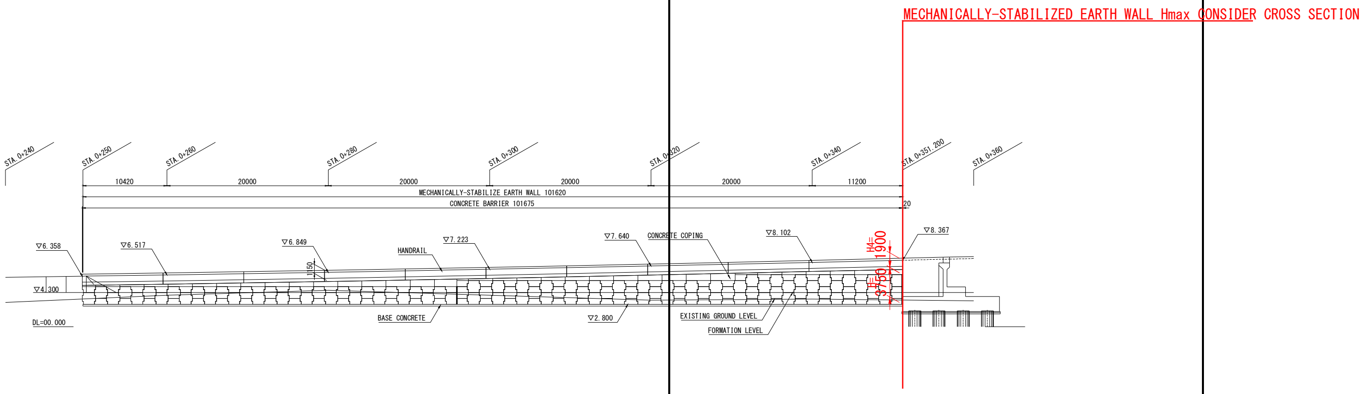


Talble4-5　And the backfill soil strength parameters of embankment

And the Backfill Soil embankment	Shear Resistance Angle (ϕ)	Adhesive Force (C)※2
Gravelly soils	35°	—
Sandy soils※1	30°	—
Cohesive soils (However w_L＜50%)	25°	—

※1　Little fines sand, gravelly soils of good use.

※2　To estimate the soil parameters from the table above to ignore sticky C.



Main Road

Mechanically-Stabilized Earth Wall

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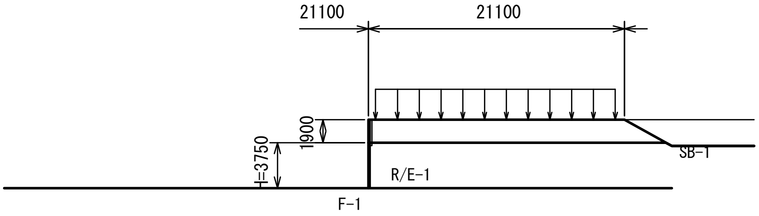
1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H = 3.750 (m)

HEIGHT OF COPING : H4 = 1.900 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	0.000	0.000	3.750

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	3.750
	2	0.000	5.650
	3	21.100	5.650
	4	25.000	3.483

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	0.000
	2	25.000	0.000

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE : RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 : MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION

PERFORMANCE 2 : KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL

PERFORMANCE 3 : KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	ces (kN/m^2)	fo*	tanϕ_1
SB－1	1.900	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	3.750	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB : SURCHARGE BACKFILL, R/E : R/E, BM : BACKSIDE EMBANKMENT, F : FOUNDATION

Hs : LAYER THICKNESS (m)

γ : UNIT WEIGHT OF SOIL (kN/m^3)

γ' : UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE (°)

c : COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)

ces : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)

(FOR SANDY SOIL : ces＝10kN/m^2)

fo* : APPARENT COEFFICIENT OF FRICTION

tanϕ_1 : APPARENT COEFFICIENT OF FRICTION

1.5　SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP $L+B_G$(m)	WIDTH OF LOAD B_L(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	LIVE LOAD1	0.600	19.750	11.60	—

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6　HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1}= c_z \cdot k_{ho} = 0.18$

k_{h1}　: HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL
k_{ho}　: STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18
［ CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ ］
※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012"
c_z　: AREA CORRECTION COEFFICIENT = 1.00 ［ AREA : A ］

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2}= c_z \cdot k_{ho} \cdot \nu = 0.13$

k_{h2}　: HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF
k_{ho}　: STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18
［ CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ ］
※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012
c_z　: AREA CORRECTION COEFFICIENT = 1.00 ［ AREA : A ］
ν　: CORRECTION COEFFICIENT = 0.70

(3) OVERALL STABILITY EXAMINATION INCLUDING R/E

(EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

$k_{h3} = c_z \cdot k_{ho} = 0.12$

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.12

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR EMBANKMENT CONSTRUCTION 2012″

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT h_c (m)	UNIT WEIGHT γ_c (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σ_a=140.0	σ_{aE}=210.0
BOLT SHEAR STRESS (N/mm^2)	τ_a=200.00	τ_{aE}=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$
CIRCULAR SLIP	$F_s \geqq 1.20$	$F_{sE} \geqq 1.00$

1.9　MATERIAL

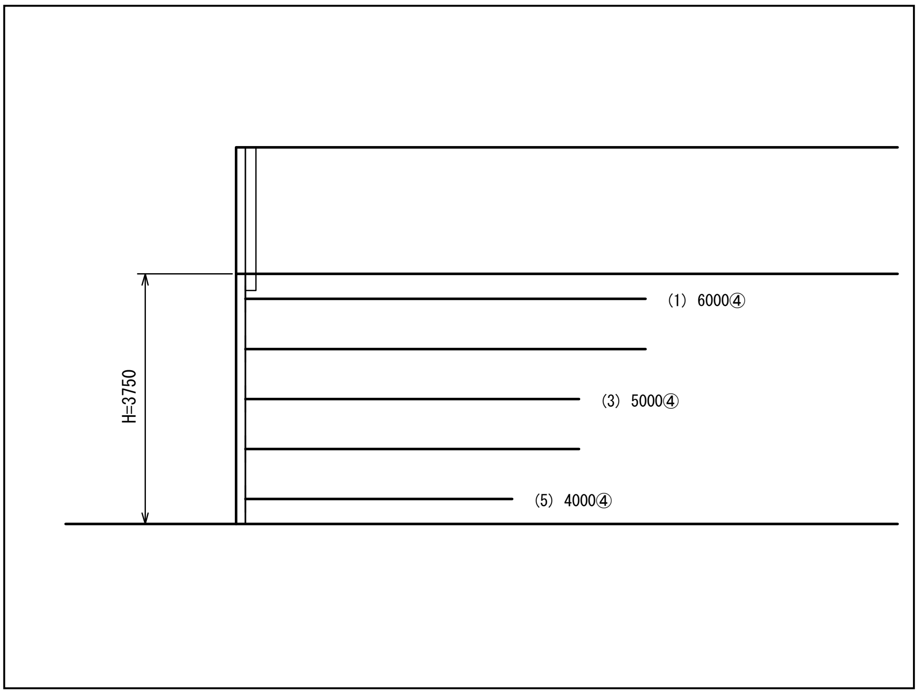
(1) SKIN　　　　: CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	Ae	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n’	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	Cm	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z_i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB(m)	DESIGN LENGTH L (m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.275	0.750	0.750	6.000	5.125	5.209	2.510	1.516	○
2	3.025	0.750	0.750	6.000	4.706	4.586	2.827	1.869	○
3	3.775	0.750	0.750	5.000	4.094	3.745	2.610	1.860	○
4	4.525	0.750	0.750	5.000	4.000	4.000	2.904	2.187	○
5	5.275	0.750	0.750	4.000	4.000	4.000	2.421	1.892	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	σtEi (N/mm^2)	$\sigma otEi$ (N/mm^2)	τoEi (N/mm^2)	
1	74.4	68.7	79.5	87.1	80.4	93.0	○
2	78.8	72.7	84.1	89.9	83.0	96.0	○
3	82.3	76.0	87.9	92.1	85.0	98.4	○
4	87.5	80.8	93.4	99.0	91.4	105.7	○
5	93.9	86.7	100.3	107.4	99.1	114.6	○

σti , σtEi : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σoti, $\sigma otEi$: TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τoi , τoEi : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3　EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.097 (1.500)	○	2.702 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.928 (0.648)	○	-0.467 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	179.781	—	158.477	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	144.112	—	129.498	—

2.4　OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R(m)	Fsmin	Fs	JUDGE
		X(m)	Y(m)				
WITHIN REINFORCED AREA	ORDINARY-1	-11.900	15.500	19.541	1.437	1.200	○
	EARTHQUAKE-1	-4.500	16.500	17.103	1.553	1.000	○

2.5 RESULTS OF EXAMINATION OF INTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

2.5.1 LIST OF WALL HEIGHTS AND STRIP STRENGTHS

NOTE) HORIZONTAL SPACING OF STRIP FOR EACH ORDINAL STAGE NUMBER IS REPRESENTED IN THE FOLLOWING SYMBOLS:

④= 0.750 (m),

i : STRIPSTAGE

H : HEIGHT OF R/E(m)

VALUES IN THE TABLE MINIMUM STRIP LENGTH(m)

() IN THE EXECUTION STRIP LENGTH(m)

i	(1)H=3.750(m)	(2)H=3.000(m)	(3)H=2.250(m)
1	④ 5.209 (6.0)	④ 4.939 (5.0)	④ 4.534 (5.0)
2	④ 4.706	④ 4.265	④ 3.815
3	④ 4.094 (5.0)	④ 3.670	④ 3.220 (4.0)
4	④ 3.654	④ 3.241 (4.0)	
5	④ 3.343 (4.0)		

2.5.2 LIST OF WALL HEIGHTS AND STRUCTURAL DETAIL SPECIFICS

BASED ON: DESIGN AND CONSTRUCTION MANUAL FOR REINFORCED EARTHWORK

H : HEIGHT OF R/E (m)
H4 : HEIGHT OF COPING (m)
Ha : VIRTUAL WALL HEIGHT (m)
H1 : HEIGHT OF SURCHARGE BACKFILL IN THE UPPER PART OF R/E (m)
n : SLOPE INCLINE
Bb : FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN (m)
L1 : DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER (m)
QLj_W1 : LIVE LOAD AFFECTING DIFFERENT STAGE STRIP (ORDINARY) (kN/m^2)
B_G : HORIZONTAL DISTANCE FROM ROAD SHOULDER OF SKIN ELEMENT TO SURCHARGE LOAD (m)
B_L : WIDTH OF LOAD (m)
Lsd : MINIMUM STRIP LENGTH (m)
Hsd : PLACEMENT AREA OF MINIMUM STRIP LENGTH (m)

(1) H=3.750 (m), H4=1.900 (m), Ha=5.650 (m), H1=1.900 (m)
1 : n=1 : -1.800, Bb =20.960 (m), L1 =20.960 (m)
QL1_W1= 11.600 (kN/m^2), B_G+L1= 0.600 (m), B_L=19.750 (m)
NEAR THE TOP : Lsd = 0.7 × Ha = 3.955 (m)
NEAR THE TOP : Hsd = 0.5 × Ha = 2.825 (m) ・・・ < 1TH TO 1TH STAGES >
NEAR THE LOWER : Lsd = 0.4 × Ha = 2.260 ≧ 4.000 (m)
NEAR THE LOWER : Hsd = 0.3 × Ha = 1.695 (m) ・・・ < 4TH TO 5TH STAGES >

(2) H=3.000 (m), H4=2.100 (m), Ha=5.100 (m), H1=2.100 (m)
1 : n=1 : 0.000, Bb =0.000 (m), L1 =0.000 (m)
QL1_W1= 11.600 (kN/m^2), B_G+L1= 0.600 (m), B_L=19.750 (m)
NEAR THE TOP : Lsd = 0.7 × Ha = 3.570 (m)
NEAR THE TOP : Hsd = 0.6 × Ha = 3.060 (m) ・・・ < 1TH TO 1TH STAGES >
NEAR THE LOWER : Lsd = 0.4 × Ha = 2.040 ≧ 4.000 (m)
NEAR THE LOWER : Hsd = 0.3 × Ha = 1.530 (m) ・・・ < 3TH TO 4TH STAGES >

(3)　Ｈ＝2.250 (m), Ｈ4＝2.100 (m), Ｈａ＝4.350 (m), Ｈ1＝2.100 (m)

　１：ｎ＝1：0.000, Ｂｂ＝0.000 (m), Ｌ1＝0.000 (m)

　ＱL1_W1＝ 11.600 (kN/m^2), B_G＋Ｌ1＝ 0.600 (m), B_L＝19.750 (m)

　NEAR THE TOP：Ｌsd ＝ 0.7 × Ｈa ＝ 3.045 (m)

　NEAR THE TOP：Ｈsd ＝ 0.6 × Ｈa ＝ 2.610 (m)・・・＜ 1TH TO 1TH STAGES ＞

　NEAR THE LOWER：Ｌsd ＝ 0.4 × Ｈa ＝ 1.740 ≧ 4.000 (m)

　NEAR THE LOWER：Ｈsd ＝ 0.3 × Ｈa ＝ 1.305 (m)・・・＜ 2TH TO 3TH STAGES ＞

2.6 RESULTS OF EXAMINATION OF EXTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

(1) Ｈ＝3.750 (m)

・VIRTUAL RETAINING WALL R/E BOTTOM WIDTH OF THE VIRTUAL WALL : Ｂ ＝3.890 (m)
・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION : ｂ ＝0.400 (m)
HEIGHT OF THE FOUNDATION : ｈｆ ＝0.200 (m)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Ｆｓ	10.097 (1.500)	○	2.702 (1.200)	○
OVERTUENING	ECCENTLICTY ｅ (m)	-0.928 (0.648)	○	-0.467 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION ｑｓ (kN/m^2)	179.781	—	158.477	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION ｑｗ (kN/m^2)	144.112	—	129.498	—

(2) Ｈ＝3.000 (m)

・VIRTUAL RETAINING WALL R/E BOTTOM WIDTH OF THE VIRTUAL WALL : Ｂ ＝3.973 (m)
・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION : ｂ ＝0.400 (m)
HEIGHT OF THE FOUNDATION : ｈｆ ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Ｆｓ	6.485 (1.500)	○	2.410 (1.200)	○
OVERTUENING	ECCENTLICTY ｅ (m)	-0.477 (0.662)	○	-0.093 (1.324)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION ｑｓ (kN/m^2)	142.183	—	126.014	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION ｑｗ (kN/m^2)	121.044	—	109.287	—

(3)　Ｈ＝2.250 (m)

・VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL : Ｂ ＝3.890 (m)

・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION : ｂ ＝0.400 (m)

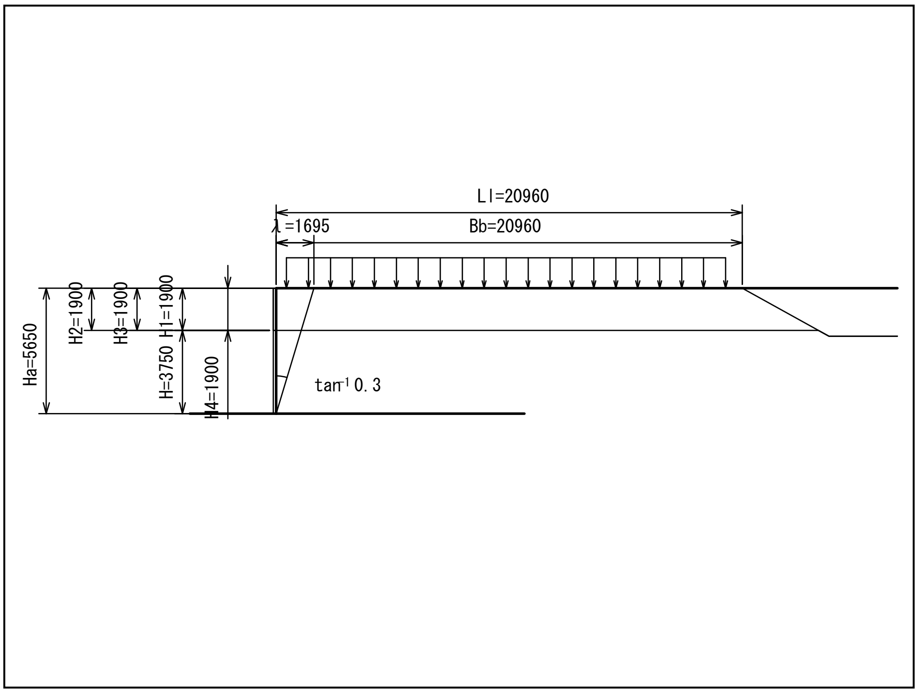
HEIGHT OF THE FOUNDATION : ｈｆ ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Ｆｓ	11.524 (1.500)	○	2.893 (1.200)	○
OVERTUENING	ECCENTLICTY ｅ (m)	-0.711 (0.648)	○	-0.372 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION ｑｓ (kN/m^2)	131.551	—	112.956	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION ｑｗ (kN/m^2)	95.073	—	86.567	—

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 3.750 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : H4 = 1.900 (m)

HEIGHT OF SURCHARGE BACKFILL : H1 = 1.900 (m)

SLOPE INCLINE (1∶n) : n = -1.800

FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN : Bb = 20.960 (m)

DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER : Ll = 20.960 (m)

3.1.2 VIRTUAL WALL HEIGHT

$$Ha = H+H2 = 5.650 \ (m)$$

Ha : VIRTUAL WALL HEIGHT (m)

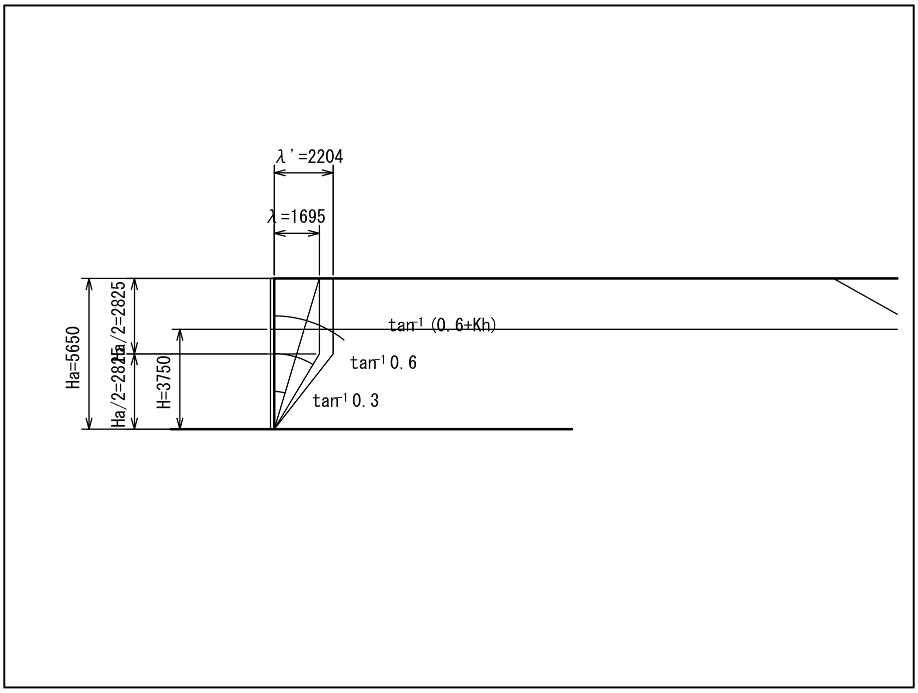
H : HEIGHT OF R/E = 3.750 (m)

H2 : DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 1.900$

H4 : HEIGHT OF COPING = 1.900 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$zi = \Delta H \cdot (i-1/2)+H2$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

H2 : $H2+Ha=0.3(H+H4)-Bb/n-0.3+H4$ (m) = 1.900 (m)

STAGE (i)	zi (m)
1	2.275
2	3.025
3	3.775
4	4.525
5	5.275

3.2　LOAD

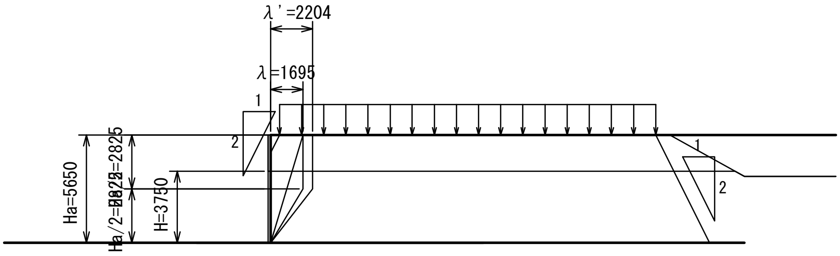
3.2.1　SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

$q1 = \gamma 2 \cdot H3 = 36.100$ (kN/m^2)

$q1$: SURCHARGE SOIL LOAD (kN/m^2)
$\gamma 2$: UNIT WEIGHT OF SURCHARGE SOIL = 19.0 (kN/m^3)
$H3$: ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 1.900 (m)

3.2.2 CALCULATION OF LIVE LOAD APPLIED TO EACH STAGE



THE LIVE LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA.WHEN THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE LIVE LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{Li} = q_L \cdot \frac{B_L}{B_{Li}}$$

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_L : LIVE LOAD (kN/m^2)

B_L : LIVE LOAD WIDTH (m)

B_{Li} : DISPERSIVE WIDTH OF THE LIVE LOAD AT iTH STAGE STRIP (m)

(1) LIVE LOAD-1

q_L ＝ 11.600 (kN/m^2)　　B_L ＝ 19.750 (m)

STAGE (i)	z_i (m)	B_{Li} (m)	q_{Li} (kN/m^2)
1	2.275	21.348	10.732
2	3.025	21.723	10.547
3	3.775	22.098	10.368
4	4.525	22.473	10.195
5	5.275	22.848	10.027

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY（V_i）AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE（V_{Ei}）.

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.275	6.300	8.118
2	3.025	5.037	6.444
3	3.775	4.203	5.338
4	4.525	3.602	4.567
5	5.275	3.150	3.978

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$$q_i = q_1 + q_{Li} + V_i$$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 36.100 (kN/m^2)

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{Li} (kN/m^2)	V_i (kN/m^2)	q_i (kN/m^2)
1	2.275	10.732	6.300	53.132
2	3.025	10.547	5.037	51.684
3	3.775	10.368	4.203	50.671
4	4.525	10.195	3.602	49.897
5	5.275	10.027	3.150	49.277

(2) EARTHQUAKE

$$q_{Ei} = q_1 + V_{Ei}$$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 36.100 (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	V_{Ei} (kN/m^2)	q_{Ei} (kN/m^2)
1	2.275	8.118	44.218
2	3.025	6.444	42.544
3	3.775	5.338	41.438
4	4.525	4.567	40.667
5	5.275	3.978	40.078

3.3　EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（Hi）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_Ei）.

STAGE (i)	zi (m)	ORDINARY Hi (kN/m)	EARTH QUAKE H_Ei (kN/m)
1	2.275	4.075	6.162
2	3.025	2.417	3.495
3	3.775	0.765	0.837
4	4.525	0.000	0.000
5	5.275	0.000	0.000

3.4　EARTH PRESSURE

3.4.1　COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq z_o = 6.0$ (m)

$$K_i = K_o \cdot \left(1 - \frac{z_i}{z_o}\right) + K_A \cdot \frac{z_i}{z_o}$$

IF $z_i > z_o = 6.0$ (m)

$$K_i = K_A$$

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

K_o : COEFFICIENT OF EARTH PRESSURE AT REST

$$K_o = 1 - \sin\phi$$

K_A : COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

z_o : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE ＝ 6.000 (m)

STAGE (i)	z_i (m)	ϕ (°)	K_o	K_A	K_i
1	2.275	30.0	0.5000	0.3333	0.4368
2	3.025	30.0	0.5000	0.3333	0.4160
3	3.775	30.0	0.5000	0.3333	0.3951
4	4.525	30.0	0.5000	0.3333	0.3743
5	5.275	30.0	0.5000	0.3333	0.3535

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$Pi = Ki \cdot \sigma vi \cdot \Delta H + Hi$$

$$To = 0.75 \cdot Pi$$

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

Hi : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma vi = \gamma 1 \cdot xi + qi$

$\gamma 1$: UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

xi : $xi = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

qi : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	xi (m)	Ki	qi (kN/m^2)	σvi (kN/m^2)	Hi (kN/m)	Pi (kN/m)	To (kN/m)
1	0.375	0.437	53.132	60.257	4.075	23.815	17.862
2	1.125	0.416	51.684	73.059	2.417	25.210	18.907
3	1.875	0.395	50.671	86.296	0.765	26.339	19.754
4	2.625	0.374	49.897	99.772	0.000	28.009	21.007
5	3.375	0.353	49.277	113.402	0.000	30.063	22.548

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = P_i + \Delta P_i$$

$$\Delta P_i = \frac{1}{2} \cdot \left(1 + \frac{z_i}{H_a}\right) \cdot \alpha \cdot k_h \cdot P_n$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$P_i = K_i \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$

ΔP_i : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_n : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 27.625 (kN/m)

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

H_a : VIRTUAL WALL HEIGHT = 5.650 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{VEi} = \gamma_1 \cdot x_i + q_{Ei}$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.437	44.218	51.343	6.162	22.982
2	1.125	0.416	42.544	63.919	3.495	23.436
3	1.875	0.395	41.438	77.063	0.837	23.675
4	2.625	0.374	40.667	90.542	0.000	25.418
5	3.375	0.353	40.078	104.203	0.000	27.625

STAGE (i)	z i (m)	Δ P i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.275	4.882	27.864	20.898
2	3.025	5.344	28.781	21.585
3	3.775	5.806	29.481	22.111
4	4.525	6.268	31.686	23.765
5	5.275	6.730	34.355	25.766

3.5　HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS ＝ 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT ＝ 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP ＝ 80.0 (mm)

t : THE THICKNESS OF STRIP ＝ 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP ＝ 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP ＝ 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER ＝ 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

j　　:THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM = 2

n　　:THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS = 1 (PIECE)

Ae　　:SCREW EFFECTIVE AREA OF MOUNTING BOLTS= 84.3 (mm^2)

STAGE (i)	Pi (kN/m)	To (kN/m)	ΔB1i (m)	ΔB2i (m)	ΔB3i (m)
1	23.815	17.862	1.410	1.528	1.887
2	25.210	18.907	1.332	1.443	1.783
3	26.339	19.754	1.275	1.381	1.706
4	28.009	21.007	1.199	1.299	1.605
5	30.063	22.548	1.117	1.210	1.495

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)

τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)

P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	27.864	20.898	1.808	1.959	2.420
2	28.781	21.585	1.751	1.897	2.343
3	29.481	22.111	1.709	1.852	2.287
4	31.686	23.765	1.590	1.723	2.128
5	34.355	25.766	1.467	1.589	1.963

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF 1.000 ＞ $\Delta B_i \geqq 0.750$　　$\Delta B_i = 0.750$

STAGE (i)	ΔB_{1i} (m)	ΔB_{2i} (m)	ΔB_{3i} (m)	ΔB_{1Ei} (m)	ΔB_{2Ei} (m)	ΔB_{3Ei} (m)	ΔB_i (m)
1	1.410	1.528	1.887	1.808	1.959	2.420	0.750
2	1.332	1.443	1.783	1.751	1.897	2.343	0.750
3	1.275	1.381	1.706	1.709	1.852	2.287	0.750
4	1.199	1.299	1.605	1.590	1.723	2.128	0.750
5	1.117	1.210	1.495	1.467	1.589	1.963	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	23.815	17.862	17.862	13.396	27.864	20.898	20.898	15.674
2	0.750	25.210	18.907	18.907	14.181	28.781	21.585	21.585	16.189
3	0.750	26.339	19.754	19.754	14.816	29.481	22.111	22.111	16.583
4	0.750	28.009	21.007	21.007	15.755	31.686	23.765	23.765	17.823
5	0.750	30.063	22.548	22.548	16.911	34.355	25.766	25.766	19.325

3.7　STRIP LENGTH

3.7.1　FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^* = fo^* \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^* = \tan\phi 1$$

fi^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^* = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^*	$\tan\phi 1$	fi^*
1	2.275	1.500	0.727	1.2067
2	3.025	1.500	0.727	1.1100
3	3.775	1.500	0.727	1.0134
4	4.525	1.500	0.727	0.9167
5	5.275	1.500	0.727	0.8200

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 2.825$ (m)

$$L_{oi} = 0.3 \cdot Ha$$

IF $z_i > Ha/2 = 2.825$ (m)

$$L_{oi} = 0.6 \cdot (Ha - z_i)$$

L_{oi} : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Ha : VIRTUAL WALL HEIGHT = 5.650 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 2.825$ (m)

$$L_{oEi} = (0.6 + kh1) \cdot (Ha/2)$$

IF $z_i > Ha/2 = 2.825$ (m)

$$L_{oEi} = (0.6 + kh1) \cdot (Ha - z_i)$$

L_{oEi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)
Ha : VIRTUAL WALL HEIGHT = 5.650 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
$kh1$: HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	z_i (m)	L_{oi} (m)	L_{oEi} (m)
1	2.275	1.695	2.204
2	3.025	1.575	2.048
3	3.775	1.125	1.463
4	4.525	0.675	0.878
5	5.275	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqi} = \frac{Fs \cdot Ti}{2 \cdot fi^{*} \cdot (\sigma vi - Vi) \cdot b}$$

Lreqi : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

Fs : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 2.00

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

Vi : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	Ti (kN/PIECE)	fi^{*}	$\sigma vi-Vi$ (kN/m^2)	b (m)	Lreqi (m)
1	17.862	1.2067	53.957	0.0800	3.430
2	18.907	1.1100	68.022	0.0800	3.131
3	19.754	1.0134	82.093	0.0800	2.969
4	21.007	0.9167	96.170	0.0800	2.979
5	22.548	0.8200	110.252	0.0800	3.118

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^* \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT = 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^*	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	20.898	1.2067	43.225	0.0800	3.005
2	21.585	1.1100	57.475	0.0800	2.538
3	22.111	1.0134	71.725	0.0800	2.282
4	23.765	0.9167	85.975	0.0800	2.262
5	25.766	0.8200	100.225	0.0800	2.352

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$L_{mini} = L_{oi} + L_{reqi}$$

EARTHQUAKE

$$L_{minEi} = L_{oEi} + L_{reqEi}$$

L_{mini} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)

L_{oi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

L_{minEi} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{oEi} : LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	L_{oi} (m)	L_{reqi} (m)	L_{mini} (m)	L_{oEi} (m)	L_{reqEi} (m)	L_{minEi} (m)
1	1.695	3.430	5.125	2.204	3.005	5.209
2	1.575	3.131	4.706	2.048	2.538	4.586
3	1.125	2.969	4.094	1.463	2.282	3.745
4	0.675	2.979	3.654	0.878	2.262	3.140
5	0.225	3.118	3.343	0.293	2.352	2.645

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

WHEN SURCHERGE BACKFILL HEIGHT IS UNDER 2M ($0\text{m} \leqq H1 = 1.900 < 2\text{m}$)

0.5 Ha OR MORE FROM UPPER EDGE OF Ha

$L_{sd} = 0.7 \cdot Ha = 3.955$ (m)

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$L_{sd} = 0.4 \cdot Ha = 2.260$ (m)

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

Ha : VIRTUAL WALL HEIGHT = 5.650 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)

L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)

L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)

L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	5.125	5.209	3.955	6.000
2	4.706	4.586	—	6.000
3	4.094	3.745	—	5.000
4	3.654	3.140	4.000	5.000
5	3.343	2.645	4.000	4.000

3.8　CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$Fsi = \frac{Si}{Ti} \geqq Fs = 2.00$$

Fsi　：SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

Fs　：DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 2.0

Ti　：PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Si　：FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$Si = 2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei$$

fi^{*}　：APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$\sigma vi'$　：VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b　：STRIP WIDTH AT iTH STAGE (m)

Lei　：EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$Lei = Li - Loi$$

Li　：DETERMINED STRIP LENGTH AT iTH STAGE (m)

Loi　：ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	Loi (m)	Lei (m)	fi*	σvi' (kN/m²)	b (m)	Si (kN/PIECE)	Ti (kN/PIECE)	Fsi	JUDGE
1	6.000	1.695	4.305	1.2067	53.957	0.0800	44.849	17.862	2.510	○
2	6.000	1.575	4.425	1.1100	68.022	0.0800	53.459	18.907	2.827	○
3	5.000	1.125	3.875	1.0134	82.093	0.0800	51.578	19.754	2.610	○
4	5.000	0.675	4.325	0.9167	96.170	0.0800	61.005	21.007	2.904	○
5	4.000	0.225	3.775	0.8200	110.252	0.0800	54.606	22.548	2.421	○

(2) EARTHQUAKE

$$F_{SEi} = \frac{S_{Ei}}{T_{Ei}} \geqq F_{SE} = 1.20$$

F_{siEi} : SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP
F_{SE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2
T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
S_{Ei} : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)
$S_{Ei} = 2 \cdot f i^{*} \cdot \sigma_{VEi} \cdot b \cdot L_{eEi}$
$f i^{*}$: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
σ_{VEi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)
b : STRIP WIDTH AT iTH STAGE (m)
L_{eEi} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)
$L_{eEi} = L i - L_{OEi}$
$L i$: DETERMINED STRIP LENGTH AT iTH STAGE (m)
L_{OEi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	$L i$ (m)	L_{OEi} (m)	L_{eEi} (m)	$f 1^{*}$	σ_{VEi}' (kN/m^2)	b (m)	S_{Ei} (kN/PIECE)	T_{Ei} (kN/PIECE)	F_{SEi}	JUDGE
1	6.000	2.204	3.797	1.2067	43.225	0.0800	31.685	20.898	1.516	○
2	6.000	2.048	3.953	1.1100	57.475	0.0800	40.347	21.585	1.869	○
3	5.000	1.463	3.538	1.0134	71.725	0.0800	41.139	22.111	1.860	○
4	5.000	0.878	4.123	0.9167	85.975	0.0800	51.984	23.765	2.187	○
5	4.000	0.293	3.708	0.8200	100.225	0.0800	48.752	25.766	1.892	○

3.9 CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \text{ (N/mm}^2\text{)}$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \text{ (N/mm}^2\text{)}$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \text{ (N/mm}^2\text{)}$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)
σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)
τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)
σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)
τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)
Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)
An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)
$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	17.862	13.396	74.4	68.7	79.5	○
2	18.907	14.181	78.8	72.7	84.1	○
3	19.754	14.816	82.3	76.0	87.9	○
4	21.007	15.755	87.5	80.8	93.4	○
5	22.548	16.911	93.9	86.7	100.3	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma_{tEi} = \frac{T_{Ei}}{Ag} \leqq \sigma_{aE} = 210.0\ (\mathrm{N/mm^2})$$

$$\sigma_{otEi} = \frac{T_{fEi}}{An} \leqq \sigma_{aE} = 210.0\ (\mathrm{N/mm^2})$$

$$\tau_{oEi} = \frac{T_{fEi}}{A\tau} \leqq \tau_{aE} = 300.0\ (\mathrm{N/mm^2})$$

σ_{tEi} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σ_{otEi} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τ_{oEi} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σ_{aE} : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τ_{aE} : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fEi} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

Aτ : EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)	σ_{tEi} (N/mm^2)	σ_{otEi} (N/mm^2)	τ_{oEi} (N/mm^2)	JUDGE
1	20.898	15.674	87.1	80.4	93.0	○
2	21.585	16.189	89.9	83.0	96.0	○
3	22.111	16.583	92.1	85.0	98.4	○
4	23.765	17.823	99.0	91.4	105.7	○
5	25.766	19.325	107.4	99.1	114.6	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2
CASE 1-1	ORDINARY	○		○	
CASE 1-2	ORDINARY	○		○	
CASE 2-3	EARTHQUAKE	○	○		○

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

CASE 1-1 : OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)

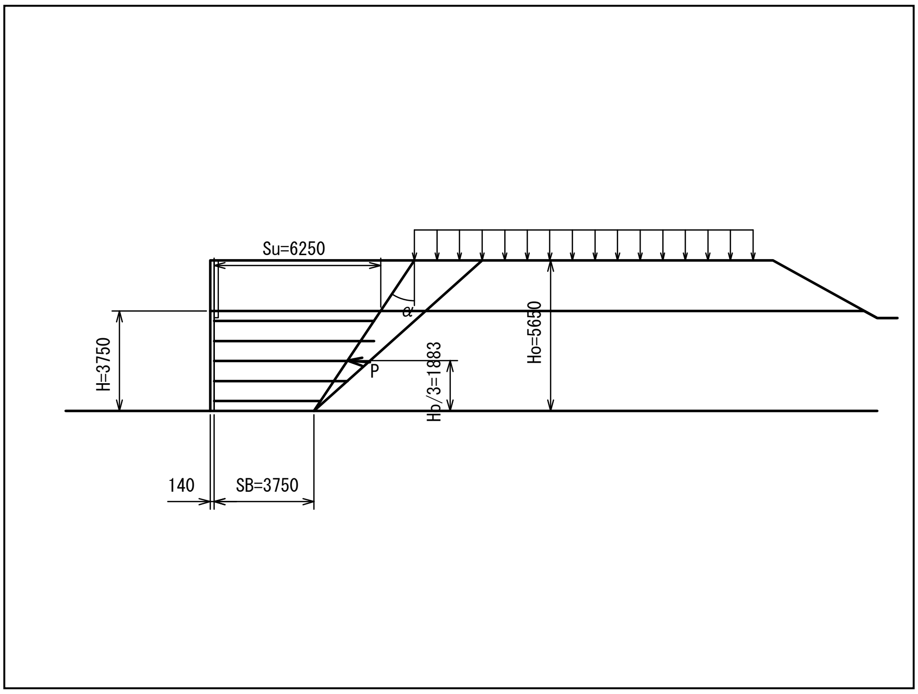
CASE 1-2 : OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 : ORDINARY LOAD

LOAD2 : EARTHQUAKELOAD

4.2 EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1 CASE 1 CALCULATION OF LOAD



H_o : WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 5.650 (m)

H : R/E WALL HEIGHT＝ 3.750 (m)

h_o : CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

S_u : REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 6.250 (m)

S_B : REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.750 (m)

B : R/E BOTTOM WIDTH ＝ 3.890 (m)

α : VIRTUAL BACK SLOPE INCLINE ＝ -33.690 (°)

$$\alpha = -\tan^{-1}\frac{S_u - S_B}{H} = -33.690\ (°)$$

$W1$: OWN WEIGHT OF R/E ＝ 619.767 (kN/m)

W_q : LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2\cdot c/\gamma)\cdot\tan(45° + \phi/2) = (2\cdot c/\gamma)\cdot\tan(45° + \phi/2) = 0.000$ (m)

$\delta 1$: FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

$\omega 1$: ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 41.9 (°)

$W2$ ：OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) ＝ 165.604 (kN/m)

$L1$ ：LENGTH OF SLIP SURFACE ＝ 8.465 (m)

P_A ：ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 35.359$$

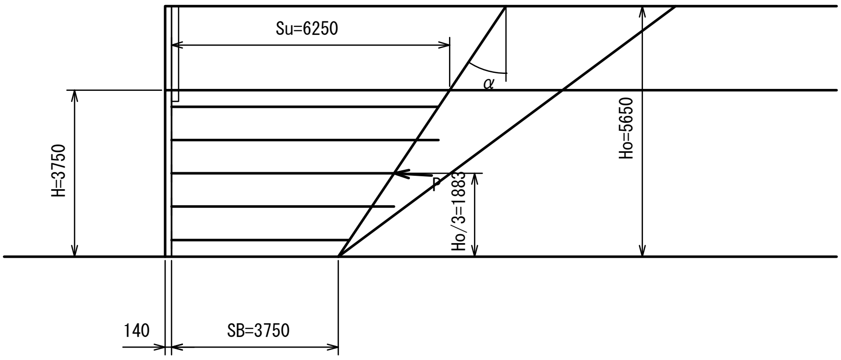
P_{Ah} ：HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 35.286$ (kN/m)

P_{AV} ：VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = -2.276$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK = -33.690 (°)

W1 : OWN WEIGHT OF R/E = 619.767 (kN/m)

Wq : LOADED WEIGHT ON R/E = 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL = 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL = 30.0 (°)

c : COHESION OF BACK FILL = 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45° + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK = 30.000 (°)

$kh2$: HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM ″DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY″)

θ : EARTHQUAKE COMPOUND ANGLE = $\tan^{-1} kh2$ = 7.407 (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE = 36.7 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE = 204.537 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE = 9.452 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 51.147$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 51.040$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = -3.292$ (kN/m)

4.3　AGGREGATION OF LOAD

4.3.1　ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V(kN/m)	ACTION POSITION X(m)	POSITION MOMENT Mx(kNm/m)	HORIZONTAL FORCE H(kN/m)	ACTION POSITION Y(m)	POSITION MOMENT My(kNm/m)
VIRTUAL WALL OF WEIGHT	619.767	2.989	1852.52	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-2.276	5.146	-11.71	35.286	1.883	66.45
TOTAL	617.492		1840.81	35.286		66.45

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V(kN/m)	ACTION POSITION X(m)	POSITION MOMENT Mx(kNm/m)	HORIZONTAL FORCE H(kN/m)	ACTION POSITION Y(m)	POSITION MOMENT My(kNm/m)
VIRTUAL WALL OF WEIGHT	619.767	2.989	1852.52	0.000	0.000	0.00
LIVE LOAD	81.857	4.128	337.93	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-2.276	5.146	-11.71	35.286	1.883	66.45
TOTAL	699.349		2178.75	35.286		66.45

4.3.2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	619.767	2.989	1852.52	80.570	3.132	252.36
REAR EARTH PRESSURE 2	-3.292	5.146	-16.94	51.040	1.883	96.13
TOTAL	616.476		1835.58	131.610		348.49

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	619.767	2.989	1852.52	80.570	3.132	252.36
REAR EARTH PRESSURE 2	-3.292	5.146	-16.94	51.040	1.883	96.13
TOTAL	616.476		1835.58	131.610		348.49

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 10.097 \geqq F_{sa} = 1.50 \cdots OK$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 617.492 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 35.286 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.702 \geqq F_{saE} = 1.20 \cdots OK$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 616.476 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 131.610 (kN/m)

4.6 STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.873 \ (m)$$

d : R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

ΣMr : R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1840.81 (kN·m/m)

ΣMo : R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 66.45 (kN·m/m)

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 617.492 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d = -0.928 \leqq B/6 = 0.648 \ (m) \quad \cdot\cdot\cdot \ OK$$

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.7 EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.412 \ (m)$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1835.58 (kN·m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 348.49 (kN·m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 616.476 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$e = B/2 - d3_E = -0.467 \leqq B/3 = 1.297$ (m) ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.8 STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$$qs = \Sigma V / B = 179.781 \text{ (kN/m}^2\text{)}$$

qs : VERTICAL SUBGRADE REACTION ON R/E BOTTOM (kN/m^2)

ΣV : TOTAL VERTICAL LOAD ON R/E BOTTOM = 699.349 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.9　STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1　VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_{SE} = \Sigma V_E / B = 158.477$ (kN/m^2)

q_{SE} : VERTICAL SUBGRADE REACTION ON R/E BOTTOM
DURING EARTHQUAKE (kN/m^2)

ΣV_E : TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE
$= 616.476$ (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B : R/E BOTTOM WIDTH $= 3.890$ (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma (To \cdot \tan\delta) = 36.425 \ (kN/m)$$

To : TENSILE FORCE AGAINST WALL (kN/m)

δ : ANGLE OF WALL FRICTION (°) = $2/3\phi$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	ϕ (°)	tan δ	Vci (kN/m)
1	17.862	30.0	0.364	6.501
2	18.907	30.0	0.364	6.882
3	19.754	30.0	0.364	7.190
4	21.007	30.0	0.364	7.646
5	22.548	30.0	0.364	8.207
TOTAL				36.425

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 144.112\ (kN/m^2)$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 19.380\ (kN/m)$

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL $= 24.5\ (kN/m^3)$

Hta : TOTAL HEIGHT OF R/E $= 5.650$ (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL $= 0.140$ (m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE $= hf \cdot b \cdot \gamma c2 = 1.840\ (kN/m)$

b : WIDTH OF CONCRETE FOUNDATION $= 0.400$ (m)

hf : HEIGHT OF CONCRETE FOUNDATION $= 0.200$ (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION $= 23.0\ (kN/m^3)$

Vc : VERTICAL COMPONENT AGAINST WALL $= 36.425\ (kN/m)$

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$V_{cE} = \Sigma (T_{oE} \cdot \tan\delta_E) = 30.580 \text{ (kN/m)}$$

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	20.898	30.0	0.268	5.600
2	21.585	30.0	0.268	5.784
3	22.111	30.0	0.268	5.925
4	23.765	30.0	0.268	6.368
5	25.766	30.0	0.268	6.904
TOTAL				30.580

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 129.498\ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL $= 19.380\ (kN/m)$

$Wc2$: WEIGHT OF FOUNDATION CONCRETE $= 1.840\ (kN/m)$

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE $= 30.580\ (kN/m)$

5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

5.1 ORDINARY

5.1.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{Ai} = \frac{\sigma a \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
σa : ALLOWABLE TENSILE STRENGTH OF STRIP (N/mm^2)
Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)
t : THE THICKNESS OF THE STRIP (mm)
Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)
ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{Ai} (kN/m)
1	140.0	80.0	4.0	1.0	240.0	0.750	44.800
2	140.0	80.0	4.0	1.0	240.0	0.750	44.800
3	140.0	80.0	4.0	1.0	240.0	0.750	44.800
4	140.0	80.0	4.0	1.0	240.0	0.750	44.800
5	140.0	80.0	4.0	1.0	240.0	0.750	44.800

5.1.2 FORMULA FOR SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

$$F_s = \frac{R\Sigma\{c_{es}\cdot l + (W'\cdot\cos\alpha + T_{avail}\cdot\sin\theta)\tan\phi + T_{avail}\cdot\cos\theta\}}{R\Sigma(W\cdot\sin\alpha)}$$

F_s : SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

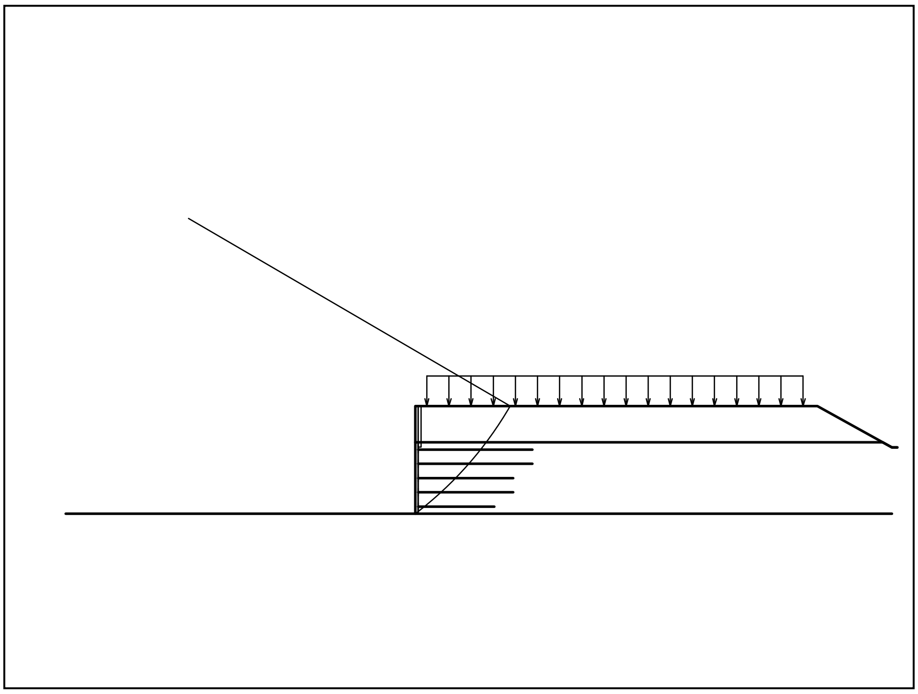
T_{avail} : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

R : RADIUS OF SLIP CIRCULAR ARC (m)

5.1.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [ORDINARY-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pi},\ T_{Ai})$$

$$T_{pi} = \frac{2 \cdot f_i^* \cdot \sigma_{vi} \cdot b \cdot L_{ei}}{F_s} \times \frac{1}{\triangle B}$$

T_{availi} : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP (kN/m)

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{vi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE $= \gamma \cdot z_i$ (kN/m^2)

z_i : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

L_{ei} : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

$\triangle B$: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_s : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

STAGE (i)	f i*	γ (kN/m^3)	z i (m)	σ vi (kN/m^2)	b (m)	L ei (m)	⊿B (m)	T pi (kN/m)	T Ai (kN/m)	T availi (kN/m)
1	1.2067	19.0 19.0	1.900 0.375	43.225	0.0800	2.715	0.750	15.106	44.800	15.106
2	1.1100	19.0 19.0	1.900 1.125	57.475	0.0800	3.339	0.750	22.723	44.800	22.723
3	1.0134	19.0 19.0	1.900 1.875	71.725	0.0800	3.032	0.750	23.507	44.800	23.507
4	0.9167	19.0 19.0	1.900 2.625	85.975	0.0800	3.802	0.750	31.962	44.800	31.962
5	0.8200	19.0 19.0	1.900 3.375	100.225	0.0800	3.666	0.750	32.138	44.800	32.138
								Σ Tavail = 125.435		

(3) LIST OF SAFETY FACTOR

Fs		COORDINATE OF CIRCULAR CENTER X						
		-12.2m	-12.1m	-12.0m	-11.9m	-11.8m	-11.7m	-11.6m
Y	15.8m	1.437	1.437	1.437	1.437	1.437	1.438	1.438
	15.7m	1.437	1.437	1.437	1.437	1.437	1.437	1.438
	15.6m	1.437	1.437	1.437	1.437	1.437	1.437	1.437
	15.5m	1.437	1.437	1.437	1.437	1.437	1.437	1.437
	15.4m	1.437	1.437	1.437	1.437	1.437	1.437	1.437
	15.3m	1.437	1.437	1.437	1.437	1.437	1.437	1.437
	15.2m	1.438	1.438	1.437	1.437	1.437	1.437	1.437

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	ORDINARY-1	JUDGE
MINIMUM SAFETY FACTOR	Fsmin Fs		1.437 (1.200)	○
Σ Tavail	kN/m		125.435	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN·m/m	[1480.6] [2790.6]	—
	M_R	kN·m/m	4271.3	—
	M_T	kN·m/m	2732.0	—
ACTIVE MOMENT	M_D	kN·m/m	4875.2	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-11.900 15.500	—
RADIUS	R	m	19.541	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN·m/m)

M_R : RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN·m/m)

M_T : RESISTER MOMENT OF STRIP (kN·m/m)

Fsmin : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP = $(M_R+M_T) / M_D$

5.2　EARTHQUAKE

5.2.1　THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{AEi} = \frac{\sigma_{aE} \cdot A_g}{\Delta B} \times 10^{-3}$$

T_{AEi} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σ_{aE} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH OF STRIP (N/mm^2)

A_g : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$A_g = b \cdot (t - C_m)$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

C_m : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σ_{aE} (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{AEi} (kN/m)
1	210.0	80.0	4.0	1.0	240.0	0.750	67.200
2	210.0	80.0	4.0	1.0	240.0	0.750	67.200
3	210.0	80.0	4.0	1.0	240.0	0.750	67.200
4	210.0	80.0	4.0	1.0	240.0	0.750	67.200
5	210.0	80.0	4.0	1.0	240.0	0.750	67.200

5.2.2 FORMULA FOR SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

$$F_{SE} = \frac{R\Sigma\{c_{es}\cdot l + (W'\cdot\cos\alpha - k_{h3}\cdot W\cdot\sin\alpha)\tan\phi\} + R\cdot\Sigma T_{avail}(\cos\theta + \sin\theta\cdot\tan\phi)}{\Sigma(R\cdot W\cdot\sin\alpha\ +\ k_{h3}\cdot W\cdot y_G)}$$

F_{SE} : SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

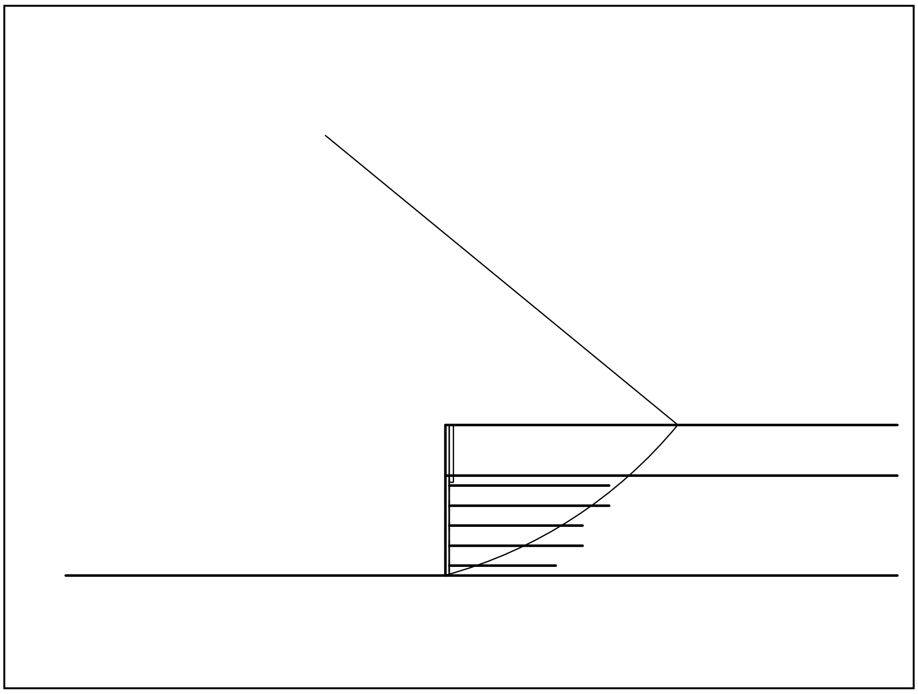
R : RADIUS OF SLIP CIRCULAR ARC (m)

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT = 0.12 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

y_G : VERTICAL DISTANCE FROM CENTER OF CIRCULAR ARC TO CENTER OF GRAVITY OF SPLIT PIECE (m)

5.2.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [EARTHQUAKE-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$Tavaili = \min(T_{pEi},\ T_{AEi})$$

$$T_{pEi} = \frac{2 \cdot fi^{*} \cdot \sigma_{VEi} \cdot b \cdot Lei}{F_{SE}} \times \frac{1}{\angle B}$$

$Tavaili$: AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pEi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

T_{AEi} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot zi$ (kN/m^2)

zi : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

Lei : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

$\angle B$: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ON THE EARTHQUAKE = 1.20

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{VEi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pEi} (kN/m)	T_{AEi} (kN/m)	T_{availi} (kN/m)
1	1.2067			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
2	1.1100	19.0 19.0	1.900 1.125	57.475	0.0800	0.640	0.750	7.259	67.200	7.259
3	1.0134	19.0 19.0	1.900 1.875	71.725	0.0800	0.773	0.750	9.988	67.200	9.988
4	0.9167	19.0 19.0	1.900 2.625	85.975	0.0800	2.149	0.750	30.110	67.200	30.110
5	0.8200	19.0 19.0	1.900 3.375	100.225	0.0800	2.940	0.750	42.955	67.200	42.955
								ΣT_{avail} = 90.312		

(3) LIST OF SAFETY FACTOR

F_{SE}		COORDINATE OF CIRCULAR CENTER X						
		-10.5m	-8.5m	-6.5m	-4.5m	-2.5m	-0.5m	1.5m
Y	22.5m	1.601	1.566	1.563	*****	*****	*****	*****
	20.5m	1.626	1.584	1.559	*****	*****	*****	*****
	18.5m	1.660	1.610	1.565	*****	*****	*****	*****
	16.5m	1.707	1.645	1.591	1.553	*****	*****	*****
	14.5m	1.776	1.698	1.629	1.569	*****	*****	*****
	12.5m	1.877	1.776	1.687	1.610	*****	*****	*****
	10.5m	2.036	1.901	1.780	1.677	*****	*****	*****

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	EARTHQUAKE-1	JUDGE
MINIMUM SAFETY FACTOR	F_{smin} F_{SE}		1.553 (1.000)	○
HORIZONTAL SEISMIC COEFFICIENT	k_{h3}		0.12	—
ΣT_{avail}	kN/m		90.312	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN·m/m	[1805.6] [4694.6]	—
	M_R	kN·m/m	6500.2	—
	M_T	kN·m/m	1767.0	—
ACTIVE MOMENT	M_D	kN·m/m	5326.8	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-4.500 16.500	—
RADIUS	R	m	17.103	—

M_{RC}	: RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)
M_{RF}	: RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN·m/m)
M_R	: RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN·m/m)
M_T	: RESISTER MOMENT OF STRIP (kN·m/m)
F_{smin}	: MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP = $(M_R+M_T) / M_D$

Main Road

Concrete Barrier

1 Design Conditons

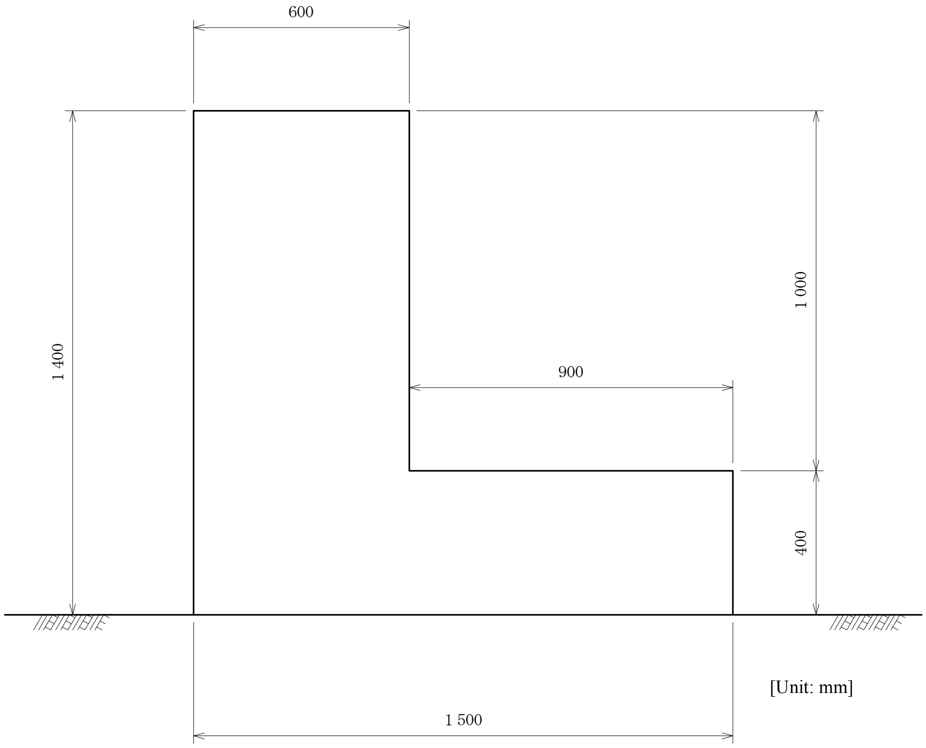
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 10000(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: $\sigma ck = 24$ (N/mm^2)
RC Bottom Slab: $\sigma ck = 24$ (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】 Soil at Rear Side: 30°

【Unit Weight】

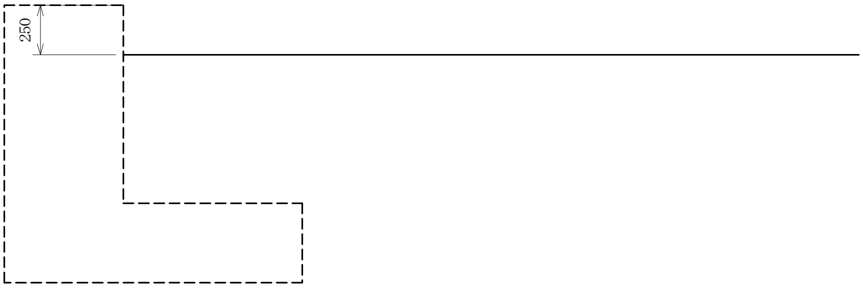
(kN/m^3)

Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】
Frame: Kh = 0.18
Soil (Front side): Kh = 0.18
Soil (Rear side): Kh = 0.18

1.6 Soil

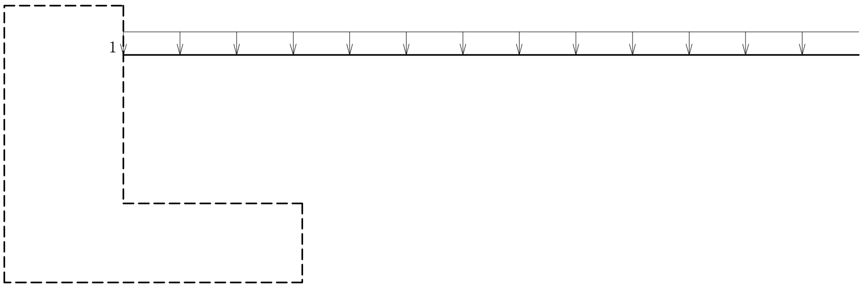
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr. In the stability Analysis (m)	0.000
In the vertical wall design(m)	0.000

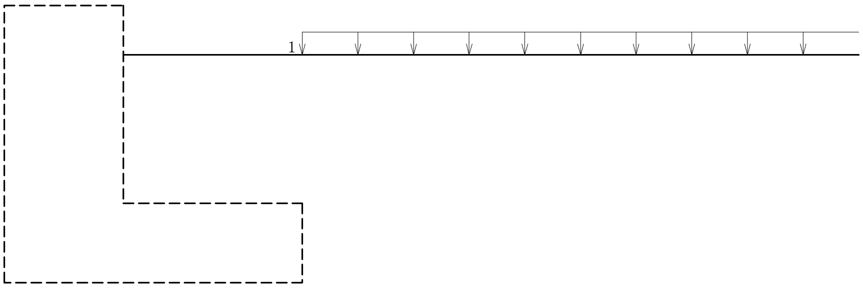
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

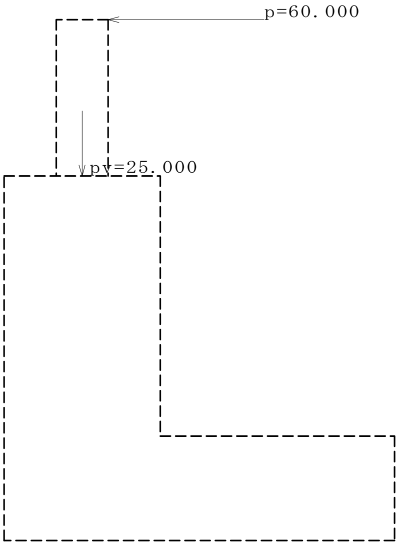


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.900	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.600	0.750	60.000

Vertical Force

Effective Width	Loading location x (m)	Front Wheel Load P (kN)
Same as horizontal force	0.300	25.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method

- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

- In the stability analysis the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).

- In the stability analysis, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	———
Earthquake	0.000	0.000	———

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	——	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	——	——	——	——
	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, tanφ_B	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B ／ V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	Shear Stress τ_{a2}	Rebar Compressive Stress σ_{sna}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700	200.000
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700	200.000
Earthquake	1.50	12.000	300.000	0.350	2.550	200.000
Collision	1.50	12.000	300.000	0.345	2.550	200.000

2) Bottom Slab (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}	Rebar Compressive Stress σ_{sna}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700	————
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700	————
Earthquake	1.50	12.000	300.000	0.350	2.550	————
Collision	1.50	12.000	300.000	0.345	2.550	————

Where,

τ_{a1} : Shear stress when shear force is bored by concrete

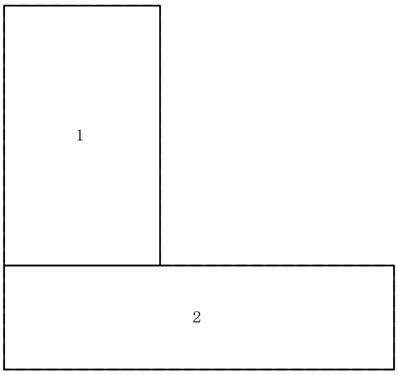
τ_{a2} : Shear stress when shared with diagonal tension bars

2 Stability Analysis

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



2) Volume・Center of Gravity

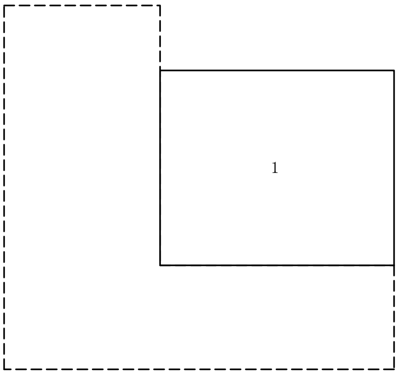
Classification	Width x Height x Depth	Vol. Vi (m³)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.600× 1.000× 1.000 1.500× 0.400× 1.000	0.600 0.600	0.300 0.750	0.900 0.200	0.180 0.450	0.540 0.120	
Σ		1.200	——	——	0.630	0.660	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.630／ 1.200 = 0.525 (m)

YG = Σ（Vi・Yi）／ΣVi = 0.660／ 1.200 = 0.550 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m³)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.900× 0.750× 1.000	0.675	1.050	0.775	0.709	0.523	
Σ		0.675	——	——	0.709	0.523	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.709／ 0.675 = 1.050 (m)

YG = Σ（Vi・Yi）／ΣVi = 0.523／ 0.675 = 0.775 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.200 = 29.400	0.525
Soil (Rear Side)	19.000 × 0.675 = 12.825	1.050
Total	42.225	0.684

[2] Earthquake

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.200 = 29.400	0.525
Soil (Rear Side)	19.000 × 0.675 = 12.825	1.050
Total	42.225	0.684

Location	Horizontal Force $H = W \cdot kh$ (kN)	Location Y (m)
Frame (Rebar)	29.400 × 0.18 = 5.292	0.550
Soil (Rear Side)	12.825 × 0.18 = 2.309	0.775
Total	7.601	0.618

2.3 Ground Surface Loading, Snow Load

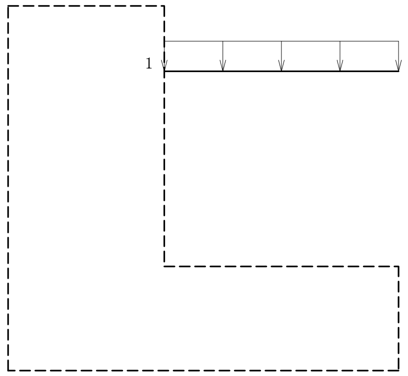
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

q : Load
L : Length of load
X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.900	10.440	1.050

2.4 Collision Load

Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 10.000(m)

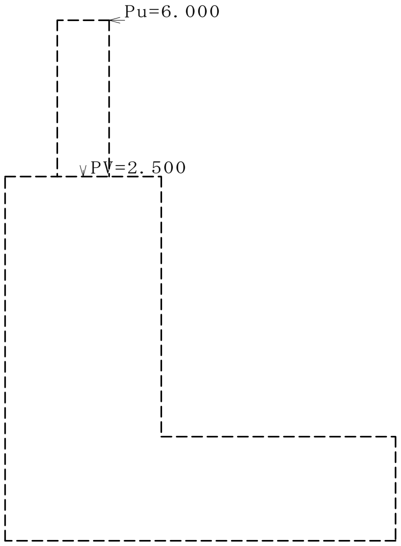
Vertical Force

$$Pv = \frac{pv}{L}$$

Where,

Pv : Front wheel load per unit width (kN/m)
pv : Front wheel load per block (kN)
L : Block length (m), L = 10.000(m)

[1] Collision



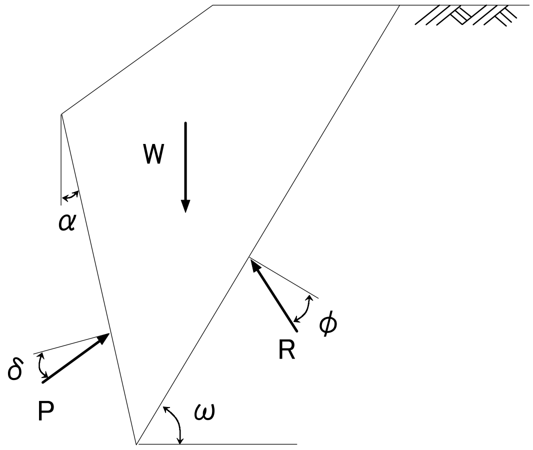
Horizontal Force

p (kN)	Pu (kN/m)	Location Y (m)
50.000	5.000	2.000

Vertical Force

pv (kN)	Pv (kN/m)	Location X (m)
25.000	2.500	0.300

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	8.015	15.564	8.627
60.00	7.254	0.000	7.702	14.956	8.635
61.00	6.965	0.000	7.394	14.359	8.628

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 8.635 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{14.956 \times \sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

$$= 8.635 \text{ kN}$$

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 8.635 \times \cos(0.000^\circ + 0.000^\circ) = 8.635$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 8.635 \times \sin(0.000^\circ + 0.000^\circ) = 0.000$ kN

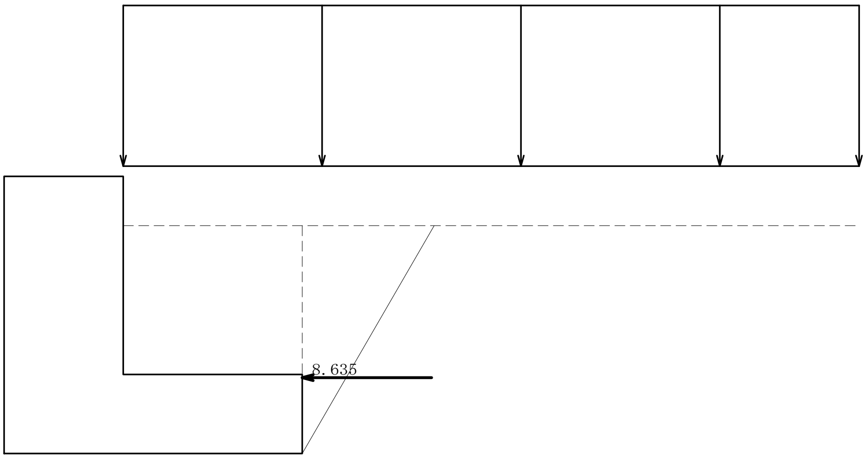
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000^\circ = 1.500$ m

$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	$xp = 1.500$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 1.150$ m
Angle to the Vertical of Earth Pressure Surface	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Angle of Ground Surface from the Horizontal	$\beta = 0.000$ °
Composite angle during earthquake	$\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204^\circ$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle $\omega i =$ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω ＝ 44.00° P ＝ 5.508 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{13.011/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

＝ 5.508 kN

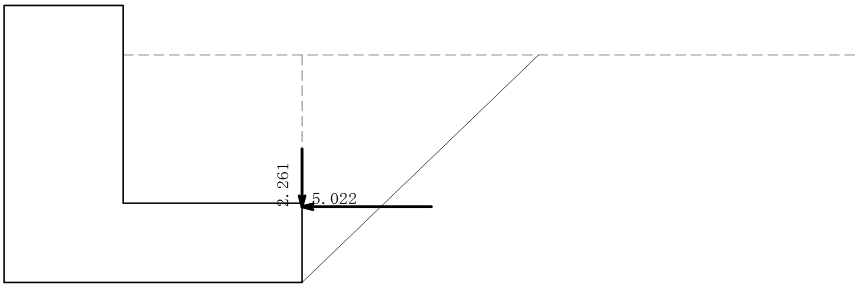
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 5.508 \times \cos(0.000° + 24.239°) = 5.022$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 5.508 \times \sin(0.000° + 24.239°) = 2.261$ kN
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m
$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

· Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	0.000	7.549	4.184
60.00	7.254	0.000	0.000	7.254	4.188
61.00	6.965	0.000	0.000	6.965	4.185

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 4.188 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.254 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

＝ 4.188 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 4.188 \times \cos(0.000° + 0.000°) = 4.188$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 4.188 \times \sin(0.000° + 0.000°) = 0.000$ kN

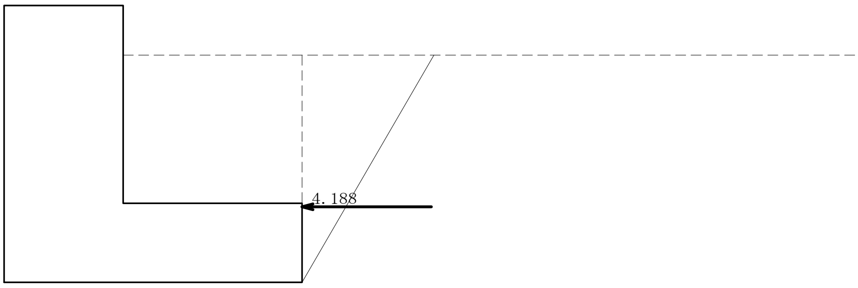
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m

$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

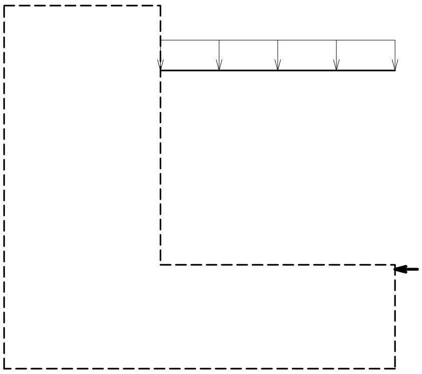
- Earth Pressure Diagram



2.6 Total Acting Force

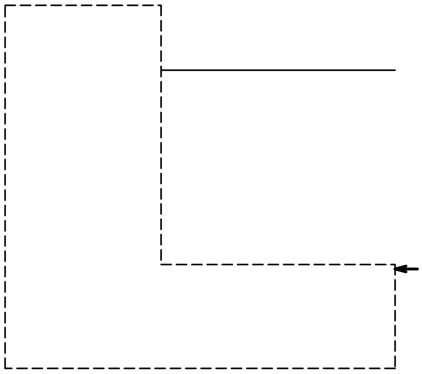
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



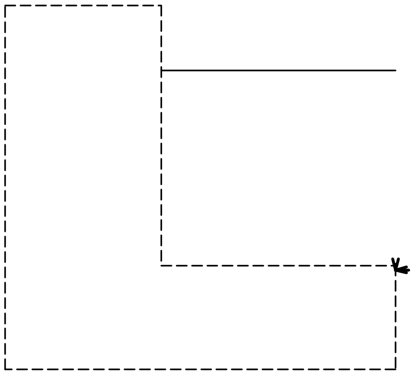
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	42.225	0.000	0.684	0.000	28.901	0.000
Loading, Snow	10.440	0.000	1.050	0.000	10.962	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	52.665	8.635	———	———	39.863	3.307

[2] Ordinary Condition (Rear Side)



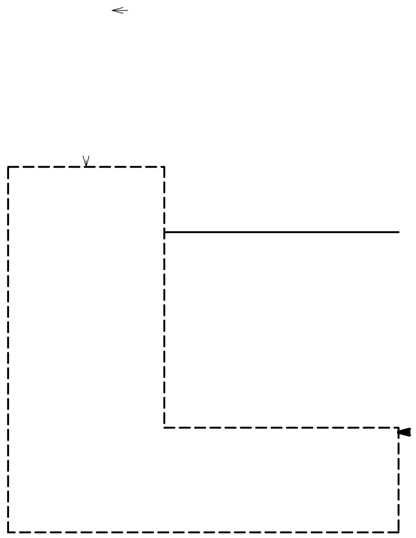
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	42.225	0.000	0.684	0.000	28.901	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	42.225	8.635	———	———	28.901	3.307

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	42.225	7.601	0.684	0.618	28.901	4.700
Earth Pressure	2.261	5.022	1.500	0.383	3.391	1.923
Total	44.486	12.622	———	———	32.293	6.623

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	42.225	0.000	0.684	0.000	28.901	0.000
Loading, Snow	0.000	4.188	1.500	0.383	0.000	1.604
Collision Load	2.500	5.000	0.300	2.000	0.750	12.000
Total	44.725	10.188	———	———	29.651	13.604

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	52.665	8.635	36.556
Ordinary (Rear Side)	42.225	8.635	25.594
Earthquake	44.486	12.622	25.670
Collision	44.725	10.188	16.047

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)

Horizontal Force : $H_c = H_o$ (kN)

Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,

Footing Earth Pressure Width : $B_j = 1.500$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	52.665	8.635	2.943
Ordinary (Rear Side)	42.225	8.635	6.075
Earthquake	44.486	12.622	7.695
Collision	44.725	10.188	17.497

- Full Width per 10m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	526.650	86.350	29.428
Ordinary (Rear Side)	422.250	86.350	60.748
Earthquake	444.860	126.225	76.949
Collision	447.250	101.880	174.966

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B ＝ 1.500

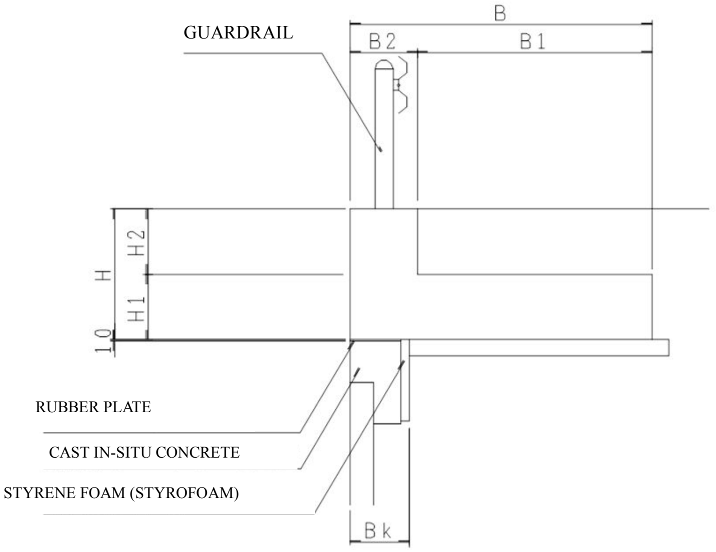
$$e_a = B/n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	39.863	3.307	52.665	0.694	0.056	≦	0.250	○
Ordinary (Rear Side)	28.901	3.307	42.225	0.606	0.144	≦	0.250	○
Earthquake	32.293	6.623	44.486	0.577	0.173	≦	0.500	○
Collision	29.651	13.604	44.725	0.359	0.391	≦	0.500	○

2.7.2 Stability from Sliding



H = 1.150 m
H 1 = 0.400 m
H 2 = 0.750 m

B = 1.500 m
B 1 = 0.900 m
B 2 = 0.600 m

B k = 0.350 m

Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m^2)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.144	1.500	11.951	44.349	44.349	36.789
Earthquake	0.173	1.500	9.138	50.177	50.177	40.601
Collision	0.346	1.212	0.000	73.804	73.804	52.491

Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Ｎｎａ (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN'=ΣN-Ｎｎａ	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	42.225	14.199	28.600	8.635
Earthquake	44.486	15.886	28.600	12.622
Collision	44.725	24.346	20.379	10.188

Factor of safety for sliding, Fs:

$$Fs = \Sigma N' \cdot \mu / \Sigma H$$

Load Condition	Factor of Safety Fs	Required Factor of Safety Fsa
Ordinary (Rear Side)	1.947	≧1.5
Earthquake	1.360	≧1.2
Collision	1.200	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

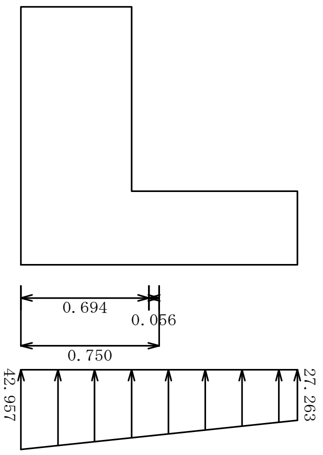
2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

Where,

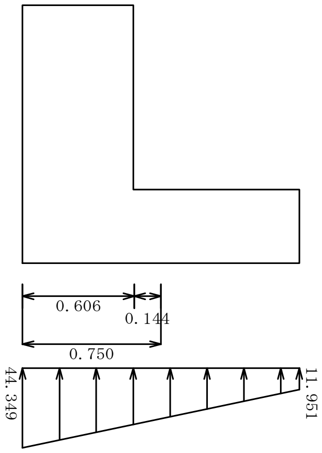
ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)
B : Bottom plate width (m), B = 1.500
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



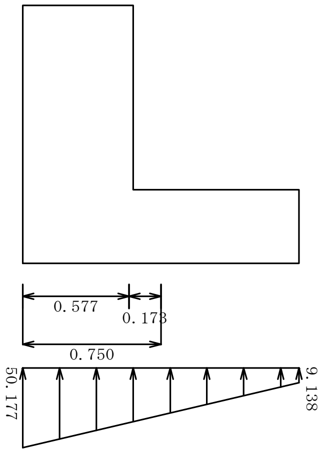
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.500	Trapezoid	27.263	42.957	≦ 200.000	○

[2] Ordinary Condition (Rear Side)



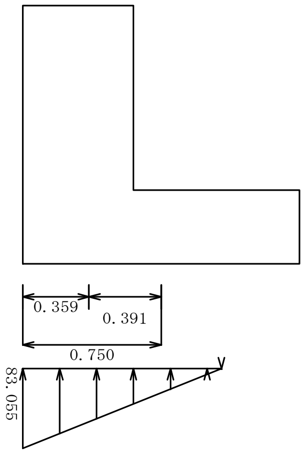
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.500	Trapezoid	11.951	44.349 ≦	200.000	○

[3] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.500	Trapezoid	9.138	50.177 ≦	300.000	○

[4] Collision



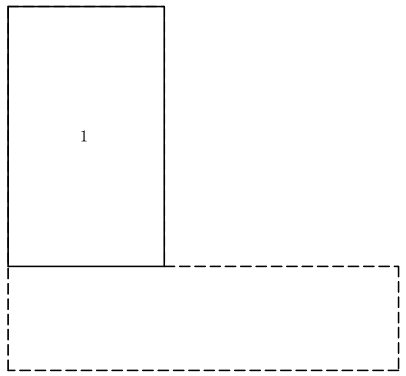
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.077	Triangle	0.000	83.055 ≦	300.000	○

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m³)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.600× 1.000× 1.000	0.600	0.300	0.500	0.180	0.300	
Σ		0.600	—	—	0.180	0.300	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.180／ 0.600 = 0.300 (m)

YG = Σ（Vi・Yi）／ΣVi = 0.300／ 0.600 = 0.500 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W = γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.600 = 14.700	0.000

Force Location

X ＝ Xc－XG ＝ 0.300－0.300

＝ 0.000 m

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W = γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.600 = 14.700	0.000

Location	H ＝ W ・ kh (kN)	Location Y (m)
Frame(Rebar)	14.700 × 0.180 ＝ 2.646	0.500

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

Horizontal Force

$$Pu = \frac{p}{\lambda + y}$$

Where,

Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
y : Distance from the tip of section being checked (m), y ＝ 1.000(m)

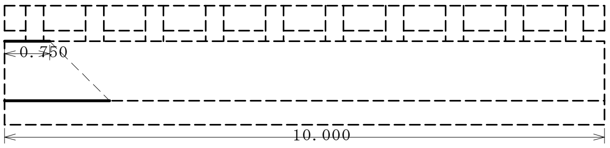
Vertical Force

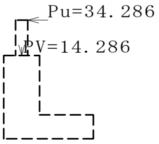
$$Pv = \frac{pv}{\lambda + y}$$

Where,

Pv : Front wheel load of the effective width (kN/m)
pv : Front wheel load per block (kN)

[1] Collision





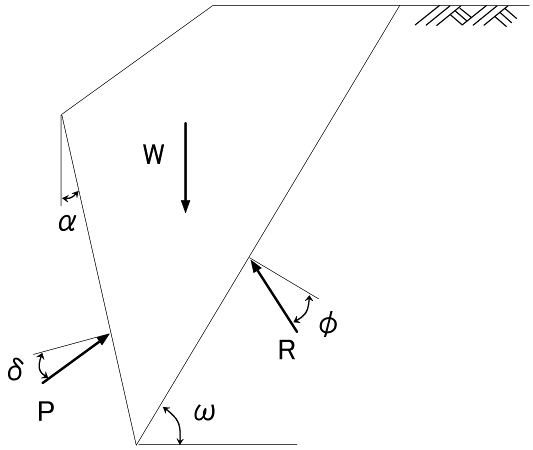
Horizontal Force

p (kN)	λ (m)	Pu (kN/m)	Location Y (m)
60.000	0.750	34.286	1.600

Vertical Force

pv (kN)	λorL (m)	Pu (kN/m)	Location X (m)
25.000	0.750	14.286	0.000

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.300 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.750 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	6.092	9.834	4.172
56.00	3.605	0.000	5.868	9.473	4.176
57.00	3.471	0.000	5.650	9.121	4.172

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 4.176 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{9.473 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

$$= 4.176 \text{ kN}$$

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 4.176 \times \cos(0.000° + 20.000°) = 3.924$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 4.176 \times \sin(0.000° + 20.000°) = 1.428$ kN

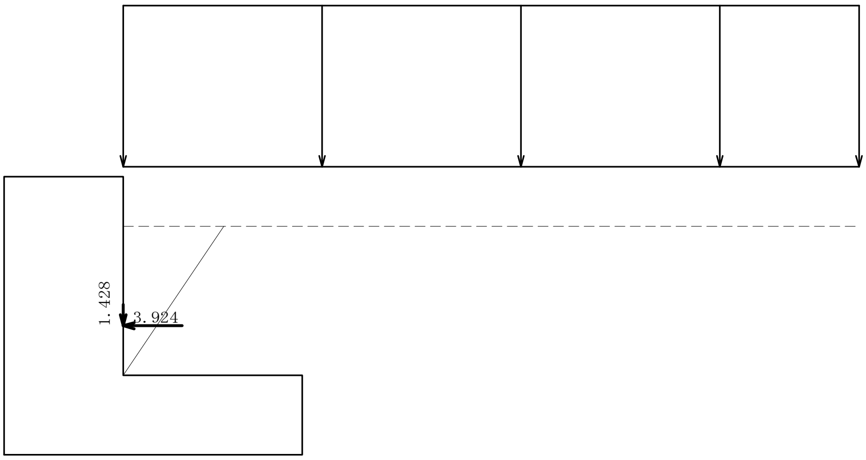
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.300 = -0.300$ m

$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

- Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	$xp = 0.300$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 0.750$ m
Angle of Earth Pressure Acting Surface to the Vertical	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Wall Friction Angle	$\delta = 2/3\varphi = 20.000$ °
Rate of Change of Slip Angle	$\omega i = 10.00$ °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	0.000	3.742	1.587
56.00	3.605	0.000	0.000	3.605	1.589
57.00	3.471	0.000	0.000	3.471	1.588

Earth pressure becomes maximum when:
$\omega = 56.00°$ $P = 1.589$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{3.605 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

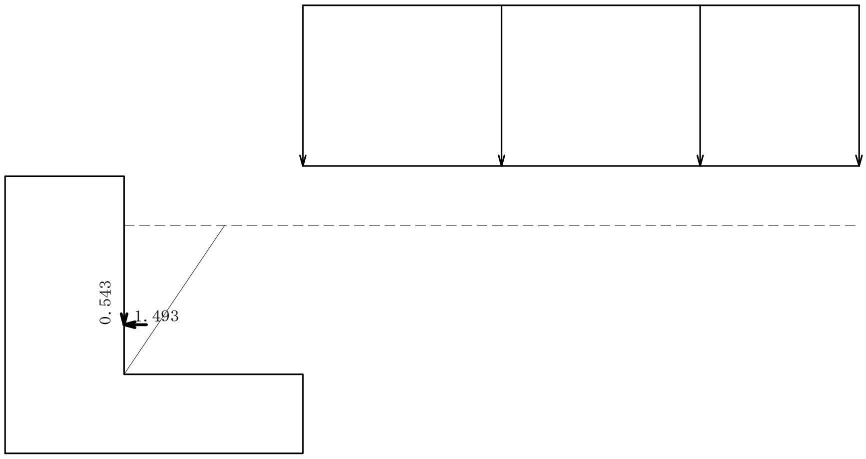
$= 1.589$ kN

Earth Pressure's location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 1.589 \times \cos(0.000° + 20.000°) = 1.493$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 1.589 \times \sin(0.000° + 20.000°) = 0.543$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.300$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.750$ m
Angle to the Vertical of Earth Pressure Surface $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 1/2\varphi = 15.000$ °
Composite angle during earthquake $\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204°$

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °
Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
45.00	5.344	0.000	0.000	5.344	2.312
46.00	5.161	0.000	0.000	5.161	2.316
47.00	4.983	0.000	0.000	4.983	2.316

Earth pressure becomes maximum when:
ω = 46.00° P = 2.316 kN

土圧力

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.161/\cos 10.204^\circ \times \sin(46.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(46.00^\circ - 30.00^\circ - 0.000^\circ - 15.000^\circ)}$$

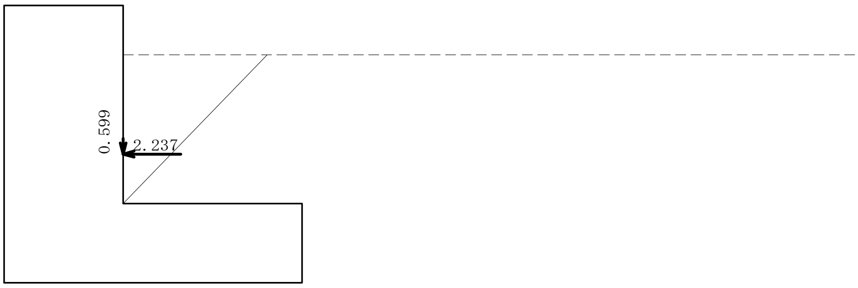
= 2.316 kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 2.316 \times \cos(0.000^\circ + 15.000^\circ) = 2.237$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 2.316 \times \sin(0.000^\circ + 15.000^\circ) = 0.599$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000^\circ - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

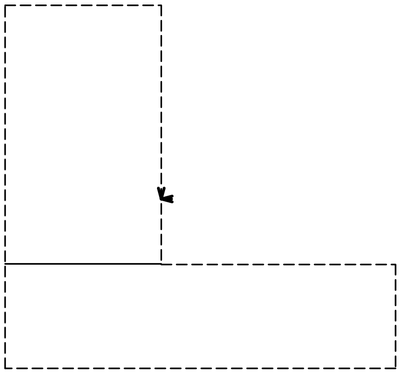
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

[1] Ordinary Condition (Surcharge)

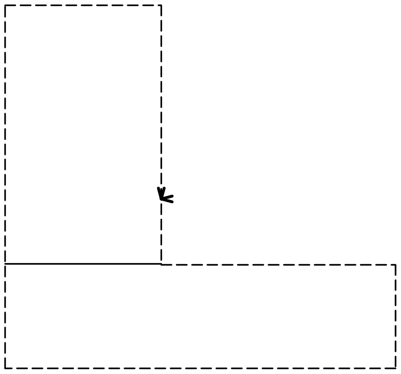


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	14.700	0.000	0.000	0.000	0.000
Earth Pressure	1.428	3.924	-0.300	0.250	0.981
Total	0.000	3.924	―――	―――	0.981

項　目	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN. m)
自　重	14. 700	0. 000	0. 000	0. 000	0. 000
土　圧	1. 428	3. 924	-0. 300	0. 250	0. 981
合　計	0. 000	3. 924	―――	―――	0. 981

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

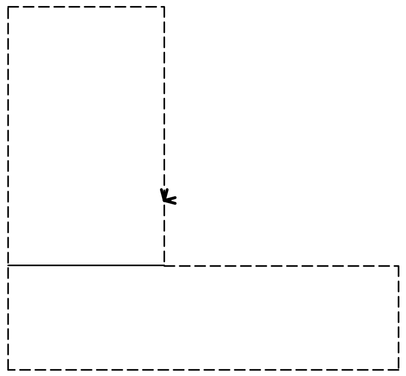
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	14.700	0.000	0.000	0.000	0.000
Earth Pressure	0.543	1.493	-0.300	0.250	0.373
Total	0.000	1.493	―――	―――	0.373

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

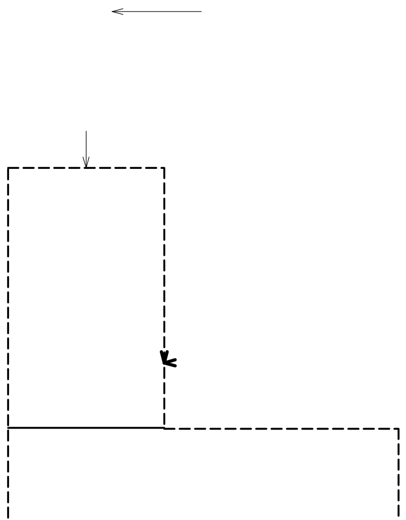
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	14.700	2.646	0.000	0.500	1.323
Earth Pressure	0.599	2.237	-0.300	0.250	0.559
Total	0.000	4.883	———	———	1.882

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

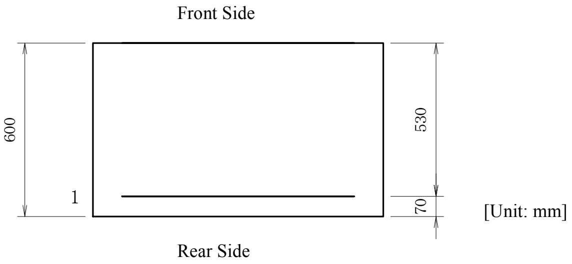


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	14.700	0.000	0.000	0.000	0.000
Earth Pressure	0.543	1.493	-0.300	0.250	0.373
Collision Load	14.286	34.286	0.000	1.600	54.857
Total	0.000	35.779	———	———	55.230

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	7.0	D13	1.267	4.000	5.068
	2	—	–	—	—	—

Required rebar amount at tension side: 3.636 (cm^2)

(1) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 60000.0 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 600000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 600.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judge ment
Ordinary (Surcharge)	5.068	1.668	≦	114.821	0.000	5.000	○
Ordinary (Rear Side)	5.068	0.635	≦	114.821	0.000	5.000	○
Earthquake	5.068	3.200	≦	114.821	0.000	5.000	○
Collision	5.068	93.892	≦	114.821	0.000	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 （$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2 （$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars （$d_i \leqq x$）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars （$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), σ_{sy} = 345.00
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 530.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), σ_{ck} = 24.00
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	169.941	○
Ordinary (Rear Side)	5.068	≦	169.941	○
Earthquake	5.068	≦	169.941	○
Collision	5.068	≦	169.941	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

Stress Calculation

$$\sigma_c=\frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s=n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h ＝ 600.000
b : Member cross-section width (mm), b ＝ 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n ＝ 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
M : Bending moment (N.mm)

Load Conditon	M	N	x	Comp. Stress (N/mm^2)	Tensile Stress (N/mm^2)	Judgement

(Water Level)	(kN.m)	(kN)	(cm)	Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	0.000	8.247	0.047	≦	8.000	3.852	≦	180.000	○
Ordinary (Rear Side)	0.373	0.000	8.247	0.018	≦	8.000	1.466	≦	180.000	○
Earthquake	1.882	0.000	8.247	0.091	≦	12.000	7.391	≦	300.000	○
Collision	55.230	0.000	8.247	2.664	≦	12.000	216.879	≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)
S_h : Shear force (N)
d : Sectional member’s effective height (mm)
b : Sectional member’s effective width (mm)
τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

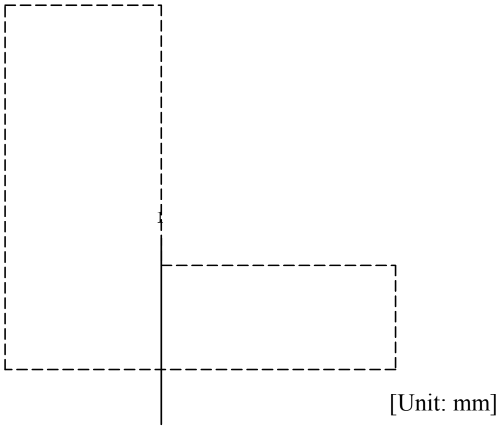
τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)
Ce : Correction factor for the sectional member’s effective height
Cpt : Correction factor for the tensile main reinforcement ratio, Pt
CN : Correction factor from axial compression force
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	3.924	53.000	0.007	≦	0.202	1.700	1.27	0.69	1.00	○
Ordinary(Rear Side)	1.493	53.000	0.003	≦	0.202	1.700	1.27	0.69	1.00	○
Earthquake	4.883	53.000	0.009	≦	0.307	2.550	1.27	0.69	1.00	○
Collision	35.779	53.000	0.068	≦	0.303	2.550	1.27	0.69	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

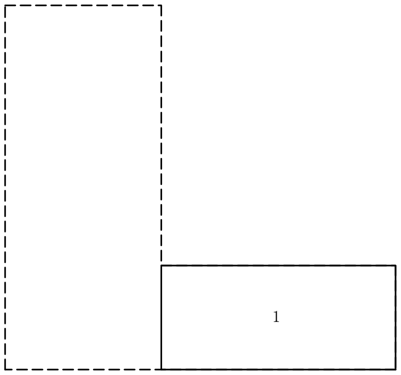
Distance from joint: 0.000 m



4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



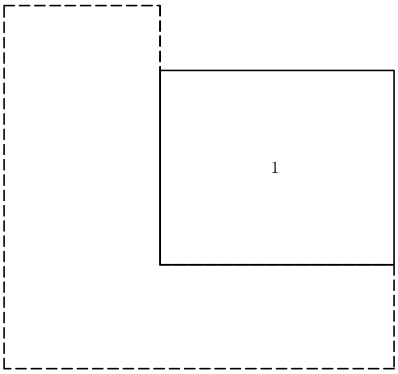
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.900× 0.400× 1.000	0.360	0.450	0.162	
Σ		0.360	—	0.162	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.162／ 0.360 ＝ 0.450 (m)

(2) Soil at Rear Side

1) Block Division



1) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.900× 0.750× 1.000	0.675	0.450	0.304	
Σ		0.675	——	0.304	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.304／0.675 ＝ 0.450 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force W ＝ γ ・ V (kN)	Location X (m)
Frame	24.500 × 0.360 ＝ 8.820	0.450
Soil(Rear Side)	19.000 × 0.675 ＝ 12.825	0.450

4.1.3 Ground Surface Load・Snow Load

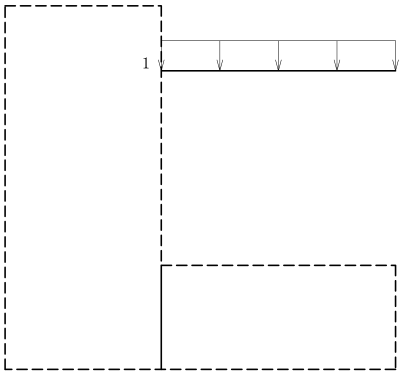
Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

Where,

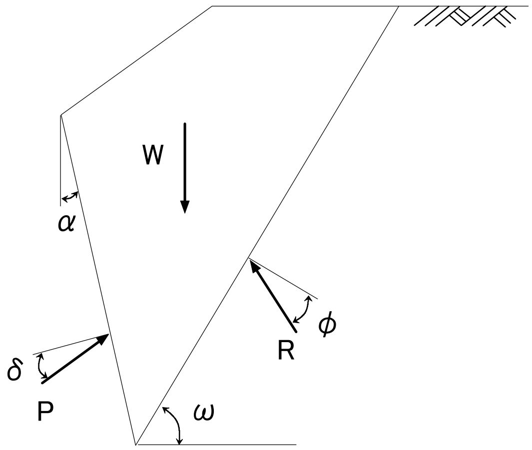
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1]　Ordinary Condition (Surcharge)



No.	q1 (kN/m²)	q2 (kN/m²)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.900	10.440	0.450

4.1.4　Earth Pressure



[1]　Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	$xp = 1.500$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 1.150$ m
Angle to the vertical of the Earth Pressure Surface	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Angle to the Horizontal of the Ground Surface	$\beta = 0.000$ °
Composite Angle during Earthquake	$\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi\cdot\sin(\theta+\Delta-\beta)}{1-\sin\phi\cdot\cos(\theta+\Delta-\beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times\sin(10.204° +20.751° -0.000°)}{1-\sin 30.00° \times\cos(10.204° +20.751° -0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta+\theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° +10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
Wen ω ＝ 44.00°, P ＝ 5.508 kN

Earth Pressure

$$P = \frac{W/\cos\theta\cdot\sin(\omega-\phi+\theta)}{\cos(\omega-\phi-\alpha-\delta)}$$

$$= \frac{13.011/\cos 10.204° \times\sin(44.00° -30.00° +10.204°)}{\cos(44.00° -30.00° -0.000° -24.239°)}$$

$$= 5.508 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component

$Pv = P\cdot\sin(\alpha+\delta) = 5.508\times\sin(0.000°+24.239°) = 2.261$ kN

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2\cdot Pv}{L} = \frac{2\times 2.261}{0.900} = 5.024 \text{ kN/m}$$

Where,

pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.000 m
Width of distributed load perpendicular to the design section L2 = 0.900 m

Distributed Load at the Design $\mathrm{pd} = \frac{\mathrm{pv}}{\mathrm{L}} \cdot \mathrm{L1} = \frac{5.024}{0.900} \times 0.000 = 0.000$ kN/m

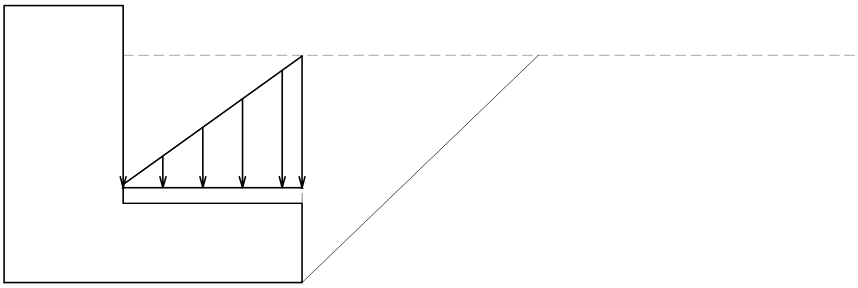
Vertical Force

$$N = \frac{1}{2} \cdot (\mathrm{pd}+\mathrm{pv}) \cdot \mathrm{L2} = \frac{1}{2} \times (0.000+5.024) \times 0.900 = 2.261 \text{ kN}$$

Location

$$x = \frac{\mathrm{pd}+2 \cdot \mathrm{pv}}{\mathrm{pd}+\mathrm{pv}} \cdot \frac{\mathrm{L2}}{3} = \frac{0.000+2\times5.024}{0.000+5.024} \times \frac{0.900}{3} = 0.600 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(\mathrm{q1}+\mathrm{q2}) \cdot \mathrm{L}$$

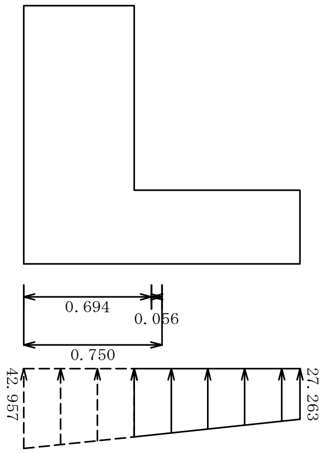
Location

$$X = \frac{2 \cdot \mathrm{q1}+\mathrm{q2}}{3 \cdot (\mathrm{q1}+\mathrm{q2})} \cdot \mathrm{L}$$

Where,

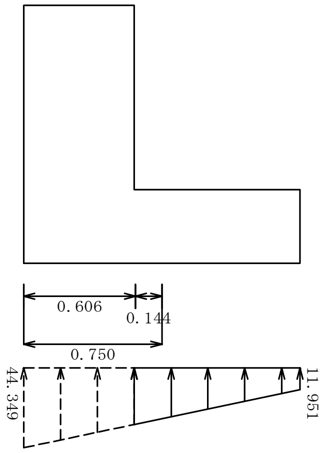
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



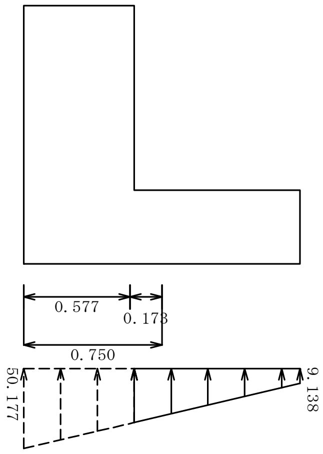
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
27.263	36.679	0.900	28.774	0.428

[2] Ordinary Condition (Rear Side)



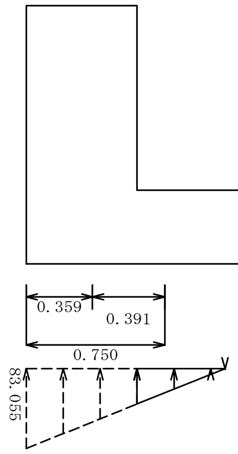
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
11.951	31.390	0.900	19.503	0.383

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
9.138	33.761	0.900	19.305	0.364

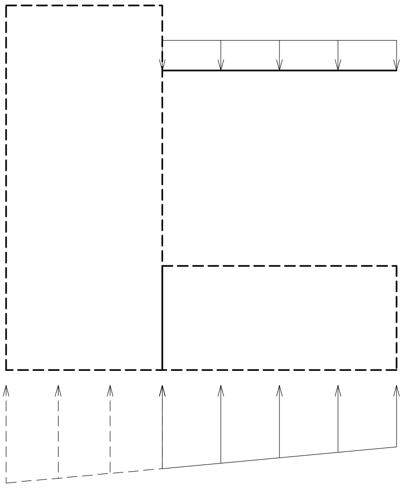
[4] Collision



Ground Reaction (kN/m²)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	36.785	0.477	8.773	0.159

4.1.6 Total Sectional Forces

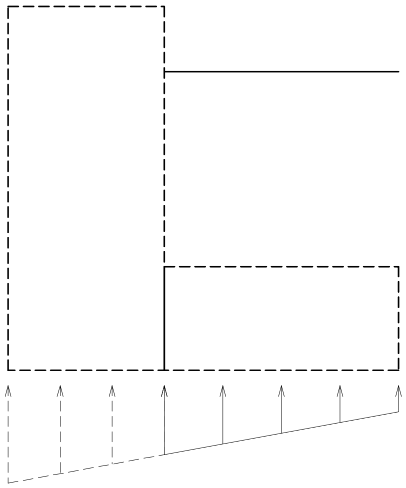
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	21.645	0.450	9.740
Loading, Snow Load	10.440	0.450	4.698
Ground Reaction	-28.774	0.428	-12.313
Total	3.311	———	2.125

Sectional force of vertical wall foundation, M1 ＝ 0.981 kN.m
Sectional force of the base of heel, M3 ＝ 2.125 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

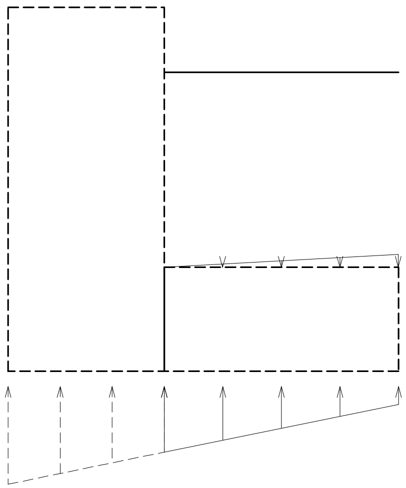
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	21.645	0.450	9.740
Loading, Snow Load	-19.503	0.383	-7.464
Ground Reaction	2.142	———	2.276
Total	21.645	0.450	9.740

Sectional force of vertical wall foundation, M1 ＝ 0.373 kN.m
Sectional force of the base of heel, M3 ＝ 2.276 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

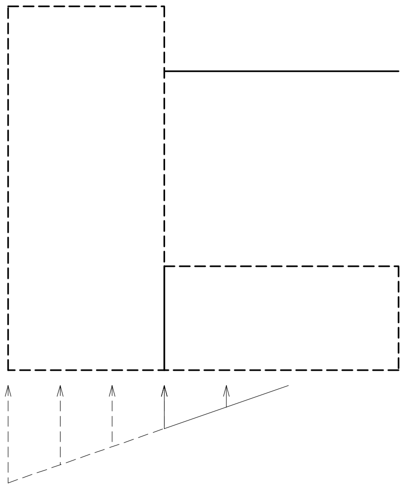
[3] Earthquake



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	21.645	0.450	9.740
Loading, Snow Load	2.261	0.600	1.357
Ground Reaction	-19.305	0.364	-7.025
Total	4.601	———	4.072

Sectional force of vertical wall foundation, M1 ＝ 1.882 kN.m
Sectional force of the base of heel, M3 ＝ 4.072 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

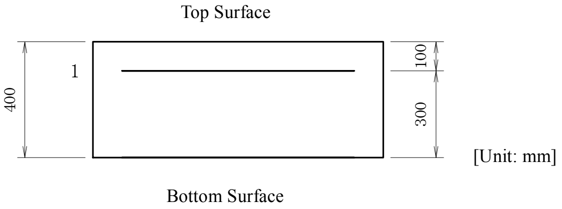
[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	21.645	0.450	9.740
Ground Reaction	-8.773	0.159	-1.395
Total	12.872	——	8.345

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.0	D13	1.267	4.000	5.068
	2	—	–	——	—	——
Bottom Surface	1'	—	–	——	—	——
	2'	—	–	——	—	——

Required tension side rebar amount 0.957 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 400.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m) ×1.7 M_c (kN.m)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	1.668 ≦ 51.031	5.000	○
Ordinary (Rear Side)	5.068	0.635 ≦ 51.031	5.000	○
Earthquake	5.068	3.200 ≦ 51.031	5.000	○
Collision	5.068	12.603 ≦ 51.031	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1（$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$y2 = x - y1$

$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$

Compressive force at the compression side rebars（$d_i \leqq x$）

$\varepsilon_i = A \cdot (x - d_i)$

$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$

$S_{ci} = \sigma si \cdot Asi$

$S1 = \Sigma S_{ci}$

Tensile force at the tension side rebars（$d_j > x$）

$\varepsilon_j = A \cdot (d_j - x)$

$\sigma_{sj} = \varepsilon_j \cdot E_s$

$S_{tj} = \sigma_{sj} \cdot A_{sj}$

$S2 = \Sigma S_{tj}$

From the axial force balance

$N = C1 + C2 + S1 - \alpha \cdot S2$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$A_{sb} = \alpha \cdot \Sigma A_{sj}$

Where,

A_{sb} : Balanced rebar amount (mm^2)

σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$

E_s : Rebar Young’s Modulus (N/mm^2), $Es = 200000$

d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0

b : Width of member (mm), b = 1000.0

σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$

d_i : Distance fr. compression side edge to the “ith” compression side rebar (mm)

A_{si} : Area enclosed by the compression side edge to the “ith” compression side rebar (mm^2)

σ_{si} : Compressive stress from compression side edge to the “ith” compression side rebar (N/mm^2)

d_j : Distance fr. compression side edge to the “jth” tension side rebar (mm)

A_{sj} : Area eclosed by the compression side edge to the “jth” tension side rebar (mm^2)

σ_{sj} : Tensile stress from compression side edge to the “jth” tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2 \cdot n}{b}\left\{As \cdot (x-d)\right\} = 0.0$$

よりxを求める。

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2} \cdot \left(\frac{h}{2} - \frac{x}{3}\right) + n \cdot As \cdot \frac{(x-d) \cdot (h/2-d)}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h ＝ 400.000

b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^2)

n : Rebar and concrete's coefficient of Young's modulus, n ＝ 15.00

e : Distance from the sectional member's centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^2)

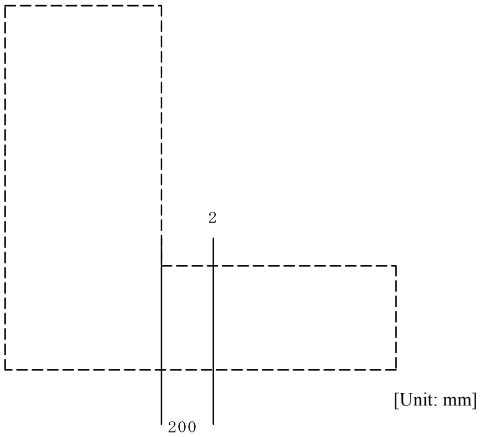
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	6.035	0.116	≦	8.000	6.916	≦	180.000	○
Ordinary (Rear Side)	0.373	6.035	0.044	≦	8.000	2.632	≦	180.000	○
Earthquake	1.882	6.035	0.223	≦	12.000	13.270	≦	300.000	○
Collision	8.345	6.035	0.988	≦	12.000	58.837	≦	300.000	○

4.2 Design of Section [2]

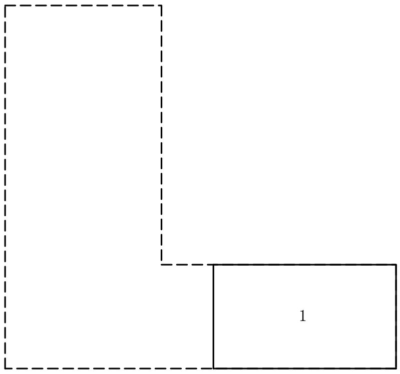
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



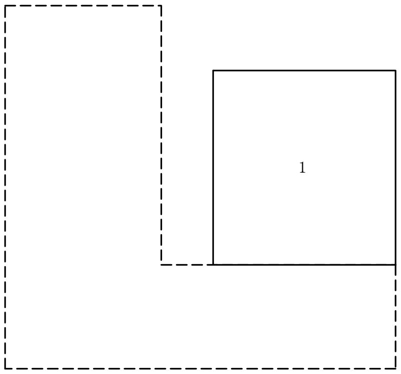
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.700× 0.400× 1.000	0.280	0.350	0.098	
Σ		0.280	——	0.098	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.098／ 0.280 ＝ 0.350 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.700× 0.750× 1.000	0.525	0.350	0.184	
Σ		0.525	——	0.184	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.184／ 0.525 ＝ 0.350 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.280 ＝ 6.860	0.350
Soil(Rear Side)	19.000 × 0.525 ＝ 9.975	0.350

4.2.3 Ground Surface Load・Snow Load

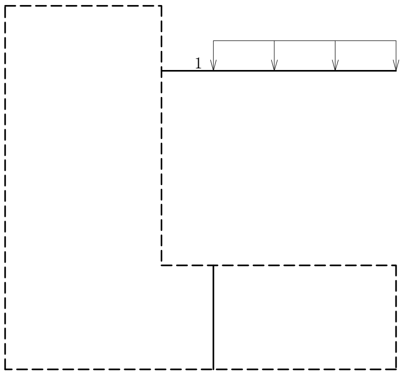
Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

Where,

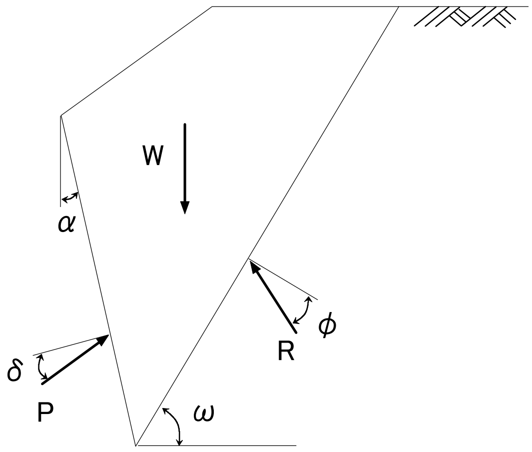
q : Ground surface loading
L : Length of ground surface load
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m²)	q2 (kN/m²)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.700	8.120	0.350

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) xp = 1.500 m

yp = 0.000 m

Height of Assumed Rear Surface H = 1.150 m

Angle to the vertical of the Earth Pressure Surface α = 0.000 °

Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3

Angle of Internal Friction of Soil at Rear Side φ = 30.000 °

Angle to the Horizontal of the Ground Surface β = 0.000 °

Composite Angle during Earthquake θ = $\tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta+\Delta-\beta)}{1-\sin\phi \cdot \cos(\theta+\Delta-\beta)}$$

$$= \tan^{-1}\frac{\sin 30.00^\circ \times \sin(10.204^\circ + 20.751^\circ - 0.000^\circ)}{1-\sin 30.00^\circ \times \cos(10.204^\circ + 20.751^\circ - 0.000^\circ)}$$

$$= 24.239^\circ$$

$$\Delta = \sin^{-1}\frac{\sin(\beta+\theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000^\circ + 10.204^\circ)}{\sin 30.00^\circ} = 20.751^\circ$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω ＝ 44.00°, P ＝ 5.508 kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega-\phi+\theta)}{\cos(\omega-\phi-\alpha-\delta)}$$

$$= \frac{13.011/\cos 10.204^\circ \times \sin(44.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(44.00^\circ - 30.00^\circ - 0.000^\circ - 24.239^\circ)}$$

$$= 5.508 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component

$$Pv = P \cdot \sin(\alpha+\delta) = 5.508\times\sin(0.000^\circ+24.239^\circ) = 2.261 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2 \cdot Pv}{L} = \frac{2\times\ 2.261}{0.900} = 5.024 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.200 m
Width of distributed load perpendicular to the design section L2 = 0.700 m

Distributed Load at the Design Section, pd

Distributed Load at the Design Section $$pd = \frac{pv}{L} \cdot L1 = \frac{5.024}{0.900} \times 0.200 = 1.117 \text{ kN/m}$$

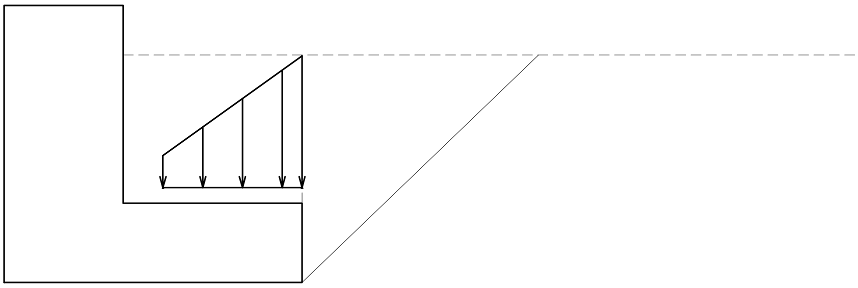
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (1.117 + 5.024) \times 0.700 = 2.149 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{1.117 + 2 \times 5.024}{1.117 + 5.024} \times \frac{0.700}{3} = 0.424 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

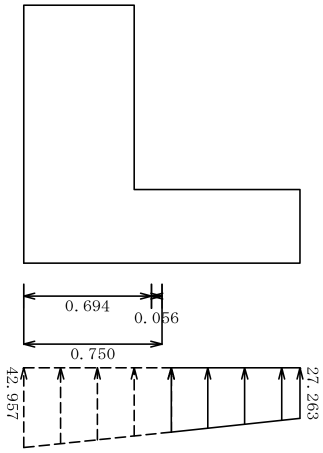
Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

Where,

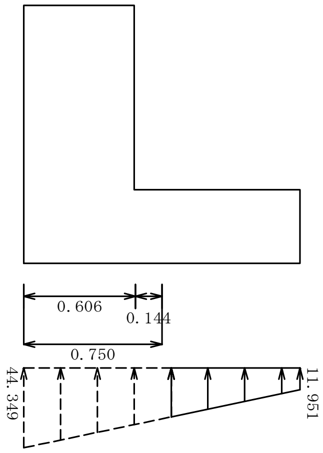
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



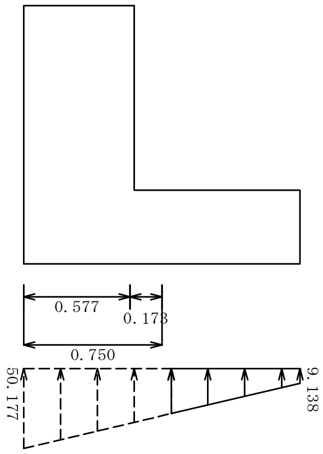
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
27.263	34.587	0.700	21.647	0.336

[2] Ordinary Condition (Rear Side)



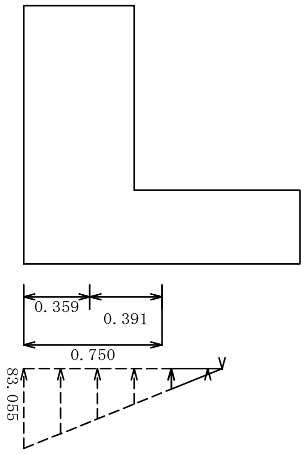
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
11.951	27.070	0.700	13.657	0.305

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
9.138	28.290	0.700	13.100	0.290

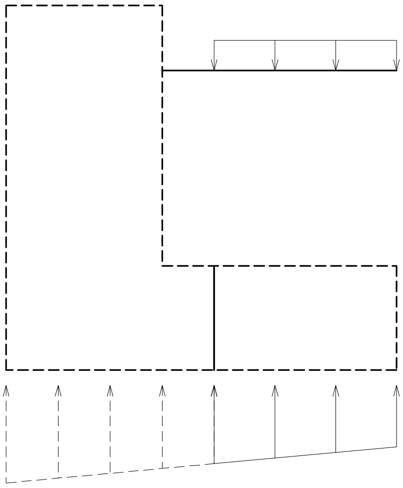
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	21.361	0.277	2.959	0.092

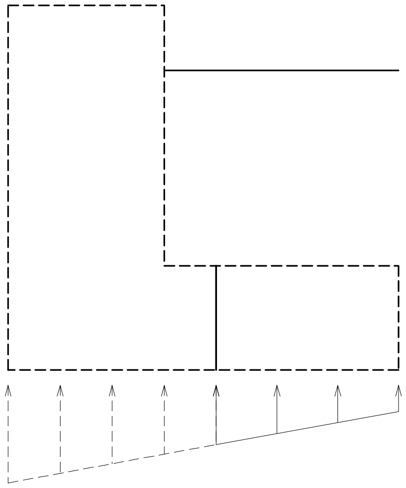
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



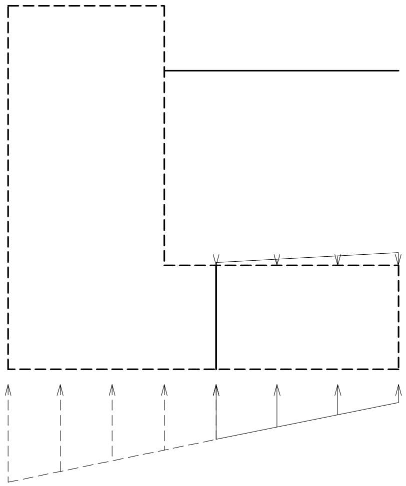
Item	N_i (kN)	X_i (m)	$M\ = N_i$・X_i (kN.m)
Self-weight	16.835	0.350	5.892
Loading, Snow Load	8.120	0.350	2.842
Ground Reaction	-21.647	0.336	-7.278
Total	3.308	———	1.457

[2] Ordinary Condition (Rear Side)



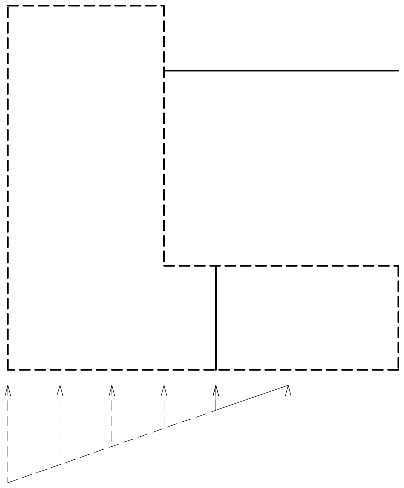
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	16.835	0.350	5.892
Ground Reaction	-13.657	0.305	-4.163
Total	3.178	———	1.730

[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	16.835	0.350	5.892
Loading, Snow Load	2.149	0.424	0.912
Ground Reaction	-13.100	0.290	-3.803
Total	5.885	———	3.001

[4] Collision



Item	N_i (kN)	X_i (m)	M $=N_i$・X_i (kN.m)
Self-weight	16.835	0.350	5.892
Ground Reaction	-2.959	0.092	-0.273
Total	13.876	———	5.619

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ $a > 2.5d$,

when $a \leqq 2.5d$, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)

d : Effective height of footing calculated from the column, front wall or rear wall (mm)

d' : Effective height of sectional member (mm)

b : Effective width of sectional member (mm)

S : Shear force acting on the sectional member (N)

M : Bending moment acting on the sectional member (N.mm)

θ : Angle of the top surface footing to the horizontal, $\theta = 0.000$, $\tan\theta = 0.000$

a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span 2.5・d a (cm)	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000 ≦ 94.200	3.308	1.457	0.000	3.308
Ordinary (Rear Side)	30.000	75.000 ≦ 120.000	3.178	1.730	0.000	3.178
Earthquake	30.000	75.000 ≦ 118.490	5.885	3.001	0.000	5.885
Collision	30.000	75.000 ≦ 94.830	13.876	5.619	0.000	13.876

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm：Average shear stress of concrete (N/mm2)

Sh：Actual Shear Force (N)

d：Effective height of footing calculated from the column, front wall or rear wall (mm)

d'：Effective height of sectional member (mm)

b：Effective width of sectional member (mm)

S：Shear force acting on the sectional member (N)

τa1：Allowable shear stress (N/mm2)

$$\tau_{a1} = Ce \cdot Cpt \cdot Cdc \cdot \tau_{a1}'$$

Where,

τa1'：Allowable shear stress when shear force acts in concrete (N/mm2)

Ce ：Correction factor for the sectional member's effective height

Cpt：Correction factor for the tensile main reinforcement ratio, Pt

Cdc：Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	3.308	30.000	0.011	≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	3.178	30.000	0.011	≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	5.885	30.000	0.020	≦	0.411	2.550	1.40	0.84	1.00	○
Collision	13.876	30.000	0.046	≦	0.405	2.550	1.40	0.84	1.00	○

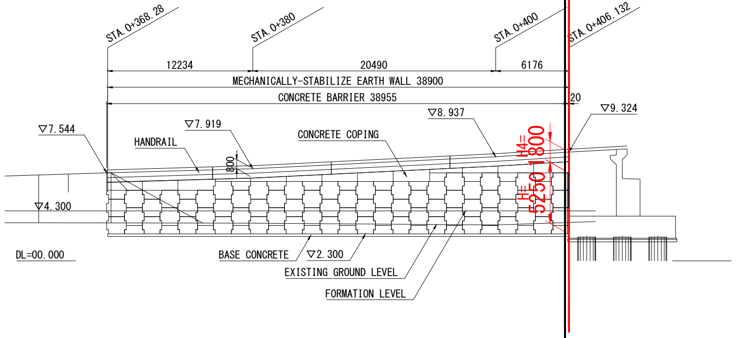
Ramp Road

Design Conditions

THANLYIN RAMP DESIGN CONDITIONS

ITEM	DESIGN CRITERIA	(SUPPORTING DOCUMENTATION)	NOTE	ONSULTATION RESUL
1. Application criteria	1) Japan Road Association (2012). Road Earthwork: Retaining Wall Guidelines 2012. 2) Japan Road Association (2012). Specifications for Highway Bridges, the Commentary. 3) Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed. 4) American Association of State Highway and Transportation Officials (2012) , AASHTO LRFD Bridge Design Specifications, 6th Ed (US)	【Road Earthwork: Retaining Wall Guidelines　P.66】		
2. Soil Condition 1). Embankment Material	γ=19kN/m^3,　φ=30°,　C=0kN/m^2			
3. Earthquake resistant design	Horizontal Seismic Coefficient **kh = Cz・kho = 1.0×0.18 = 0.18**			
4. Loading Conditions 1). Main Load Loading	Road Department　11.6kN/m^2　(AASHTO LRFD 2012 Bridge)			
2). Special Loads Collision Load	Mechanically -stabilised earth wall　Handrail（A）　43kN/m (Only horizontal force taken into account)			
5. Study Model	MSE TYPE-1　=5250(Full Height)　H=5230(Finished Height)　＋　H4=1800　＝　70 MSE TYPE-2　=5250(Full Height)　H=5230(Finished Height)　＋　H4=1800　＝　70 (LEFT SIDE)	Concrete Barrier Height h=900 Concrete Barrier Height h=800		

MECHANICALLY-STABILIZED EARTH WALL Hmax CONSIDER CROSS SECTION



Talble4-6　Unit weight of soil　(kN/m2)

Ground	Soils and foundations	One with a loose	A dense
Natural Ground	Sand and gravel	18	20
	Sandy soils	17	19
	Cohesive soils	14	18
And the Backfill Soil embankment	Sand and gravel	20	
	Sandy soils	19	
	Cohesive soils (However w_L＜50%)	18	

※　Groundwater less than weight per unit volume of soil is good as 9(kN/m3) minus the value in the table.

Talble4-5　And the backfill soil strength parameters of embankment

And the Backfill Soil embankment	Shear Resistance Angle (ϕ)	Adhesive Force (C)※2
Gravelly soils	35°	—
Sandy soils※1	30°	—
Cohesive soils (However w_L＜50%)	25°	—

※1　Little fines sand, gravelly soils of good use.

※2　To estimate the soil parameters from the table above to ignore sticky C.

Thanlyin Ramp

Mechanically-Stabilized Earth Wall Type-1

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4.8 STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

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5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

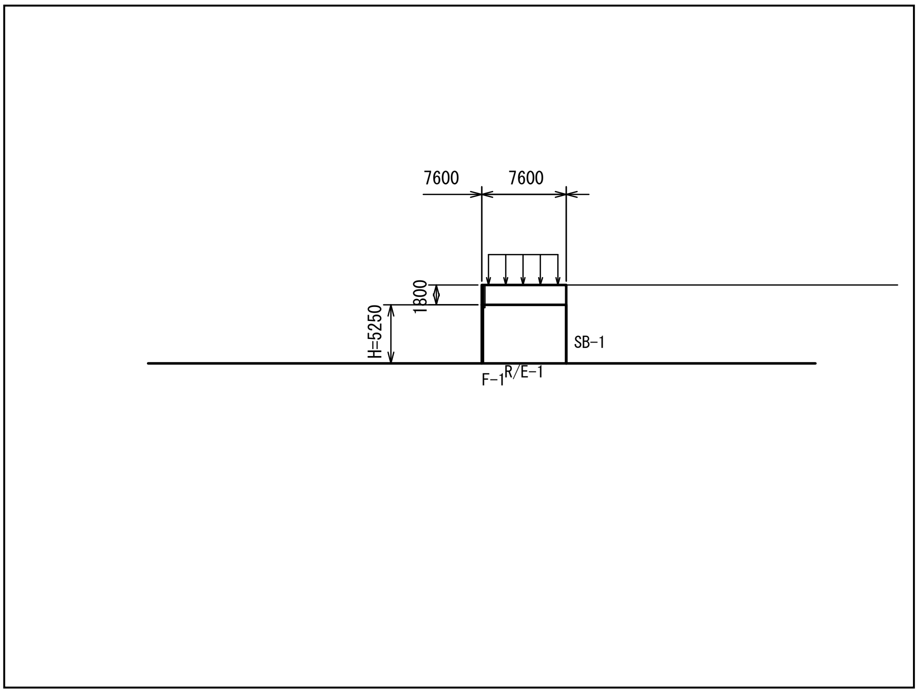
5.1 ORDINARY

5.2 EARTHQUAKE

1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H = 5.250 (m)

HEIGHT OF COPING : H4 = 1.800 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	0.000	0.000	5.250

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	5.250
	2	0.000	7.050
	3	7.600	7.050
	4	7.600	0.000

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	0.000
	2	30.000	0.000

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE : RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 : MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION

PERFORMANCE 2 : KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL

PERFORMANCE 3 : KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	c es (kN/m^2)	fo*	tanϕ1
SB－1	1.800	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	5.250	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB : SURCHARGE BACKFILL, R/E : R/E, BE : BACKSIDE EMBANKMENT, F : FOUNDATION

Hs : LAYER THICKNESS (m)

γ : UNIT WEIGHT OF SOIL (kN/m^3)

γ' : UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE (°)

c : COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)

c es : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c es＝10kN/m^2)

fo* : APPARENT COEFFICIENT OF FRICTION

tanϕ_1 : APPARENT COEFFICIENT OF FRICTION

1.5 SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP $L+B_G$(m)	WIDTH OF LOAD B_L(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	ARBITRARY LOAD1	0.600	6.250	11.60	0.00

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6 HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1}= c_z \cdot k_{ho}$ = 0.18

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012″

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2}= c_z \cdot k_{ho} \cdot \nu$ = 0.13

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

ν : CORRECTION COEFFICIENT = 0.70

(3) OVERALL STABILITY EXAMINATION INCLUDING R/E

(EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

$k_{h3} = c_z \cdot k_{ho} = 0.12$

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.12

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR EMBANKMENT CONSTRUCTION 2012″

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT hc (m)	UNIT WEIGHT γc (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σa=140.0	σa_E=210.0
BOLT SHEAR STRESS (N/mm^2)	τa=200.00	τa_E=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$
CIRCULAR SLIP	$F_s \geqq 1.20$	$F_{sE} \geqq 1.00$

1.9 MATERIAL

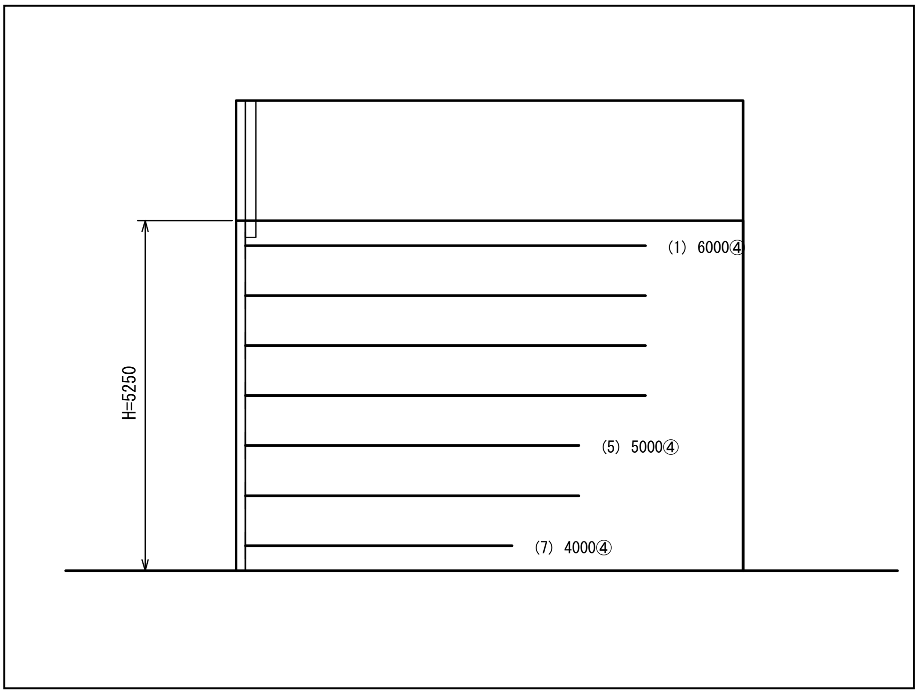
(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	Ae	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n’	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	Cm	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z_i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB (m)	DESIGN LENGTH L (m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.175	0.750	0.750	6.000	5.262	5.569	2.469	1.383	○
2	2.925	0.750	0.750	6.000	5.115	5.241	2.590	1.566	○
3	3.675	0.750	0.750	6.000	4.973	4.964	2.697	1.733	○
4	4.425	0.750	0.750	6.000	4.531	4.314	2.994	2.094	○
5	5.175	0.750	0.750	5.000	4.209	4.000	2.512	1.805	○
6	5.925	0.750	0.750	5.000	4.000	4.000	2.646	1.993	○
7	6.675	0.750	0.750	4.000	4.000	4.000	2.302	1.807	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σ_{ti} (N/mm^2)	σ_{oti} (N/mm^2)	τ_{oi} (N/mm^2)	σ_{tEi} (N/mm^2)	σ_{otEi} (N/mm^2)	τ_{oEi} (N/mm^2)	
1	67.7	62.5	72.3	78.9	72.9	84.3	○
2	75.4	69.6	80.5	86.3	79.7	92.2	○
3	82.1	75.8	87.7	92.8	85.7	99.1	○
4	87.6	80.9	93.5	98.3	90.8	105.0	○
5	94.1	86.9	100.5	106.9	98.7	114.2	○
6	99.6	91.9	106.3	114.3	105.5	122.0	○
7	109.9	101.5	117.4	126.0	116.3	134.5	○

σ_{ti} , σ_{tEi} : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σ_{oti}, σ_{otEi} : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τ_{oi} , τ_{oEi} : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3　EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.511 (1.500)	○	2.750 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.819 (0.662)	○	-0.188 (1.324)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	206.209	—	188.375	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	199.682	—	178.169	—

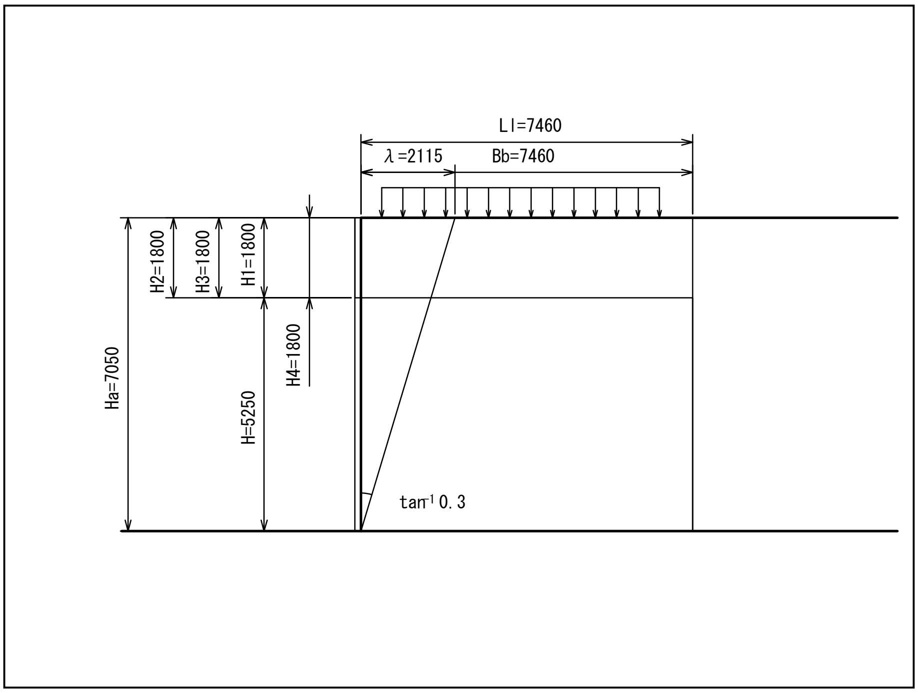
2.4　OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R(m)	Fsmin	Fs	JUDGE
		X(m)	Y(m)				
WITHIN REINFORCED AREA	ORDINARY-1	-12.500	18.000	21.915	1.302	1.200	○
	EARTHQUAKE-1	-5.000	15.000	15.811	1.345	1.000	○

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 5.250 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : $H4$ = 1.800 (m)

HEIGHT OF SURCHARGE BACKFILL : $H1$ = 1.800 (m)

SLOPE INCLINE (1:n) : n = 0.000

FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN : Bb = 7.460 (m)

DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER : Ll = 7.460 (m)

3.1.2 VIRTUAL WALL HEIGHT

$$Ha = H + H2 = 7.050 \text{ (m)}$$

Ha : VIRTUAL WALL HEIGHT (m)

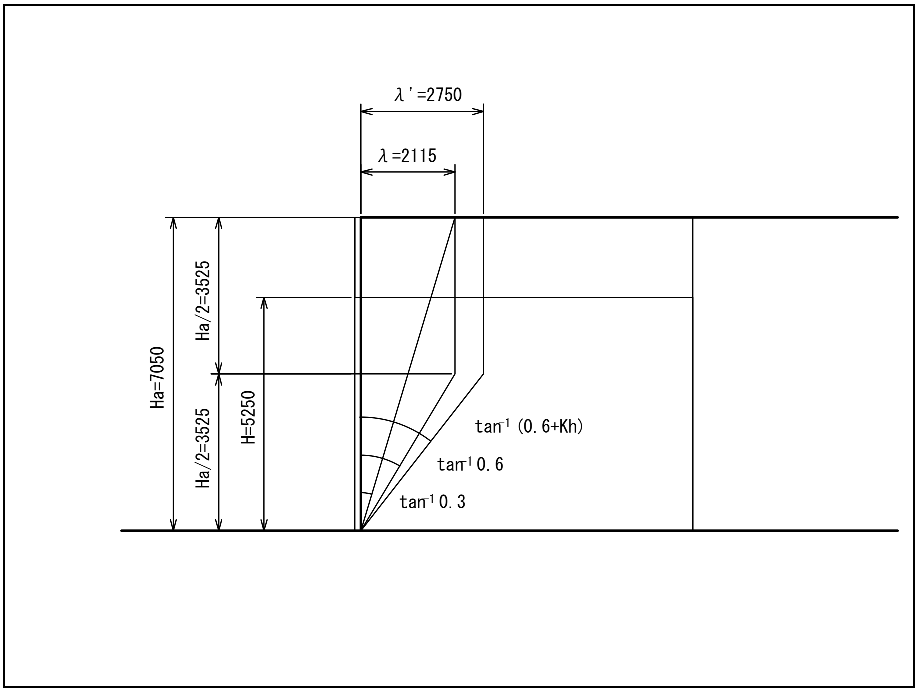
H : HEIGHT OF R/E = 5.250 (m)

H2 : DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 1.800$

H4 : HEIGHT OF COPING = 1.800 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$zi = \Delta H \cdot (i-1/2) + H2$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)
$H2$: $H2 + Ha = 0.3(H+H4) - Bb/n - 0.3 + H4$ (m) = 1.800 (m)

STAGE (i)	zi (m)
1	2.175
2	2.925
3	3.675
4	4.425
5	5.175
6	5.925
7	6.675

3.2 LOAD

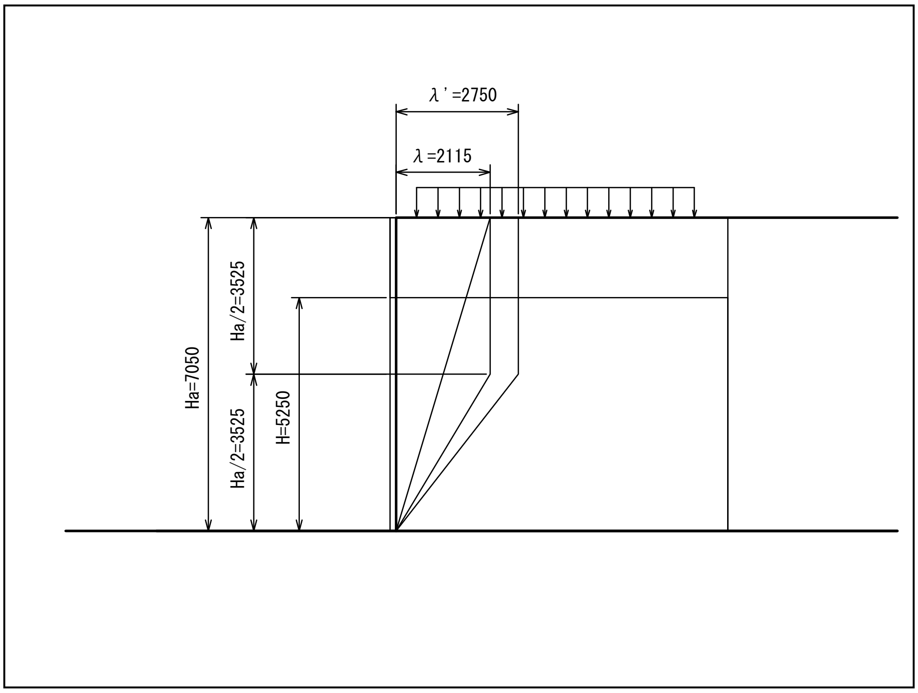
3.2.1 SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

$$q1 = \gamma 2 \cdot H3 = 34.200 \ (kN/m^2)$$

$q1$: SURCHARGE SOIL LOAD (kN/m^2)
$\gamma 2$: UNIT WEIGHT OF SURCHARGE SOIL = 19.0 (kN/m^3)
$H3$: ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 1.800 (m)

3.2.2 CALCULATION OF ARBITRARY LOAD APPLIED TO EACH STAGE



THE ARBITRARY LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE ARBITRARY LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA. WHEN THE ARBITRARY LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE ARBITRARY LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{di} = q_d \cdot \frac{B_d}{B_{di}}$$

ここに, q_{di} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_d : ARBITRARY LOAD (kN/m^2)

B_d : ARBITRARY LOAD WIDTH (m)

B_{di} : DISPERSIVE WIDTH OF THE ARBITRARY LOAD AT iTH STAGE STRIP (m)

(1) ARBITRARY LOAD-1

q_d (ORDINARY) = 11.600 (kN/m^2) B_d = 6.250 (m)

q_{dE} (EARTHQUAKE) = 0.000 (kN/m^2)

STAGE (i)	z_i (m)	B_{di} (m)	ORDINARY q_{di} (kN/m^2)	EARTH QUAKE q_{dEi} (kN/m^2)
1	2.175	6.250	11.600	0.000
2	2.925	6.250	11.600	0.000
3	3.675	6.250	11.600	0.000
4	4.425	6.250	11.600	0.000
5	5.175	6.250	11.600	0.000
6	5.925	6.250	11.600	0.000
7	6.675	6.250	11.600	0.000

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY (V_i) AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE (V_{Ei}) .

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.175	5.207	6.214
2	2.925	4.289	5.082
3	3.675	3.640	4.295
4	4.425	3.165	3.726
5	5.175	2.797	3.283
6	5.925	2.510	2.938
7	6.675	2.272	2.661

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$q_i = q_1 + q_{di} + V_i$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)
q_1 : SURCHARGE SOIL LOAD = 34.200 (kN/m^2)
q_{di} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)
V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z i (m)	q di (kN/m^2)	V i (kN/m^2)	q i (kN/m^2)
1	2.175	11.600	5.207	51.007
2	2.925	11.600	4.289	50.089
3	3.675	11.600	3.640	49.440
4	4.425	11.600	3.165	48.965
5	5.175	11.600	2.797	48.597
6	5.925	11.600	2.510	48.310
7	6.675	11.600	2.272	48.072

(2) EARTHQUAKE

$q_{Ei} = q_1 + q_{dEi} + V_{Ei}$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)
q_1 : SURCHARGE SOIL LOAD = 34.200 (kN/m^2)
q_{dEi} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)
V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z i (m)	q dEi (kN/m^2)	V Ei (kN/m^2)	q Ei (kN/m^2)
1	2.175	0.000	6.214	40.414
2	2.925	0.000	5.082	39.282
3	3.675	0.000	4.295	38.495
4	4.425	0.000	3.726	37.926
5	5.175	0.000	3.283	37.483
6	5.925	0.000	2.938	37.138
7	6.675	0.000	2.661	36.861

3.3　EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（Ｈi）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_Ei）.

STAGE (i)	ｚi (m)	ORDINARY Ｈi (kN/m)	EARTH QUAKE H_Ei (kN/m)
1	2.175	2.502	4.246
2	2.925	1.694	2.807
3	3.675	0.886	1.369
4	4.425	0.081	0.000
5	5.175	0.000	0.000
6	5.925	0.000	0.000
7	6.675	0.000	0.000

3.4 EARTH PRESSURE

3.4.1 COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq zo = 6.0$ (m)

$$Ki = Ko \cdot \left(1 - \frac{zi}{zo}\right) + K_A \cdot \frac{zi}{zo}$$

IF $zi > zo = 6.0$ (m)

$$Ki = K_A$$

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

Ko : COEFFICIENT OF EARTH PRESSURE AT REST

$$Ko = 1 - \sin\phi$$

K_A : COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	φ (°)	Ko	KA	Ki
1	2.175	30.0	0.5000	0.3333	0.4396
2	2.925	30.0	0.5000	0.3333	0.4188
3	3.675	30.0	0.5000	0.3333	0.3979
4	4.425	30.0	0.5000	0.3333	0.3771
5	5.175	30.0	0.5000	0.3333	0.3562
6	5.925	30.0	0.5000	0.3333	0.3354
7	6.675	30.0	0.5000	0.3333	0.3333

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_i = K_i \cdot \sigma_{vi} \cdot \Delta H + H_i$$

$$T_o = 0.75 \cdot P_i$$

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

H_i : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σ_{vi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{vi} = \gamma_1 \cdot x_i + q_i$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : $x_i = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	x_i (m)	K_i	q_i (kN/m^2)	σ_{vi} (kN/m^2)	H_i (kN/m)	P_i (kN/m)	T_o (kN/m)
1	0.375	0.440	51.007	58.132	2.502	21.667	16.251
2	1.125	0.419	50.089	71.464	1.694	24.138	18.104
3	1.875	0.398	49.440	85.065	0.886	26.273	19.704
4	2.625	0.377	48.965	98.840	0.081	28.034	21.026
5	3.375	0.356	48.597	112.722	0.000	30.118	22.588
6	4.125	0.335	48.310	126.685	0.000	31.869	23.902
7	4.875	0.333	48.072	140.697	0.000	35.174	26.381

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = P_i + \Delta P_i$$

$$\Delta P_i = \frac{1}{2} \cdot \left(1 + \frac{z_i}{H_a}\right) \cdot \alpha \cdot k_h \cdot P_n$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$P_i = K_i \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$

ΔP_i : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_n : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 32.371 (kN/m)

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

H_a : VIRTUAL WALL HEIGHT = 7.050 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{VEi} = \gamma_1 \cdot x_i + q_{Ei}$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.440	40.414	47.539	4.246	19.919
2	1.125	0.419	39.282	60.657	2.807	21.857
3	1.875	0.398	38.495	74.120	1.369	23.489
4	2.625	0.377	37.926	87.801	0.000	24.831
5	3.375	0.356	37.483	101.608	0.000	27.148
6	4.125	0.335	37.138	115.513	0.000	29.059
7	4.875	0.333	36.861	129.486	0.000	32.371

STAGE (i)	z_i (m)	ΔP_i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.175	5.337	25.256	18.942
2	2.925	5.771	27.628	20.721
3	3.675	6.205	29.694	22.271
4	4.425	6.639	31.470	23.603
5	5.175	7.073	34.221	25.666
6	5.925	7.507	36.565	27.424
7	6.675	7.941	40.312	30.234

3.5 HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS = 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT = 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP = 80.0 (mm)

t : THE THICKNESS OF STRIP = 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP = 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP = 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER = 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

ｊ　　：THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM ＝ 2

ｎ　　：THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS ＝ 1 (PIECE)

Ａe　　：SCREW EFFECTIVE AREA OF MOUNTING BOLTS＝ 84.3 (mm^2)

STAGE (i)	Ｐi (kN/m)	Ｔo (kN/m)	ΔＢ1i (m)	ΔＢ2i (m)	ΔＢ3i (m)
1	21.667	16.251	1.550	1.679	2.075
2	24.138	18.104	1.391	1.507	1.862
3	26.273	19.704	1.278	1.385	1.711
4	28.034	21.026	1.198	1.298	1.603
5	30.118	22.588	1.115	1.208	1.492
6	31.869	23.902	1.054	1.142	1.410
7	35.174	26.381	0.955	1.034	1.278

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)
$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)
$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)
σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)
τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)
P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)
T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)
Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)
An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)
$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	25.256	18.942	1.995	2.161	2.670
2	27.628	20.721	1.824	1.976	2.440
3	29.694	22.271	1.697	1.838	2.271
4	31.470	23.603	1.601	1.734	2.142
5	34.221	25.666	1.472	1.595	1.970
6	36.565	27.424	1.378	1.493	1.844
7	40.312	30.234	1.250	1.354	1.672

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF 1.000 ＞ $\Delta B_i \geqq 0.750$　　$\Delta B_i = 0.750$

STAGE (i)	ΔB_{1i} (m)	ΔB_{2i} (m)	ΔB_{3i} (m)	ΔB_{1Ei} (m)	ΔB_{2Ei} (m)	ΔB_{3Ei} (m)	ΔB_i (m)
1	1.550	1.679	2.075	1.995	2.161	2.670	0.750
2	1.391	1.507	1.862	1.824	1.976	2.440	0.750
3	1.278	1.385	1.711	1.697	1.838	2.271	0.750
4	1.198	1.298	1.603	1.601	1.734	2.142	0.750
5	1.115	1.208	1.492	1.472	1.595	1.970	0.750
6	1.054	1.142	1.410	1.378	1.493	1.844	0.750
7	0.955	1.034	1.278	1.250	1.354	1.672	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	21.667	16.251	16.251	12.188	25.256	18.942	18.942	14.207
2	0.750	24.138	18.104	18.104	13.578	27.628	20.721	20.721	15.541
3	0.750	26.273	19.704	19.704	14.778	29.694	22.271	22.271	16.703
4	0.750	28.034	21.026	21.026	15.769	31.470	23.603	23.603	17.702
5	0.750	30.118	22.588	22.588	16.941	34.221	25.666	25.666	19.249
6	0.750	31.869	23.902	23.902	17.926	36.565	27.424	27.424	20.568
7	0.750	35.174	26.381	26.381	19.786	40.312	30.234	30.234	22.676

3.7 STRIP LENGTH

3.7.1 FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^{*} = fo^{*} \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^{*} = \tan\phi 1$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^{*} = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^{*}	$\tan\phi 1$	fi^{*}
1	2.175	1.500	0.727	1.2196
2	2.925	1.500	0.727	1.1229
3	3.675	1.500	0.727	1.0263
4	4.425	1.500	0.727	0.9296
5	5.175	1.500	0.727	0.8329
6	5.925	1.500	0.727	0.7362
7	6.675	1.500	0.727	0.7265

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 3.525$ (m)

$$L_{oi} = 0.3 \cdot Ha$$

IF $z_i > Ha/2 = 3.525$ (m)

$$L_{oi} = 0.6 \cdot (Ha - z_i)$$

L_{oi} : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Ha : VIRTUAL WALL HEIGHT = 7.050 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 3.525$ (m)

$$L_{oEi} = (0.6 + k_{h1}) \cdot (Ha/2)$$

IF $z_i > Ha/2 = 3.525$ (m)

$$L_{oEi} = (0.6 + k_{h1}) \cdot (Ha - z_i)$$

L_{oEi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)
Ha : VIRTUAL WALL HEIGHT = 7.050 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	z_i (m)	L_{oi} (m)	L_{oEi} (m)
1	2.175	2.115	2.750
2	2.925	2.115	2.750
3	3.675	2.025	2.633
4	4.425	1.575	2.048
5	5.175	1.125	1.463
6	5.925	0.675	0.878
7	6.675	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqi} = \frac{Fs \cdot Ti}{2 \cdot fi^{*} \cdot (\sigma vi - Vi) \cdot b}$$

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

Fs : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

Vi : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	Ti (kN/PIECE)	fi*	σvi-Vi (kN/m^2)	b (m)	Lreqi (m)
1	16.251	1.2196	52.925	0.0800	3.147
2	18.104	1.1229	67.175	0.0800	3.000
3	19.704	1.0263	81.425	0.0800	2.948
4	21.026	0.9296	95.675	0.0800	2.956
5	22.588	0.8329	109.925	0.0800	3.084
6	23.902	0.7362	124.175	0.0800	3.269
7	26.381	0.7265	138.425	0.0800	3.279

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^* \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^*	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	18.942	1.2196	41.325	0.0800	2.819
2	20.721	1.1229	55.575	0.0800	2.491
3	22.271	1.0263	69.825	0.0800	2.331
4	23.603	0.9296	84.075	0.0800	2.266
5	25.666	0.8329	98.325	0.0800	2.351
6	27.424	0.7362	112.575	0.0800	2.482
7	30.234	0.7265	126.825	0.0800	2.461

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$L_{\mathrm{min}i} = L_{\mathrm{o}i} + L_{\mathrm{req}i}$$

EARTHQUAKE

$$L_{\mathrm{min}Ei} = L_{\mathrm{o}Ei} + L_{\mathrm{req}Ei}$$

$L_{\mathrm{min}i}$: MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)

$L_{\mathrm{o}i}$: LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

$L_{\mathrm{req}i}$: LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

$L_{\mathrm{min}Ei}$: MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)

$L_{\mathrm{o}Ei}$: LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

$L_{\mathrm{req}Ei}$: LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	$L_{\mathrm{o}i}$ (m)	$L_{\mathrm{req}i}$ (m)	$L_{\mathrm{min}i}$ (m)	$L_{\mathrm{o}Ei}$ (m)	$L_{\mathrm{req}Ei}$ (m)	$L_{\mathrm{min}Ei}$ (m)
1	2.115	3.147	5.262	2.750	2.819	5.569
2	2.115	3.000	5.115	2.750	2.491	5.241
3	2.025	2.948	4.973	2.633	2.331	4.964
4	1.575	2.956	4.531	2.048	2.266	4.314
5	1.125	3.084	4.209	1.463	2.351	3.814
6	0.675	3.269	3.944	0.878	2.482	3.360
7	0.225	3.279	3.504	0.293	2.461	2.754

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

WHEN SURCHERGE BACKFILL HEIGHT IS UNDER 2M ($0m \leqq H1 = 1.800 < 2m$)

0.5 Ha OR MORE FROM UPPER EDGE OF Ha

$L_{sd} = 0.7 \cdot Ha = 4.935$ (m)

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$L_{sd} = 0.4 \cdot Ha = 2.820$ (m)

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

Ha : VIRTUAL WALL HEIGHT = 7.050 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)

L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)

L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)

L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	5.262	5.569	4.935	6.000
2	5.115	5.241	4.935	6.000
3	4.973	4.964	—	6.000
4	4.531	4.314	—	6.000
5	4.209	3.814	4.000	5.000
6	3.944	3.360	4.000	5.000
7	3.504	2.754	4.000	4.000

3.8 CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$Fsi = \frac{Si}{Ti} \geqq Fs = 2.00$$

Fsi : SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

Fs : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 2.0

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Si : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$Si = 2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$\sigma vi'$: VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

Lei : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$Lei = Li - Loi$$

Li : DETERMINED STRIP LENGTH AT iTH STAGE (m)

Loi : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	Loi (m)	Lei (m)	fi*	σvi' (kN/m^2)	b (m)	Si (kN/PIECE)	Ti (kN/PIECE)	Fsi	JUDGE
1	6.000	2.115	3.885	1.2196	52.925	0.0800	40.123	16.251	2.469	○
2	6.000	2.115	3.885	1.1229	67.175	0.0800	46.889	18.104	2.590	○
3	6.000	2.025	3.975	1.0263	81.425	0.0800	53.146	19.704	2.697	○
4	6.000	1.575	4.425	0.9296	95.675	0.0800	62.967	21.026	2.994	○
5	5.000	1.125	3.875	0.8329	109.925	0.0800	56.765	22.588	2.512	○
6	5.000	0.675	4.325	0.7362	124.175	0.0800	63.262	23.902	2.646	○
7	4.000	0.225	3.775	0.7265	138.425	0.0800	60.745	26.381	2.302	○

(2) EARTHQUAKE

$$F_{sEi} = \frac{S_{Ei}}{T_{Ei}} \geqq F_{sE} = 1.20$$

F_{siEi} : SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP

F_{sE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

S_{Ei} : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$S_{Ei} = 2 \cdot f_i^* \cdot \sigma_{VEi} \cdot b \cdot L_{eEi}$$

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

L_{eEi} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$L_{eEi} = L_i - L_{oEi}$$

L_i : DETERMINED STRIP LENGTH AT iTH STAGE (m)

L_{oEi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	L_i (m)	L_{oEi} (m)	L_{eEi} (m)	f_1^*	σ_{VEi}' (kN/m^2)	b (m)	S_{Ei} (kN/PIECE)	T_{Ei} (kN/PIECE)	F_{sEi}	JUDGE
1	6.000	2.750	3.251	1.2196	41.325	0.0800	26.212	18.942	1.383	○
2	6.000	2.750	3.251	1.1229	55.575	0.0800	32.457	20.721	1.566	○
3	6.000	2.633	3.368	1.0263	69.825	0.0800	38.610	22.271	1.733	○
4	6.000	2.048	3.953	0.9296	84.075	0.0800	49.425	23.603	2.094	○
5	5.000	1.463	3.538	0.8329	98.325	0.0800	46.352	25.666	1.805	○
6	5.000	0.878	4.123	0.7362	112.575	0.0800	54.667	27.424	1.993	○
7	4.000	0.293	3.708	0.7265	126.825	0.0800	54.660	30.234	1.807	○

3.9　CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \ (N/mm^2)$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS ＝ 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP ＝ 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT ＝ 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	16.251	12.188	67.7	62.5	72.3	○
2	18.104	13.578	75.4	69.6	80.5	○
3	19.704	14.778	82.1	75.8	87.7	○
4	21.026	15.769	87.6	80.9	93.5	○
5	22.588	16.941	94.1	86.9	100.5	○
6	23.902	17.926	99.6	91.9	106.3	○
7	26.381	19.786	109.9	101.5	117.4	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma t_{Ei} = \frac{T_{Ei}}{Ag} \leqq \sigma a_{E} = 210.0 \ (N/mm^2)$$

$$\sigma ot_{Ei} = \frac{Tf_{Ei}}{An} \leqq \sigma a_{E} = 210.0 \ (N/mm^2)$$

$$\tau o_{Ei} = \frac{Tf_{Ei}}{A\tau} \leqq \tau a_{E} = 300.0 \ (N/mm^2)$$

σt_{Ei} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σot_{Ei} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τo_{Ei} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa_{E} : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa_{E} : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tf_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	T_{Ei} (kN/PIECE)	Tf_{Ei} (kN/PIECE)	σt_{Ei} (N/mm^2)	σot_{Ei} (N/mm^2)	τo_{Ei} (N/mm^2)	JUDGE
1	18.942	14.207	78.9	72.9	84.3	○
2	20.721	15.541	86.3	79.7	92.2	○
3	22.271	16.703	92.8	85.7	99.1	○
4	23.603	17.702	98.3	90.8	105.0	○
5	25.666	19.249	106.9	98.7	114.2	○
6	27.424	20.568	114.3	105.5	122.0	○
7	30.234	22.676	126.0	116.3	134.5	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2	ARBITRARY LOAD
CASE 1-1	ORDINARY	○		○		○
CASE 1-2	ORDINARY	○		○		○
CASE 2-3	EARTHQUAKE	○	○		○	

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

CASE 1-1 : OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)

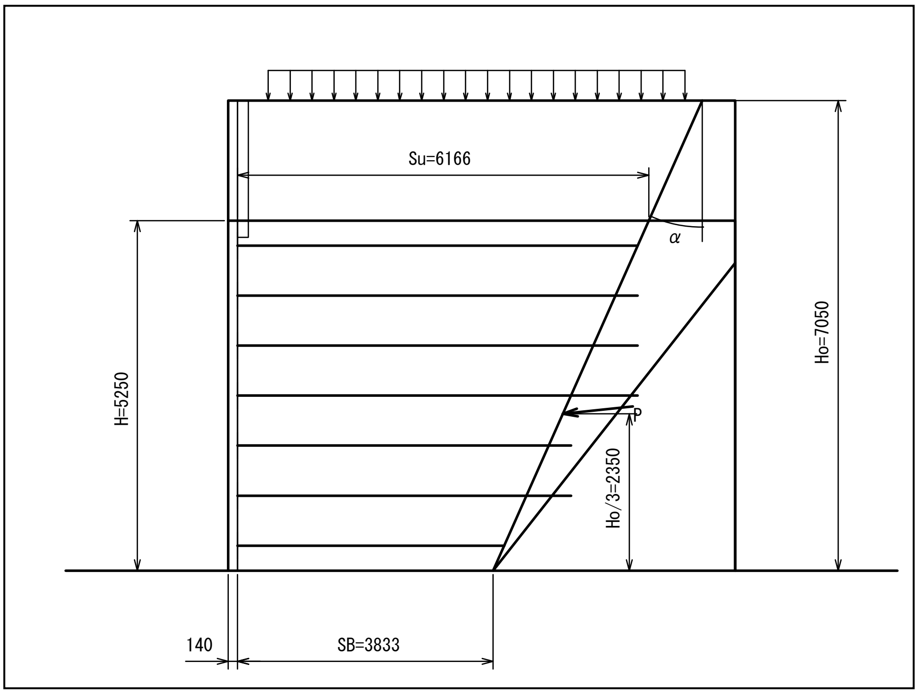
CASE 1-2 : OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 : ORDINARY LOAD

LOAD2 : EARTHQUAKELOAD

4.2 EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1 CASE 1 CALCULATION OF LOAD



H_o : WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 7.050 (m)

H : R/E WALL HEIGHT＝ 5.250 (m)

h_o : CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

S_u : REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 6.166 (m)

S_B : REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.833 (m)

B : R/E BOTTOM WIDTH ＝ 3.973 (m)

α : VIRTUAL BACK SLOPE INCLINE ＝ -23.959 (°)

$$\alpha = -\tan^{-1}\frac{S_u - S_B}{H} = -23.959\ (°\ )$$

$W1$: OWN WEIGHT OF R/E ＝ 742.008 (kN/m)

W_q : LOADED WEIGHT ON R/E ＝ 72.500 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma)\cdot\tan(45° + \phi/2) = (2 \cdot c/\gamma)\cdot\tan(45° + \phi/2) = 0.000$ (m)

$\delta 1$: FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

$\omega 1$: ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 51.8 (°)

$W2$ ：OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) ＝ 116.967 (kN/m)

$L1$ ：LENGTH OF SLIP SURFACE ＝ 5.870 (m)

P_A ：ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 45.223$$

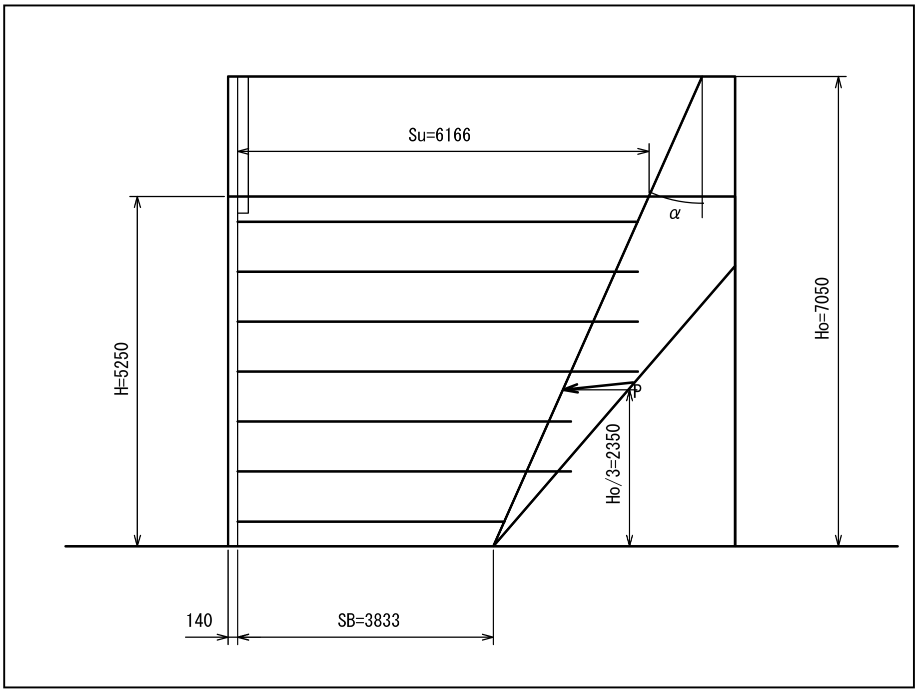
P_{Ah} ：HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 44.972$ (kN/m)

P_{AV} ：VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = 4.759$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK ＝ -23.959 (°)

W1 : OWN WEIGHT OF R/E ＝ 742.008 (kN/m)

Wq : LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45^\circ + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

kh2 : HORIZONTAL SEISMIC COEFFICIENT ＝ 0.13 (QUOTED FROM ″DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY″)

θ : EARTHQUAKE COMPOUND ANGLE ＝ $\tan^{-1} kh2 = 7.407$ (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE ＝ 49.2 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE ＝ 131.021 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE ＝ 5.555 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 60.865$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 60.527$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = 6.405$ (kN/m)

4.3　AGGREGATION OF LOAD

4.3.1　ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V(kN/m)	ACTION POSITION X(m)	POSITION MOMENT Mx(kNm/m)	HORIZONTAL FORCE H(kN/m)	ACTION POSITION Y(m)	POSITION MOMENT My(kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	0.000	0.000	0.00
ARBITRARY LOAD	72.500	3.725	270.06	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	4.759	5.017	23.88	44.972	2.350	105.68
TOTAL	819.267		2403.88	44.972		105.68

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V(kN/m)	ACTION POSITION X(m)	POSITION MOMENT Mx(kNm/m)	HORIZONTAL FORCE H(kN/m)	ACTION POSITION Y(m)	POSITION MOMENT My(kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	0.000	0.000	0.00
ARBITRARY LOAD	72.500	3.725	270.06	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	4.759	5.017	23.88	44.972	2.350	105.68
TOTAL	819.267		2403.88	44.972		105.68

4.3.2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	96.461	3.857	372.08
REAR EARTH PRESSURE 2	6.405	5.017	32.14	60.527	2.350	142.24
TOTAL	748.413		2142.07	156.989		514.32

kh2 : HORIZONTAL SEISMIC COEFFICIENT ＝ 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	96.461	3.857	372.08
REAR EARTH PRESSURE 2	6.405	5.017	32.14	60.527	2.350	142.24
TOTAL	748.413		2142.07	156.989		514.32

kh2 : HORIZONTAL SEISMIC COEFFICIENT ＝ 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 10.511 \geqq F_{sa} = 1.50 \cdots \text{OK}$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.973 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 819.267 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 44.972 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.750 \geqq F_{saE} = 1.20 \cdots \text{OK}$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.973 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 748.413 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 156.989 (kN/m)

4.6 STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.805 \ (m)$$

d : R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

ΣMr : R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 2403.88 (kN·m/m)

ΣMo : R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 105.68 (kN·m/m)

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 819.267 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$e = B/2 - d = -0.819 \leqq B/6 = 0.662$ (m) ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.7　EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.175\ (m)$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 2142.07 (kN･m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 514.32 (kN･m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 748.413 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$e = B/2 - d3_E = -0.188 \leqq B/3 = 1.324\ (m)$ ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.8　STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1　VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$$q_s = \Sigma V / B = 206.209 \ (\mathrm{kN/m^2})$$

q_s : VERTICAL SUBGRADE REACTION ON R/E BOTTOM ($\mathrm{kN/m^2}$)

ΣV : TOTAL VERTICAL LOAD ON R/E BOTTOM = 819.267 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.9 STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_{SE} = \Sigma V_E / B = 188.375$ (kN/m^2)

q_{SE} : VERTICAL SUBGRADE REACTION ON R/E BOTTOM
DURING EARTHQUAKE (kN/m^2)

ΣV_E : TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE
= 748.413 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma (To \cdot \tan\delta) = 53.851 \text{ (kN/m)}$$

To : TENSILE FORCE AGAINST WALL (kN/m)

δ : ANGLE OF WALL FRICTION (°) = $2/3\phi$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	ϕ (°)	$\tan\delta$	Vci (kN/m)
1	16.251	30.0	0.364	5.915
2	18.104	30.0	0.364	6.589
3	19.704	30.0	0.364	7.172
4	21.026	30.0	0.364	7.653
5	22.588	30.0	0.364	8.222
6	23.902	30.0	0.364	8.700
7	26.381	30.0	0.364	9.602
TOTAL				53.851

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 199.682 \ (kN/m^2)$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

Wc1 : THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 24.182 \ (kN/m)$

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL = 24.5 (kN/m^3)

Hta : TOTAL HEIGHT OF R/E = 7.050 (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL = 0.140 (m)

Wc2 : WEIGHT OF FOUNDATION CONCRETE = $hf \cdot b \cdot \gamma c2$ = 1.840 (kN/m)

b : WIDTH OF CONCRETE FOUNDATION = 0.400 (m)

hf : HEIGHT OF CONCRETE FOUNDATION = 0.200 (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION = 23.0 (kN/m^3)

Vc : VERTICAL COMPONENT AGAINST WALL = 53.851 (kN/m)

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$V_{cE} = \Sigma (T_{oE} \cdot \tan \delta_E) = 45.246 \text{ (kN/m)}$$

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	18.942	30.0	0.268	5.076
2	20.721	30.0	0.268	5.552
3	22.271	30.0	0.268	5.967
4	23.603	30.0	0.268	6.324
5	25.666	30.0	0.268	6.877
6	27.424	30.0	0.268	7.348
7	30.234	30.0	0.268	8.101
TOTAL				45.246

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 178.169\ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL = 24.182 (kN/m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE = 1.840 (kN/m)

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE = 45.246 (kN/m)

5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

5.1 ORDINARY

5.1.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{Ai} = \frac{\sigma a \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σa : ALLOWABLE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{Ai} (kN/m)
1	140.0	80.0	4.0	1.0	240.0	0.750	44.800
2	140.0	80.0	4.0	1.0	240.0	0.750	44.800
3	140.0	80.0	4.0	1.0	240.0	0.750	44.800
4	140.0	80.0	4.0	1.0	240.0	0.750	44.800
5	140.0	80.0	4.0	1.0	240.0	0.750	44.800
6	140.0	80.0	4.0	1.0	240.0	0.750	44.800
7	140.0	80.0	4.0	1.0	240.0	0.750	44.800

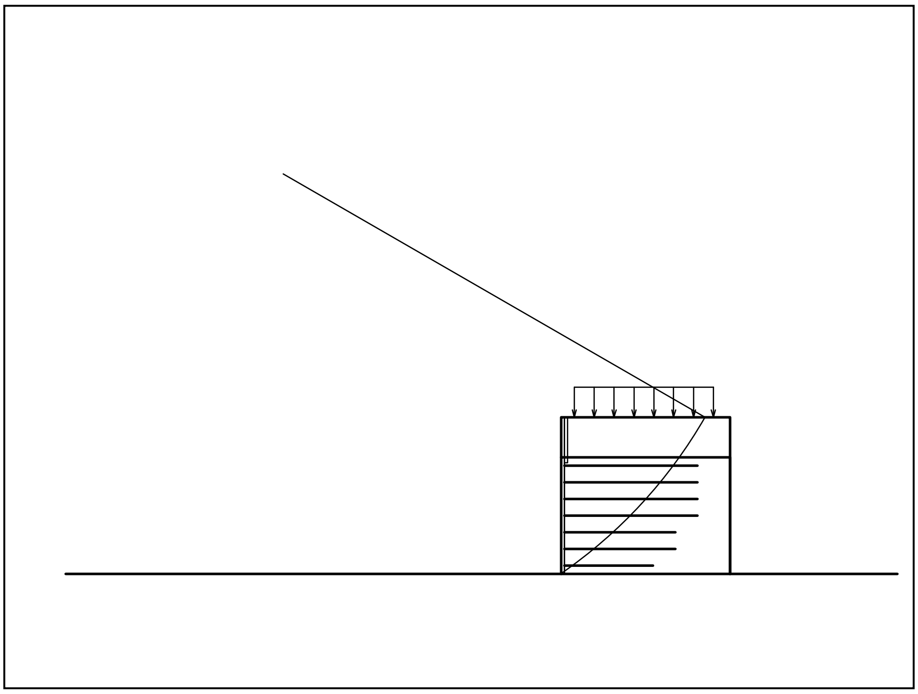
5.1.2 FORMULA FOR SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

$$Fs = \frac{R\Sigma\{ces \cdot l + (W' \cdot \cos\alpha + Tavail \cdot \sin\theta)\tan\phi + Tavail \cdot \cos\theta\}}{R\Sigma(W \cdot \sin\alpha)}$$

Fs	: SAFETY FACTOR OF ORDINARY CIRCULAR SLIP
l	: LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)
W	: TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)
W'	: TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)
α	: ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)
ces	: COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2) (FOR SANDY SOIL : $ces = 10$kN/m^2)
ϕ	: ANGLE OF SHEARING RESISTANCE OF SOIL (°)
Tavail	: AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(Tp), WHICHEVER SMALLER) (kN/m)
θ	: ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)
R	: RADIUS OF SLIP CIRCULAR ARC (m)

5.1.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [ORDINARY-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$Tavaili = \min(Tpi,\ T_{Ai})$$

$$Tpi = \frac{2 \cdot fi* \cdot \sigma vi \cdot b \cdot Lei}{Fs} \times \frac{1}{\triangle B}$$

Tavaili	: AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
Tpi	: DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP (kN/m)
T_{Ai}	: ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
fi*	: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
σvi	: VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE $= \gamma \cdot zi$ (kN/m^2)
zi	: DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)
b	: WIDTH OF STRIP (m)
Lei	: ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)
⊿B	: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)
Fs	: SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{vi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pi} (kN/m)	T_{Ai} (kN/m)	T_{availi} (kN/m)
1	1.2196	19.0 19.0	1.800 0.375	41.325	0.0800	1.090	0.750	5.860	44.800	5.860
2	1.1229	19.0 19.0	1.800 1.125	55.575	0.0800	1.677	0.750	11.163	44.800	11.163
3	1.0263	19.0 19.0	1.800 1.875	69.825	0.0800	2.319	0.750	17.725	44.800	17.725
4	0.9296	19.0 19.0	1.800 2.625	84.075	0.0800	3.024	0.750	25.209	44.800	25.209
5	0.8329	19.0 19.0	1.800 3.375	98.325	0.0800	2.799	0.750	24.450	44.800	24.450
6	0.7362	19.0 19.0	1.800 4.125	112.575	0.0800	3.658	0.750	32.338	44.800	32.338
7	0.7265	19.0 19.0	1.800 4.875	126.825	0.0800	3.616	0.750	35.540	44.800	35.540
								ΣT_{avail} = 152.287		

(3) LIST OF SAFETY FACTOR

Fs		COORDINATE OF CIRCULAR CENTER X						
		-14.0m	-13.5m	-13.0m	-12.5m	-12.0m	-11.5m	-11.0m
Y	19.5m	1.302	1.302	1.303	1.309	1.314	1.321	1.328
	19.0m	1.303	1.302	1.302	1.303	1.309	1.315	1.322
	18.5m	1.304	1.303	1.302	1.302	1.303	1.309	1.315
	18.0m	1.306	1.304	1.303	1.302	1.302	1.303	1.309
	17.5m	1.308	1.306	1.304	1.303	1.302	1.302	1.303
	17.0m	1.312	1.309	1.306	1.304	1.303	1.302	1.303
	16.5m	1.316	1.312	1.309	1.306	1.304	1.303	1.303

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	ORDINARY-1	JUDGE
MINIMUM SAFETY FACTOR	Fsmin Fs		1.302 (1.200)	○
ΣTavail	kN/m		152.287	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN･m/m	[2116.0] [5182.8]	—
	M_R	kN･m/m	7298.8	—
	M_T	kN･m/m	3745.8	—
ACTIVE MOMENT	M_D	kN･m/m	8484.2	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-12.500 18.000	—
RADIUS	R	m	21.915	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN･m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN･m/m)

M_R : RESISTER MOMENT OF SOIL $= M_{RC} + M_{RF}$ (kN･m/m)

M_T : RESISTER MOMENT OF STRIP (kN･m/m)

Fsmin : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP $= (M_R + M_T) / M_D$

5.2　EARTHQUAKE

5.2.1　THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{AEi} = \frac{\sigma_{aE} \cdot A_g}{\Delta B} \times 10^{-3}$$

T_{AEi} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σ_{aE} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH OF STRIP (N/mm^2)

A_g : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$A_g = b \cdot (t - C_m)$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

C_m : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σ_{aE} (N/mm^2)	b (mm)	t (mm)	C_m (mm)	A_g (mm^2)	ΔB (m)	T_{AEi} (kN/m)
1	210.0	80.0	4.0	1.0	240.0	0.750	67.200
2	210.0	80.0	4.0	1.0	240.0	0.750	67.200
3	210.0	80.0	4.0	1.0	240.0	0.750	67.200
4	210.0	80.0	4.0	1.0	240.0	0.750	67.200
5	210.0	80.0	4.0	1.0	240.0	0.750	67.200
6	210.0	80.0	4.0	1.0	240.0	0.750	67.200
7	210.0	80.0	4.0	1.0	240.0	0.750	67.200

5.2.2 FORMULA FOR SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

$$F_{SE} = \frac{R\Sigma\{\mathrm{ces}\cdot l+(W'\cdot\cos\alpha-\mathrm{kh3}\cdot W\cdot\sin\alpha)\tan\phi\}+R\cdot\Sigma\,\mathrm{Tavail}(\cos\theta+\sin\theta\cdot\tan\phi)}{\Sigma(R\cdot W\cdot\sin\alpha\ +\ \mathrm{kh3}\cdot W\cdot y\mathrm{G})}$$

F_{SE} : SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

ces : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP ($\mathrm{kN/m^2}$)
(FOR SANDY SOIL : ces = 10$\mathrm{kN/m^2}$)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(Tp), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

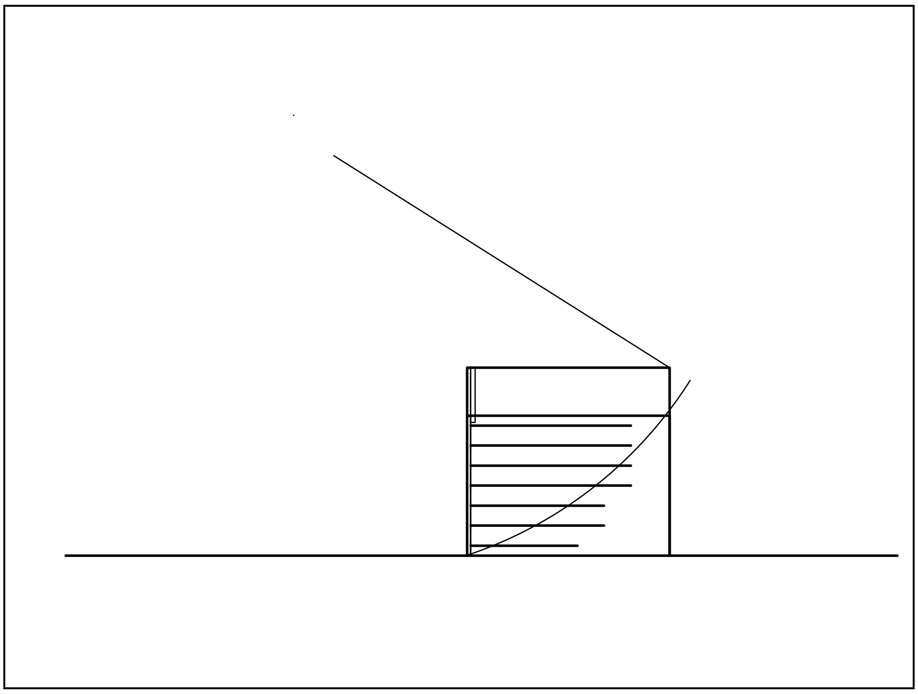
R : RADIUS OF SLIP CIRCULAR ARC (m)

kh3 : HORIZONTAL SEISMIC COEFFICIENT = 0.12 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

yG : VERTICAL DISTANCE FROM CENTER OF CIRCULAR ARC TO CENTER OF GRAVITY OF SPLIT PIECE (m)

5.2.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [EARTHQUAKE-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pEi},\ T_{AEi})$$

$$T_{pEi} = \frac{2 \cdot f_i^* \cdot \sigma_{VEi} \cdot b \cdot L_{ei}}{F_{SE}} \times \frac{1}{\Delta B}$$

Tavaili : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pEi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

T_{AEi} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

fi* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot z_i$ (kN/m^2)

zi : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

Lei : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

⊿B : HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ON THE EARTHQUAKE = 1.20

STAGE (i)	fi^*	γ (kN/m^3)	zi (m)	σ_{VEi} (kN/m^2)	b (m)	Lei (m)	⊿B (m)	T_{pEi} (kN/m)	T_{AEi} (kN/m)	Tavaili (kN/m)
1	1.2196			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
2	1.1229			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
3	1.0263	19.0 19.0	1.800 1.875	69.825	0.0800	0.422	0.750	5.376	67.200	5.376
4	0.9296	19.0 19.0	1.800 2.625	84.075	0.0800	1.298	0.750	18.034	67.200	18.034
5	0.8329	19.0 19.0	1.800 3.375	98.325	0.0800	1.323	0.750	19.262	67.200	19.262
6	0.7362	19.0 19.0	1.800 4.125	112.575	0.0800	2.558	0.750	37.690	67.200	37.690
7	0.7265	19.0 19.0	1.800 4.875	126.825	0.0800	3.130	0.750	51.273	67.200	51.273
								Σ Tavail = 131.634		

(3) LIST OF SAFETY FACTOR

F_{SE}		COORDINATE OF CIRCULAR CENTER X						
		-11.0m	-9.0m	-7.0m	-5.0m	-3.0m	-1.0m	1.0m
Y	21.0m	1.378	1.351	1.376	*****	*****	*****	*****
	19.0m	1.435	1.364	1.352	*****	*****	*****	*****
	17.0m	1.498	1.421	1.349	1.365	*****	*****	*****
	15.0m	1.582	1.492	1.406	1.345	*****	*****	*****
	13.0m	1.700	1.590	1.486	1.392	*****	*****	*****
	11.0m	1.874	1.735	1.604	1.483	1.375	*****	*****
	9.0m	2.158	1.969	1.793	1.633	1.491	*****	*****

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	EARTHQUAKE-1	JUDGE
MINIMUM SAFETY FACTOR	F_{smin} F_{SE}		1.345 (1.000)	○
HORIZONTAL SEISMIC COEFFICIENT	k_{h3}		0.12	—
ΣT_{avail}	kN/m		131.634	—
RESISTER MOMENT	$[M_{RC}]$ $[M_{RF}]$	kN·m/m	[1501.0] [5063.8]	—
	M_R	kN·m/m	6564.8	—
	M_T	kN·m/m	2387.9	—
ACTIVE MOMENT	M_D	kN·m/m	6658.9	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-5.000 15.000	—
RADIUS	R	m	15.811	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN·m/m)

M_R : RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN·m/m)

M_T : RESISTER MOMENT OF STRIP (kN·m/m)

F_{smin} : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP
= (M_R+M_T) / M_D

Thanlyin Ramp

Mechanically-Stabilized Earth Wall Type-2

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5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

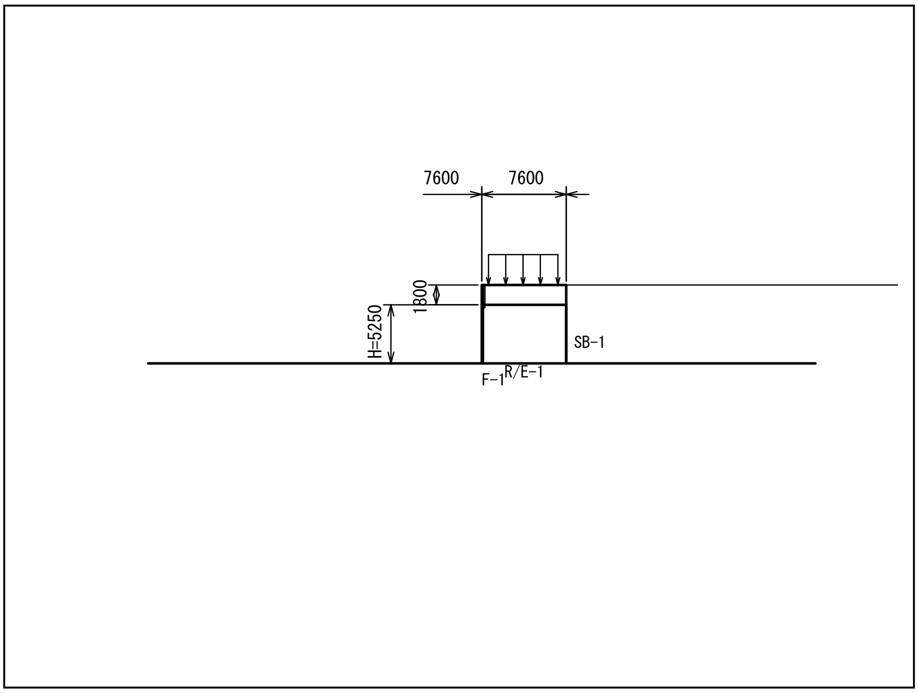
5.1 ORDINARY

5.2 EARTHQUAKE

1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H = 5.250 (m)

HEIGHT OF COPING : H4 = 1.800 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	0.000	0.000	5.250

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	5.250
	2	0.000	7.050
	3	7.600	7.050
	4	7.600	0.000

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	0.000
	2	30.000	0.000

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE：RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 ：MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION
PERFORMANCE 2 ：KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL
PERFORMANCE 3 ：KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	c es (kN/m^2)	fo*	tanϕ1
SB－1	1.800	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	5.250	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB：SURCHARGE BACKFILL, R/E：R/E, BE：BACKSIDE EMBANKMENT, F：FOUNDATION

Hs ：LAYER THICKNESS (m)
γ ：UNIT WEIGHT OF SOIL (kN/m^3)
γ' ：UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)
ϕ ：ANGLE OF SHEARING RESISTANCE (°)
c ：COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)
c es ：COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL： c es＝10kN/m^2)
fo* ：APPARENT COEFFICIENT OF FRICTION
tanϕ_1 ：APPARENT COEFFICIENT OF FRICTION

1.5　SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP L+B_G(m)	WIDTH OF LOAD B_L(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	ARBITRARY LOAD1	0.600	6.250	11.60	0.00

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6　HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1} = c_z \cdot k_{ho} = 0.18$

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

［ CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ ］

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 ［ AREA : A ］

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2} = c_z \cdot k_{ho} \cdot \nu = 0.13$

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

［ CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ ］

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012

c_z : AREA CORRECTION COEFFICIENT = 1.00 ［ AREA : A ］

ν : CORRECTION COEFFICIENT = 0.70

(3) OVERALL STABILITY EXAMINATION INCLUDING R/E

(EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

$k_{h3} = c_z \cdot k_{ho} = 0.12$

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.12

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR EMBANKMENT CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT h_c (m)	UNIT WEIGHT γ_c (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σ_a=140.0	σ_{aE}=210.0
BOLT SHEAR STRESS (N/mm^2)	τ_a=200.00	τ_{aE}=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$
CIRCULAR SLIP	$F_s \geqq 1.20$	$F_{sE} \geqq 1.00$

1.9 MATERIAL

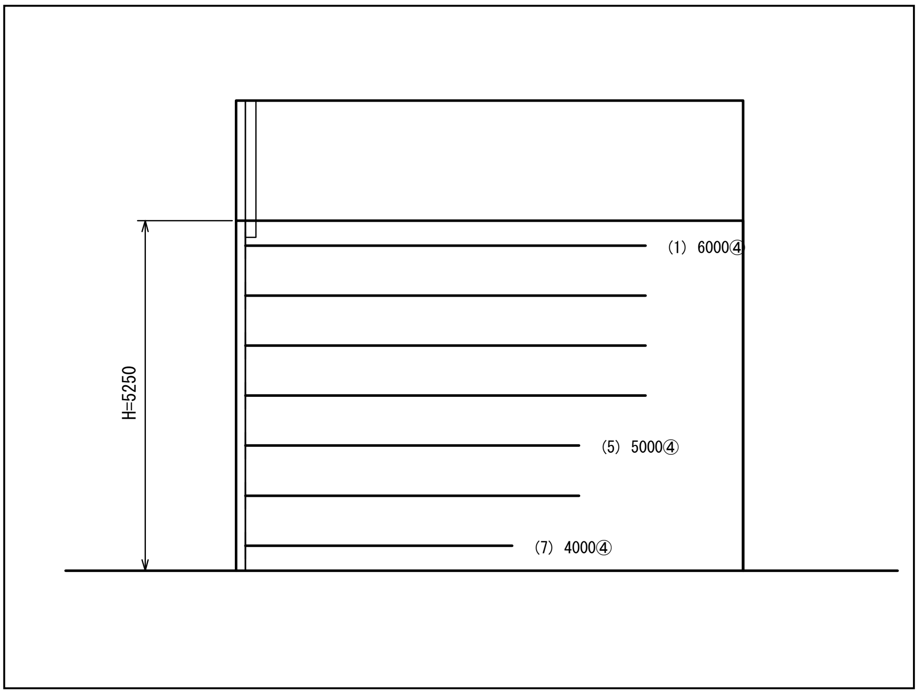
(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	Ae	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n'	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	Cm	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z_i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB (m)	DESIGN LENGTH L (m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.175	0.750	0.750	6.000	5.195	5.489	2.523	1.424	○
2	2.925	0.750	0.750	6.000	5.076	5.198	2.624	1.593	○
3	3.675	0.750	0.750	6.000	4.954	4.946	2.714	1.747	○
4	4.425	0.750	0.750	6.000	4.528	4.308	2.997	2.099	○
5	5.175	0.750	0.750	5.000	4.207	4.000	2.515	1.809	○
6	5.925	0.750	0.750	5.000	4.000	4.000	2.648	1.996	○
7	6.675	0.750	0.750	4.000	4.000	4.000	2.304	1.810	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	σtEi (N/mm^2)	σotEi (N/mm^2)	τoEi (N/mm^2)	
1	66.3	61.2	70.7	76.7	70.8	81.9	○
2	74.4	68.7	79.5	84.8	78.3	90.6	○
3	81.6	75.3	87.1	92.0	85.0	98.3	○
4	87.5	80.8	93.4	98.1	90.6	104.7	○
5	94.0	86.8	100.4	106.7	98.5	114.0	○
6	99.5	91.9	106.2	114.1	105.3	121.8	○
7	109.9	101.4	117.3	125.8	116.1	134.3	○

σti , σtEi : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σoti, σotEi : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τoi , τoEi : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3 EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.511 (1.500)	○	2.750 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.819 (0.662)	○	-0.188 (1.324)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	206.209	—	188.375	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	198.960	—	177.321	—

2.4 OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R(m)	Fsmin	Fs	JUDGE
		X(m)	Y(m)				
WITHIN REINFORCED AREA	ORDINARY-1	-12.500	18.000	21.915	1.302	1.200	○
	EARTHQUAKE-1	-5.000	15.000	15.811	1.345	1.000	○

2.5 RESULTS OF EXAMINATION OF INTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

2.5.1 LIST OF WALL HEIGHTS AND STRIP STRENGTHS

NOTE) HORIZONTAL SPACING OF STRIP FOR EACH ORDINAL STAGE NUMBER IS REPRESENTED IN THE FOLLOWING SYMBOLS:

④= 0.750 (m),

i : STRIPSTAGE

H : HEIGHT OF R/E(m)

VALUES IN THE TABLE MINIMUM STRIP LENGTH(m)

() IN THE EXECUTION STRIP LENGTH(m)

i	(1) H=5.250(m)	(2) H=4.500(m)	(3) H=3.750(m)
1	④ 5.489 (6.0)	④ 5.156 (6.0)	④ 4.846 (6.0)
2	④ 5.198	④ 4.885	④ 4.539
3	④ 4.954	④ 4.510 (5.0)	④ 4.060 (5.0)
4	④ 4.528	④ 4.086	④ 3.636
5	④ 4.207 (5.0)	④ 3.768	④ 3.318 (4.0)
6	④ 3.941	④ 3.506 (4.0)	
7	④ 3.502 (4.0)		

2.5.2 LIST OF WALL HEIGHTS AND STRUCTURAL DETAIL SPECIFICS

BASED ON: DESIGN AND CONSTRUCTION MANUAL FOR REINFORCED EARTHWORK

Ｈ : HEIGHT OF R/E (m)
Ｈ4 : HEIGHT OF COPING (m)
Ｈａ : VIRTUAL WALL HEIGHT (m)
Ｈ1 : HEIGHT OF SURCHARGE BACKFILL IN THE UPPER PART OF R/E (m)
ｎ : SLOPE INCLINE
Ｂｂ : FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN (m)
Ｌ1 : DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER (m)
Ｑｄｊ_W1 : ARBITRARY LOAD AFFECTING DIFFERENT STAGE STRIP (ORDINARY) (kN/m^2)
Ｑｄｊ_W2 : ARBITRARY LOAD AFFECTING DIFFERENT STAGE STRIP (SEISMIC) (kN/m^2)
B_G : HORIZONTAL DISTANCE FROM ROAD SHOULDER OF SKIN ELEMENT TO SURCHARGE LOAD (m)
B_L : WIDTH OF LOAD (m)
Ｌｓｄ : MINIMUM STRIP LENGTH (m)
Ｈｓｄ : PLACEMENT AREA OF MINIMUM STRIP LENGTH (m)

(1) Ｈ＝5.250 (m), Ｈ4＝1.800 (m), Ｈａ＝7.050 (m), Ｈ1＝1.800 (m)
１ : ｎ＝1 : 0.000, Ｂｂ ＝7.460 (m), Ｌ1 ＝7.460 (m)
Ｑｄ1_W1＝ 11.600 (kN/m^2), Ｑｄ1_W2＝ 0.000 (kN/m^2), B_G＋Ｌ1＝ 0.600 (m), B_L＝ 6.250 (m)
NEAR THE TOP : Ｌｓｄ ＝ 0.7 × Ｈａ ＝ 4.935 (m)
NEAR THE TOP : Ｈｓｄ ＝ 0.5 × Ｈａ ＝ 3.525 (m)・・・ < 1TH TO 2TH STAGES >
NEAR THE LOWER : Ｌｓｄ ＝ 0.4 × Ｈａ ＝ 2.820 ≧ 4.000 (m)
NEAR THE LOWER : Ｈｓｄ ＝ 0.3 × Ｈａ ＝ 2.115 (m)・・・ < 5TH TO 7TH STAGES >

(2) Ｈ＝4.500 (m), Ｈ4＝1.850 (m), Ｈａ＝6.350 (m), Ｈ1＝1.850 (m)
１ : ｎ＝1 : 0.000, Ｂｂ ＝0.000 (m), Ｌ1 ＝0.000 (m)
Ｑｄ1_W1＝ 11.600 (kN/m^2), Ｑｄ1_W2＝ 0.000 (kN/m^2), B_G＋Ｌ1＝ 0.600 (m), B_L＝ 6.250 (m)
NEAR THE TOP : Ｌｓｄ ＝ 0.7 × Ｈａ ＝ 4.445 (m)
NEAR THE TOP : Ｈｓｄ ＝ 0.5 × Ｈａ ＝ 3.175 (m)・・・ < 1TH TO 2TH STAGES >
NEAR THE LOWER : Ｌｓｄ ＝ 0.4 × Ｈａ ＝ 2.540 ≧ 4.000 (m)
NEAR THE LOWER : Ｈｓｄ ＝ 0.3 × Ｈａ ＝ 1.905 (m)・・・ < 4TH TO 6TH STAGES >

(3)　H＝3.750 (m), H4＝1.850 (m), Ha＝5.600 (m), H1＝1.850 (m)

1 : n＝1 : 0.000, Bb ＝0.000 (m), L1 ＝0.000 (m)

Qd1_W1＝ 11.600 (kN/m^2), Qd1_W2＝ 0.000 (kN/m^2), B_G＋L1＝ 0.600 (m), B_L＝ 6.250 (m)

NEAR THE TOP : Lsd ＝ 0.7 × Ha ＝ 3.920 (m)

NEAR THE TOP : Hsd ＝ 0.5 × Ha ＝ 2.800 (m)・・・＜ 1TH TO 1TH STAGES ＞

NEAR THE LOWER : Lsd ＝ 0.4 × Ha ＝ 2.240 ≧ 4.000 (m)

NEAR THE LOWER : Hsd ＝ 0.3 × Ha ＝ 1.680 (m)・・・＜ 4TH TO 5TH STAGES ＞

2.6　RESULTS OF EXAMINATION OF EXTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

(1)　H＝5.250 (m)

・VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.973 (m)
・THE SHAPE OF THE FOUNDATION　WIDTH OF THE FOUNDATION : b ＝0.400 (m)
HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.511 (1.500)	○	2.750 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.819 (0.662)	○	-0.188 (1.324)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	206.209	—	188.375	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	198.960	—	177.321	—

(2)　H＝4.500 (m)

・VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.940 (m)
・THE SHAPE OF THE FOUNDATION　WIDTH OF THE FOUNDATION : b ＝0.400 (m)
HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	9.141 (1.500)	○	2.379 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.882 (0.657)	○	-0.248 (1.313)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	191.310	—	173.162	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	169.702	—	150.541	—

(3)　Ｈ＝3.750 (m)

・VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL：Ｂ　＝3.890 (m)

・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION　：ｂ　＝0.400 (m)

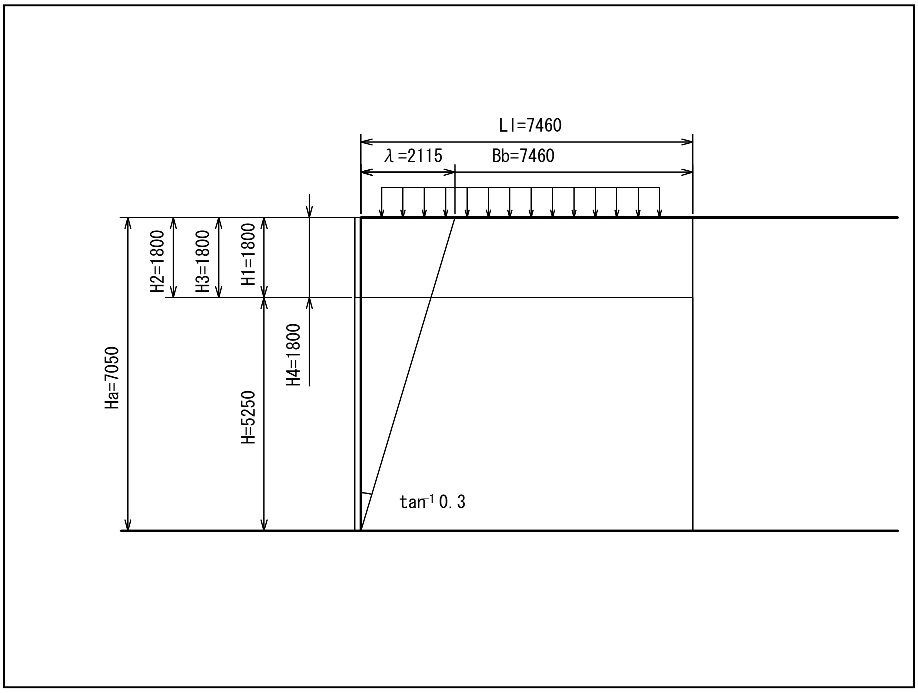
HEIGHT OF THE FOUNDATION：ｈｆ ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Ｆｓ	13.829 (1.500)	○	2.709 (1.200)	○
OVERTUENING	ECCENTLICTY ｅ (m)	-1.030 (0.648)	○	-0.464 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION ｑｓ (kN/m^2)	175.622	―	156.626	―
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION ｑｗ (kN/m^2)	141.467	―	125.625	―

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 5.250 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : H4 = 1.800 (m)

HEIGHT OF SURCHARGE BACKFILL : H1 = 1.800 (m)

SLOPE INCLINE (1：n) : n = 0.000

FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN : Bb = 7.460 (m)

DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER : Ll = 7.460 (m)

3.1.2 VIRTUAL WALL HEIGHT

$$Ha = H + H2 = 7.050\ (m)$$

Ha : VIRTUAL WALL HEIGHT (m)

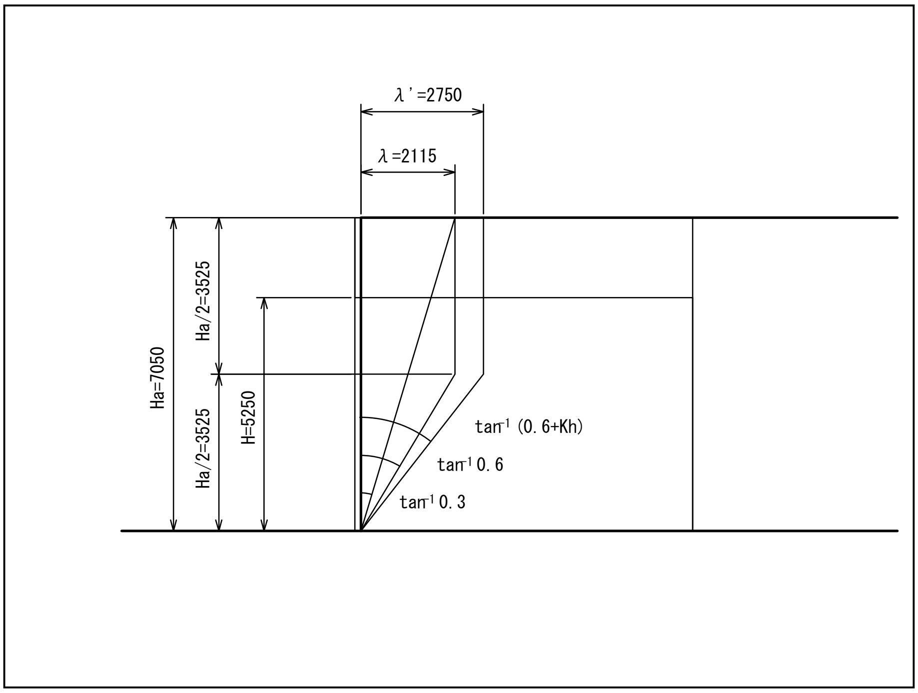
H : HEIGHT OF R/E = 5.250 (m)

H2 : DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 1.800$

H4 : HEIGHT OF COPING = 1.800 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$zi = \Delta H \cdot (i-1/2) + H2$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

ΔH : VERTICAL SPACING OF STRIP ＝ 0.750 (m)

H2 : $H2+Ha=0.3(H+H4)-Bb/n-0.3+H4$ (m) ＝ 1.800 (m)

STAGE (i)	zi (m)
1	2.175
2	2.925
3	3.675
4	4.425
5	5.175
6	5.925
7	6.675

3.2 LOAD

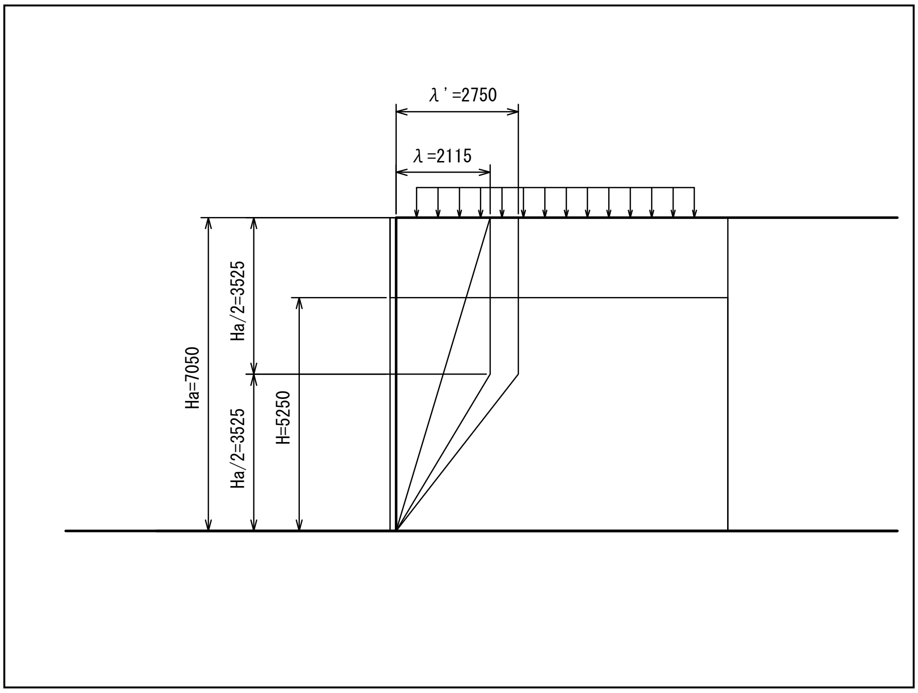
3.2.1 SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

$q1 = \gamma 2 \cdot H3 = 34.200$ (kN/m^2)

$q1$: SURCHARGE SOIL LOAD (kN/m^2)
$\gamma 2$: UNIT WEIGHT OF SURCHARGE SOIL = 19.0 (kN/m^3)
$H3$: ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 1.800 (m)

3.2.2 CALCULATION OF ARBITRARY LOAD APPLIED TO EACH STAGE



THE ARBITRARY LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE ARBITRARY LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA.WHEN THE ARBITRARY LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE ARBITRARY LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{di} = q_d \cdot \frac{B_d}{B_{di}}$$

ここに, q_{di} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_d : ARBITRARY LOAD (kN/m^2)

B_d : ARBITRARY LOAD WIDTH (m)

B_{di} : DISPERSIVE WIDTH OF THE ARBITRARY LOAD AT iTH STAGE STRIP (m)

(1) ARBITRARY LOAD-1

q_d (ORDINARY) ＝ 11.600 (kN/m^2)　B_d ＝ 6.250 (m)

q_{dE} (EARTHQUAKE) ＝ 0.000 (kN/m^2)

STAGE (i)	z_i (m)	B_{di} (m)	ORDINARY q_{di} (kN/m^2)	EARTH QUAKE q_{dEi} (kN/m^2)
1	2.175	6.250	11.600	0.000
2	2.925	6.250	11.600	0.000
3	3.675	6.250	11.600	0.000
4	4.425	6.250	11.600	0.000
5	5.175	6.250	11.600	0.000
6	5.925	6.250	11.600	0.000
7	6.675	6.250	11.600	0.000

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY (V_i) AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE (V_{Ei}) .

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.175	4.989	5.794
2	2.925	4.111	4.742
3	3.675	3.495	4.021
4	4.425	3.041	3.484
5	5.175	2.689	3.079
6	5.925	2.408	2.751
7	6.675	2.187	2.492

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$$q_i = q_1 + q_{di} + V_i$$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 34.200 (kN/m^2)

q_{di} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{di} (kN/m^2)	V_i (kN/m^2)	q_i (kN/m^2)
1	2.175	11.600	4.989	50.789
2	2.925	11.600	4.111	49.911
3	3.675	11.600	3.495	49.295
4	4.425	11.600	3.041	48.841
5	5.175	11.600	2.689	48.489
6	5.925	11.600	2.408	48.208
7	6.675	11.600	2.187	47.987

(2) EARTHQUAKE

$$q_{Ei} = q_1 + q_{dEi} + V_{Ei}$$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 34.200 (kN/m^2)

q_{dEi} : ARBITRARY LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{dEi} (kN/m^2)	V_{Ei} (kN/m^2)	q_{Ei} (kN/m^2)
1	2.175	0.000	5.794	39.994
2	2.925	0.000	4.742	38.942
3	3.675	0.000	4.021	38.221
4	4.425	0.000	3.484	37.684
5	5.175	0.000	3.079	37.279
6	5.925	0.000	2.751	36.951
7	6.675	0.000	2.492	36.692

3.3　EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（H_i）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_{Ei}）.

STAGE (i)	z_i (m)	ORDINARY H_i (kN/m)	EARTH QUAKE H_{Ei} (kN/m)
1	2.175	2.108	3.671
2	2.925	1.433	2.445
3	3.675	0.759	1.218
4	4.425	0.087	0.000
5	5.175	0.000	0.000
6	5.925	0.000	0.000
7	6.675	0.000	0.000

3.4　EARTH PRESSURE

3.4.1　COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq z_o = 6.0$ (m)

$$K_i = K_o \cdot \left(1 - \frac{z_i}{z_o}\right) + K_A \cdot \frac{z_i}{z_o}$$

IF $z_i > z_o = 6.0$ (m)

$$K_i = K_A$$

K_i　: COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

K_o　: COEFFICIENT OF EARTH PRESSURE AT REST

$$K_o = 1 - \sin\phi$$

K_A　: COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

z_i　: STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

z_o　: DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE = 6.000 (m)

STAGE (i)	z_i (m)	ϕ (°)	K_o	K_A	K_i
1	2.175	30.0	0.5000	0.3333	0.4396
2	2.925	30.0	0.5000	0.3333	0.4188
3	3.675	30.0	0.5000	0.3333	0.3979
4	4.425	30.0	0.5000	0.3333	0.3771
5	5.175	30.0	0.5000	0.3333	0.3562
6	5.925	30.0	0.5000	0.3333	0.3354
7	6.675	30.0	0.5000	0.3333	0.3333

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$Pi = Ki \cdot \sigma vi \cdot \Delta H + Hi$$

$$To = 0.75 \cdot Pi$$

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

Hi : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$$\sigma vi = \gamma 1 \cdot xi + qi$$

$\gamma 1$: UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

xi : $xi = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

qi : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	xi (m)	Ki	qi (kN/m^2)	σvi (kN/m^2)	Hi (kN/m)	Pi (kN/m)	To (kN/m)
1	0.375	0.440	50.789	57.914	2.108	21.202	15.901
2	1.125	0.419	49.911	71.286	1.433	23.821	17.866
3	1.875	0.398	49.295	84.920	0.759	26.102	19.577
4	2.625	0.377	48.841	98.716	0.087	28.005	21.004
5	3.375	0.356	48.489	112.614	0.000	30.089	22.567
6	4.125	0.335	48.208	126.583	0.000	31.844	23.883
7	4.875	0.333	47.987	140.612	0.000	35.153	26.365

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = P_i + \Delta P_i$$

$$\Delta P_i = \frac{1}{2} \cdot (1 + \frac{z_i}{H_a}) \cdot \alpha \cdot k_h \cdot P_n$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$P_i = K_i \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$

ΔP_i : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_n : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 32.329 (kN/m)

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

H_a : VIRTUAL WALL HEIGHT = 7.050 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{VEi} = \gamma_1 \cdot x_i + q_{Ei}$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.440	39.994	47.119	3.671	19.206
2	1.125	0.419	38.942	60.317	2.445	21.388
3	1.875	0.398	38.221	73.846	1.218	23.256
4	2.625	0.377	37.684	87.559	0.000	24.763
5	3.375	0.356	37.279	101.404	0.000	27.094
6	4.125	0.335	36.951	115.326	0.000	29.012
7	4.875	0.333	36.692	129.317	0.000	32.329

STAGE (i)	z_i (m)	ΔP_i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.175	5.330	24.536	18.402
2	2.925	5.764	27.152	20.364
3	3.675	6.197	29.453	22.090
4	4.425	6.630	31.393	23.545
5	5.175	7.064	34.157	25.618
6	5.925	7.497	36.509	27.381
7	6.675	7.930	40.260	30.195

3.5 HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS = 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT = 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP = 80.0 (mm)

t : THE THICKNESS OF STRIP = 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP = 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP = 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER = 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

j　　：THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM ＝ 2

n　　：THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS ＝ 1 (PIECE)

Ae　　：SCREW EFFECTIVE AREA OF MOUNTING BOLTS＝ 84.3 (mm^2)

STAGE (i)	Pi (kN/m)	To (kN/m)	ΔB1i (m)	ΔB2i (m)	ΔB3i (m)
1	21.202	15.901	1.584	1.716	2.120
2	23.821	17.866	1.410	1.528	1.887
3	26.102	19.577	1.287	1.394	1.722
4	28.005	21.004	1.199	1.299	1.605
5	30.089	22.567	1.116	1.209	1.494
6	31.844	23.883	1.055	1.143	1.411
7	35.153	26.365	0.955	1.035	1.278

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)

τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)

P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	24.536	18.402	2.054	2.225	2.748
2	27.152	20.364	1.856	2.010	2.483
3	29.453	22.090	1.711	1.853	2.289
4	31.393	23.545	1.605	1.739	2.148
5	34.157	25.618	1.475	1.598	1.974
6	36.509	27.381	1.380	1.495	1.847
7	40.260	30.195	1.251	1.356	1.675

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF $1.000 > \Delta Bi \geqq 0.750$　　$\Delta Bi = 0.750$

STAGE (i)	$\Delta B1i$ (m)	$\Delta B2i$ (m)	$\Delta B3i$ (m)	$\Delta B1_{E}i$ (m)	$\Delta B2_{E}i$ (m)	$\Delta B3_{E}i$ (m)	ΔBi (m)
1	1.584	1.716	2.120	2.054	2.225	2.748	0.750
2	1.410	1.528	1.887	1.856	2.010	2.483	0.750
3	1.287	1.394	1.722	1.711	1.853	2.289	0.750
4	1.199	1.299	1.605	1.605	1.739	2.148	0.750
5	1.116	1.209	1.494	1.475	1.598	1.974	0.750
6	1.055	1.143	1.411	1.380	1.495	1.847	0.750
7	0.955	1.035	1.278	1.251	1.356	1.675	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)
T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)
P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)
T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)
ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	21.202	15.901	15.901	11.926	24.536	18.402	18.402	13.801
2	0.750	23.821	17.866	17.866	13.399	27.152	20.364	20.364	15.273
3	0.750	26.102	19.577	19.577	14.683	29.453	22.090	22.090	16.567
4	0.750	28.005	21.004	21.004	15.753	31.393	23.545	23.545	17.659
5	0.750	30.089	22.567	22.567	16.925	34.157	25.618	25.618	19.214
6	0.750	31.844	23.883	23.883	17.912	36.509	27.381	27.381	20.536
7	0.750	35.153	26.365	26.365	19.774	40.260	30.195	30.195	22.646

3.7　STRIP LENGTH

3.7.1　FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^{*} = fo^{*} \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^{*} = \tan\phi 1$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^{*} = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^{*}	$\tan\phi 1$	fi^{*}
1	2.175	1.500	0.727	1.2196
2	2.925	1.500	0.727	1.1229
3	3.675	1.500	0.727	1.0263
4	4.425	1.500	0.727	0.9296
5	5.175	1.500	0.727	0.8329
6	5.925	1.500	0.727	0.7362
7	6.675	1.500	0.727	0.7265

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 3.525$ (m)

$$L_{oi} = 0.3 \cdot Ha$$

IF $z_i > Ha/2 = 3.525$ (m)

$$L_{oi} = 0.6 \cdot (Ha - z_i)$$

L_{oi} : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Ha : VIRTUAL WALL HEIGHT = 7.050 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 3.525$ (m)

$$L_{oEi} = (0.6 + k_{h1}) \cdot (Ha/2)$$

IF $z_i > Ha/2 = 3.525$ (m)

$$L_{oEi} = (0.6 + k_{h1}) \cdot (Ha - z_i)$$

L_{oEi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)
Ha : VIRTUAL WALL HEIGHT = 7.050 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	z_i (m)	L_{oi} (m)	L_{oEi} (m)
1	2.175	2.115	2.750
2	2.925	2.115	2.750
3	3.675	2.025	2.633
4	4.425	1.575	2.048
5	5.175	1.125	1.463
6	5.925	0.675	0.878
7	6.675	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqi} = \frac{Fs \cdot Ti}{2 \cdot fi^{*} \cdot (\sigma vi - Vi) \cdot b}$$

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)
Fs : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00
Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
b : STRIP WIDTH (m)
σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)
Vi : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	Ti (kN/PIECE)	fi*	σvi-Vi (kN/m^2)	b (m)	Lreqi (m)
1	15.901	1.2196	52.925	0.0800	3.080
2	17.866	1.1229	67.175	0.0800	2.961
3	19.577	1.0263	81.425	0.0800	2.929
4	21.004	0.9296	95.675	0.0800	2.953
5	22.567	0.8329	109.925	0.0800	3.082
6	23.883	0.7362	124.175	0.0800	3.266
7	26.365	0.7265	138.425	0.0800	3.277

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^* \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^*	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	18.402	1.2196	41.325	0.0800	2.739
2	20.364	1.1229	55.575	0.0800	2.448
3	22.090	1.0263	69.825	0.0800	2.313
4	23.545	0.9296	84.075	0.0800	2.260
5	25.618	0.8329	98.325	0.0800	2.347
6	27.381	0.7362	112.575	0.0800	2.478
7	30.195	0.7265	126.825	0.0800	2.458

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$\mathrm{Lmini} = \mathrm{Loi} + \mathrm{Lreqi}$$

EARTHQUAKE

$$\mathrm{Lmin_{E}i} = \mathrm{Lo_{E}i} + \mathrm{Lreq_{E}i}$$

Lmini : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)
Loi : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Lreqi : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)
Lmin$_{E}$i : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)
Lo$_{E}$i : LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)
Lreq$_{E}$i : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	Loi (m)	Lreqi (m)	Lmini (m)	Lo$_{E}$i (m)	Lreq$_{E}$i (m)	Lmin$_{E}$i (m)
1	2.115	3.080	5.195	2.750	2.739	5.489
2	2.115	2.961	5.076	2.750	2.448	5.198
3	2.025	2.929	4.954	2.633	2.313	4.946
4	1.575	2.953	4.528	2.048	2.260	4.308
5	1.125	3.082	4.207	1.463	2.347	3.810
6	0.675	3.266	3.941	0.878	2.478	3.356
7	0.225	3.277	3.502	0.293	2.458	2.751

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

WHEN SURCHERGE BACKFILL HEIGHT IS UNDER 2M ($0\,\mathrm{m} \leqq H1 = 1.800 < 2\,\mathrm{m}$)

0.5 Ha OR MORE FROM UPPER EDGE OF Ha

$L_{sd} = 0.7 \cdot Ha = 4.935$ (m)

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$L_{sd} = 0.4 \cdot Ha = 2.820$ (m)

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

Ha : VIRTUAL WALL HEIGHT = 7.050 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)

L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)

L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)

L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	5.195	5.489	4.935	6.000
2	5.076	5.198	4.935	6.000
3	4.954	4.946	—	6.000
4	4.528	4.308	—	6.000
5	4.207	3.810	4.000	5.000
6	3.941	3.356	4.000	5.000
7	3.502	2.751	4.000	4.000

3.8　CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$Fsi = \frac{Si}{Ti} \geqq Fs = 2.00$$

Fsi　：SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

Fs　：DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 2.0

Ti　：PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Si　：FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$Si = 2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei$$

fi^{*}　：APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$\sigma vi'$　：VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b　：STRIP WIDTH AT iTH STAGE (m)

Lei　：EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$Lei = Li - Loi$$

Li　：DETERMINED STRIP LENGTH AT iTH STAGE (m)

Loi　：ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	Loi (m)	Lei (m)	fi^{*}	$\sigma vi'$ (kN/m^2)	b (m)	Si (kN/PIECE)	Ti (kN/PIECE)	Fsi	JUDGE
1	6.000	2.115	3.885	1.2196	52.925	0.0800	40.123	15.901	2.523	○
2	6.000	2.115	3.885	1.1229	67.175	0.0800	46.889	17.866	2.624	○
3	6.000	2.025	3.975	1.0263	81.425	0.0800	53.146	19.577	2.714	○
4	6.000	1.575	4.425	0.9296	95.675	0.0800	62.967	21.004	2.997	○
5	5.000	1.125	3.875	0.8329	109.925	0.0800	56.765	22.567	2.515	○
6	5.000	0.675	4.325	0.7362	124.175	0.0800	63.262	23.883	2.648	○
7	4.000	0.225	3.775	0.7265	138.425	0.0800	60.745	26.365	2.304	○

(2) EARTHQUAKE

$$F_{sE}i = \frac{S_{E}i}{T_{E}i} \geqq F_{sE} = 1.20$$

$F_{si}{}_{E}i$: SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP

F_{sE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2

$T_{E}i$: PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

$S_{E}i$: FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$S_{E}i = 2 \cdot fi^{*} \cdot \sigma_{VE}i \cdot b \cdot L_{eE}i$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$\sigma_{VE}i'$: VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

$L_{eE}i$: EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$L_{eE}i = Li - L_{OE}i$

Li : DETERMINED STRIP LENGTH AT iTH STAGE (m)

$L_{OE}i$: ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	$L_{OE}i$ (m)	$L_{eE}i$ (m)	f 1*	$\sigma_{VE}i'$ (kN/m^2)	b (m)	$S_{E}i$ (kN/PIECE)	$T_{E}i$ (kN/PIECE)	$F_{sE}i$	JUDGE
1	6.000	2.750	3.251	1.2196	41.325	0.0800	26.212	18.402	1.424	○
2	6.000	2.750	3.251	1.1229	55.575	0.0800	32.457	20.364	1.593	○
3	6.000	2.633	3.368	1.0263	69.825	0.0800	38.610	22.090	1.747	○
4	6.000	2.048	3.953	0.9296	84.075	0.0800	49.425	23.545	2.099	○
5	5.000	1.463	3.538	0.8329	98.325	0.0800	46.352	25.618	1.809	○
6	5.000	0.878	4.123	0.7362	112.575	0.0800	54.667	27.381	1.996	○
7	4.000	0.293	3.708	0.7265	126.825	0.0800	54.660	30.195	1.810	○

3.9 CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \ (N/mm^2)$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	15.901	11.926	66.3	61.2	70.7	○
2	17.866	13.399	74.4	68.7	79.5	○
3	19.577	14.683	81.6	75.3	87.1	○
4	21.004	15.753	87.5	80.8	93.4	○
5	22.567	16.925	94.0	86.8	100.4	○
6	23.883	17.912	99.5	91.9	106.2	○
7	26.365	19.774	109.9	101.4	117.3	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma t_{Ei} = \frac{T_{Ei}}{Ag} \leqq \sigma a_{E} = 210.0\ (\mathrm{N/mm^2})$$

$$\sigma ot_{Ei} = \frac{Tf_{Ei}}{An} \leqq \sigma a_{E} = 210.0\ (\mathrm{N/mm^2})$$

$$\tau o_{Ei} = \frac{Tf_{Ei}}{A\tau} \leqq \tau a_{E} = 300.0\ (\mathrm{N/mm^2})$$

σt_{Ei} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

σot_{Ei} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

τo_{Ei} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

σa_{E} : ALLOWABLE TENSILE STRESS OF STRIP ($\mathrm{N/mm^2}$)

τa_{E} : ALLOWABLE SHEARING STRESS OF BOLT ($\mathrm{N/mm^2}$)

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tf_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 ($\mathrm{mm^2}$)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 ($\mathrm{mm^2}$)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 ($\mathrm{mm^2}$)

STAGE (i)	T_{Ei} (kN/PIECE)	Tf_{Ei} (kN/PIECE)	σt_{Ei} ($\mathrm{N/mm^2}$)	σot_{Ei} ($\mathrm{N/mm^2}$)	τo_{Ei} ($\mathrm{N/mm^2}$)	JUDGE
1	18.402	13.801	76.7	70.8	81.9	○
2	20.364	15.273	84.8	78.3	90.6	○
3	22.090	16.567	92.0	85.0	98.3	○
4	23.545	17.659	98.1	90.6	104.7	○
5	25.618	19.214	106.7	98.5	114.0	○
6	27.381	20.536	114.1	105.3	121.8	○
7	30.195	22.646	125.8	116.1	134.3	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2	ARBITRARY LOAD
CASE 1-1	ORDINARY	○		○		○
CASE 1-2	ORDINARY	○		○		○
CASE 2-3	EARTHQUAKE	○	○		○	

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

CASE 1-1 : OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)

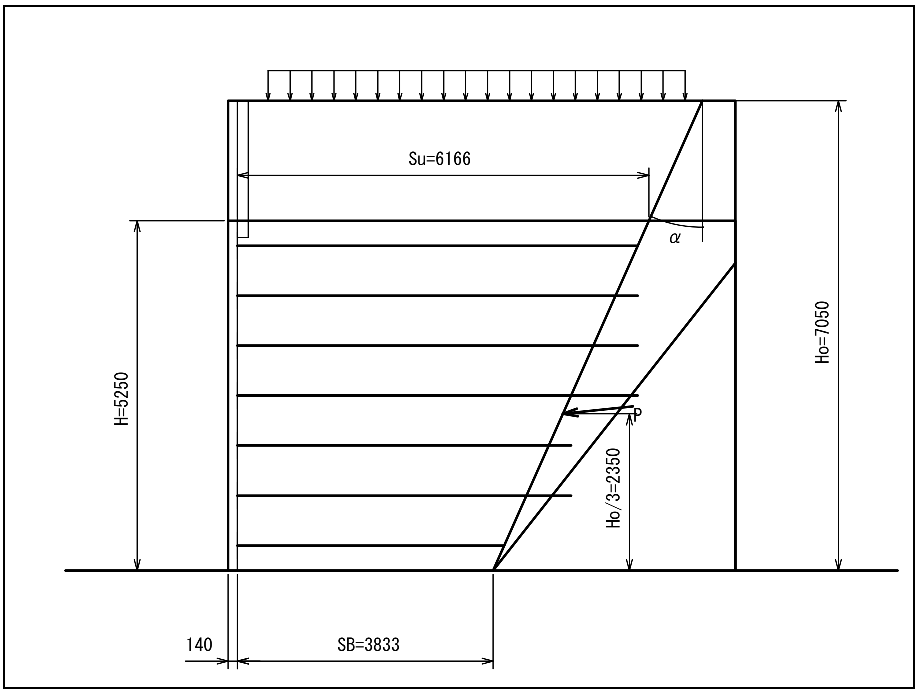
CASE 1-2 : OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 : ORDINARY LOAD

LOAD2 : EARTHQUAKELOAD

4.2 EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1 CASE 1 CALCULATION OF LOAD



H_o : WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 7.050 (m)

H : R/E WALL HEIGHT＝ 5.250 (m)

h_o : CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

S_u : REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 6.166 (m)

S_B : REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.833 (m)

B : R/E BOTTOM WIDTH ＝ 3.973 (m)

α : VIRTUAL BACK SLOPE INCLINE ＝ -23.959 (°)

$$\alpha = -\tan^{-1}\frac{S_u - S_B}{H} = -23.959\ (^\circ)$$

$W1$: OWN WEIGHT OF R/E ＝ 742.008 (kN/m)

Wq : LOADED WEIGHT ON R/E ＝ 72.500 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = (2 \cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = 0.000$ (m)

$\delta 1$: FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

$\omega 1$: ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 51.8 (°)

$W2$: OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) = 116.967 (kN/m)

$L1$: LENGTH OF SLIP SURFACE = 5.870 (m)

P_A : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 45.223$$

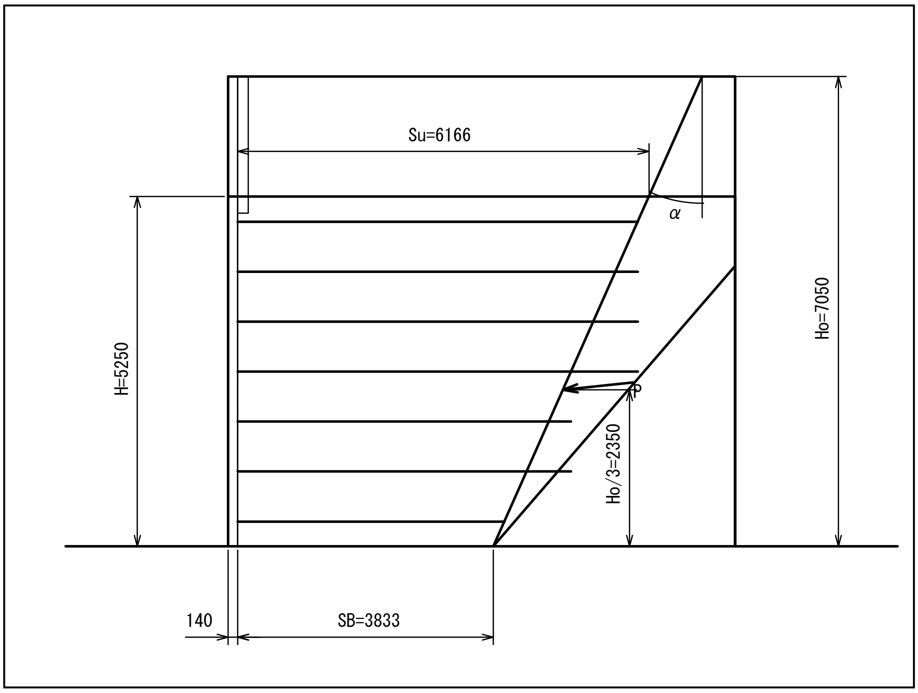
P_{Ah} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 44.972$ (kN/m)

P_{AV} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = 4.759$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK = -23.959 (°)

W1 : OWN WEIGHT OF R/E = 742.008 (kN/m)

Wq : LOADED WEIGHT ON R/E = 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL = 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL = 30.0 (°)

c : COHESION OF BACK FILL = 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45° + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK = 30.000 (°)

kh2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY")

θ : EARTHQUAKE COMPOUND ANGLE = $\tan^{-1} kh2$ = 7.407 (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE = 49.2 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE = 131.021 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE = 5.555 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 60.865$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 60.527$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = 6.405$ (kN/m)

4.3 AGGREGATION OF LOAD

4.3.1 ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	0.000	0.000	0.00
ARBITRARY LOAD	72.500	3.725	270.06	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	4.759	5.017	23.88	44.972	2.350	105.68
TOTAL	819.267		2403.88	44.972		105.68

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	0.000	0.000	0.00
ARBITRARY LOAD	72.500	3.725	270.06	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	4.759	5.017	23.88	44.972	2.350	105.68
TOTAL	819.267		2403.88	44.972		105.68

4.3.2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	96.461	3.857	372.08
REAR EARTH PRESSURE 2	6.405	5.017	32.14	60.527	2.350	142.24
TOTAL	748.413		2142.07	156.989		514.32

$kh2$: HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	742.008	2.844	2109.94	96.461	3.857	372.08
REAR EARTH PRESSURE 2	6.405	5.017	32.14	60.527	2.350	142.24
TOTAL	748.413		2142.07	156.989		514.32

$kh2$: HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 10.511 \geqq F_{sa} = 1.50 \cdots OK$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.973 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 819.267 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 44.972 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.750 \geqq F_{saE} = 1.20 \cdots OK$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.973 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 748.413 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 156.989 (kN/m)

4.6 STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.805 \text{ (m)}$$

d : R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

ΣMr : R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 2403.88 (kN·m/m)

ΣMo : R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 105.68 (kN·m/m)

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 819.267 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$e = B/2 - d = -0.819 \leqq B/6 = 0.662$ (m) ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.7　EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.175\ (m)$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 2142.07 (kN･m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 514.32 (kN･m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 748.413 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d3_E = -0.188 \leqq B/3 = 1.324\ (m)$$ ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.8 STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_s = \Sigma V / B = 206.209$ (kN/m^2)

q_s : VERTICAL SUBGRADE REACTION ON R/E BOTTOM (kN/m^2)

ΣV : TOTAL VERTICAL LOAD ON R/E BOTTOM = 819.267 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.9 STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_{SE} = \Sigma V_E / B = 188.375$ (kN/m^2)

q_{SE} : VERTICAL SUBGRADE REACTION ON R/E BOTTOM
DURING EARTHQUAKE (kN/m^2)

ΣV_E : TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE
$= 748.413$ (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B : R/E BOTTOM WIDTH = 3.973 (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma (To \cdot \tan\delta) = 53.563 \text{ (kN/m)}$$

To : TENSILE FORCE AGAINST WALL (kN/m)

δ : ANGLE OF WALL FRICTION (°) = 2/3ϕ

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	ϕ (°)	tanδ	Vci (kN/m)
1	15.901	30.0	0.364	5.788
2	17.866	30.0	0.364	6.503
3	19.577	30.0	0.364	7.125
4	21.004	30.0	0.364	7.645
5	22.567	30.0	0.364	8.214
6	23.883	30.0	0.364	8.693
7	26.365	30.0	0.364	9.596
TOTAL				53.563

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 198.960\ (kN/m^2)$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

Wc1 : THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 24.182\ (kN/m)$

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL $= 24.5\ (kN/m^3)$

Hta : TOTAL HEIGHT OF R/E = 7.050 (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL = 0.140 (m)

Wc2 : WEIGHT OF FOUNDATION CONCRETE $= hf \cdot b \cdot \gamma c2 = 1.840\ (kN/m)$

b : WIDTH OF CONCRETE FOUNDATION = 0.400 (m)

hf : HEIGHT OF CONCRETE FOUNDATION = 0.200 (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION $= 23.0\ (kN/m^3)$

Vc : VERTICAL COMPONENT AGAINST WALL = 53.563 (kN/m)

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$V_{cE} = \Sigma\ (T_{oE} \cdot \tan\delta_E) = 44.907\ (\text{kN/m})$$

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	18.402	30.0	0.268	4.931
2	20.364	30.0	0.268	5.456
3	22.090	30.0	0.268	5.919
4	23.545	30.0	0.268	6.309
5	25.618	30.0	0.268	6.864
6	27.381	30.0	0.268	7.337
7	30.195	30.0	0.268	8.091
TOTAL				44.907

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 177.321\ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL ＝ 24.182 (kN/m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE ＝ 1.840 (kN/m)

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE ＝ 44.907 (kN/m)

5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

5.1 ORDINARY

5.1.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{Ai} = \frac{\sigma a \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σa : ALLOWABLE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{Ai} (kN/m)
1	140.0	80.0	4.0	1.0	240.0	0.750	44.800
2	140.0	80.0	4.0	1.0	240.0	0.750	44.800
3	140.0	80.0	4.0	1.0	240.0	0.750	44.800
4	140.0	80.0	4.0	1.0	240.0	0.750	44.800
5	140.0	80.0	4.0	1.0	240.0	0.750	44.800
6	140.0	80.0	4.0	1.0	240.0	0.750	44.800
7	140.0	80.0	4.0	1.0	240.0	0.750	44.800

5.1.2 FORMULA FOR SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

$$Fs = \frac{R\Sigma\{ces\cdot l + (W'\cdot\cos\alpha + Tavail\cdot\sin\theta)\tan\phi + Tavail\cdot\cos\theta\}}{R\Sigma(W\cdot\sin\alpha)}$$

Fs : SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

ces : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : ces = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

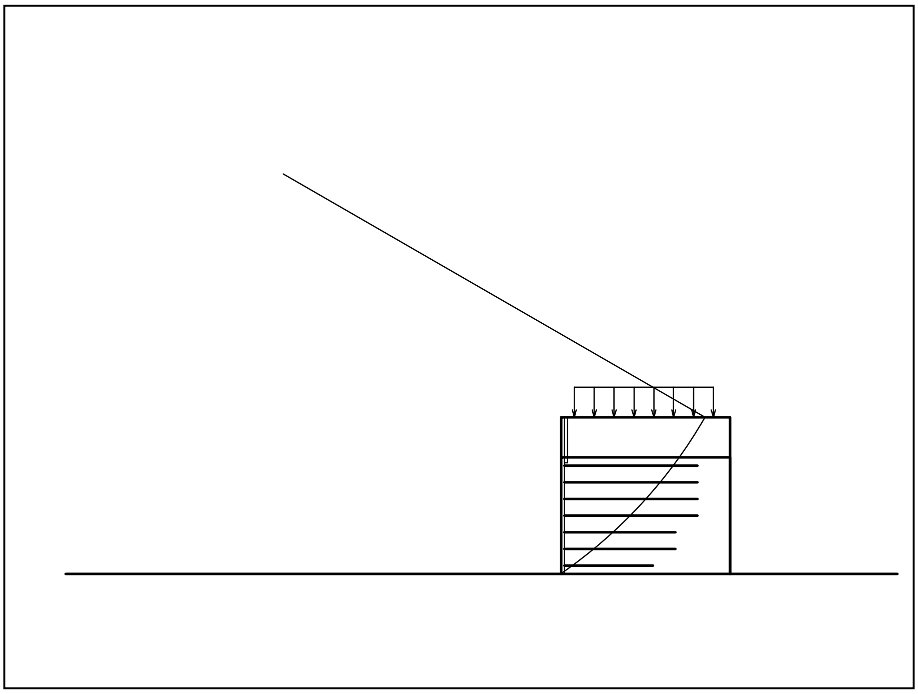
Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

R : RADIUS OF SLIP CIRCULAR ARC (m)

5.1.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [ORDINARY-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pi},\ T_{Ai})$$

$$T_{pi} = \frac{2 \cdot f_i^* \cdot \sigma_{vi} \cdot b \cdot L_{ei}}{F_s} \times \frac{1}{\Delta B}$$

T_{availi}	: AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
T_{pi}	: DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP (kN/m)
T_{Ai}	: ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
f_i^*	: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
σ_{vi}	: VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot z_i$ (kN/m^2)
z_i	: DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)
b	: WIDTH OF STRIP (m)
L_{ei}	: ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)
ΔB	: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)
F_s	: SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{vi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pi} (kN/m)	T_{Ai} (kN/m)	T_{availi} (kN/m)
1	1.2196	19.0 19.0	1.800 0.375	41.325	0.0800	1.090	0.750	5.860	44.800	5.860
2	1.1229	19.0 19.0	1.800 1.125	55.575	0.0800	1.677	0.750	11.163	44.800	11.163
3	1.0263	19.0 19.0	1.800 1.875	69.825	0.0800	2.319	0.750	17.725	44.800	17.725
4	0.9296	19.0 19.0	1.800 2.625	84.075	0.0800	3.024	0.750	25.209	44.800	25.209
5	0.8329	19.0 19.0	1.800 3.375	98.325	0.0800	2.799	0.750	24.450	44.800	24.450
6	0.7362	19.0 19.0	1.800 4.125	112.575	0.0800	3.658	0.750	32.338	44.800	32.338
7	0.7265	19.0 19.0	1.800 4.875	126.825	0.0800	3.616	0.750	35.540	44.800	35.540
								ΣT_{avail} = 152.287		

(3) LIST OF SAFETY FACTOR

Fs		COORDINATE OF CIRCULAR CENTER X						
		-18.5m	-16.5m	-14.5m	-12.5m	-10.5m	-8.5m	-6.5m
Y	24.0m	1.304	1.313	1.326	1.351	1.416	1.531	1.716
	22.0m	1.308	1.303	1.313	1.327	1.364	1.453	1.613
	20.0m	1.320	1.308	1.303	1.314	1.332	1.387	1.514
	18.0m	1.345	1.323	1.308	1.302	1.316	1.341	1.425
	16.0m	1.388	1.354	1.327	1.310	1.303	1.320	1.357
	14.0m	1.459	1.410	1.368	1.335	1.313	1.305	1.328
	12.0m	1.577	1.507	1.444	1.390	1.347	1.319	1.313

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	ORDINARY-1	JUDGE
MINIMUM SAFETY FACTOR	Fsmin Fs		1.302 (1.200)	○
ΣTavail	kN/m		152.287	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN･m/m	[2116.0] [5182.8]	—
	M_R	kN･m/m	7298.8	—
	M_T	kN･m/m	3745.8	—
ACTIVE MOMENT	M_D	kN･m/m	8484.2	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-12.500 18.000	—
RADIUS	R	m	21.915	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN･m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN･m/m)

M_R : RESISTER MOMENT OF SOIL $= M_{RC} + M_{RF}$ (kN･m/m)

M_T : RESISTER MOMENT OF STRIP (kN･m/m)

Fsmin : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP $= (M_R + M_T) / M_D$

5.2 EARTHQUAKE

5.2.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{AEi} = \frac{\sigma_{aE} \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{AEi} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σ_{aE} : ALLOWABLE EARTHQUAKE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σ_{aE} (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{AEi} (kN/m)
1	210.0	80.0	4.0	1.0	240.0	0.750	67.200
2	210.0	80.0	4.0	1.0	240.0	0.750	67.200
3	210.0	80.0	4.0	1.0	240.0	0.750	67.200
4	210.0	80.0	4.0	1.0	240.0	0.750	67.200
5	210.0	80.0	4.0	1.0	240.0	0.750	67.200
6	210.0	80.0	4.0	1.0	240.0	0.750	67.200
7	210.0	80.0	4.0	1.0	240.0	0.750	67.200

5.2.2 FORMULA FOR SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

$$F_{SE} = \frac{R\Sigma\{c_{es}\cdot l+(W'\cdot\cos\alpha-k_{h3}\cdot W\cdot\sin\alpha)\tan\phi\}+R\cdot\Sigma T_{avail}(\cos\theta+\sin\theta\cdot\tan\phi)}{\Sigma(R\cdot W\cdot\sin\alpha\ +\ k_{h3}\cdot W\cdot y_G)}$$

F_{SE} : SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

T_{avail} : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

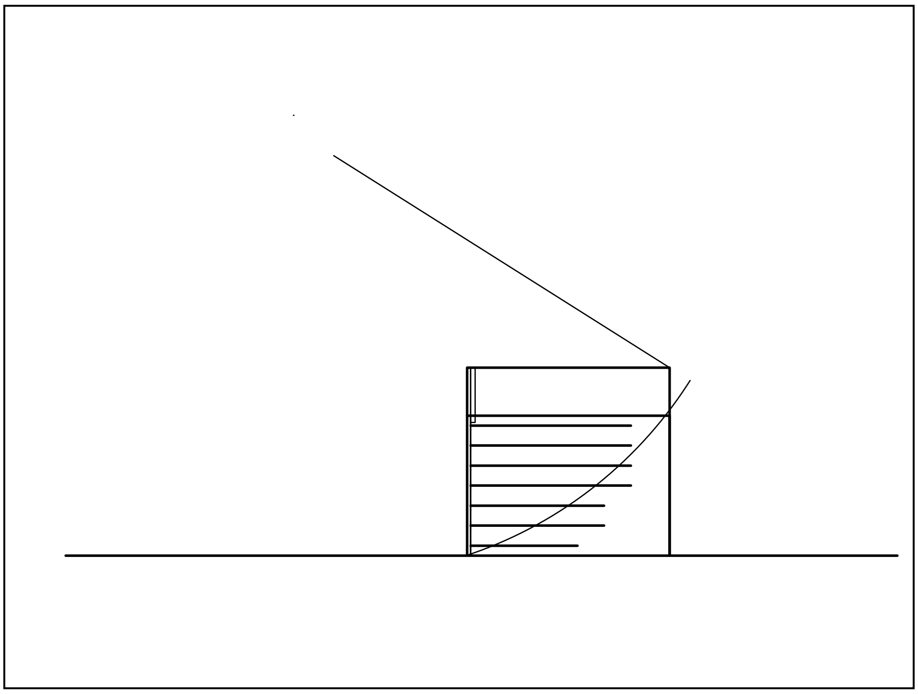
R : RADIUS OF SLIP CIRCULAR ARC (m)

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT = 0.12 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

y_G : VERTICAL DISTANCE FROM CENTER OF CIRCULAR ARC TO CENTER OF GRAVITY OF SPLIT PIECE (m)

5.2.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [EARTHQUAKE-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pEi},\ T_{AEi})$$

$$T_{pEi} = \frac{2 \cdot f_i{*} \cdot \sigma_{VEi} \cdot b \cdot L_{ei}}{F_{SE}} \times \frac{1}{\Delta B}$$

T_{availi} : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pEi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

T_{AEi} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

$f_i{*}$: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE $= \gamma \cdot z_i$ (kN/m^2)

z_i : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

L_{ei} : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ON THE EARTHQUAKE $= 1.20$

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{VEi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pEi} (kN/m)	T_{AEi} (kN/m)	T_{availi} (kN/m)
1	1.2196			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
2	1.1229			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
3	1.0263	19.0 19.0	1.800 1.875	69.825	0.0800	0.422	0.750	5.376	67.200	5.376
4	0.9296	19.0 19.0	1.800 2.625	84.075	0.0800	1.298	0.750	18.034	67.200	18.034
5	0.8329	19.0 19.0	1.800 3.375	98.325	0.0800	1.323	0.750	19.262	67.200	19.262
6	0.7362	19.0 19.0	1.800 4.125	112.575	0.0800	2.558	0.750	37.690	67.200	37.690
7	0.7265	19.0 19.0	1.800 4.875	126.825	0.0800	3.130	0.750	51.273	67.200	51.273
								ΣT_{avail} = 131.634		

(3) LIST OF SAFETY FACTOR

F_{SE}		COORDINATE OF CIRCULAR CENTER X						
		-11.0m	-9.0m	-7.0m	-5.0m	-3.0m	-1.0m	1.0m
Y	21.0m	1.378	1.351	1.376	*****	*****	*****	*****
	19.0m	1.435	1.364	1.352	*****	*****	*****	*****
	17.0m	1.498	1.421	1.349	1.365	*****	*****	*****
	15.0m	1.582	1.492	1.406	1.345	*****	*****	*****
	13.0m	1.700	1.590	1.486	1.392	*****	*****	*****
	11.0m	1.874	1.735	1.604	1.483	1.375	*****	*****
	9.0m	2.158	1.969	1.793	1.633	1.491	*****	*****

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	EARTHQUAKE-1	JUDGE
MINIMUM SAFETY FACTOR	F_{smin} F_{SE}		1.345 (1.000)	○
HORIZONTAL SEISMIC COEFFICIENT	k_{h3}		0.12	—
ΣTavail	kN/m		131.634	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN･m/m	[1501.0] [5063.8]	—
	M_R	kN･m/m	6564.8	—
	M_T	kN･m/m	2387.9	—
ACTIVE MOMENT	M_D	kN･m/m	6658.9	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-5.000 15.000	—
RADIUS	R	m	15.811	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN･m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN･m/m)

M_R : RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN･m/m)

M_T : RESISTER MOMENT OF STRIP (kN･m/m)

F_{smin} : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP
= (M_R+M_T) / M_D

Thanlyin Ramp

Concrete Barrier Type-1

1 Design Conditons

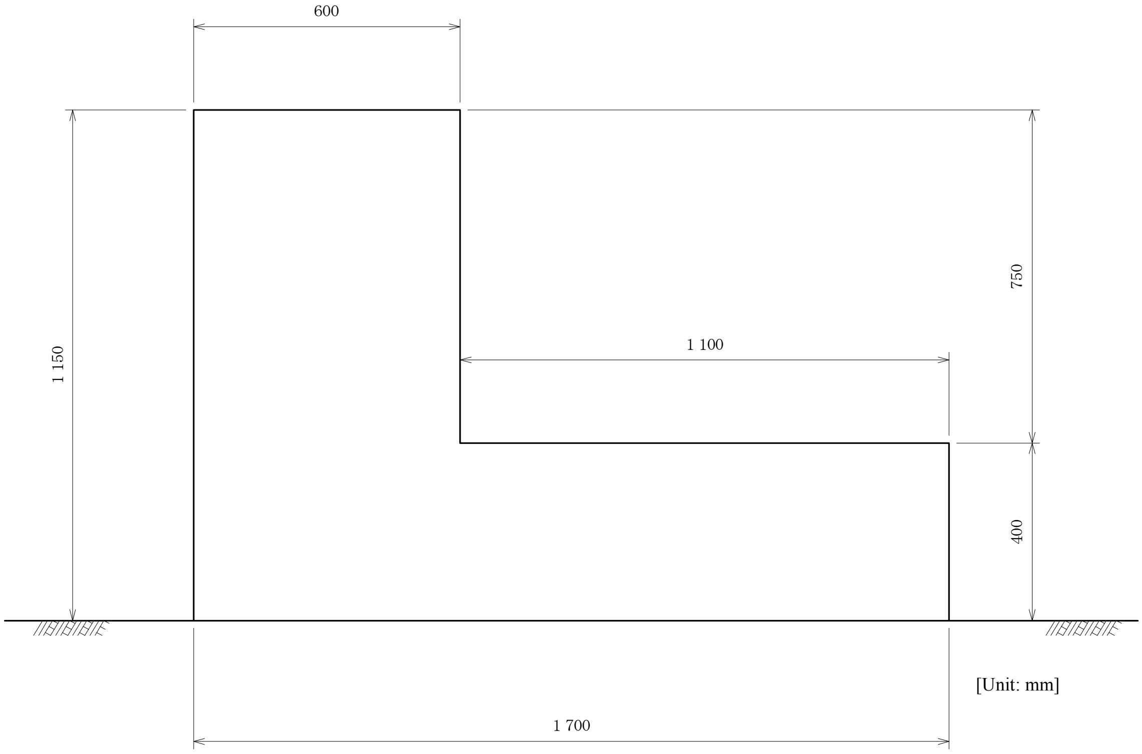
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 10000(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)
RC Bottom Slab: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】Soil at Rear Side: 30°

【Unit Weight】

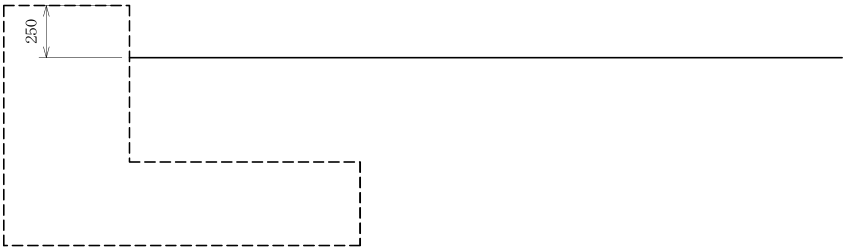
(kN/m^3)

Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】
Frame: Kh ＝ 0.18
Soil (Front side): Kh ＝ 0.18
Soil (Rear side): Kh ＝ 0.18

1.6 Soil

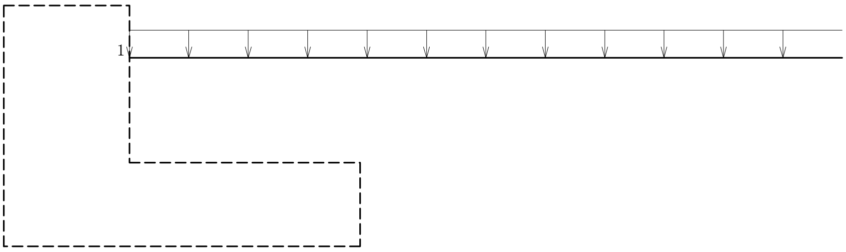
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr. In the stability Analysis (m)	0.000
In the vertical wall design(m)	0.000

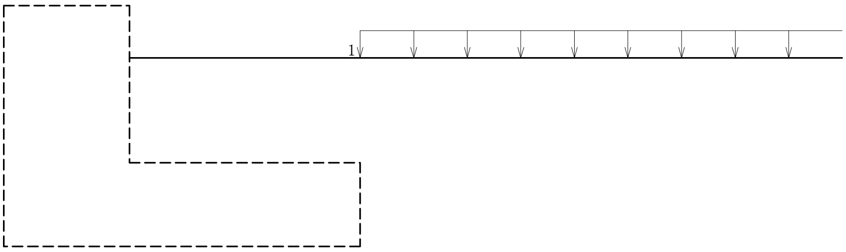
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m²)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

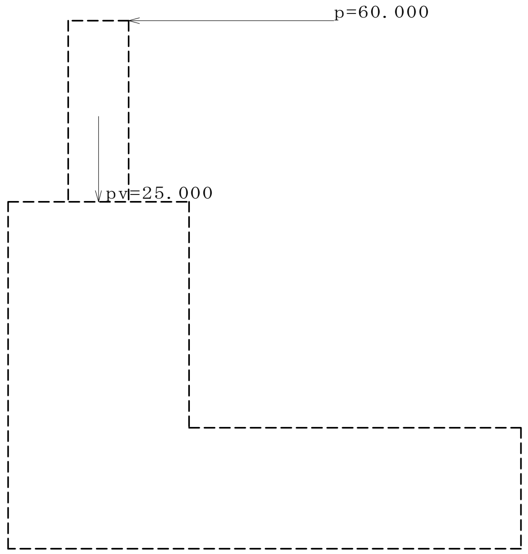


Number	Load Location (m)	Load Width (m)	Load (kN/m²)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	1.100	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.600	0.400	60.000

Vertical Force

Effective Width	Loading location x (m)	Front Wheel Load P (kN)
Same as horizontal force	0.300	25.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

・Earth Pressure Analysis Method: Trial Wedge Method

・Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

- In the stability analysis the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).

- In the stability analysis, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	———
Earthquake	0.000	0.000	———

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	——	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	——	——	——	——
	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, tanφ_B	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B / V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

2) Bottom Slab (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

τ_{a1} : Shear stress when shear force is bored by concrete

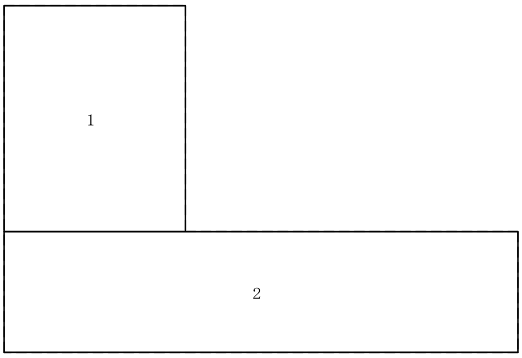
τ_{a2} : Shear stress when shared with diagonal tension bars

2 Stability Analysis

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



2) Volume・Center of Gravity

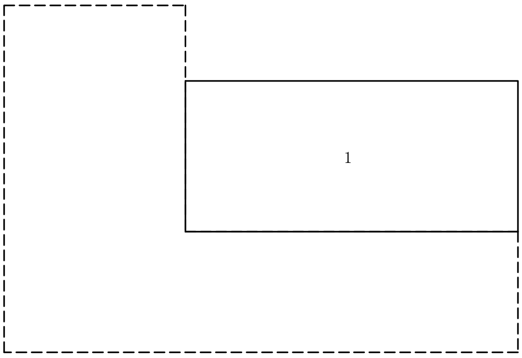
Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.600× 0.750× 1.000 1.700× 0.400× 1.000	0.450 0.680	0.300 0.850	0.775 0.200	0.135 0.578	0.349 0.136	
Σ		1.200	——	——	0.713	0.485	

Center of GravityXG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.713／ 1.130 ＝ 0.631 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.485／ 1.130 ＝ 0.429 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.900× 0.750× 1.000	0.675	1.150	0.650	0.633	0.357	
Σ		0.675	——	——	0.632	0.357	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.632／ 0.550 ＝ 1.150 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.357／ 0.550 ＝ 0.650 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.130 = 27.685	0.631
Soil (Rear Side)	19.000 × 0.550 = 10.450	1.150
Total	38.135	0.773

[2] Earthquake

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.130 = 27.685	0.631
Soil (Rear Side)	19.000 × 0.550 = 10.450	1.150
Total	38.135	0.773

Location	Horizontal Force $H = W \cdot kh$ (kN)	Location Y (m)
Frame (Rebar)	27.685 × 0.18 = 4.983	0.429
Soil (Rear Side)	10.450 × 0.18 = 1.881	0.650
Total	6.684	0.489

2.3 Ground Surface Loading, Snow Load

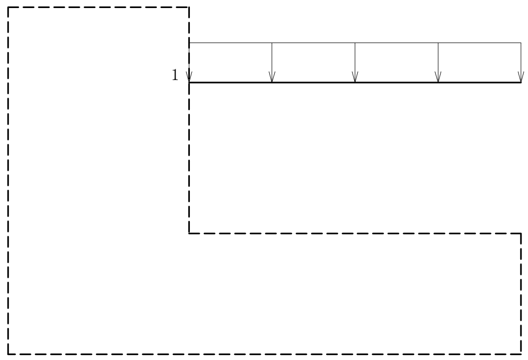
Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

Where,

q : Load
L : Length of load
X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	1.150

2.4 Collision Load

Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 8.500(m)

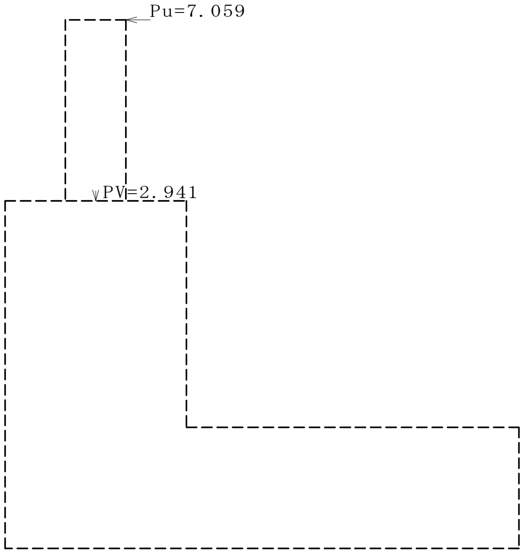
Vertical Force

$$Pv = \frac{pv}{L}$$

Where,

Pv : Front wheel load per unit width (kN/m)
pv : Front wheel load per block (kN)
L : Block length (m), L = 8.500(m)

[1] Collision



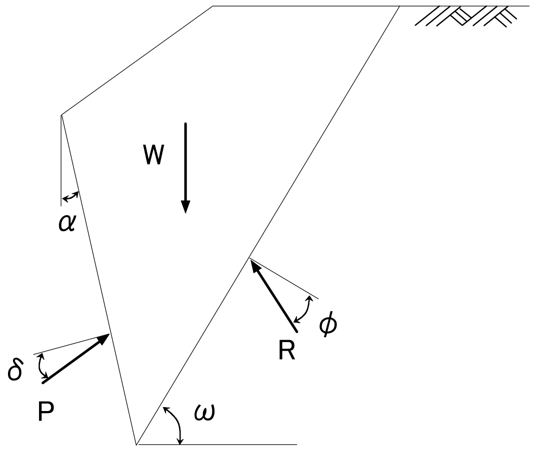
Horizontal Force

p (kN)	Pu (kN/m)	Location Y (m)
60.000	7.059	1.750

Vertical Force

pv (kN)	Pv (kN/m)	Location X (m)
25.000	2.941	0.300

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.700 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.900 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	4.624	0.000	6.273	10.897	8.627
60.00	4.443	0.000	6.028	10.471	8.635
61.00	4.266	0.000	5.787	10.053	6.040

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 6.045 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{10.471 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

$= 6.045$ kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 6.045 \times \cos(0.000° + 0.000°) = 6.045$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 6.045 \times \sin(0.000° + 0.000°) = 0.000$ kN

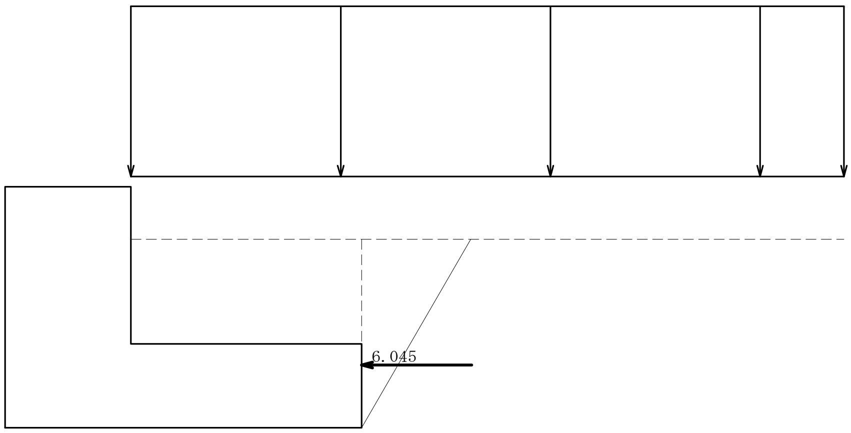
Location

$$Ho = \frac{H}{3} = \frac{0.900}{3} = 0.300 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.300 \times \tan 0.000° = 1.700$ m

$y = yp + Ho = 0.000 + 0.300 = 0.300$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	$xp = 1.700$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 0.900$ m
Angle to the Vertical of Earth Pressure Surface	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Angle of Ground Surface from the Horizontal	$\beta = 0.000$ °
Composite angle during earthquake	$\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1-\sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1-\sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle $\omega i =$ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	8.252	0.000	0.000	8.252	3.368
44.00	7.969	0.000	0.000	7.969	3.373
45.00	7.695	0.000	0.000	7.695	3.373

Earth pressure becomes maximum when:
$\omega = 44.00°$ $P = 3.373$ kN

Earth Pressure, P

$$P = \frac{W / \cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.969 / \cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$= 3.373$ kN

Earth Pressure's location and horizontal and vertical components:

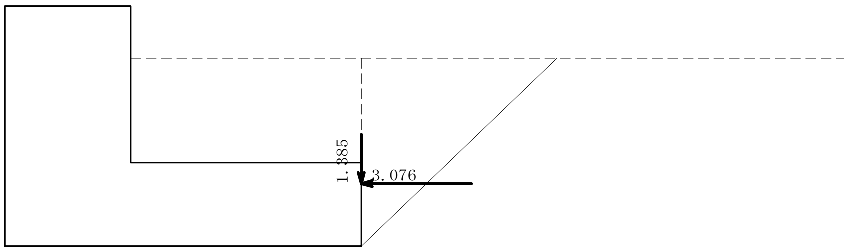
Horizontal Component
$Ph = P \cdot \cos(\alpha+\delta) = 3.373 \times \cos(0.000° + 24.239°) = 3.076$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha+\delta) = 3.373 \times \sin(0.000° + 24.239°) = 1.385$ kN

Location

$$Ho = \frac{H}{3} = \frac{0.900}{3} = 0.300 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.300 \times \tan 0.000° = 1.700$ m
$y = yp + Ho = 0.000 + 0.300 = 0.300$ m

Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe) xp = 1.700 m

yp = 0.000 m

Height of Assumed Rear Surface H = 0.900 m

Angle of Earth Pressure Acting Surface to the Vertical α = 0.000 °

Unit Weight of Soil at Rear Side $\gamma s = 19.000\ kN/m^3$

Angle of Internal Friction of Soil at Rear Side φ = 30.000 °

Angle of Ground Surface from the Horizontal β = 0.000 °

Wall Friction Angle δ = β = 0.000 °

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw = 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	4.624	0.000	0.000	4.624	2.563
60.00	4.443	0.000	0.000	4.443	2.565
61.00	4.266	0.000	0.000	4.266	2.563

Earth pressure becomes maximum when:

ω = 60.00° P = 2.565 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{4.443 \times \sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

= 2.565 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 2.565 \times \cos(0.000° + 0.000°) = 2.565$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 2.565 \times \sin(0.000° + 0.000°) = 0.000$ kN

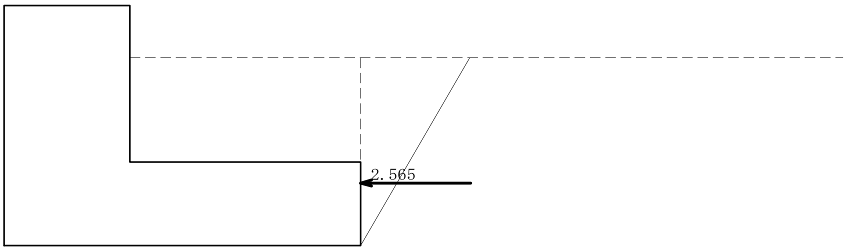
Location

$$Ho = \frac{H}{3} = \frac{0.900}{3} = 0.300 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.300 \times \tan 0.000° = 1.700$ m

$y = yp + Ho = 0.000 + 0.300 = 0.300$ m

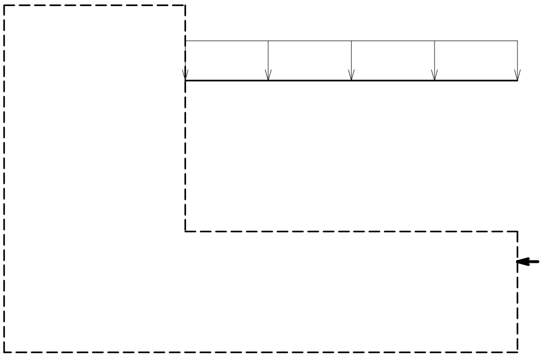
- Earth Pressure Diagram



2.6 Total Acting Force

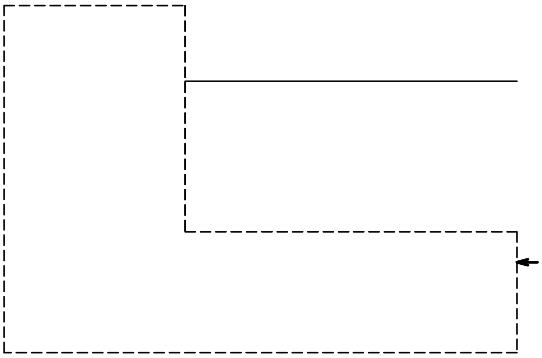
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



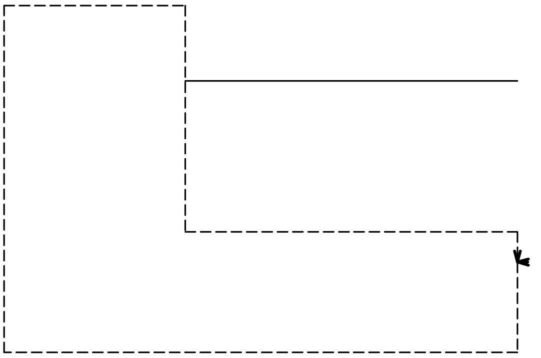
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	38.135	0.000	0.773	0.000	29.486	0.000
Loading, Snow	12.760	0.000	1.150	0.000	14.674	0.000
Earth Pressure	0.000	6.045	1.700	0.300	0.000	1.814
Total	50.895	6.045	―――	―――	44.160	1.814

[2] Ordinary Condition (Rear Side)



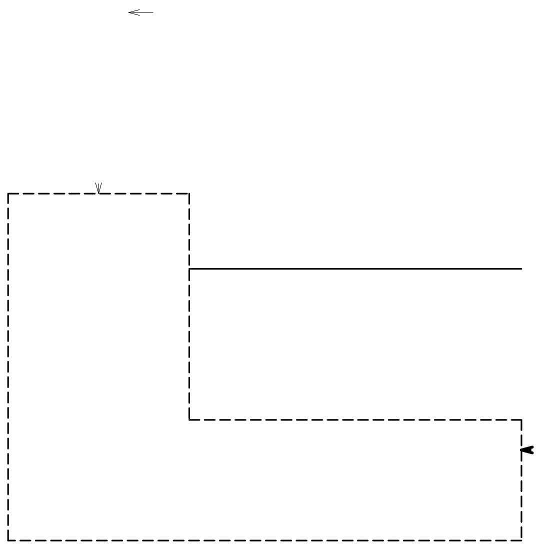
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	38.135	0.000	0.773	0.000	29.486	0.000
Earth Pressure	0.000	6.045	1.700	0.300	0.000	1.814
Total	38.135	6.045	———	———	29.486	1.814

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	38.135	6.864	0.773	0.489	29.486	3.360
Earth Pressure	1.385	3.076	1.700	0.300	2.355	0.923
Total	39.520	9.940	———	———	31.840	4.283

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	38.135	0.000	0.773	0.000	29.486	0.000
Loading, Snow	0.000	2.565	1.700	0.300	0.000	0.770
Collision Load	2.941	7.059	0.300	1.750	0.882	12.353
Total	41.076	9.624	―――	―――	30.368	13.122

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	50.895	6.045	42.347
Ordinary (Rear Side)	38.135	6.045	27.673
Earthquake	39.520	9.940	27.557
Collision	41.076	9.624	17.246

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,
Footing Earth Pressure Width : $B_j = 1.700$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	50.895	6.045	0.914
Ordinary (Rear Side)	38.135	6.045	4.742
Earthquake	39.520	9.940	6.035
Collision	41.076	9.624	17.669

■ Full Width per 8.500m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	432.607	51.383	7.771
Ordinary (Rear Side)	324.147	51.383	40.309
Earthquake	335.920	84.493	51.295
Collision	349.147	81.802	150.185

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B = 1.500

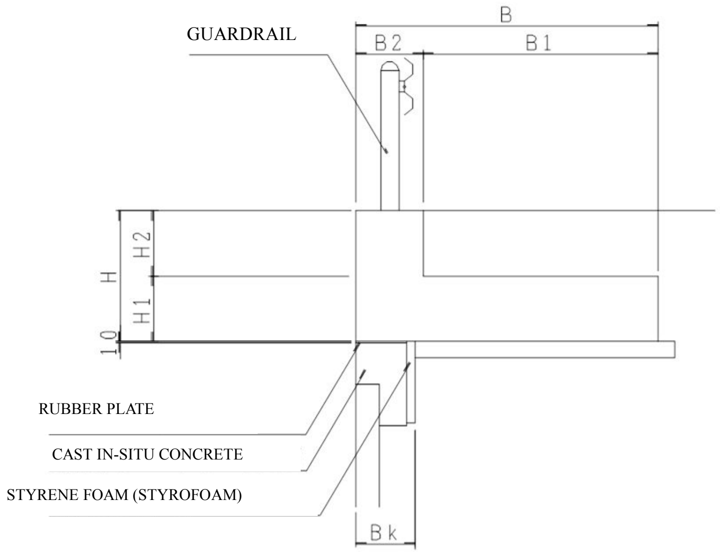
$$e_a = B／n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	44.160	1.814	50.895	0.832	0.018	≦	0.283	○
Ordinary (Rear Side)	29.486	1.814	38.135	0.726	0.124	≦	0.283	○
Earthquake	31.840	4.283	39.520	0.697	0.153	≦	0.567	○
Collision	30.368	13.122	41.076	0.420	0.430	≦	0.567	○

2.7.2 Stability from Sliding



H = 0.900 m
H1 = 0.400 m
H2 = 0.500 m

B = 1.700 m
B1 = 1.100 m
B2 = 0.600 m

Bk = 0.350 m

Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m^2)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.124	1.700	12.587	32.278	32.278	28.224
Earthquake	0.153	1.700	10.718	35.776	35.776	30.617
Collision	0.430	1.260	0.000	65.200	65.200	47.089

Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Nna (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN'=ΣN-Nna	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	38.135	10.588	27.547	6.045
Earthquake	39.520	11.619	27.901	9.940
Collision	41.076	19.651	21.425	9.624

Factor of safety for sliding, Fs:

$$Fs = \Sigma N' \cdot \mu / \Sigma H$$

Load Condition	Factor of Safety Fs	Required Factor of Safety Fsa
Ordinary (Rear Side)	2.734	≧1.5
Earthquake	1.684	≧1.2
Collision	1.336	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

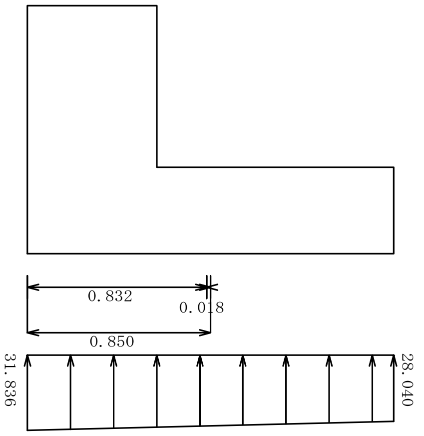
Where,

ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)

B : Bottom plate width (m), B = 1.00

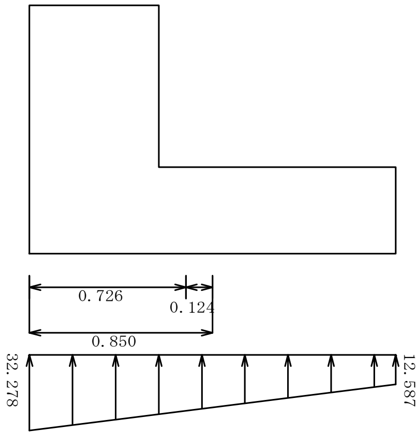
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



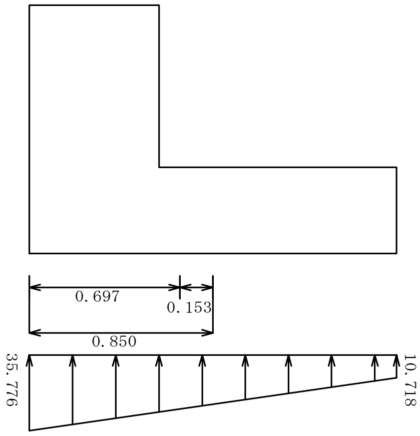
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.700	Trapezoid	28.040	31.836 ≦	200.00	○

[2] Ordinary Condition (Rear Side)



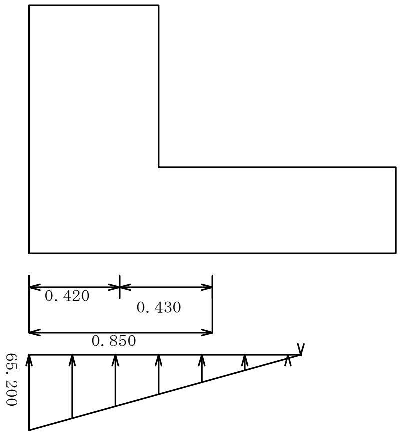
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.700	Trapezoid	12.587	32.278	≦	200.000	○

[3] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.700	Trapezoid	10.718	35.776	≦	300.000	○

[4] Collision



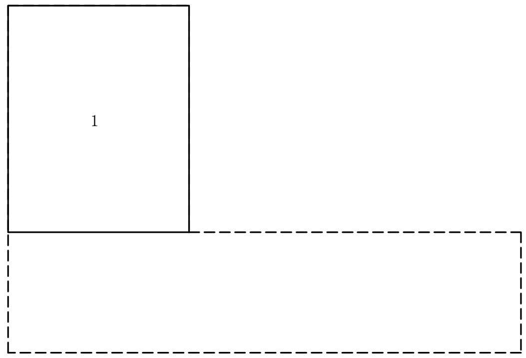
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.260	Triangle	0.000	65.200	≦	300.000	○

3 **Design of Vertical Wall**

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.600× 0.750× 1.000	0.450	0.300	0.375	0.135	0.169	
Σ		0.450	—	—	0.135	0.169	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.135／ 0.450 ＝ 0.300 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.169／ 0.450 ＝ 0.375 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.450 ＝ 11.025	0.000

Force Location

X ＝ Xc－XG ＝ 0.300－0.300

＝ 0.000 m

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.450 ＝ 11.025	0.000

Location	H = W · kh (kN)	Location Y (m)
Frame(Rebar)	11.025 × 0.180 = 1.985	0.375

Force Location

$$X = Xc - XG = 0.300 - 0.300$$
$$= 0.000 \text{ m}$$

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

Horizontal Force

$$Pu = \frac{p}{\lambda + y}$$

Where,

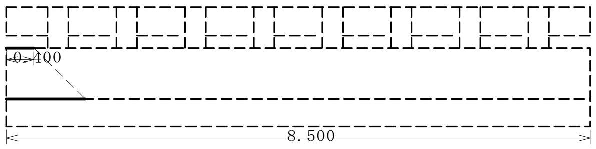
Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
y : Distance from the tip of section being checked (m), y = 0.750(m)

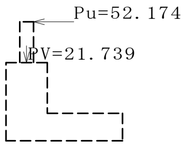
Vertical Force $Pv = \frac{pv}{\lambda + y}$

Where,

Pv : Front wheel load of the effective width (kN/m)
pv : Front wheel load per block (kN)

[1] Collision





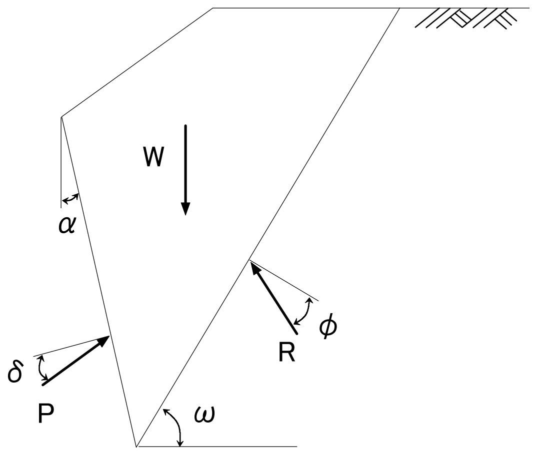
Horizontal Force

p (kN)	λ (m)	Pu (kN/m)	Location Y (m)
60.000	0.400	52.174	1.350

Vertical Force

pv (kN)	λ or L (m)	Pu (kN/m)	Location X (m)
25.000	0.400	21.739	0.000

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.300 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.500 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	1.663	0.000	4.061	5.724	2.428
56.00	1.602	0.000	3.912	5.514	2.430
57.00	1.542	0.000	3.767	5.309	2.428

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 2.430 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.514 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

＝ 2.430 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

Ph ＝ P・cos(α＋δ) ＝ 2.430×cos(0.000°＋20.000°) ＝ 2.283 kN

Vertical Component

Pv ＝ P・sin(α＋δ) ＝ 2.430×sin(0.000°＋20.000°) ＝ 0.831 kN

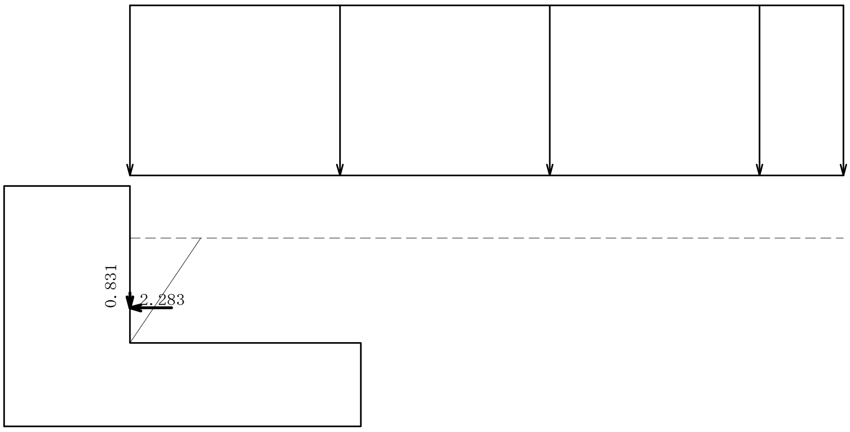
Location

$$Ho = \frac{H}{3} = \frac{0.500}{3} = 0.167 \text{ m}$$

x ＝ Ho・tanα－xp ＝ 0.167×tan0.000°－0.300 ＝ -0.300 m

y ＝ yp＋Ho ＝ 0.000＋0.167 ＝ 0.167 m

- Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) xp ＝ 0.300 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 0.500 m
Angle of Earth Pressure Acting Surface to the Vertical α ＝ 0.000 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	1.663	0.000	0.000	1.663	0.705
56.00	1.602	0.000	0.000	1.602	0.706
57.00	1.542	0.000	0.000	1.542	0.705

Earth pressure becomes maximum when:
ω = 56.00° P = 0.706 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{1.602 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

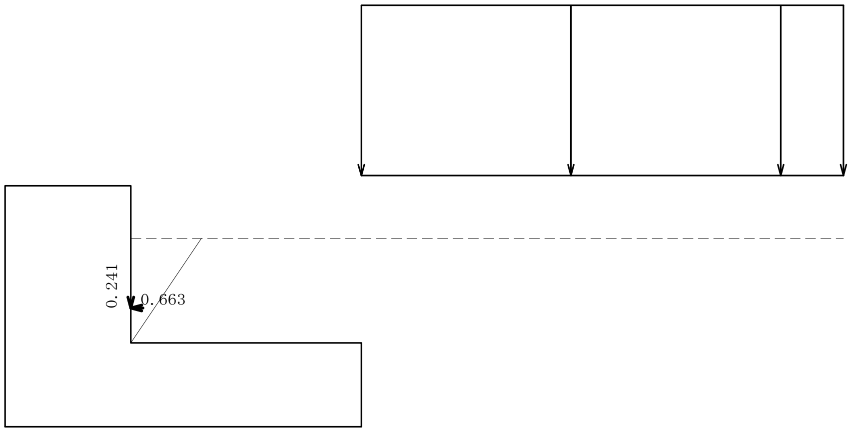
$= 0.706$ kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 0.706 \times \cos(0.000° + 20.000°) = 0.663$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 0.706 \times \sin(0.000° + 20.000°) = 0.241$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.500}{3} = 0.167 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.167 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.167 = 0.167$ m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) xp = 0.300 m
yp = 0.000 m
Height of Assumed Rear Surface H = 0.500 m
Angle to the Vertical of Earth Pressure Surface α = 0.000 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °
Wall Friction Angle δ = 1／2φ = 15.000 °
Composite angle during earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$
Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
45.00	2.293	0.000	0.000	2.293	1.029
46.00	2.215	0.000	0.000	2.215	1.030
47.00	2.138	0.000	0.000	2.138	1.028

Earth pressure becomes maximum when:
ω ＝ 47.00° P ＝ 1.030 kN

Earth Pressure

$$P = \frac{W／\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{2.215／\cos 10.204° \times \sin(47.00° - 30.00° + 10.204°)}{\cos(47.00° - 30.00° - 0.000° - 15.000°)}$$

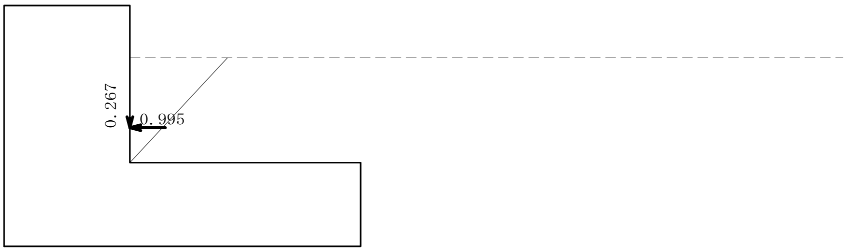
$$= 1.030 \text{ kN}$$

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 1.030 \times \cos(0.000° + 15.000°) = 0.995$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 1.030 \times \sin(0.000° + 15.000°) = 0.267$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.500}{3} = 0.167 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.167 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.167 = 0.167$ m

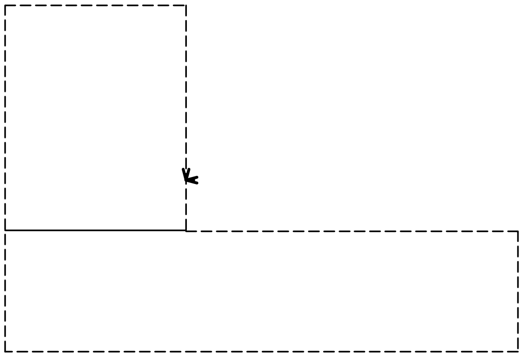
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

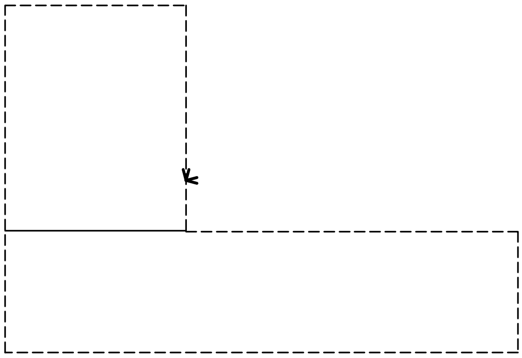
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	11.025	0.000	0.000	0.000	0.000
Earth Pressure	0.831	2.283	-0.300	0.167	0.381
Total	0.000	2.283	———	———	0.381

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

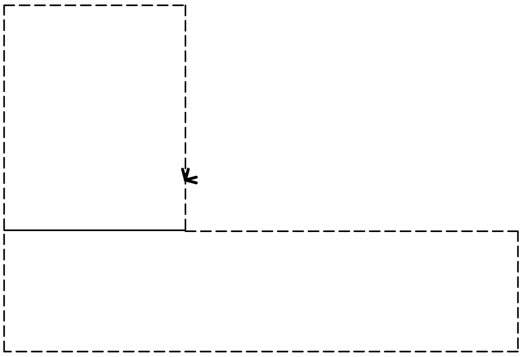
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	11.025	0.000	0.000	0.000	0.000
Earth Pressure	0.241	0.663	-0.300	0.167	0.111
Total	0.000	0.663	———	———	0.111

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

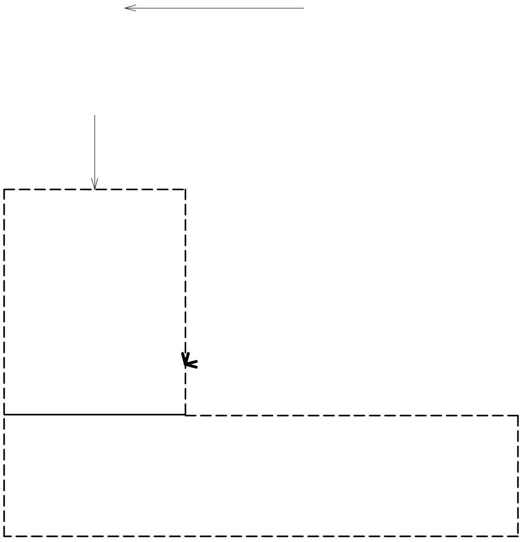
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M $=M_{xi}+M_{yi}$ (kN.m)
Self-weight	11.025	1.985	0.000	0.375	0.744
Earth Pressure	0.267	0.995	-0.300	0.167	0.166
Total	0.000	2.980	———	———	0.910

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

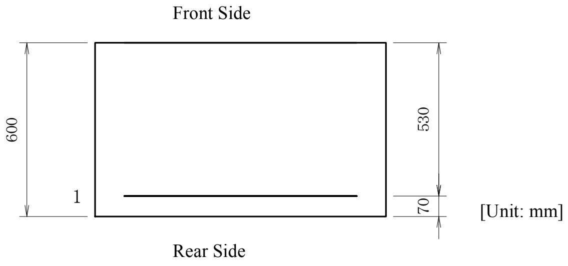


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M $=M_{xi}+M_{yi}$ (kN.m)
Self-weight	11.025	0.000	0.000	0.000	0.000
Earth Pressure	0.241	0.663	-0.300	0.167	0.111
Collision Load	21.739	52.174	0.000	1.350	70.435
Total	0.000	52.837	———	———	70.546

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	――	－	――	――	――
	2'	――	－	――	――	――
Rear Side	1	7.00	D16	1.986	4.000	7.944
	2	――	－	――	――	――

Required rebar amount at tension side: 4.670 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 60000.0 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 600000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), h = 600.000

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	7.944	0.648	≦	114.821	0.000	5.000	○
Ordinary (Rear Side)	7.944	0.188	≦	114.821	0.000	5.000	○
Earthquake	7.944	1.548	≦	114.821	0.000	5.000	○
Collision	7.944	119.927	＞	114.821	0.000	6.335	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2 $(0.002 \leqq \varepsilon(y) \leqq \varepsilon cu)$

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars $(d_i \leqq x)$

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars $(d_j > x)$

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 530.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	7.944	≦	169.941	○
Ordinary (Rear Side)	7.944	≦	169.941	○
Earthquake	7.944	≦	169.941	○
Collision	7.944	≦	169.941	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2 \cdot n}{b} \left\{ As \cdot (x - d) \right\} = 0.0$$

よりxを求める。

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2} \cdot \left(\frac{h}{2} - \frac{x}{3} \right) + n \cdot As \cdot \frac{(x - d) \cdot (h/2 - d)}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d - x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h ＝ 600.000
b : Member cross-section width (mm), b ＝ 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n ＝ 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)		Tensile Stress (N/mm^2)		Judgement
				Calculated	Allowable	Calculated	Allowable	
Ordinary (Surcharge)	0.381	0.000	10.107	0.015 ≦	8.000	0.967 ≦	180.000	○
Ordinary (Rear Side)	0.111	0.000	10.107	0.004 ≦	8.000	0.281 ≦	180.000	○
Earthquake	0.910	0.000	10.107	0.036 ≦	12.000	2.309 ≦	300.000	○
Collision	70.546	0.000	10.107	2.811 ≦	12.000	178.943 ≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member’s effective height (mm)

b : Sectional member’s effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member’s effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

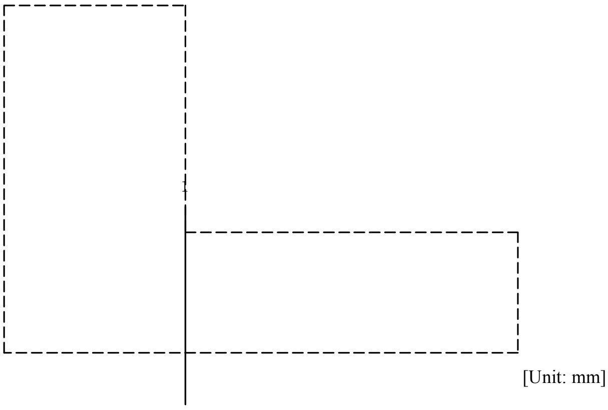
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)			Correction Factor			Judgement
			Calc. τ	Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	2.283	53.000	0.004 ≦	0.233	1.700	1.27	0.80	1.00	○
Ordinary(Rear Side)	0.663	53.000	0.001 ≦	0.233	1.700	1.27	0.80	1.00	○
Earthquake	2.979	53.000	0.006 ≦	0.355	2.550	1.27	0.80	1.00	○
Collision	52.837	53.000	0.100 ≦	0.350	2.550	1.27	0.80	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

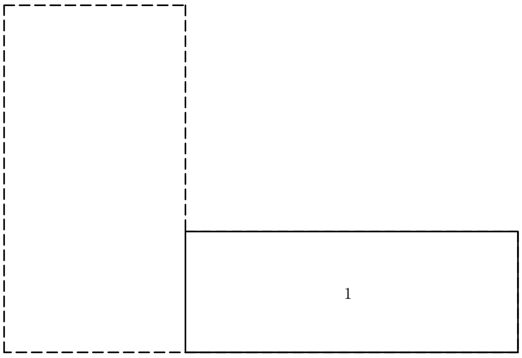
Distance from joint: 0.000 m



4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



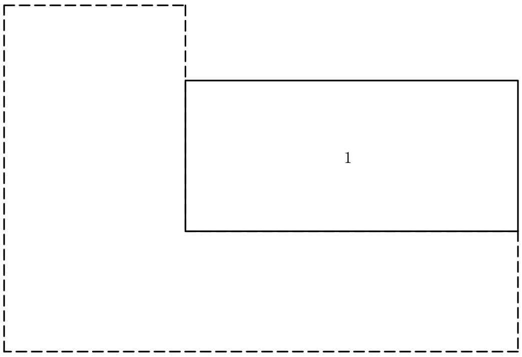
2) Volume・Center of Gravity

	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi(m)	Vi・Xi	Remarks
1	1.100× 0.400× 1.000	0.440	0.550	0.242	
Σ		0.440	———	0.242	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.242／ 0.440 ＝ 0.550 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	1.100× 0.500× 1.000	0.550	0.550	0.303	
Σ		0.550	——	0.303	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.303／0.550 ＝ 0.550 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.440 ＝ 10.780	0.550
Soil(Rear Side)	19.000 × 0.550 ＝ 10.450	0.550

4.1.3 Ground Surface Load・Snow Load

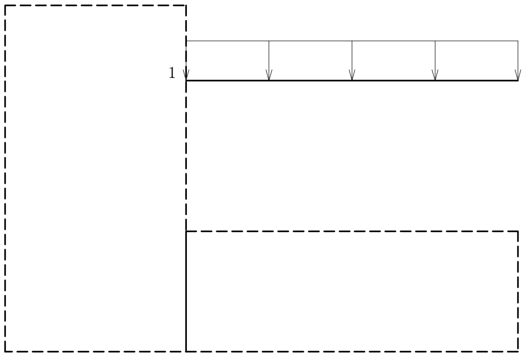
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

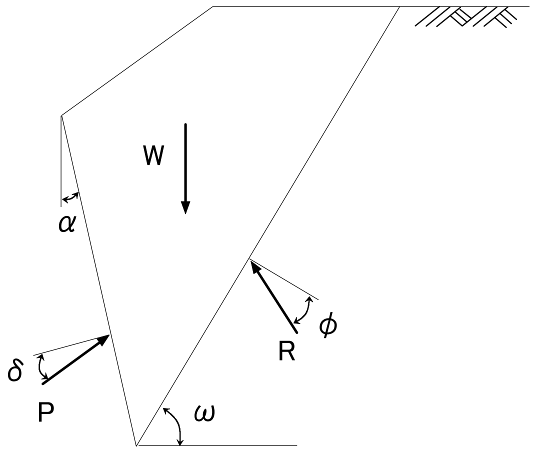
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	0.550

4.1.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) xp = 1.700 m
yp = 0.000 m
Height of Assumed Rear Surface H = 0.900 m
Angle to the vertical of the Earth Pressure Surface α = 0.000 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °
Angle to the Horizontal of the Ground Surface β = 0.000 °
Composite Angle during Earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	8.252	0.000	0.000	8.252	3.368
44.00	7.969	0.000	0.000	7.969	3.373
45.00	7.695	0.000	0.000	7.695	3.373

Earth pressure becomes maximum when:
Wen ω ＝ 44.00°, P ＝ 5.508 kN

Earth Pressure

$$P = \frac{W / \cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.969 / \cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

＝ 3.373 kN

Vertical component of Earth Pressure:

Vertical Component

$$P_v = P \cdot \sin(\alpha + \delta) = 3.373 \times \sin(0.000° + 24.239°) = 1.385 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 1.385}{1.100} = 2.518 \text{ kN/m}$$

Where,

pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.000 m

Width of distributed load perpendicular to the design section L2 = 1.100 m

Distributed Load at the Design Section, pd $pd = \frac{pv}{L} \cdot L1 = \frac{2.518}{1.100} \times 0.000 = 0.000$ kN/m

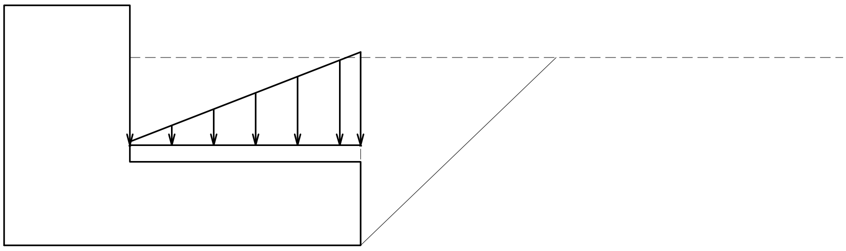
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (0.000 + 2.518) \times 1.100 = 1.385 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{0.000 + 2 \times 2.518}{0.000 + 2.518} \times \frac{1.100}{3} = 0.733 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

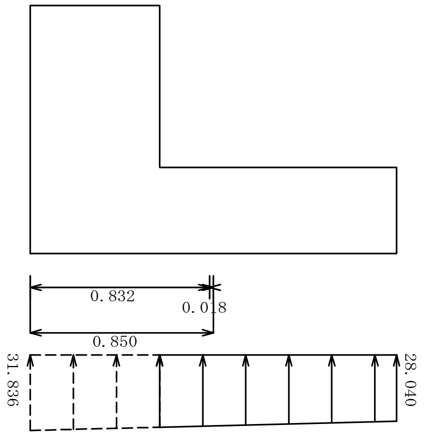
Where,

q1 : Ground reaction pressure at the front surface of heel (kN/m^2)

q2 : Ground reaction pressure at the design section of heel (kN/m^2)

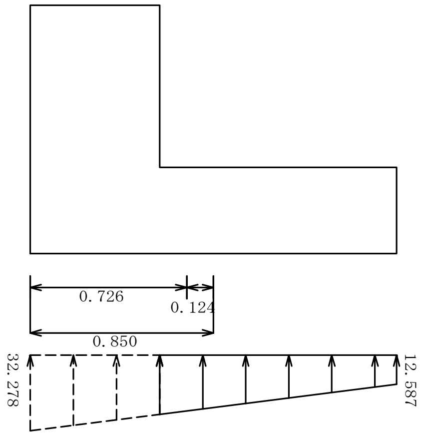
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



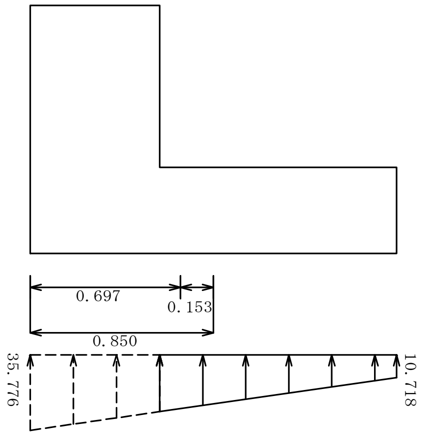
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
28.040	30.496	1.100	32.195	0.542

[2] Ordinary Condition (Rear Side)



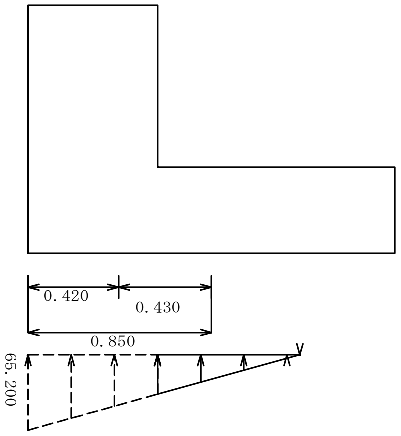
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
12.587	25.328	1.100	20.853	0.488

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
10.718	26.932	1.100	20.708	0.471

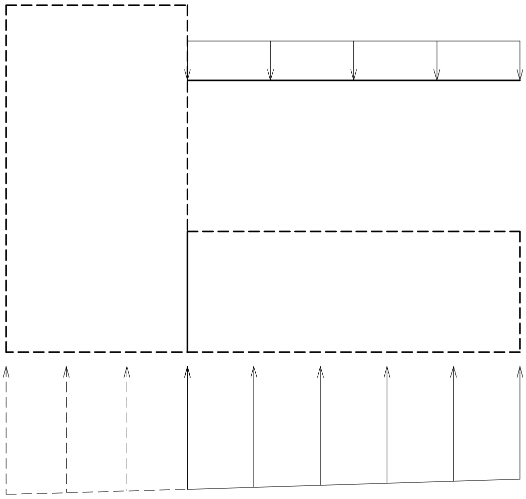
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	34.152	0.660	11.270	0.220

4.1.6 Total Sectional Forces

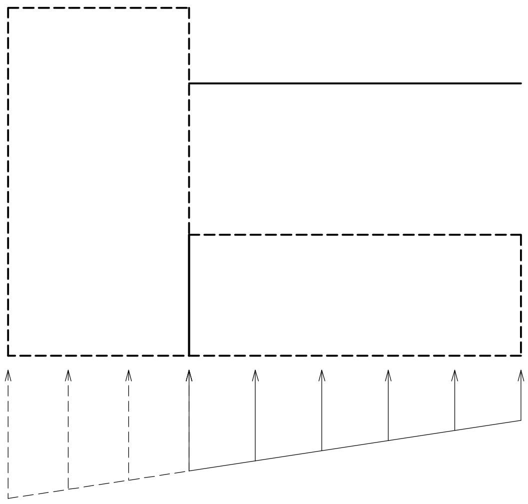
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	21.230	0.550	11.677
Loading, Snow Load	12.760	0.550	7.018
Ground Reaction	-32.195	0.542	-17.460
Total	1.795	――	1.235

Sectional force of vertical wall foundation, M1 ＝ 0.381 kN.m
Sectional force of the base of heel, M3 ＝ 1.235 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

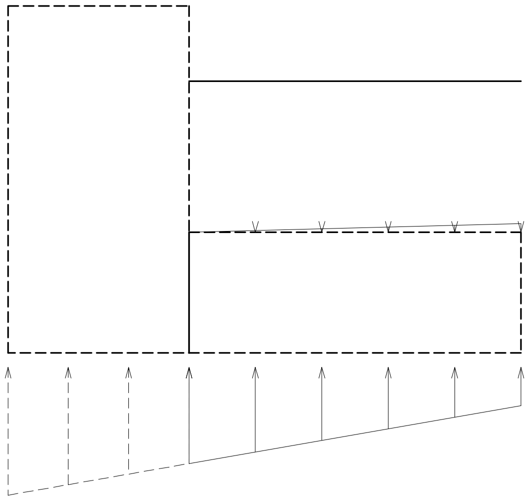
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	21.230	0.550	11.677
Loading, Snow Load	-20.853	0.488	-10.185
Ground Reaction	0.377	———	1.492
Total	21.230	0.550	11.677

Sectional force of vertical wall foundation, M1 = 0.111 kN.m
Sectional force of the base of heel, M3 = 1.492 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

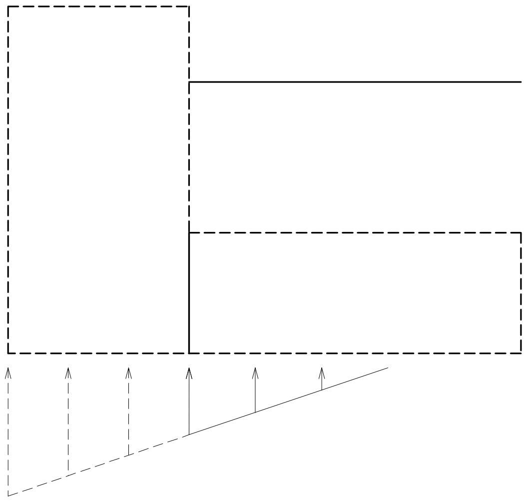
[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	21.230	0.550	11.677
Loading, Snow Load	1.385	0.733	1.016
Ground Reaction	-20.708	0.471	-9.754
Total	1.907	———	2.938

Sectional force of vertical wall foundation, M1 = 0.910 kN.m
Sectional force of the base of heel, M3 = 2.938 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

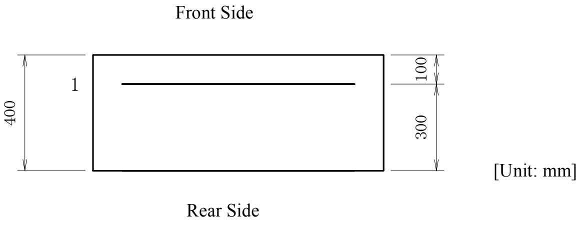
[4] Collision



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	21.230	0.550	11.677
Ground Reaction	-11.270	0.220	-2.479
Total	9.960	———	9.197

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.00	D13	1.267	4.000	5.068
	2	—	–	——	—	——
Bottom Surface	1'	—	–	——	—	——
	2'	—	–	——	—	——

Required tension side rebar amount 1.057 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), h = 400.000

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m) ×1.7 M_c (kN.m)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	0.648 ≦ 51.031	5.000	○
Ordinary (Rear Side)	5.068	0.188 ≦ 51.031	5.000	○
Earthquake	5.068	1.548 ≦ 51.031	5.000	○
Collision	5.068	15.635 ≦ 51.031	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars（$d_i \leqq x$）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars（$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), $Es = 200000$
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

よりxを求める。

Stress Calculation

$$\sigma_c=\frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s=n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h ＝ 400.000

b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^2)

n : Rebar and concrete’s coefficient of Young’s modulus, n ＝ 15.00

e : Distance from the sectional member’s centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^2)

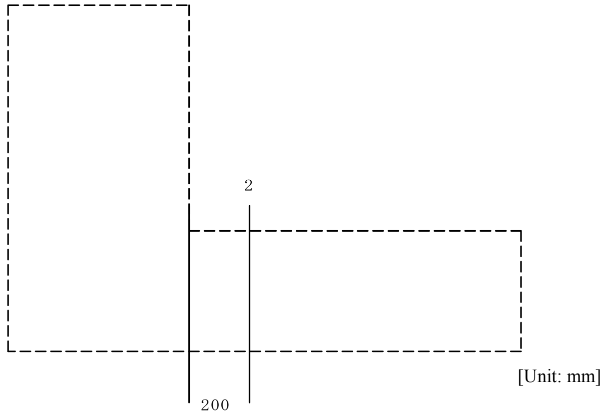
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.381	6.035	0.045	≦	8.000	2.688	≦	180.000	○
Ordinary (Rear Side)	0.111	6.035	0.013	≦	8.000	0.781	≦	180.000	○
Earthquake	0.910	6.035	0.108	≦	12.000	6.418	≦	300.000	○
Collision	9.197	6.035	1.089	≦	12.000	64.842	≦	300.000	○

4.2 Design of Section [2]

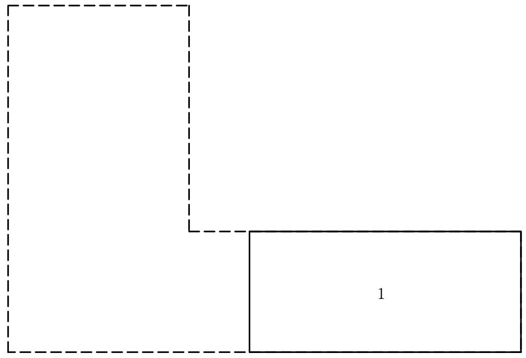
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



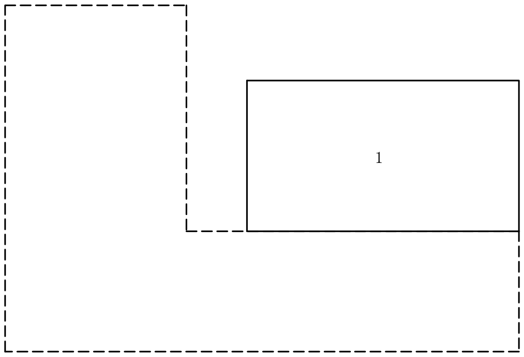
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.900× 0.400× 1.000	0.360	0.450	0.162	
Σ		0.360	——	0.162	

Center of Gravity　XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.162／ 0.360 ＝ 0.450 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.900× 0.500× 1.000	0.450	0.450	0.202	
Σ		0.450	——	0.202	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.202／ 0.450 ＝ 0.450 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.360 ＝ 8.820	0.450
Soil(Rear Side)	19.000 × 0.450 ＝ 8.550	0.450

4.2.3 Ground Surface Load・Snow Load

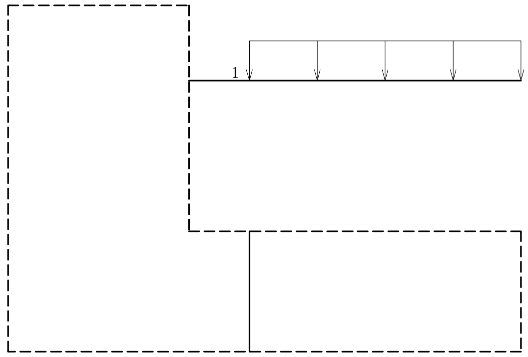
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

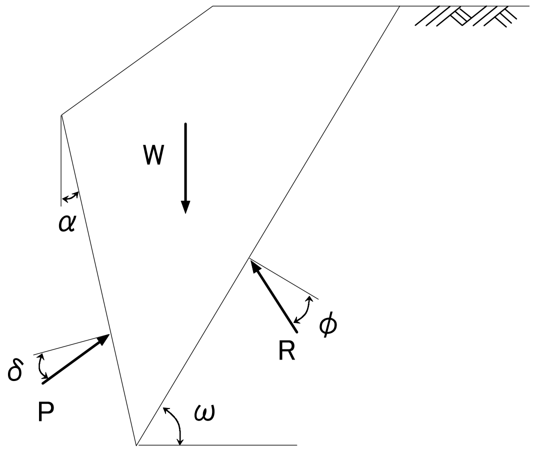
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.900	10.440	0.450

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) xp ＝ 1.700 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 0.900 m
Angle to the vertical of the Earth Pressure Surface α ＝ 0.000 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Angle to the Horizontal of the Ground Surface β ＝ 0.000 °
Composite Angle during Earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	8.252	0.000	0.000	8.252	3.368
44.00	7.969	0.000	0.000	7.969	3.373
45.00	7.695	0.000	0.000	7.695	3.373

Earth pressure becomes maximum when:
$\omega = 44.00°$, $P = 3.373$ kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.969/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$$= 3.373 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component
$$Pv = P \cdot \sin(\alpha + \delta) = 3.373 \times \sin(0.000° + 24.239°) = 1.385 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2 \cdot Pv}{L} = \frac{2 \times 1.385}{1.100} = 2.518 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.200 m

Width of distributed load perpendicular to the design section L2 = 0.700 m

Distributed Load at the Design $pd = \frac{pv}{L} \cdot L1 = \frac{2.518}{1.100} \times 0.200 = 0.458$ kN/m

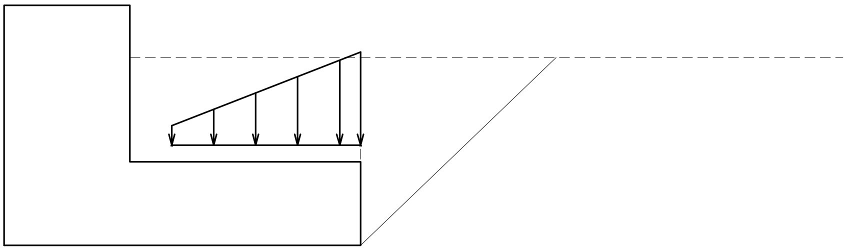
Vertical Force

$$N = \frac{1}{2} \cdot (pd+pv) \cdot L2 = \frac{1}{2} \times (0.458+2.518) \times 0.900 = 1.339 \text{ kN}$$

Location

$$x = \frac{pd+2 \cdot pv}{pd+pv} \cdot \frac{L2}{3} = \frac{0.458+2\times2.518}{0.458+2.518} \times \frac{0.900}{3} = 0.554 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2) \cdot L$$

Location

$$X = \frac{2 \cdot q1+q2}{3 \cdot (q1+q2)} \cdot L$$

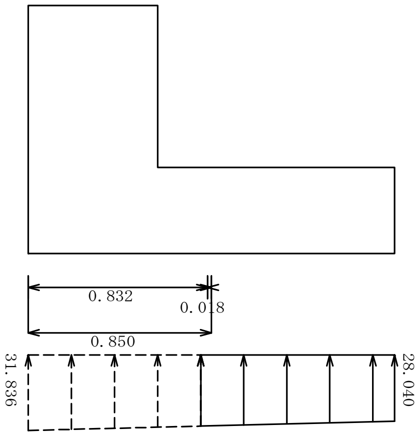
Where,

q1 : Ground reaction pressure at the front surface of heel (kN/m^2)

q2 : Ground reaction pressure at the design section of heel (kN/m^2)

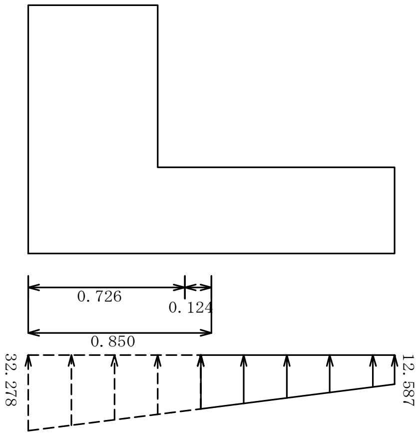
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



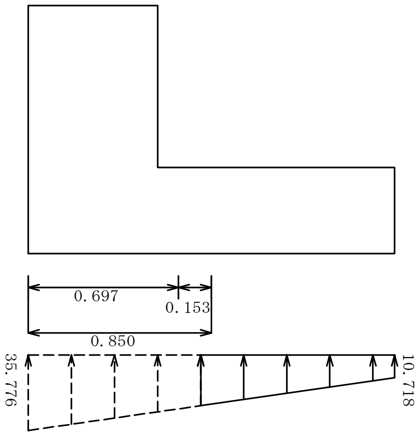
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
28.040	30.050	0.900	26.140	0.445

[2] Ordinary Condition (Rear Side)



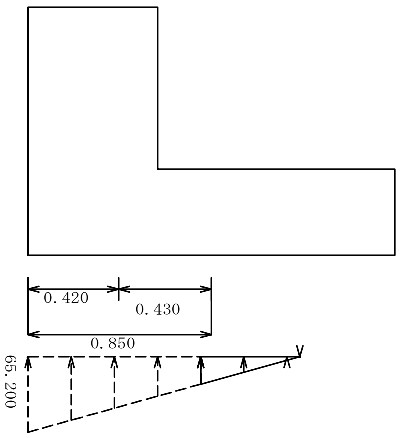
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
12.587	23.012	0.900	16.019	0.406

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
10.718	23.984	0.900	15.616	0.393

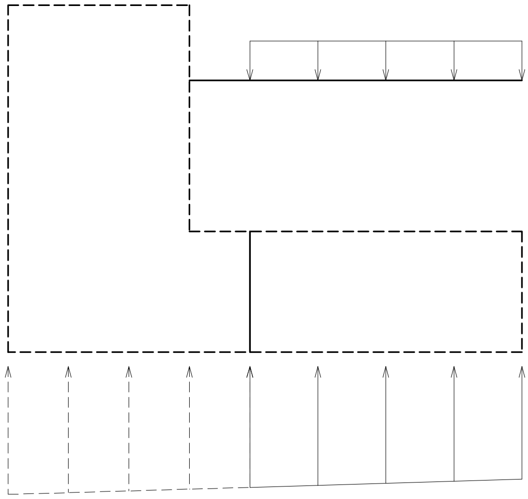
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	23.803	0.460	5.475	0.153

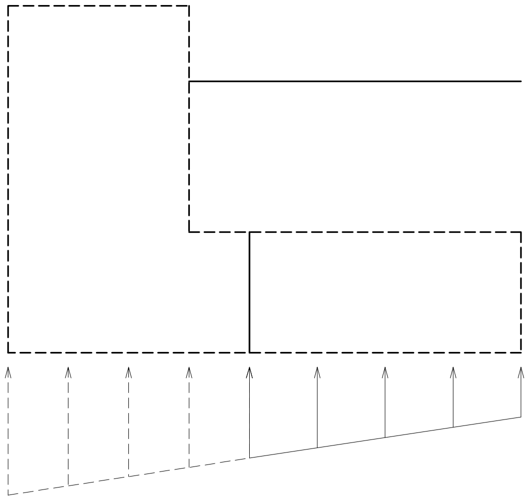
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



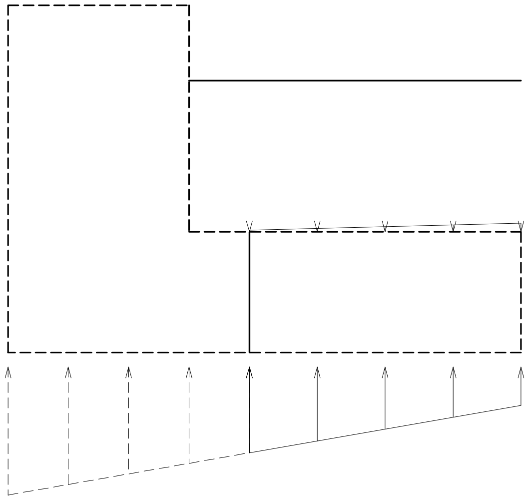
Item	N_i (kN)	X_i (m)	$M =N_i \cdot X_i$ (kN.m)
Self-weight	17.370	0.450	7.817
Loading, Snow Load	10.440	0.450	4.698
Ground Reaction	-26.140	0.445	-11.628
Total	1.670	———	0.887

[2] Ordinary Condition (Rear Side)



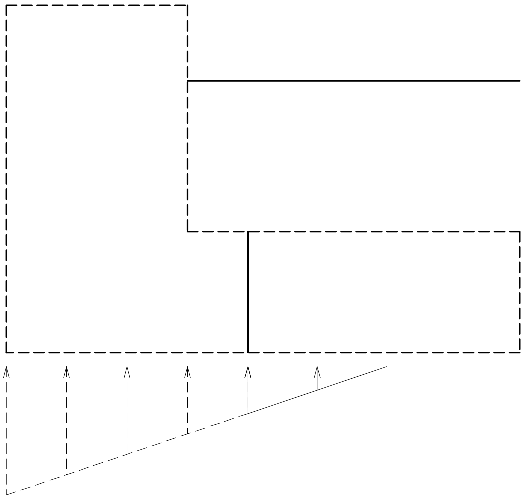
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	17.370	0.450	7.817
Ground Reaction	-16.019	0.406	-6.505
Total	1.351	———	1.311

[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	17.370	0.450	7.817
Loading, Snow Load	1.339	0.554	0.742
Ground Reaction	-15.616	0.393	-6.132
Total	3.093	———	2.427

[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	17.370	0.450	7.817
Ground Reaction	-5.475	0.153	-0.839
Total	11.895	———	6.977

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ $a > 2.5d$,

when $a \leqq 2.5d$, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
M : Bending moment acting on the sectional member (N.mm)
θ : Angle of the top surface footing to the horizontal, $\theta = 0.000$, $\tan\theta = 0.000$
a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span 2.5・d (cm)		a	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000	≦	98.800	1.670	0.887	0.000	1.670
Ordinary (Rear Side)	30.000	75.000	≦	140.000	1.351	1.311	0.000	1.351
Earthquake	30.000	75.000	≦	140.000	3.093	2.427	0.000	3.093
Collision	30.000	75.000	≦	122.340	11.895	6.977	0.000	11.895

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm : Average shear stress of concrete (N/mm2)

Sh：Actual Shear Force (N)
d：Effective height of footing calculated from the column, front wall or rear wall (mm)
d'：Effective height of sectional member (mm)
b：Effective width of sectional member (mm)
S：Shear force acting on the sectional member (N)
τa1：Allowable shear stress (N/mm2)

$$\tau a1 = Ce \cdot Cpt \cdot Cdc \cdot \tau a1'$$

Where,

τa1'：Allowable shear stress when shear force acts in concrete (N/mm2)
Ce　：Correction factor for the sectional member’s effective height
Cpt：Correction factor for the tensile main reinforcement ratio, Pt
Cdc：Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)			Correction Factor			Judgement
			Calc. τ	Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	1.670	30.000	0.006 ≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	1.351	30.000	0.005 ≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	3.093	30.000	0.010 ≦	0.411	2.550	1.40	0.84	1.00	○
Collision	11.895	30.000	0.040 ≦	0.405	2.550	1.40	0.84	1.00	○

Thanlyin Ramp

Concrete Barrier Type-2

1 Design Conditons

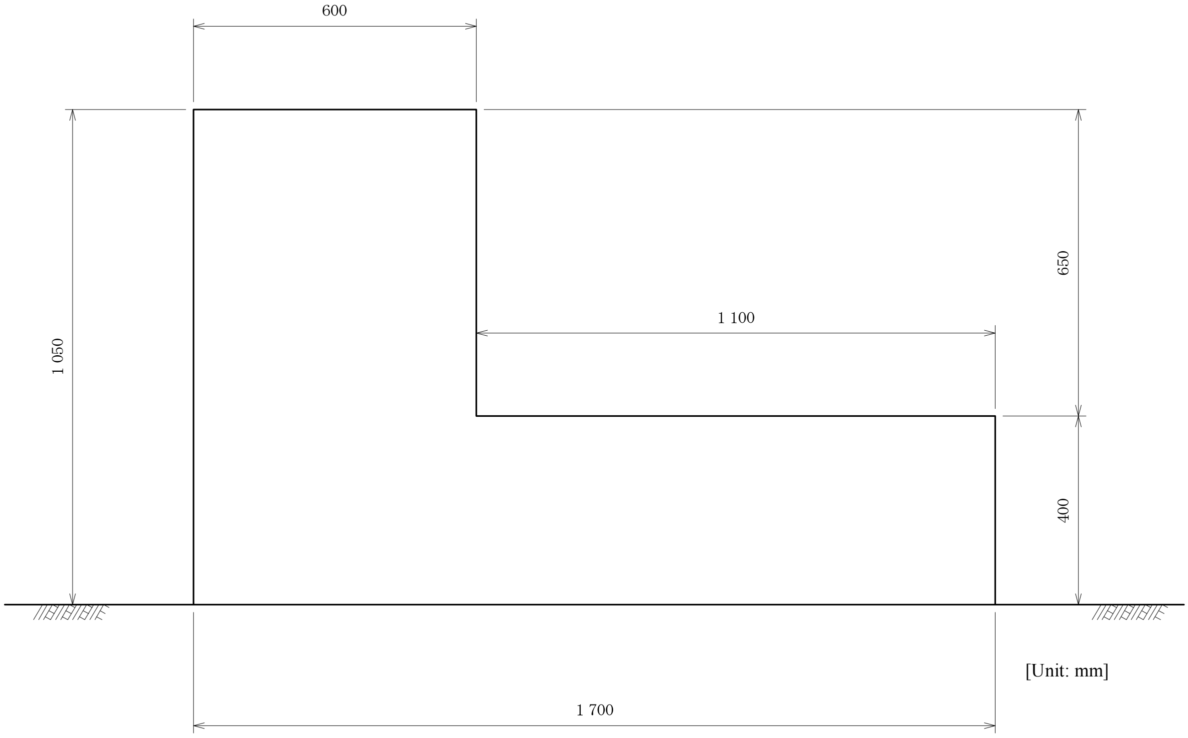
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 10000(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)
RC Bottom Slab: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】Soil at Rear Side: 30°

【Unit Weight】

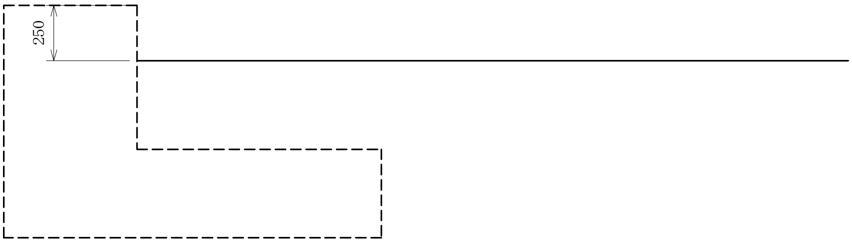
(kN/m^3)

Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】
Frame: Kh ＝ 0.18
Soil (Front side): Kh ＝ 0.18
Soil (Rear side): Kh ＝ 0.18

1.6 Soil

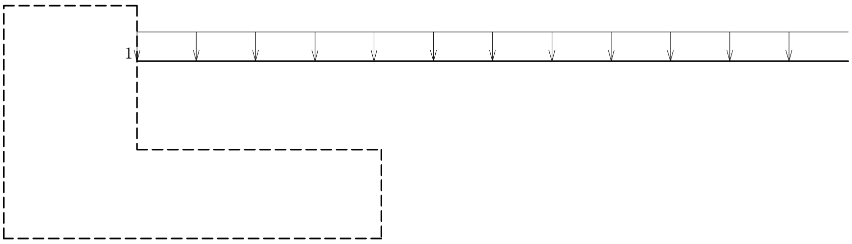
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr. In the stability Analysis (m)	0.000
In the vertical wall design(m)	0.000

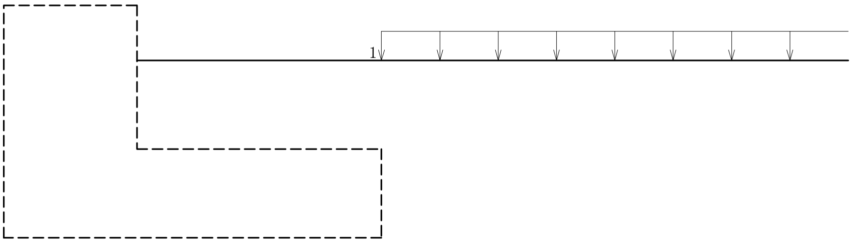
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

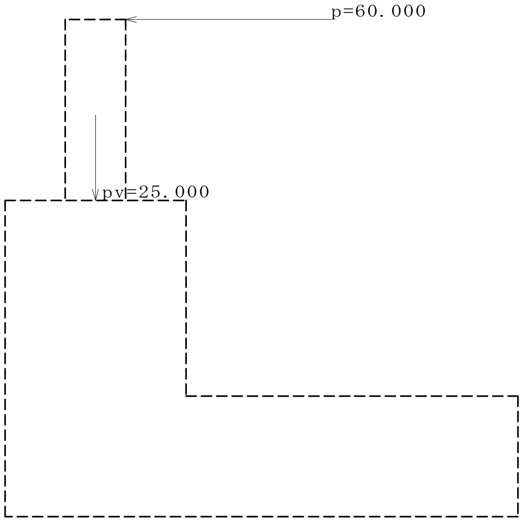


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	1.100	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.600	0.400	60.000

Vertical Force

Effective Width	Loading location x (m)	Front Wheel Load P (kN)
Same as horizontal force	0.300	25.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method
- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

・In the stability analysis the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).

・In the stability analysis, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

・In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

・Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	———
Earthquake	0.000	0.000	———

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	——	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	——	——	——	——
	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, $\tan\varphi_B$	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B ／ V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress	
				τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

2) Bottom Slab (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

τ_{a1} : Shear stress when shear force is bored by concrete

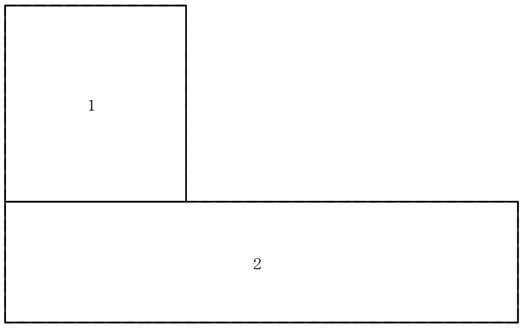
τ_{a2} : Shear stress when shared with diagonal tension bars

2 Stability Analysis

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



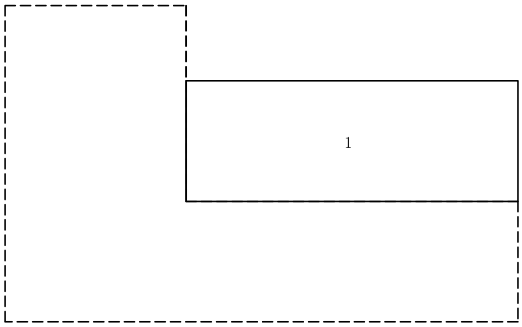
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.600× 0.650× 1.000 1.700× 0.400× 1.000	0.390 0.680	0.300 0.850	0.725 0.200	0.117 0.578	0.283 0.136	
Σ		1.070	——	——	0.695	0.419	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.695／ 1.070 = 0.650 (m)
YG = Σ（Vi・Yi）／ΣVi = 0.419／ 1.070 = 0.391 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	1.100× 0.400× 1.000	0.440	1.150	0.600	0.506	0.264	
Σ		0.440	——	——	0.506	0.264	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.506／ 0.440 = 1.150 (m)
YG = Σ（Vi・Yi）／ΣVi = 0.264／ 0.440 = 0.600 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force W = γ ・ V (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.070 = 26.215	0.650
Soil (Rear Side)	19.000 × 0.440 = 8.360	1.150
Total	34.575	0.771

[2] Earthquake

Location	Vertical Force W = γ ・ V (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.070 = 26.215	0.650
Soil (Rear Side)	19.000 × 0.440 = 8.360	1.150
Total	34.575	0.771

Location	Horizontal Force H = W ・ kh (kN)	Location Y (m)
Frame (Rebar)	26.215 × 0.18 = 4.719	0.391
Soil (Rear Side)	8.360 × 0.18 = 1.505	0.600
Total	6.224	0.442

2.3 Ground Surface Loading, Snow Load

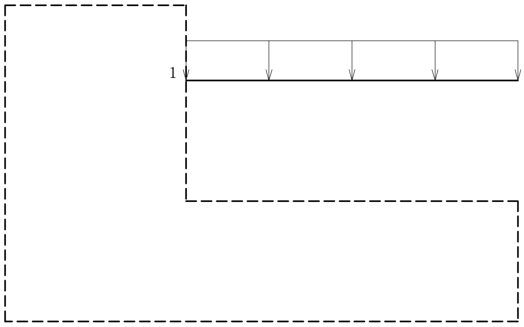
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

q : Load
L : Length of load
X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	1.150

2.4 Collision Load

Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 8.500(m)

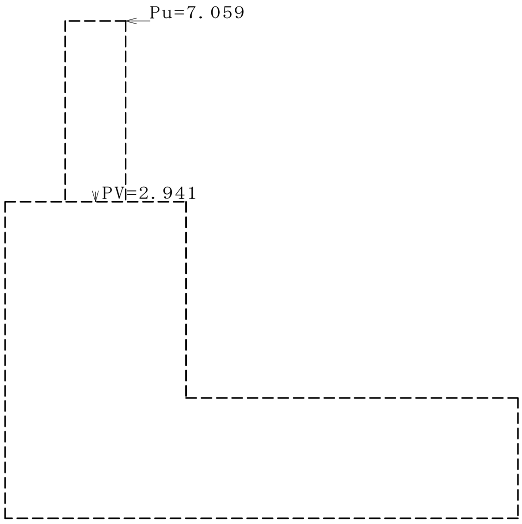
Vertical Force

$$Pv = \frac{pv}{L}$$

Where,

Pv : Front wheel load per unit width (kN/m)
pv : Front wheel load per block (kN)
L : Block length (m), L = 8.500(m)

[1] Collision



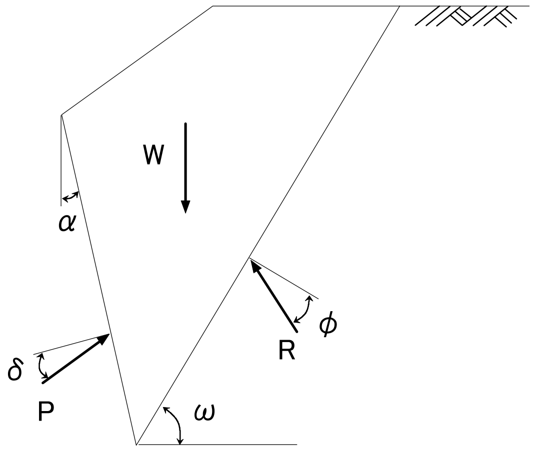
Horizontal Force

p (kN)	Pu (kN/m)	Location Y (m)
60.000	7.059	1.650

Vertical Force

pv (kN)	Pv (kN/m)	Location X (m)
25.000	2.941	0.300

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe) xp = 1.700 m
yp = 0.000 m
Height of Assumed Rear Surface H = 0.800 m
Angle of Earth Pressure Acting Surface to the Vertical α = 0.000 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °
Angle of Ground Surface from the Horizontal β = 0.000 °
Wall Friction Angle δ = β = 0.000 °
Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw = 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	3.653	0.000	5.576	9.229	5.116
60.00	3.510	0.000	5.358	8.868	5.120
61.00	3.370	0.000	5.144	8.514	5.116

Earth pressure becomes maximum when:
ω = 60.00° P = 5.120 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{8.868 \times \sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

= 5.120 kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 5.120 \times \cos(0.000^\circ + 0.000^\circ) = 5.120$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 5.120 \times \sin(0.000^\circ + 0.000^\circ) = 0.000$ kN

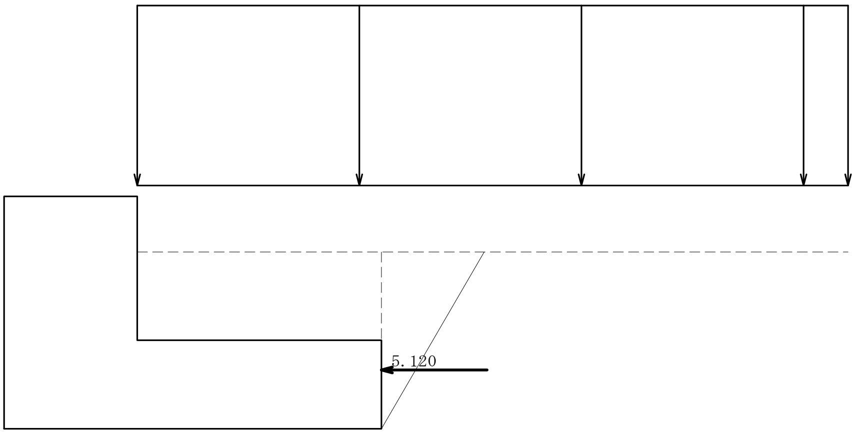
Location

$$Ho = \frac{H}{3} = \frac{0.800}{3} = 0.267 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.267 \times \tan 0.000^\circ = 1.700$ m

$y = yp + Ho = 0.000 + 0.267 = 0.267$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe) $xp = 1.700$ m

$yp = 0.000$ m

Height of Assumed Rear Surface $H = 0.800$ m

Angle to the Vertical of Earth Pressure Surface $\alpha = 0.000$ °

Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3

Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °

Angle of Ground Surface from the Horizontal $\beta = 0.000$ °

Composite angle during earthquake $\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204^\circ$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	6.520	0.000	0.000	6.520	2.661
44.00	6.296	0.000	0.000	6.296	2.665
45.00	6.080	0.000	0.000	6.080	2.665

Earth pressure becomes maximum when:
ω ＝ 44.00° P ＝ 5.508 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{6.296/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

＝ 2.665 kN

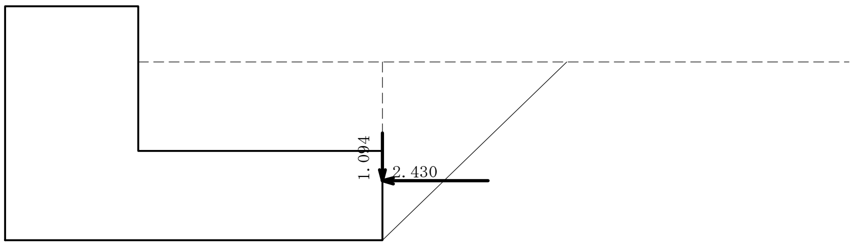
Earth Pressure's location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 2.665 \times \cos(0.000° + 24.239°) = 2.430$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 2.665 \times \sin(0.000° + 24.239°) = 1.094$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.800}{3} = 0.267 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.267 \times \tan 0.000° = 1.700$ m
$y = yp + Ho = 0.000 + 0.267 = 0.267$ m

· Earth Pressure Diagram



[3]　Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)　$xp = 1.700$ m
$yp = 0.000$ m
Height of Assumed Rear Surface　$H = 0.800$ m
Angle of Earth Pressure Acting Surface to the Vertical　$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side　$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side　$\varphi = 30.000$ °
Angle of Ground Surface from the Horizontal　$\beta = 0.000$ °
Wall Friction Angle　$\delta = \beta = 0.000$ °
Rate of Change of Slip Angle　$\omega i = 10.00$ °～　80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level　hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	3.653	0.000	0.000	3.653	2.025
60.00	3.510	0.000	0.000	3.510	2.027
61.00	3.370	0.000	0.000	3.370	2.025

Earth pressure becomes maximum when:
ω ＝ 60.00°　P ＝ 2.027 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{3.510 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

＝ 2.027 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha+\delta) = 2.027\times\cos(0.000°+0.000°) = 2.027$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha+\delta) = 2.027\times\sin(0.000°+0.000°) = 0.000$ kN

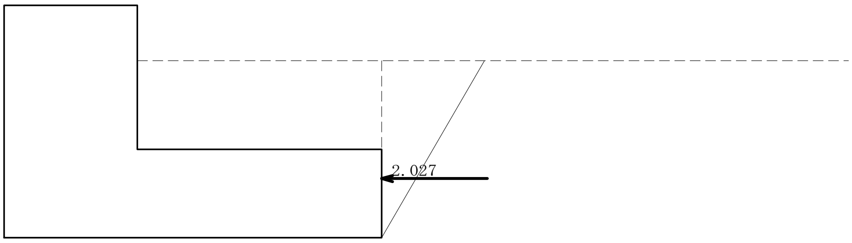
Location

$$Ho = \frac{H}{3} = \frac{0.800}{3} = 0.267 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.700 - 0.267\times\tan 0.000° = 1.700$ m

$y = yp + Ho = 0.000 + 0.267 = 0.267$ m

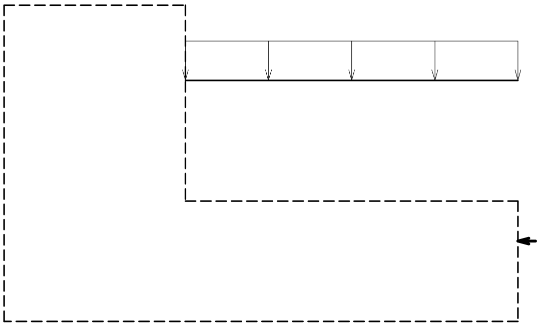
- Earth Pressure Diagram



2.6 Total Acting Force

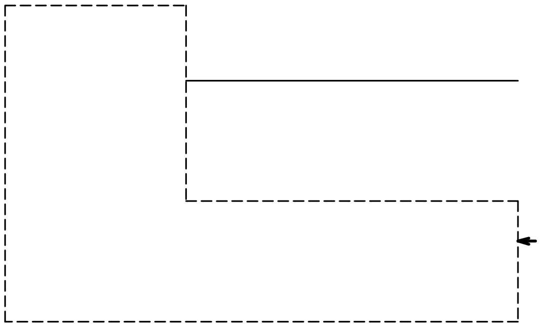
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



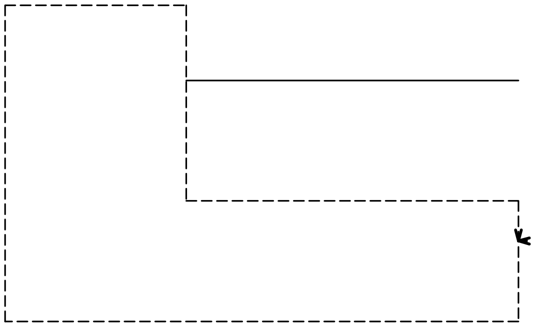
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	34.575	0.000	0.771	0.000	26.642	0.000
Loading, Snow	12.760	0.000	1.150	0.000	14.674	0.000
Earth Pressure	0.000	5.120	1.700	0.267	0.000	1.367
Total	47.335	5.120	———	———	41.316	1.367

[2] Ordinary Condition (Rear Side)



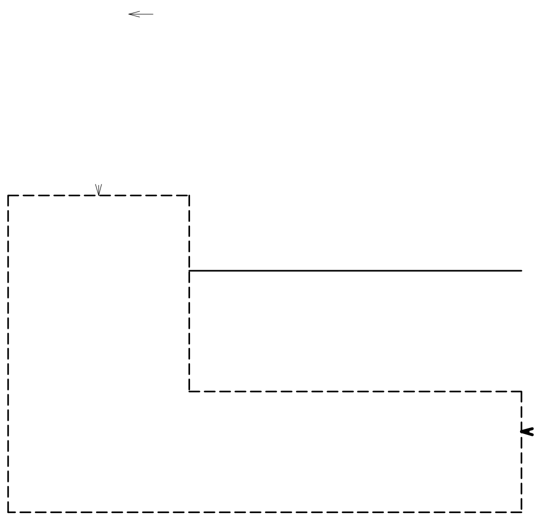
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	34.575	0.000	0.771	0.000	26.642	0.000
Earth Pressure	0.000	5.120	1.700	0.267	0.000	1.367
Total	34.575	5.120	―――	―――	26.642	1.367

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	34.575	6.224	0.771	0.442	26.642	2.750
Earth Pressure	1.094	2.430	1.700	0.267	1.860	0.649
Total	35.669	8.654	―――	―――	28.501	3.398

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	34.575	0.000	0.771	0.000	26.642	0.000
Loading, Snow	0.000	2.027	1.700	0.267	0.000	0.541
Collision Load	2.941	7.059	0.300	1.650	0.882	11.647
Total	37.516	9.086	———	———	27.524	12.188

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	47.335	5.120	39.948
Ordinary (Rear Side)	34.575	5.120	25.274
Earthquake	35.669	8.654	25.103
Collision	37.516	9.086	15.336

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,
Footing Earth Pressure Width : $B_j = 1.700$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	47.335	5.120	0.286
Ordinary (Rear Side)	34.575	5.120	4.114
Earthquake	35.669	8.654	5.216
Collision	37.516	9.086	16.553

■ Full Width per 8.500m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	402.347	43.520	2.433
Ordinary (Rear Side)	293.888	43.520	34.971
Earthquake	303.187	73.555	44.334
Collision	318.888	77.229	140.702

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B ＝ 1.700

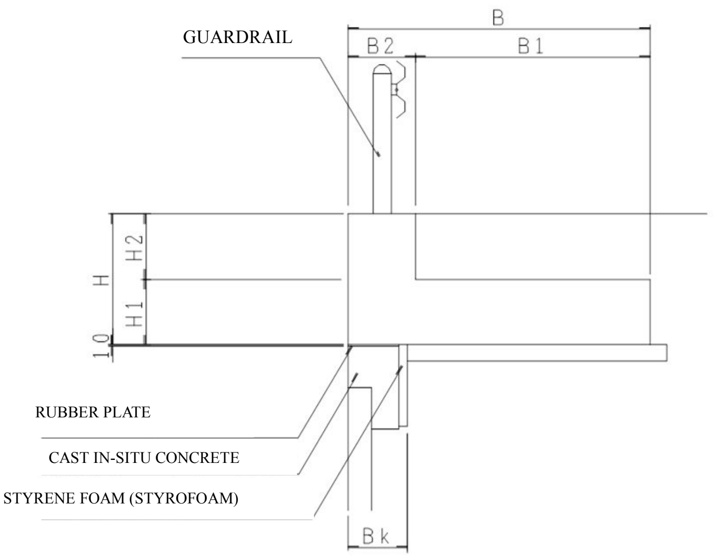
$$e_a = B／n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	41.316	1.367	47.335	0.844	0.006	≦	0.283	○
Ordinary (Rear Side)	26.642	1.367	34.575	0.731	0.119	≦	0.283	○
Earthquake	28.501	3.398	35.669	0.704	0.146	≦	0.567	○
Collision	27.524	12.188	37.516	0.409	0.441	≦	0.567	○

2.7.2 Stability from Sliding



H = 0.800 m
H1 = 0.400 m
H2 = 0.400 m

B = 1.700 m
B1 = 1.100 m
B2 = 0.600 m

Bk = 0.350 m

Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m^2)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.119	1.700	11.796	28.880	28.880	25.36
Earthquake	0.146	1.700	10.153	31.810	31.810	27.35
Collision	0.441	1.227	0.000	61.151	61.151	43.70

Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Nna (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN'=ΣN-Nna	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	34.575	9.493	25.082	5.120
Earthquake	35.669	10.353	25.316	8.654
Collision	37.516	18.350	19.166	9.086

Factor of safety for sliding, Fs:

$$Fs = \Sigma N' \cdot \mu / \Sigma H$$

Load Condition	Factor of Safety Fs	Required Factor of Safety Fsa
Ordinary (Rear Side)	2.939	≧1.5
Earthquake	1.755	≧1.2
Collision	1.266	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

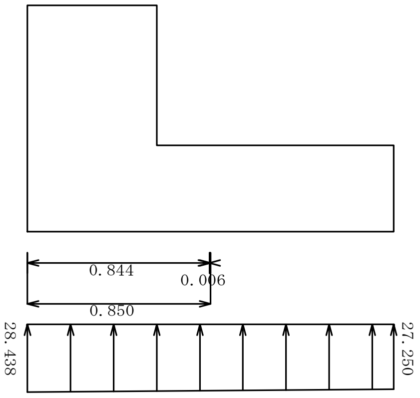
Where,

ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)

B : Bottom plate width (m), B = 1.700

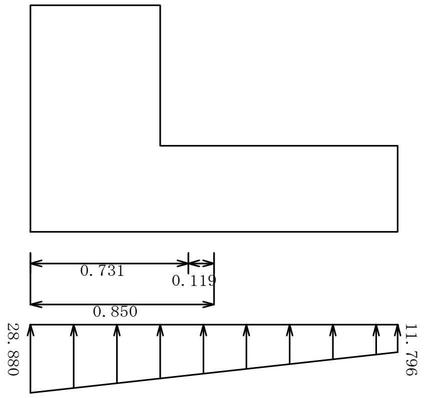
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



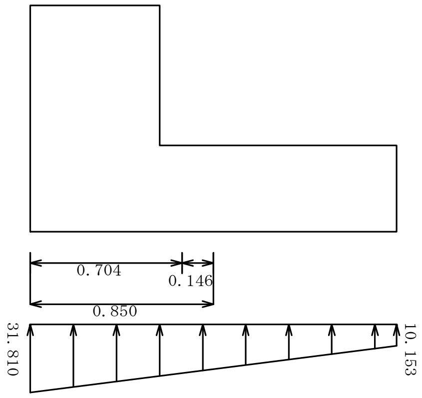
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.700	Trapezoid	27.250	28.438	≦	200.000	○

[2] Ordinary Condition (Rear Side)



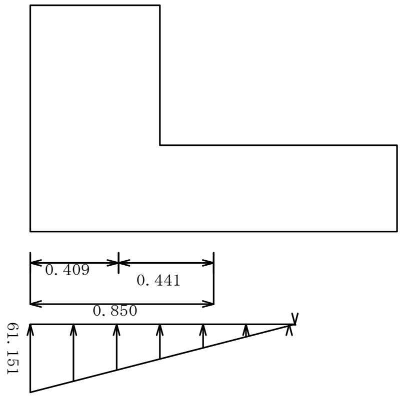
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.700	Trapezoid	11.796	28.880 ≦	200.000	○

[3] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.700	Trapezoid	10.153	31.810 ≦	300.000	○

[4] Collision



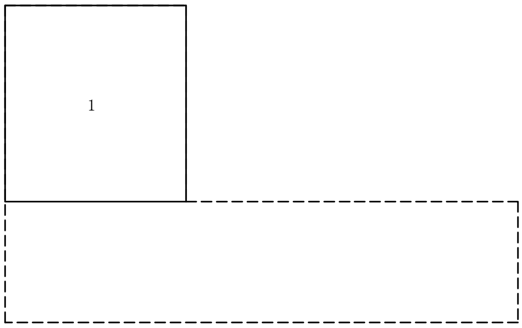
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.227	Triangle	0.000	61.151 ≦	300.000	○

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	0.600× 0.650× 1.000	0.390	0.300	0.325	0.117	0.127	
Σ		0.390	―	―	0.117	0.127	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.117／ 0.390 ＝ 0.300 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.127／ 0.390 ＝ 0.325 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.390 ＝ 9.555	0.000

Force Location

X ＝ Xc－XG ＝ 0.300－0.300

＝ 0.000 m

Where,

Xc ：Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.390 ＝ 9.555	0.000

Location	$H = W \cdot kh$ (kN)	Location Y (m)
Frame(Rebar)	9.555 × 0.180 ＝ 1.720	0.325

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

Horizontal Force

$$Pu = \frac{p}{\lambda + y}$$

Where,

Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
y : Distance from the tip of section being checked (m), y ＝ 1.000(m)

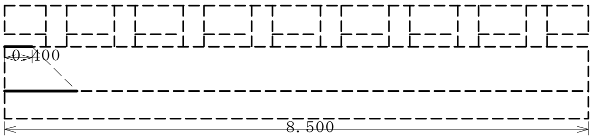
Vertical Force

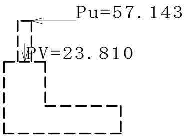
$$Pv = \frac{pv}{\lambda + y}$$

Where,

Pv : Front wheel load of the effective width (kN/m)
pv : Front wheel load per block (kN)

[1] Collision





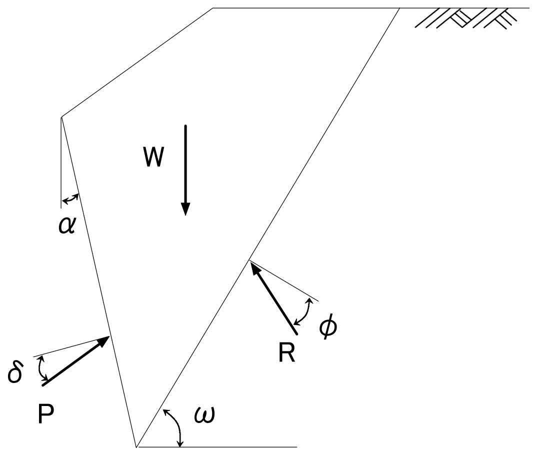
Horizontal Force

p (kN)	λ (m)	Pu (kN/m)	Location Y (m)
60.000	0.400	57.143	1.250

Vertical Force

pv (kN)	λorL (m)	Pu (kN/m)	Location X (m)
25.000	0.400	23.810	0.000

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.300 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.400 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	1.064	0.000	3.249	4.313	1.830
56.00	1.025	0.000	3.130	4.155	1.831
57.00	0.987	0.000	3.013	4.000	1.830

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 1.831 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{4.155 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

$$= 1.831 \text{ kN}$$

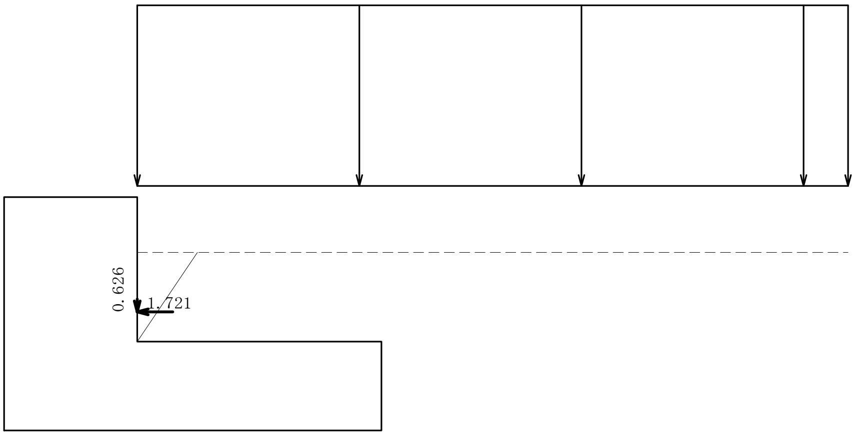
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 1.831 \times \cos(0.000° + 20.000°) = 1.721$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 1.831 \times \sin(0.000° + 20.000°) = 0.626$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.400}{3} = 0.133 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.133 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.133 = 0.133$ m

- Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.300$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.400$ m
Angle of Earth Pressure Acting Surface to the Vertical $\alpha = 0.000°$
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000°$
Wall Friction Angle $\delta = 2/3\varphi = 20.000°$
Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw = 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	1.064	0.000	0.000	1.064	0.451
56.00	1.025	0.000	0.000	1.025	0.452
57.00	0.987	0.000	0.000	0.987	0.451

Earth pressure becomes maximum when:
$\omega = 56.00°$ $P = 0.452$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{1.025 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

$= 0.452$ kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component

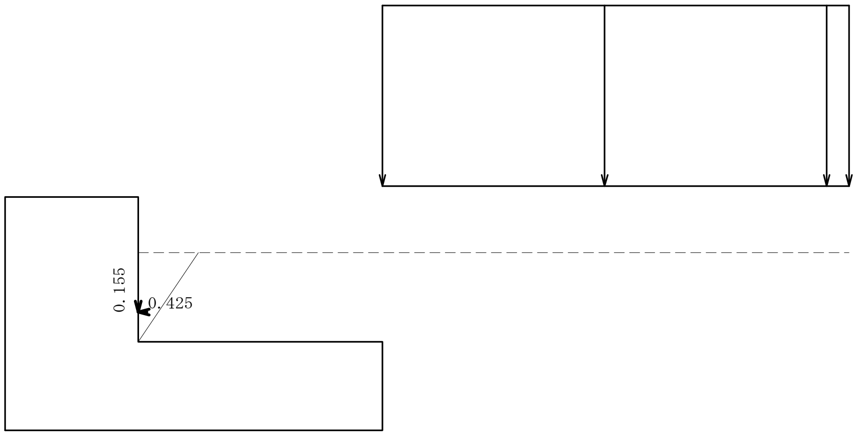
$Ph = P \cdot \cos(\alpha + \delta) = 0.452 \times \cos(0.000° + 20.000°) = 0.425$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 0.452 \times \sin(0.000° + 20.000°) = 0.155$ kN

Location

$$Ho = \frac{H}{3} = \frac{0.400}{3} = 0.133 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.133 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.133 = 0.133$ m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.300$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.400$ m
Angle to the Vertical of Earth Pressure Surface $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 1/2\varphi = 15.000$ °

Composite angle during earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)

Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
45.00	1.520	0.000	0.000	1.520	0.658
46.00	1.468	0.000	0.000	1.468	0.659
47.00	1.417	0.000	0.000	1.417	0.659

Earth pressure becomes maximum when:

$\omega = 46.00°$ $P = 0.659$ kN

Earth Pressure

$$P = \frac{W／\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{1.468／\cos 10.204° \times \sin(46.00° - 30.00° + 10.204°)}{\cos(46.00° - 30.00° - 0.000° - 15.000°)}$$

$= 0.659$ kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 0.659 \times \cos(0.000° + 15.000°) = 0.637$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 0.659 \times \sin(0.000° + 15.000°) = 0.171$ kN

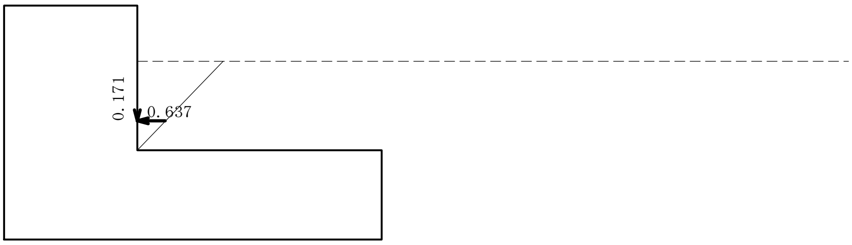
Location

$$Ho = \frac{H}{3} = \frac{0.400}{3} = 0.133 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.133 \times \tan 0.000° - 0.300 = -0.300$ m

$y = yp + Ho = 0.000 + 0.133 = 0.133$ m

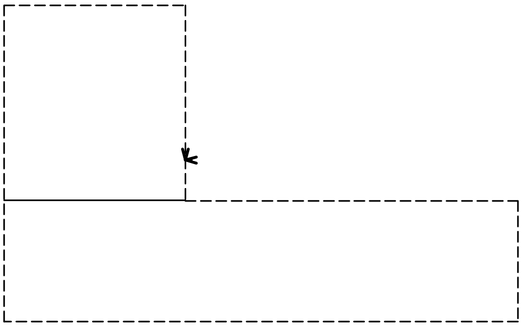
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

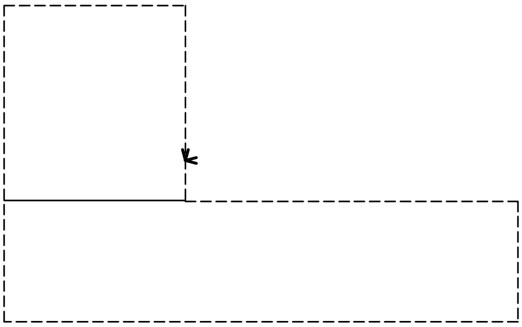
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	9.555	0.000	0.000	0.000	0.000
Earth Pressure	0.626	1.721	-0.300	0.133	0.229
Total	0.000	1.721	———	———	0.229

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

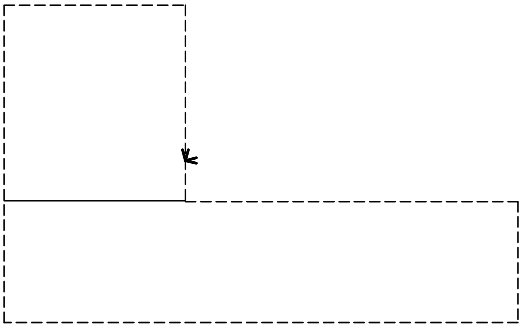
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	9.555	0.000	0.000	0.000	0.000
Earth Pressure	0.155	0.425	-0.300	0.133	0.057
Total	0.000	0.425	———	———	0.057

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

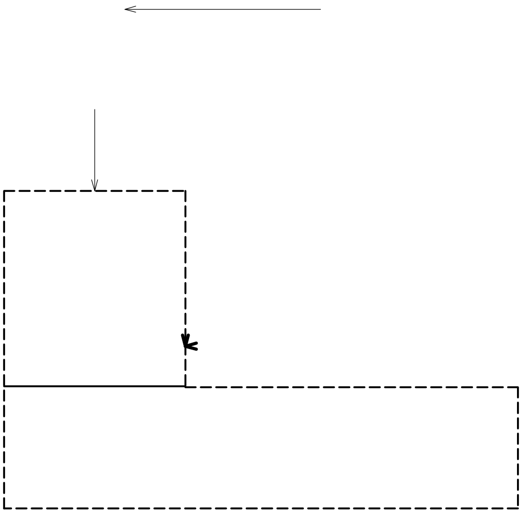
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	9.555	1.720	0.000	0.325	0.559
Earth Pressure	0.171	0.637	-0.300	0.133	0.085
Total	0.000	2.357	———	———	0.644

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

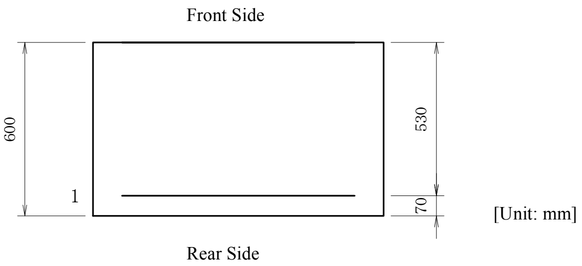


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	9.555	0.000	0.000	0.000	0.000
Earth Pressure	0.155	0.425	-0.300	0.133	0.057
Collision Load	23.810	57.143	0.000	1.250	71.429
Total	0.000	57.568	———	———	71.485

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	―	−	――	―	――
	2'	―	−	――	―	――
Rear Side	1	7.00	D16	1.986	4.000	7.944
	2	―	−	――	―	――

Required rebar amount at tension side: 4.734 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 60000.0 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 600000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 600.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	7.944	0.389	≦	114.821	0.000	5.000	○
Ordinary (Rear Side)	7.944	0.096	≦	114.821	0.000	5.000	○
Earthquake	7.944	1.094	≦	114.821	0.000	5.000	○
Collision	7.944	121.525	>	114.821	0.000	6.335	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1（$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars（$d_i \leqq x$）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars（$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 530.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	7.944	≦	169.941	○
Ordinary (Rear Side)	7.944	≦	169.941	○
Earthquake	7.944	≦	169.941	○
Collision	7.944	≦	169.941	○

(4) Check of Bending Stress

(Reference)

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\} = 0.0$$

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h = 600.000
b : Member cross-section width (mm), b = 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n = 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)	Tensile Stress (N/mm^2)		Judgement
				Calculated Allowable	Calculated	Allowable	
Ordinary (Surcharge)	0.229	0.000	10.107	0.009 ≦ 8.000	0.581 ≦	180.000	○
Ordinary (Rear Side)	0.057	0.000	10.107	0.002 ≦ 8.000	0.143 ≦	180.000	○
Earthquake	0.644	0.000	10.107	0.026 ≦ 12.000	1.633 ≦	300.000	○
Collision	71.485	0.000	10.107	2.849 ≦ 12.000	181.326 ≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member's effective height (mm)

b : Sectional member's effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member's effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

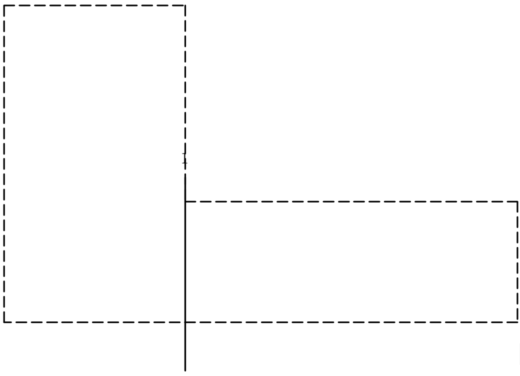
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)			Correction Factor			Judgement
			Calc. τ	Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	1.721	53.000	0.003 ≦	0.233	1.700	1.27	0.80	1.00	○
Ordinary(Rear Side)	0.425	53.000	0.000 ≦	0.233	1.700	1.27	0.80	1.00	○
Earthquake	2.357	53.000	0.004 ≦	0.355	2.550	1.27	0.80	1.00	○
Collision	57.568	53.000	0.109 ≦	0.350	2.550	1.27	0.80	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

Distance from joint: 0.000 m

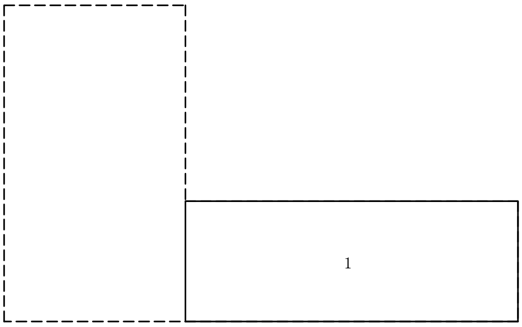


[Unit: mm]

4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



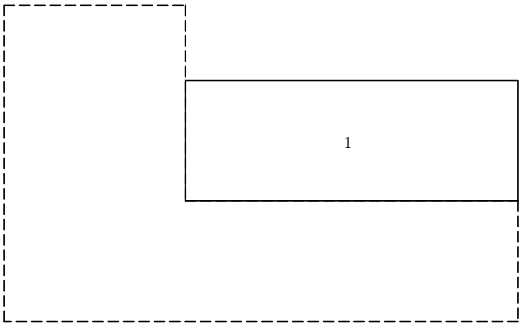
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	1.100× 0.400× 1.000	0.440	0.550	0.242	
Σ		0.440	——	0.242	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.242／ 0.440 ＝ 0.550 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	1.100× 0.400× 1.000	0.440	0.550	0.242	
Σ		0.440	——	0.242	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.242／0.440 ＝ 0.550 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.440 ＝ 10.780	0.550
Soil(Rear Side)	19.000 × 0.440 ＝ 8.360	0.550

4.1.3 Ground Surface Load・Snow Load

Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

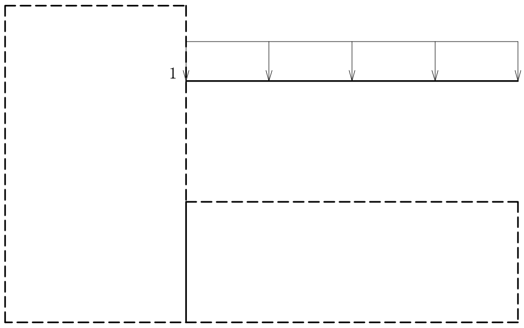
Where,

q : Ground surface loading

L : Length of ground surface load

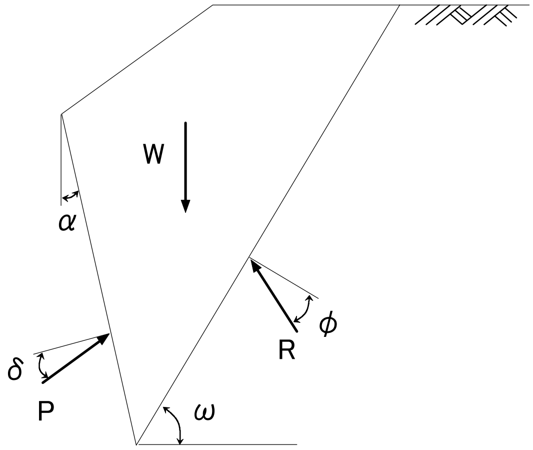
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m²)	q2 (kN/m²)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	0.550

4.1.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	xp = 1.700 m
	yp = 0.000 m
Height of Assumed Rear Surface	H = 0.800 m
Angle to the vertical of the Earth Pressure Surface	α = 0.000 °
Unit Weight of Soil at Rear Side	γs = 19.000 kN/m³
Angle of Internal Friction of Soil at Rear Side	φ = 30.000 °
Angle to the Horizontal of the Ground Surface	β = 0.000 °
Composite Angle during Earthquake	$\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	6.520	0.000	0.000	6.520	2.661
44.00	6.296	0.000	0.000	6.296	2.665
45.00	6.080	0.000	0.000	6.080	2.665

Earth pressure becomes maximum when:
Wen ω ＝ 44.00°, P ＝ 2.665 kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{6.296/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$$= 2.665 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component

$$P_v = P \cdot \sin(\alpha + \delta) = 2.665 \times \sin(0.000° + 24.239°) = 1.094 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 1.094}{1.100} = 1.989 \text{ kN/m}$$

Where,

pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section	L1	=	0.000 m
Width of distributed load perpendicular to the design section	L2	=	1.100 m

Distributed Load at the Design $pd = \frac{pv}{L} \cdot L1 = \frac{1.989}{1.100} \times 0.000 = 0.000$ kN/m

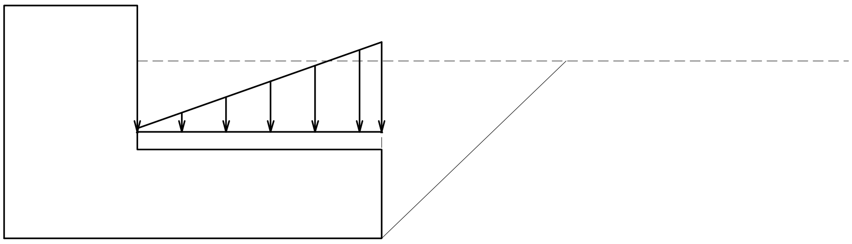
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (0.000 + 1.989) \times 1.100 = 1.094 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{0.000 + 2 \times 1.989}{0.000 + 1.989} \times \frac{1.100}{3} = 0.733 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

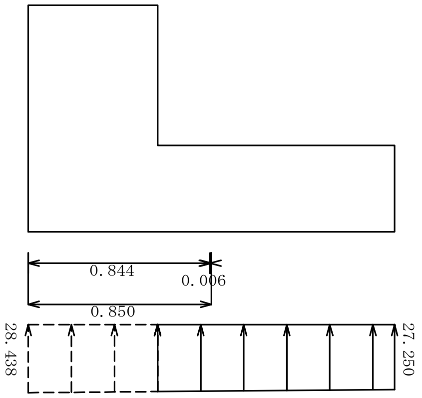
Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

Where,

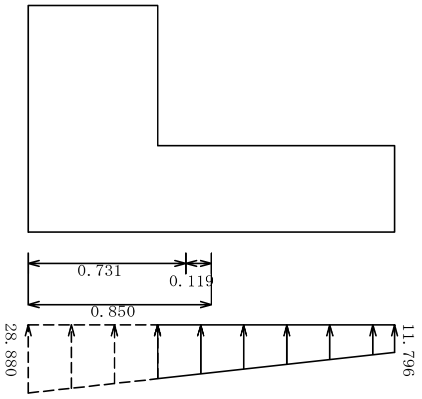
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



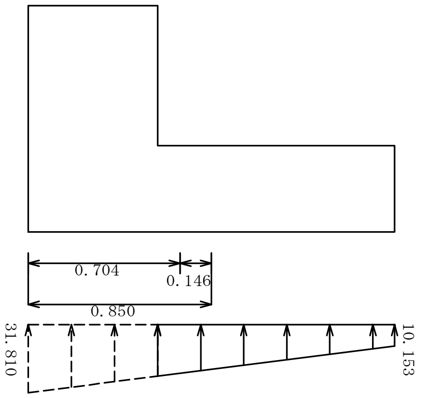
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
27.250	28.019	1.100	30.398	0.547

[2] Ordinary Condition (Rear Side)



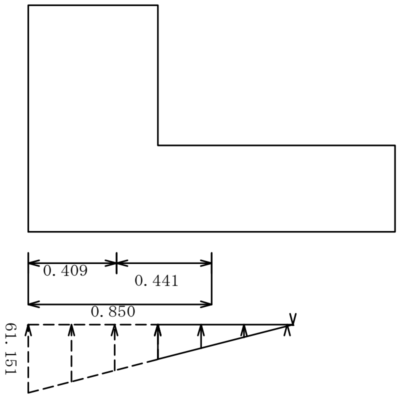
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
11.796	22.850	1.100	19.055	0.492

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
10.153	24.166	1.100	18.876	0.475

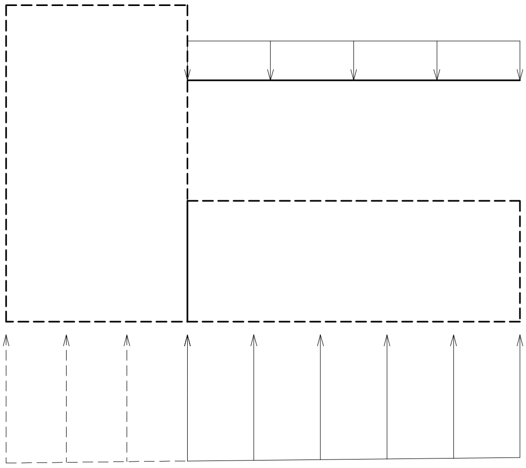
[4] Collision



Ground Reaction (kN/m²)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	31.248	0.627	9.796	0.209

4.1.6 Total Sectional Forces

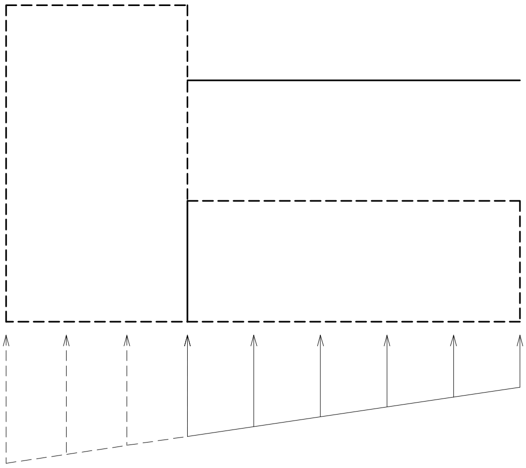
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.140	0.550	10.527
Loading, Snow Load	12.760	0.550	7.018
Ground Reaction	-30.398	0.547	-16.641
Total	1.502	———	0.904

Sectional force of vertical wall foundation, M1 ＝ 0.229 kN.m
Sectional force of the base of heel, M3 ＝0.904 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

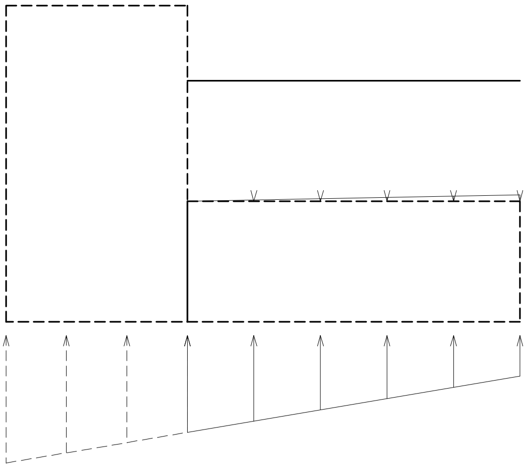
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	19.140	0.550	10.527
Loading, Snow Load	-19.055	0.492	-9.366
Ground Reaction	0.085	———	1.161
Total	19.140	0.550	10.527

Sectional force of vertical wall foundation, M1 ＝ 0.057 kN.m
Sectional force of the base of heel, M3 ＝ 1.161 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

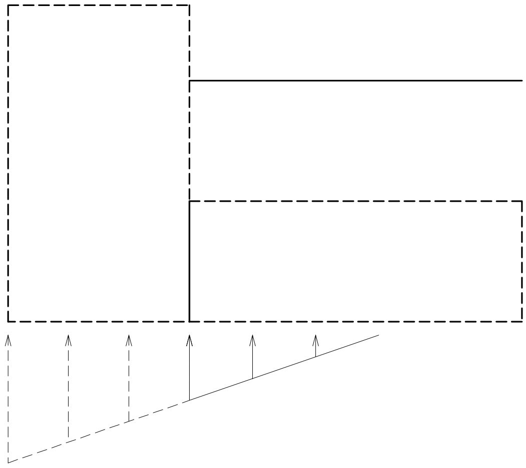
[3] Earthquake



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	19.140	0.550	10.527
Loading, Snow Load	1.094	0.733	0.802
Ground Reaction	-18.876	0.475	-8.969
Total	1.358	———	2.361

Sectional force of vertical wall foundation, M1 ＝ 0.644 kN.m
Sectional force of the base of heel, M3 ＝ 2.361 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

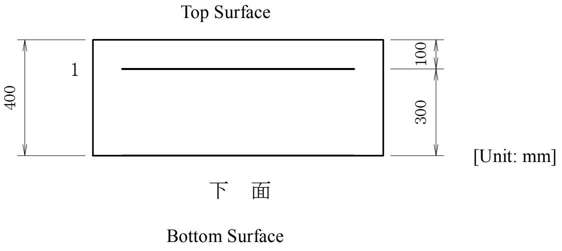
[4] Collision



Item	N_i (kN)	X_i (m)	$M =N_i \cdot X_i$ (kN.m)
Self-weight	19.140	0.550	10.527
Ground Reaction	-9.796	0.209	-2.047
Total	9.344	———	8.480

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.00	D13	1.267	4.000	5.068
	2	—	–	——	—	——
Bottom Surface	1'	—	–	——	—	——
	2'	—	–	——	—	——

Required tension side rebar amount　0.973 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), h = 400.000

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	0.389	≦	51.031	5.000	○
Ordinary (Rear Side)	5.068	0.096	≦	51.031	5.000	○
Earthquake	5.068	1.094	≦	51.031	5.000	○
Collision	5.068	14.415	≦	51.031	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1（$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$y2 = x - y1$

$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$

Compressive force at the compression side rebars（$d_i \leqq x$）

$\varepsilon_i = A \cdot (x - d_i)$

$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$

$S_{ci} = \sigma si \cdot Asi$

$S1 = \Sigma S_{ci}$

Tensile force at the tension side rebars（$d_j > x$）

$\varepsilon_j = A \cdot (d_j - x)$

$\sigma_{sj} = \varepsilon_j \cdot E_s$

$S_{tj} = \sigma_{sj} \cdot A_{sj}$

$S2 = \Sigma S_{tj}$

From the axial force balance

$N = C1 + C2 + S1 - \alpha \cdot S2$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$A_{sb} = \alpha \cdot \Sigma A_{sj}$

Where,

A_{sb} : Balanced rebar amount (mm^2)

σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$

E_s : Rebar Young’s Modulus (N/mm^2), Es = 200000

d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0

b : Width of member (mm), b = 1000.0

σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$

d_i : Distance fr. compression side edge to the “ith” compression side rebar (mm)

A_{si} : Area enclosed by the compression side edge to the “ith” compression side rebar (mm^2)

σ_{si} : Compressive stress from compression side edge to the “ith” compression side rebar (N/mm^2)

d_j : Distance fr. compression side edge to the “jth” tension side rebar (mm)

A_{sj} : Area eclosed by the compression side edge to the “jth” tension side rebar (mm^2)

σ_{sj} : Tensile stress from compression side edge to the “jth” tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2 \cdot n}{b} \left\{ As \cdot (x - d) \right\} = 0.0$$

よりxを求める。

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2} \cdot \left(\frac{h}{2} - \frac{x}{3} \right) + n \cdot As \cdot \frac{(x - d) \cdot (h / 2 - d)}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d - x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h ＝ 400.000

b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^2)

n : Rebar and concrete’s coefficient of Young’s modulus, n ＝ 15.00

e : Distance from the sectional member’s centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^2)

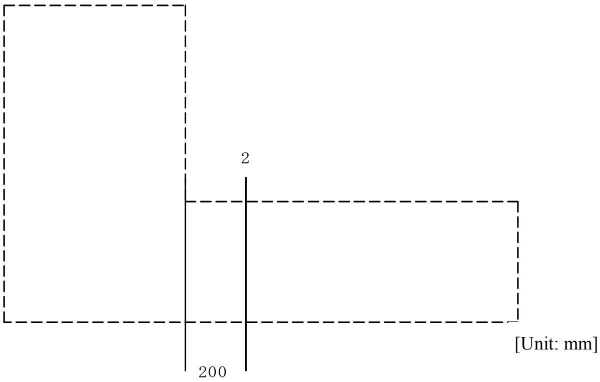
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.229	6.035	0.027	≦	8.000	1.614	≦	180.000	○
Ordinary (Rear Side)	0.057	6.035	0.007	≦	8.000	0.399	≦	180.000	○
Earthquake	0.644	6.035	0.076	≦	12.000	4.538	≦	300.000	○
Collision	8.480	6.035	1.004	≦	12.000	59.784	≦	300.000	○

4.2 Design of Section [2]

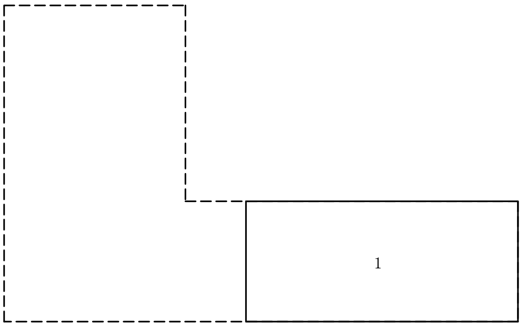
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



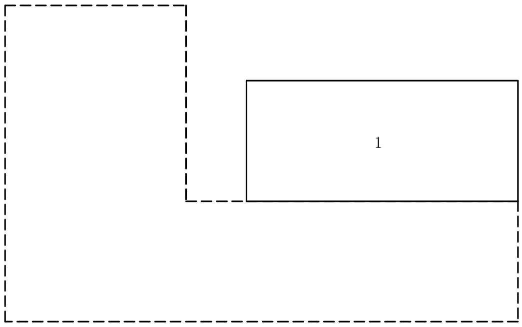
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.900× 0.400× 1.000	0.360	0.450	0.162	
Σ		0.360	——	0.162	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.162／ 0.360 ＝ 0.450 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.900× 0.400× 1.000	0.360	0.450	0.162	
Σ		0.360	——	0.162	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.162／ 0.360 ＝ 0.450 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.360 ＝ 8.820	0.450
Soil(Rear Side)	19.000 × 0.360 ＝ 6.840	0.450

4.2.3 Ground Surface Load・Snow Load

Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

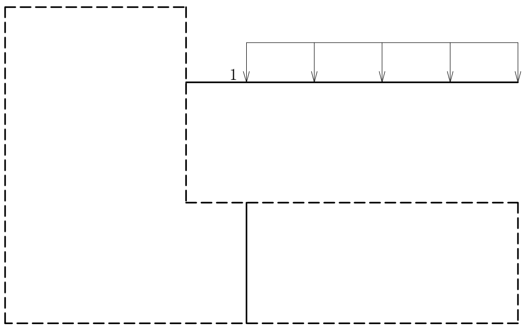
Where,

q : Ground surface loading

L : Length of ground surface load

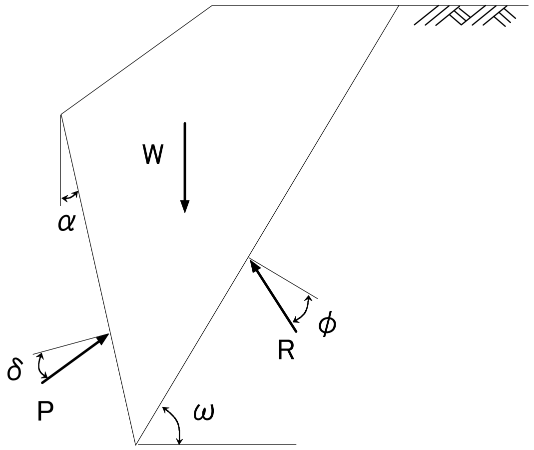
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
11.600	11.600	0.900	10.440	0.450	11.600

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	xp ＝ 1.700 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.800 m
Angle to the vertical of the Earth Pressure Surface	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle to the Horizontal of the Ground Surface	β ＝ 0.000 °
Composite Angle during Earthquake	θ ＝ $\tan^{-1}$kh＝ $\tan^{-1}$0.18 ＝10.204°

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1-\sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1-\sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	6.520	0.000	0.000	6.520	2.661
44.00	6.296	0.000	0.000	6.296	2.665
45.00	6.080	0.000	0.000	6.080	2.665

Earth pressure becomes maximum when:
$\omega = 44.00°, P = 2.665$ kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{6.296/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$$= 2.665 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component
$P_v = P \cdot \sin(\alpha + \delta) = 2.665 \times \sin(0.000° + 24.239°) = 1.094$ kN

The ff. equivalent triangular distributed load of the vertical component will be used:

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 1.094}{1.100} = 1.989 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section	L1 =	0.200 m
Width of distributed load perpendicular to the design section	L2 =	0.900 m

Distributed Load at the Design Section, pd

Distributed Load at the Design Section $pd = \frac{pv}{L} \cdot L1 = \frac{1.989}{1.100} \times 0.200 = 0.362$ kN/m

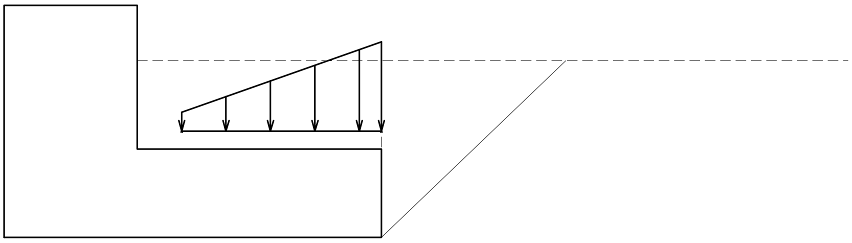
Vertical Force

$$N = \frac{1}{2} \cdot (pd+pv) \cdot L2 = \frac{1}{2} \times (0.362+1.989) \times 0.900 = 1.058 \text{ kN}$$

Location

$$x = \frac{pd+2 \cdot pv}{pd+pv} \cdot \frac{L2}{3} = \frac{0.362+2\times1.989}{0.362+1.989} \times \frac{0.900}{3} = 0.554 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2) \cdot L$$

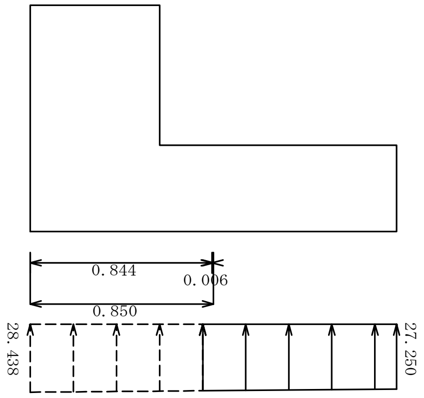
Location

$$X = \frac{2 \cdot q1+q2}{3 \cdot (q1+q2)} \cdot L$$

Where,

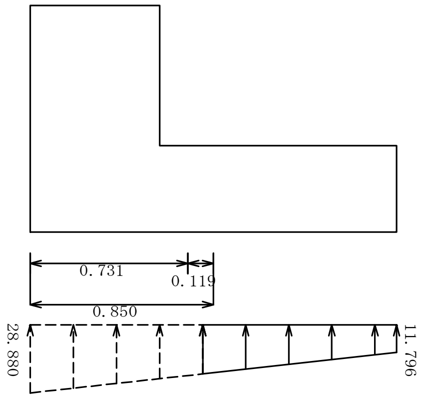
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



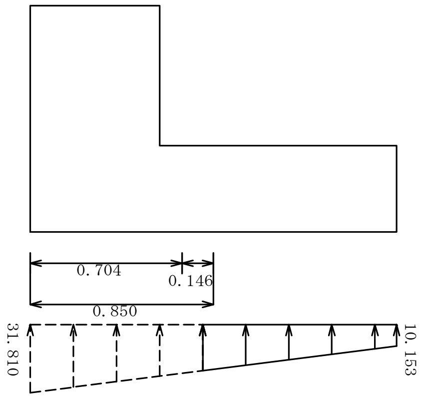
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
27.250	27.879	0.900	24.808	0.448

[2] Ordinary Condition (Rear Side)



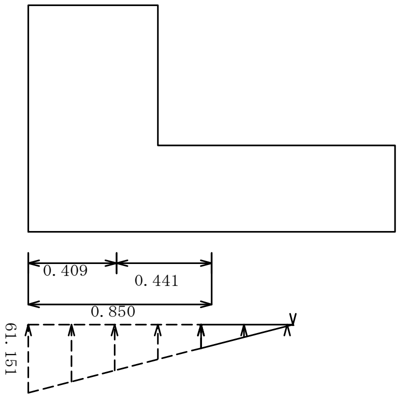
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
11.796	20.840	0.900	14.686	0.408

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
10.153	21.618	0.900	14.297	0.396

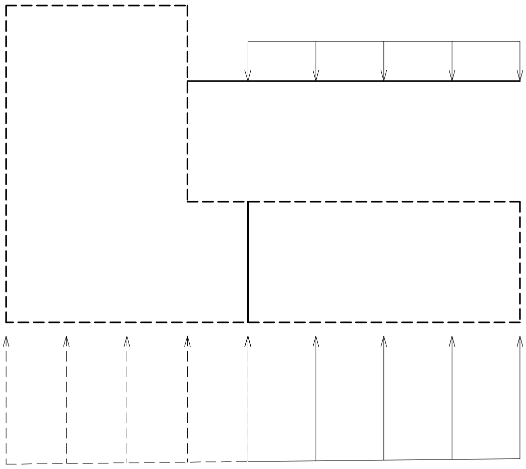
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	21.281	0.427	4.543	0.142

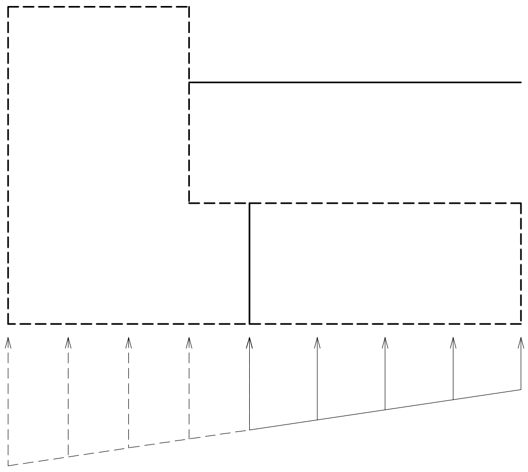
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



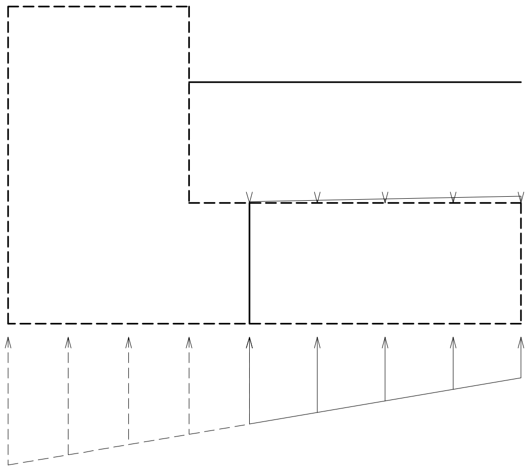
Item	N_i (kN)	X_i (m)	M =N_i・X_i (kN.m)
Self-weight	15.660	0.450	7.047
Loading, Snow Load	10.440	0.450	4.698
Ground Reaction	-24.808	0.448	-11.121
Total	1.292	———	0.624

[2] Ordinary Condition (Rear Side)



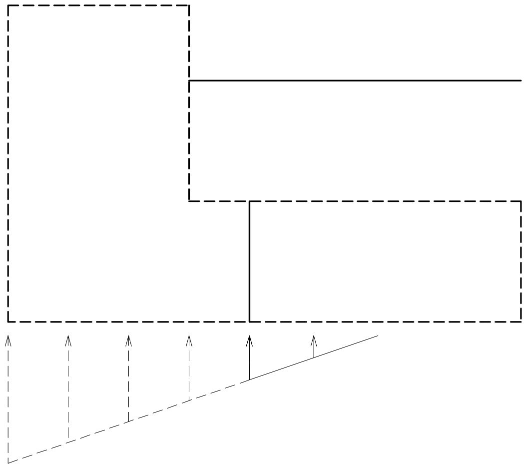
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	15.660	0.450	7.047
Ground Reaction	-14.686	0.408	-5.998
Total	0.974	———	1.049

[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	15.660	0.450	7.047
Loading, Snow Load	1.058	0.554	0.586
Ground Reaction	-14.297	0.396	-5.660
Total	2.421	———	1.973

[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	15.660	0.450	7.047
Ground Reaction	-4.543	0.142	-0.647
Total	11.117	———	6.400

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ a > 2.5d,

when a ≦ 2.5d, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
M : Bending moment acting on the sectional member (N.mm)
θ : Angle of the top surface footing to the horizontal, θ = 0.000 , tanθ = 0.000
a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span 2.5・d (cm)		a	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000	≦	90.160	1.292	0.624	0.000	1.292
Ordinary (Rear Side)	30.000	75.000	≦	140.000	0.974	1.049	0.000	0.974
Earthquake	30.000	75.000	≦	140.000	2.421	1.973	0.000	2.421
Collision	30.000	75.000	≦	120.750	11.117	6.400	0.000	11.117

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm : Average shear stress of concrete (N/mm2)
Sh : Actual Shear Force (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
τa1 : Allowable shear stress (N/mm2)

$$\tau_{a1} = Ce \cdot Cpt \cdot Cdc \cdot \tau_{a1}'$$

Where,

τa1' : Allowable shear stress when shear force acts in concrete (N/mm2)
Ce : Correction factor for the sectional member’s effective height
Cpt : Correction factor for the tensile main reinforcement ratio, Pt
Cdc : Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	1.292	30.000	0.004	≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	0.974	30.000	0.003	≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	2.421	30.000	0.008	≦	0.411	2.550	1.40	0.84	1.00	○
Collision	11.117	30.000	0.037	≦	0.405	2.550	1.40	0.84	1.00	○

ROAD DESIGN

MECHANICALLY-STABILIZED EARTH WALL AND RETAINING WALL

(PACKAGE 2)

Design Conditions

Mechanically-Stabilized Earth Wall Type-1

Mechanically-Stabilized Earth Wall Type-2

Mechanically-Stabilized Earth Wall Type-3

Concrete Barrier Type-1

Concrete Barrier Type-2

Concrete Barrier Type-3

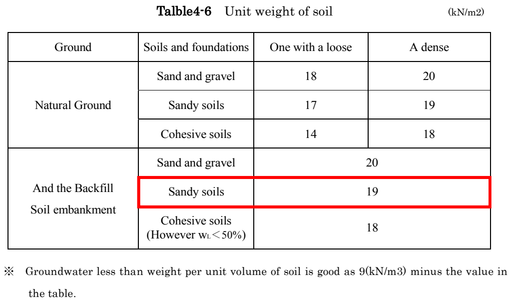
Gravity Wall (H=2800)

Gravity Wall (H=2100)

Design Conditions

THAKETA DESIGN CONDITIONS

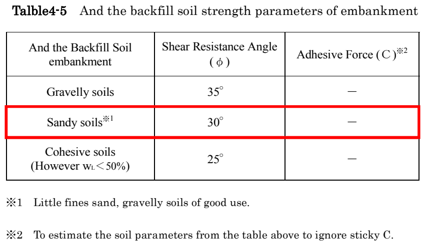
ITEM	DESIGN CRITERIA	(SUPPORTING DOCUMENTATION)	NOTE	CONSULTATION RESULT
1. Application criteria	1) Japan Road Association (2012). Road Earthwork: Retaining Wall Guidelines 2012. 2) Japan Road Association (2012). Specifications for Highway Bridges, the Commentary. 3) Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed. 4) American Association of State Highway and Transportation Officials (2012), AASHTO LRFD Bridge Design Specifications, 6th Ed (US)	【Road Earthwork: Retaining Wall Guidelines P.66】		
2. Soil Condition				
1). Embankment Material	γ=19kN/m^3, φ=30°, C=0kN/m^2			
3. Earthquake resistant design	Horizontal Seismic Coefficient kh = Cz・kho = 1.0×0.18 = 0.18			
4. Loading Conditions				
1). Main Load Loading	Road Department 11.6kN/m^2 (AASHTO LRFD 2012 Bridge)			
2). Special Loads				
Collision Load	Mechanically -stabilised earth wall Handrail（SC） 43kN/m Gravity wall (Only horizontal force taken into account)			
5. Study Model	MSE TYPE-1 H=4500(Full Height) H=4480(Finished Height) ＋ H4=1700 ＝ 6200 MSE TYPE-2 H=2250(Full Height) H=2230(Finished Height) ＋ H4=2200 ＝ 4450 Gravity Wall Hmax H=2100(Full Height) H=2061(Finished Height)	Concrete Barrier Height h=1150 Concrete Barrier Height h=1400		



Talble4-6 Unit weight of soil (kN/m2)

Ground	Soils and foundations	One with a loose	A dense
Natural Ground	Sand and gravel	18	20
	Sandy soils	17	19
	Cohesive soils	14	18
And the Backfill Soil embankment	Sand and gravel	20	
	Sandy soils	19	
	Cohesive soils (However w$_L$＜50%)	18	

※ Groundwater less than weight per unit volume of soil is good as 9(kN/m3) minus the value in the table.

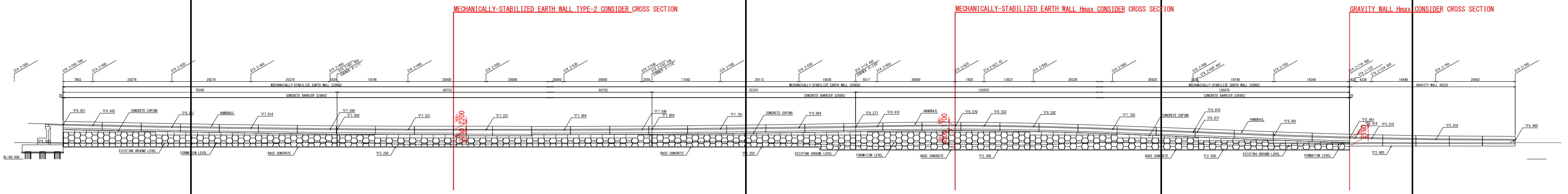


Talble4-5 And the backfill soil strength parameters of embankment

And the Backfill Soil embankment	Shear Resistance Angle (φ)	Adhesive Force (C)※2
Gravelly soils	35°	—
Sandy soils※1	30°	—
Cohesive soils (However w$_L$＜50%)	25°	—

※1 Little fines sand, gravelly soils of good use.

※2 To estimate the soil parameters from the table above to ignore sticky C.



Mechanically-Stabilized Earth Wall Type-1

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5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

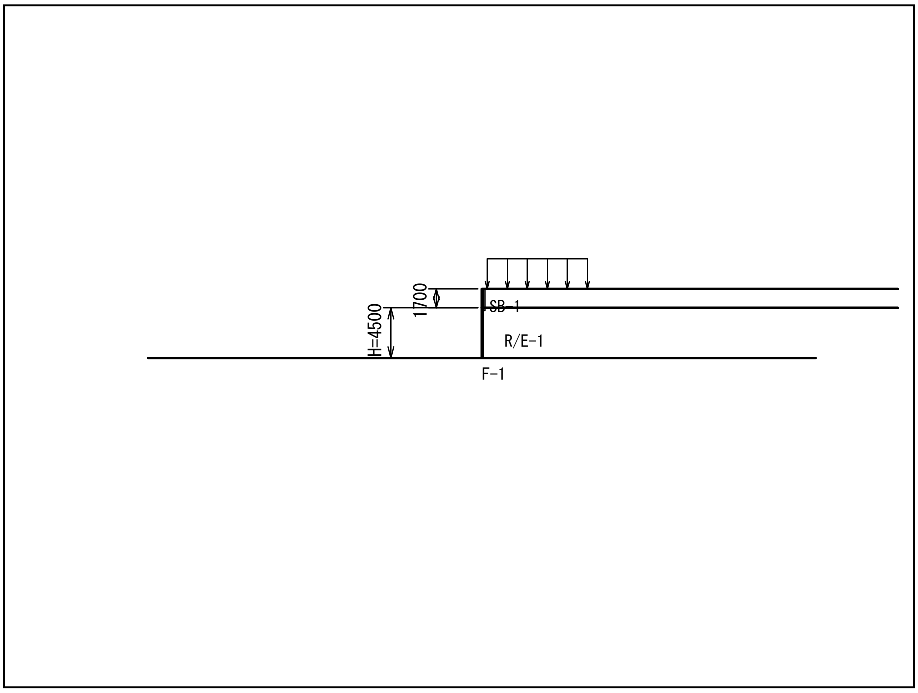
5.1 ORDINARY

5.2 EARTHQUAKE

1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H ＝ 4.500 (m)

HEIGHT OF COPING : H4 ＝ 1.700 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	0.000	0.000	4.500

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	4.500
	2	0.000	6.200

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	0.000
	2	30.000	0.000

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE：RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 ：MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION

PERFORMANCE 2 ：KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL

PERFORMANCE 3 ：KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	c es (kN/m^2)	fo*	tanϕ1
SB－1	1.700	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	4.500	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB：SURCHARGE BACKFILL, R/E：R/E, BE：BACKSIDE EMBANKMENT, F：FOUNDATION

Hs ：LAYER THICKNESS (m)

γ ：UNIT WEIGHT OF SOIL (kN/m^3)

γ' ：UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)

ϕ ：ANGLE OF SHEARING RESISTANCE (°)

c ：COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)

c es ：COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL：c es＝10kN/m^2)

fo* ：APPARENT COEFFICIENT OF FRICTION

tanϕ_1 ：APPARENT COEFFICIENT OF FRICTION

1.5　SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP $L+B_G$(m)	WIDTH OF LOAD B_L(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	LIVE LOAD1	0.500	9.000	11.60	—

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6　HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1} = c_z \cdot k_{ho} = 0.18$

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : Ａ]

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2} = c_z \cdot k_{ho} \cdot \nu = 0.13$

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : Ａ]

ν : CORRECTION COEFFICIENT = 0.70

(3) OVERALL STABILITY EXAMINATION INCLUDING R/E

(EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

$k_{h3} = c_z \cdot k_{ho} = 0.12$

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.12

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR EMBANKMENT CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : Ａ]

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT h_c (m)	UNIT WEIGHT γ_c (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σ_a=140.0	σ_{aE}=210.0
BOLT SHEAR STRESS (N/mm^2)	τ_a=200.00	τ_{aE}=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$
CIRCULAR SLIP	$F_s \geqq 1.20$	$F_{sE} \geqq 1.00$

1.9 MATERIAL

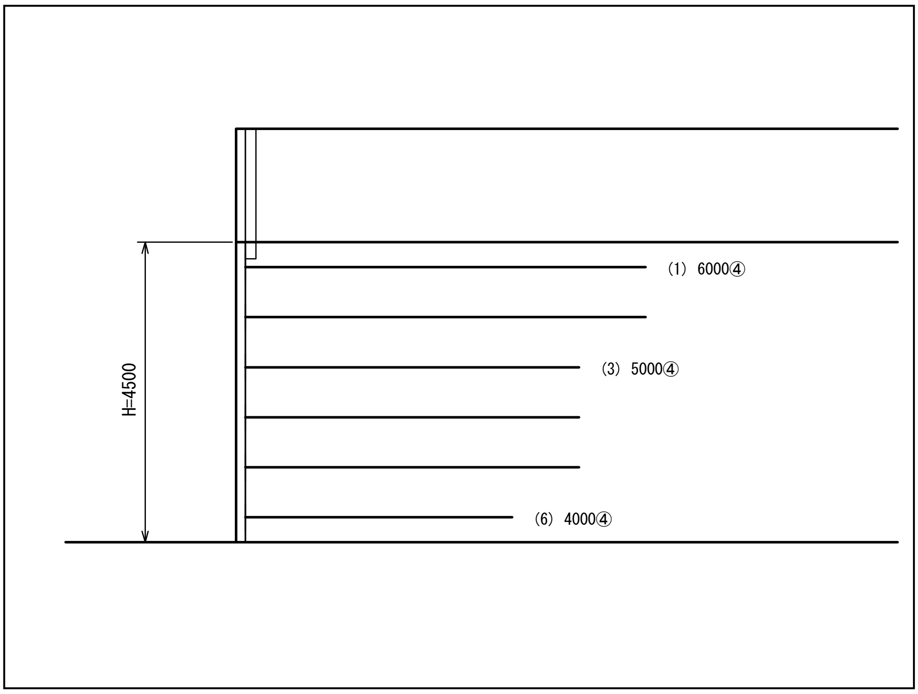
(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	Ae	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n'	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	Cm	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z_i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB(m)	DESIGN LENGTH L(m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.075	0.750	0.750	6.000	5.776	5.987	2.114	1.204	○
2	2.825	0.750	0.750	6.000	5.244	5.192	2.447	1.549	○
3	3.575	0.750	0.750	5.000	4.665	4.409	2.217	1.500	○
4	4.325	0.750	0.750	5.000	4.207	3.819	2.515	1.802	○
5	5.075	0.750	0.750	5.000	4.000	4.000	2.718	2.051	○
6	5.825	0.750	0.750	4.000	4.000	4.000	2.261	1.764	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σ_{ti} (N/mm^2)	σ_{oti} (N/mm^2)	τ_{oi} (N/mm^2)	σ_{tEi} (N/mm^2)	σ_{otEi} (N/mm^2)	τ_{oEi} (N/mm^2)	
1	79.6	73.5	85.0	96.3	88.9	102.8	○
2	81.2	74.9	86.7	93.9	86.7	100.3	○
3	82.7	76.3	88.3	92.6	85.5	98.8	○
4	88.3	81.5	94.3	101.3	93.5	108.2	○
5	94.4	87.1	100.7	109.2	100.8	116.6	○
6	99.3	91.7	106.1	116.1	107.2	124.0	○

σ_{ti} , σ_{tEi} : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σ_{oti}, σ_{otEi} : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τ_{oi} , τ_{oEi} : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3　EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	7.291 (1.500)	○	2.397 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.750 (0.657)	○	-0.245 (1.313)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	187.542	—	167.860	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	172.518	—	155.761	—

2.4　OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R (m)	Fsmin	Fs	JUDGE
		X (m)	Y (m)				
WITHIN REINFORCED AREA	ORDINARY-1	-12.000	16.500	20.402	1.360	1.200	○
	EARTHQUAKE-1	-5.500	17.000	17.868	1.420	1.000	○

2.5 RESULTS OF EXAMINATION OF INTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

2.5.1 LIST OF WALL HEIGHTS AND STRIP STRENGTHS

NOTE) HORIZONTAL SPACING OF STRIP FOR EACH ORDINAL STAGE NUMBER IS REPRESENTED IN THE FOLLOWING SYMBOLS:

④= 0.750 (m),
i : STRIPSTAGE
H : HEIGHT OF R/E(m)

VALUES IN THE TABLE MINIMUM STRIP LENGTH(m), () IN THE EXECUTION STRIP LENGTH(m)

i	(1)H=4.500(m)	(2)H=3.750(m)	(3)H=3.000(m)	(4)H=2.250(m)	(5)H=1.500(m)
1	④ 5.987 (6.0)	④ 5.628 (6.0)	④ 5.357 (6.0)	④ 4.922 (5.0)	④ 4.490 (5.0)
2	④ 5.244	④ 4.951	④ 4.502	④ 4.052	④ 3.601 (4.0)
3	④ 4.665 (5.0)	④ 4.253 (5.0)	④ 3.815 (5.0)	④ 3.365 (4.0)	
4	④ 4.207	④ 3.821	④ 3.388 (4.0)		
5	④ 3.857	④ 3.495 (4.0)			
6	④ 3.564 (4.0)				

2.5.2 LIST OF WALL HEIGHTS AND STRUCTURAL DETAIL SPECIFICS

BASED ON: DESIGN AND CONSTRUCTION MANUAL FOR REINFORCED EARTHWORK

H : HEIGHT OF R/E (m)
H4 : HEIGHT OF COPING (m)
Ha : VIRTUAL WALL HEIGHT (m)
H1 : HEIGHT OF SURCHARGE BACKFILL IN THE UPPER PART OF R/E (m)
n : SLOPE INCLINE
Bb : FLAT WIDTH OF SKIN ELEMENT LEVEE CROWN (m)
L1 : DISTANCE FROM BACK SURFACE OF SKIN ELEMENT TO ROAD SHOULDER (m)
QLj_W1 : LIVE LOAD AFFECTING DIFFERENT STAGE STRIP (ORDINARY) (kN/m^2)
B_G : HORIZONTAL DISTANCE FROM ROAD SHOULDER OF SKIN ELEMENT TO SURCHARGE LOAD (m)
B_L : WIDTH OF LOAD (m)
Lsd : MINIMUM STRIP LENGTH (m)
Hsd : PLACEMENT AREA OF MINIMUM STRIP LENGTH (m)

(1) H=4.500 (m), H4=1.700 (m), Ha=6.200 (m), H1=1.700 (m)
1 : n=1 : 0.000, Bb =0.000 (m), L1 =0.000 (m)
QL1_W1= 11.600 (kN/m^2), B_G+L1= 0.500 (m), B_L= 9.000 (m)
NEAR THE TOP : Lsd = 0.7 × Ha = 4.340 (m)
NEAR THE TOP : Hsd = 0.5 × Ha = 3.100 (m)・・・< 1TH TO 2TH STAGES >
NEAR THE LOWER : Lsd = 0.4 × Ha = 2.480 ≧ 4.000 (m)
NEAR THE LOWER : Hsd = 0.3 × Ha = 1.860 (m)・・・< 5TH TO 6TH STAGES >

(2) H=3.750 (m), H4=2.100 (m), Ha=5.850 (m), H1=2.100 (m)
1 : n=1 : 0.000, Bb =0.000 (m), L1 =0.000 (m)
QL1_W1= 11.600 (kN/m^2), B_G+L1= 0.500 (m), B_L= 9.000 (m)
NEAR THE TOP : Lsd = 0.7 × Ha = 4.095 (m)
NEAR THE TOP : Hsd = 0.6 × Ha = 3.510 (m)・・・< 1TH TO 2TH STAGES >
NEAR THE LOWER : Lsd = 0.4 × Ha = 2.340 ≧ 4.000 (m)
NEAR THE LOWER : Hsd = 0.3 × Ha = 1.755 (m)・・・< 4TH TO 5TH STAGES >

(3)　Ｈ＝3.000 (m), Ｈ4＝2.200 (m), Ｈａ＝5.200 (m), Ｈ1＝2.200 (m)
１：ｎ＝1：0.000, Ｂｂ ＝0.000 (m), Ｌ1 ＝0.000 (m)
ＱL1_W1＝ 11.600 (kN/m^2), B_G＋Ｌ1＝ 0.500 (m), B_L＝ 9.000 (m)
NEAR THE TOP：Ｌsd ＝ 0.7 × Ｈａ ＝ 3.640 (m)
NEAR THE TOP：Ｈsd ＝ 0.6 × Ｈａ ＝ 3.120 (m)・・・＜ 1TH TO 1TH STAGES ＞
NEAR THE LOWER：Ｌsd ＝ 0.4 × Ｈａ ＝ 2.080 ≧ 4.000 (m)
NEAR THE LOWER：Ｈsd ＝ 0.3 × Ｈａ ＝ 1.560 (m)・・・＜ 3TH TO 4TH STAGES ＞

(4)　Ｈ＝2.250 (m), Ｈ4＝2.200 (m), Ｈａ＝4.450 (m), Ｈ1＝2.200 (m)
１：ｎ＝1：0.000, Ｂｂ ＝0.000 (m), Ｌ1 ＝0.000 (m)
ＱL1_W1＝ 11.600 (kN/m^2), B_G＋Ｌ1＝ 0.500 (m), B_L＝ 9.000 (m)
NEAR THE TOP：Ｌsd ＝ 0.7 × Ｈａ ＝ 3.115 (m)
NEAR THE TOP：Ｈsd ＝ 0.6 × Ｈａ ＝ 2.670 (m)・・・＜ 1TH TO 1TH STAGES ＞
NEAR THE LOWER：Ｌsd ＝ 0.4 × Ｈａ ＝ 1.780 ≧ 4.000 (m)
NEAR THE LOWER：Ｈsd ＝ 0.3 × Ｈａ ＝ 1.335 (m)・・・＜ 2TH TO 3TH STAGES ＞

(5)　Ｈ＝1.500 (m), Ｈ4＝2.100 (m), Ｈａ＝3.600 (m), Ｈ1＝2.100 (m)
１：ｎ＝1：0.000, Ｂｂ ＝0.000 (m), Ｌ1 ＝0.000 (m)
ＱL1_W1＝ 11.600 (kN/m^2), B_G＋Ｌ1＝ 0.500 (m), B_L＝ 9.000 (m)
NEAR THE TOP：Ｌsd ＝ 0.7 × Ｈａ ＝ 2.520 (m)
NEAR THE TOP：Ｈsd ＝ 0.6 × Ｈａ ＝ 2.160 (m)・・・＜ 1TH TO 1TH STAGES ＞
NEAR THE LOWER：Ｌsd ＝ 0.4 × Ｈａ ＝ 1.440 ≧ 4.000 (m)
NEAR THE LOWER：Ｈsd ＝ 0.3 × Ｈａ ＝ 1.080 (m)・・・＜ 2TH TO 2TH STAGES ＞

2.6 RESULTS OF EXAMINATION OF EXTERNAL STABILITY AT CROSS SECTION OF EACH STRIPSTAGE

(1) H＝4.500 (m)

・VIRTUAL RETAINING WALL R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.940 (m)

・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION : b ＝0.400 (m)

HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	7.291 (1.500)	○	2.397 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.750 (0.657)	○	-0.245 (1.313)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	187.542	—	167.860	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	172.518	—	155.761	—

(2) H＝3.750 (m)

・VIRTUAL RETAINING WALL R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.890 (m)

・THE SHAPE OF THE FOUNDATION WIDTH OF THE FOUNDATION : b ＝0.400 (m)

HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.495 (1.500)	○	2.677 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.969 (0.648)	○	-0.482 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	188.017	—	165.961	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	152.751	—	138.636	—

(3)　H＝3.000 (m)

- VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.806 (m)
- THE SHAPE OF THE FOUNDATION　WIDTH OF THE FOUNDATION : b ＝0.400 (m)

HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	20.898 (1.500)	○	3.150 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-1.241 (0.634)	○	-0.804 (1.269)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	182.069	—	157.128	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	126.888	—	115.642	—

(4)　H＝2.250 (m)

- VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL : B ＝3.890 (m)
- THE SHAPE OF THE FOUNDATION　WIDTH OF THE FOUNDATION : b ＝0.400 (m)

HEIGHT OF THE FOUNDATION : hf ＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	11.391 (1.500)	○	2.876 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.728 (0.648)	○	-0.379 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	135.366	—	116.266	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	100.132	—	91.799	—

(5)　H＝1.500 (m)

・VIRTUAL RETAINING WALL　　R/E BOTTOM WIDTH OF THE VIRTUAL WALL：B　＝3.640 (m)

・THE SHAPE OF THE FOUNDATION　WIDTH OF THE FOUNDATION　：b　＝0.400 (m)

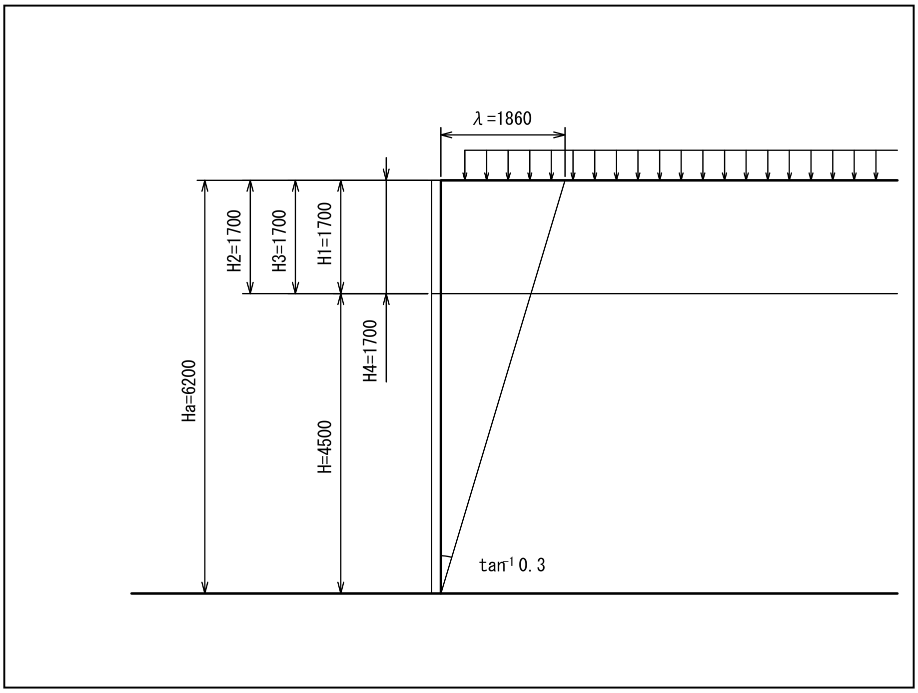
HEIGHT OF THE FOUNDATION：hf　＝0.200 (m)

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	129.169 (1.500)	○	3.940 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-1.350 (0.607)	○	-1.060 (1.213)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	138.586	—	112.750	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	72.982	—	68.659	—

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 4.500 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : H4 = 1.700 (m)

HEIGHT OF SURCHARGE BACKFILL : H1 = 1.700 (m)

SLOPE INCLINE (1∶n) : n = 0.000

3.1.2 VIRTUAL WALL HEIGHT

$$Ha = H + H2 = 6.200 \ (m)$$

Ha : VIRTUAL WALL HEIGHT (m)

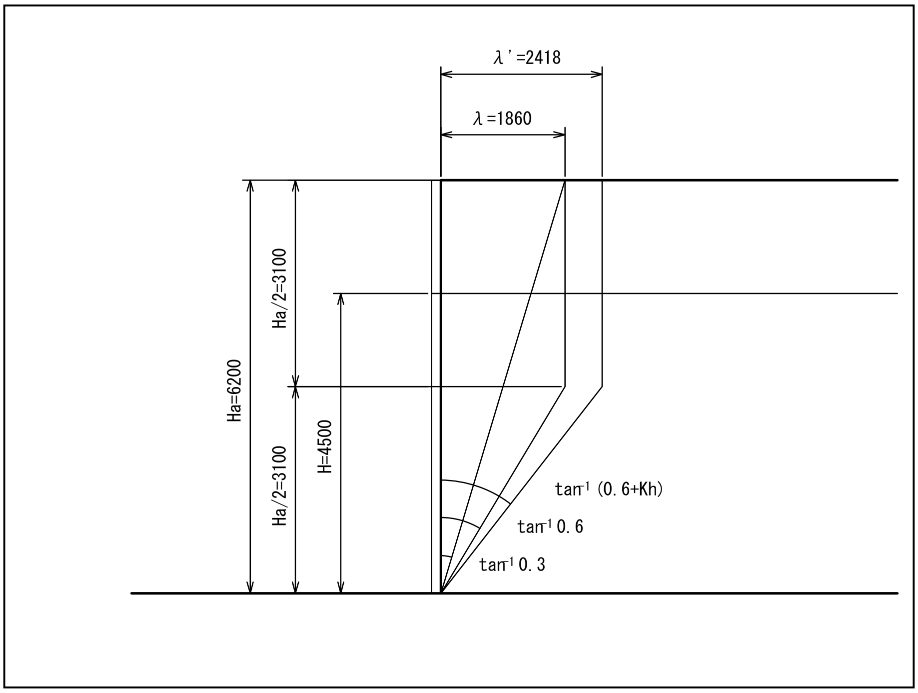
H : HEIGHT OF R/E = 4.500 (m)

H2 : DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 1.700$

H4 : HEIGHT OF COPING = 1.700 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$zi = \Delta H \cdot (i-1/2) + H2$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

$H2$: $H2 + Ha = 0.3(H+H4) - Bb / n - 0.3 + H4$ (m) = 1.700 (m)

STAGE (i)	zi (m)
1	2.075
2	2.825
3	3.575
4	4.325
5	5.075
6	5.825

3.2　LOAD

3.2.1　SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

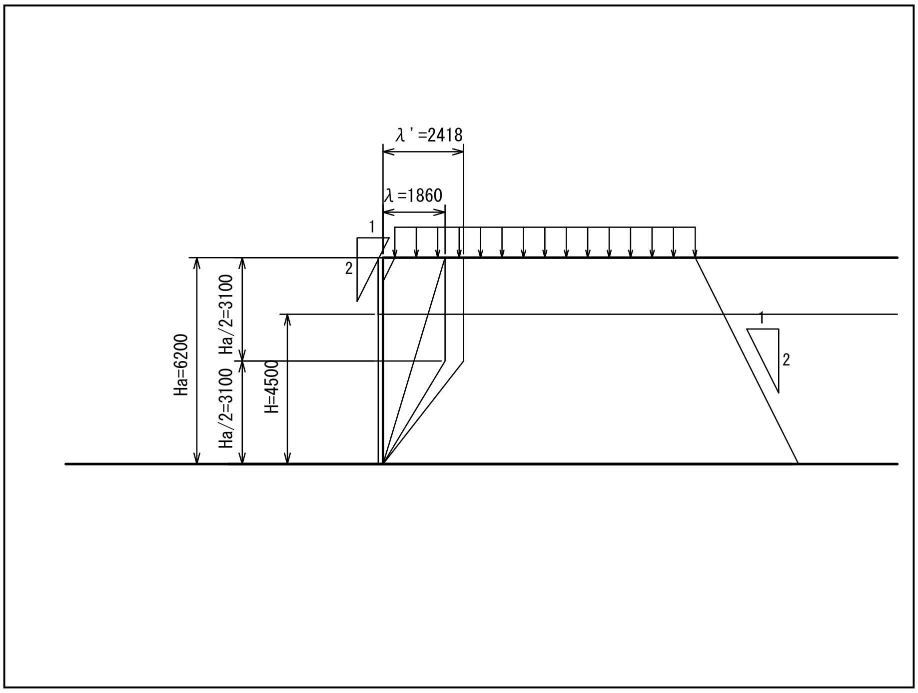
$$q_1 = \gamma_2 \cdot H_3 = 32.300 \ (\mathrm{kN/m^2})$$

q_1 : SURCHARGE SOIL LOAD ($\mathrm{kN/m^2}$)

γ_2 : UNIT WEIGHT OF SURCHARGE SOIL = 19.0 ($\mathrm{kN/m^3}$)

H_3 : ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 1.700 (m)

3.2.2 CALCULATION OF LIVE LOAD APPLIED TO EACH STAGE



THE LIVE LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA. WHEN THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE LIVE LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{Li} = q_L \cdot \frac{B_L}{B_{Li}}$$

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)
q_L : LIVE LOAD (kN/m^2)
B_L : LIVE LOAD WIDTH (m)
B_{Li} : DISPERSIVE WIDTH OF THE LIVE LOAD AT iTH STAGE STRIP (m)

(1) LIVE LOAD-1

q_L = 11.600 (kN/m^2)　　B_L = 9.000 (m)

STAGE (i)	z_i (m)	B_{Li} (m)	q_{Li} (kN/m^2)
1	2.075	10.398	10.041
2	2.825	10.773	9.691
3	3.575	11.148	9.365
4	4.325	11.523	9.061
5	5.075	11.898	8.775
6	5.825	12.273	8.507

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY (V_i) AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE (V_{Ei}).

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.075	14.324	17.179
2	2.825	11.315	13.341
3	3.575	9.351	10.916
4	4.325	7.964	9.218
5	5.075	6.937	7.983
6	5.825	6.144	7.057

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$q_i = q_1 + q_{Li} + V_i$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 32.300 (kN/m^2)

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{Li} (kN/m^2)	V_i (kN/m^2)	q_i (kN/m^2)
1	2.075	10.041	14.324	56.665
2	2.825	9.691	11.315	53.306
3	3.575	9.365	9.351	51.016
4	4.325	9.061	7.964	49.325
5	5.075	8.775	6.937	48.012
6	5.825	8.507	6.144	46.951

(2) EARTHQUAKE

$q_{Ei} = q_1 + V_{Ei}$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 32.300 (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	V_{Ei} (kN/m^2)	q_{Ei} (kN/m^2)
1	2.075	17.179	49.479
2	2.825	13.341	45.641
3	3.575	10.916	43.216
4	4.325	9.218	41.518
5	5.075	7.983	40.283
6	5.825	7.057	39.357

3.3 EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（H_i）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_{Ei}）.

STAGE (i)	z_i (m)	ORDINARY H_i (kN/m)	EARTH QUAKE H_{Ei} (kN/m)
1	2.075	4.302	7.023
2	2.825	2.363	3.399
3	3.575	0.428	0.000
4	4.325	0.000	0.000
5	5.075	0.000	0.000
6	5.825	0.000	0.000

3.4　EARTH PRESSURE

3.4.1　COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq z_o = 6.0$ (m)

$$K_i = K_o \cdot \left(1-\frac{z_i}{z_o}\right) + K_A \cdot \frac{z_i}{z_o}$$

IF $z_i > z_o = 6.0$ (m)

$$K_i = K_A$$

K_i　: COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

K_o　: COEFFICIENT OF EARTH PRESSURE AT REST

$$K_o = 1-\sin\phi$$

K_A　: COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

z_i　: STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

z_o　: DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE = 6.000 (m)

STAGE (i)	z_i (m)	ϕ (°)	K_o	K_A	K_i
1	2.075	30.0	0.5000	0.3333	0.4424
2	2.825	30.0	0.5000	0.3333	0.4215
3	3.575	30.0	0.5000	0.3333	0.4007
4	4.325	30.0	0.5000	0.3333	0.3799
5	5.075	30.0	0.5000	0.3333	0.3590
6	5.825	30.0	0.5000	0.3333	0.3382

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_i = K_i \cdot \sigma_{vi} \cdot \Delta H + H_i$$

$$T_o = 0.75 \cdot P_i$$

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

H_i : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σ_{vi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$$\sigma_{vi} = \gamma_1 \cdot x_i + q_i$$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : $x_i = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	x_i (m)	K_i	q_i (kN/m^2)	σ_{vi} (kN/m^2)	H_i (kN/m)	P_i (kN/m)	T_o (kN/m)
1	0.375	0.442	56.665	63.790	4.302	25.466	19.099
2	1.125	0.422	53.306	74.681	2.363	25.973	19.480
3	1.875	0.401	51.016	86.641	0.428	26.466	19.849
4	2.625	0.380	49.325	99.200	0.000	28.262	21.196
5	3.375	0.359	48.012	112.137	0.000	30.195	22.646
6	4.125	0.338	46.951	125.326	0.000	31.788	23.841

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = Pi + \Delta Pi$$

$$\Delta Pi = \frac{1}{2} \cdot (1 + \frac{zi}{Ha}) \cdot \alpha \cdot kh \cdot Pn$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$Pi = Ki \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$

ΔPi : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Pn : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 29.862 (kN/m)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

Ha : VIRTUAL WALL HEIGHT = 6.200 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

$kh1$: HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{VEi} = \gamma 1 \cdot xi + q_{Ei}$

$\gamma 1$: UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

xi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.442	49.479	56.604	7.023	25.803
2	1.125	0.422	45.641	67.016	3.399	24.586
3	1.875	0.401	43.216	78.841	0.000	23.693
4	2.625	0.380	41.518	91.393	0.000	26.037
5	3.375	0.359	40.283	104.408	0.000	28.114
6	4.125	0.338	39.357	117.732	0.000	29.862

STAGE (i)	z_i (m)	ΔP_i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.075	5.022	30.824	23.118
2	2.825	5.477	30.063	22.547
3	3.575	5.932	29.626	22.219
4	4.325	6.387	32.425	24.319
5	5.075	6.843	34.957	26.217
6	5.825	7.298	37.160	27.870

3.5 HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS = 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT = 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP = 80.0 (mm)

t : THE THICKNESS OF STRIP = 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP = 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP = 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER = 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

j　　：THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM ＝ 2

n　　：THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS ＝ 1 (PIECE)

Ae　　：SCREW EFFECTIVE AREA OF MOUNTING BOLTS＝ 84.3 (mm^2)

STAGE (i)	Pi (kN/m)	To (kN/m)	ΔB1i (m)	ΔB2i (m)	ΔB3i (m)
1	25.466	19.099	1.319	1.429	1.765
2	25.973	19.480	1.293	1.401	1.731
3	26.466	19.849	1.269	1.375	1.698
4	28.262	21.196	1.188	1.287	1.590
5	30.195	22.646	1.112	1.205	1.488
6	31.788	23.841	1.056	1.145	1.414

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)

τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)

P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	30.824	23.118	1.635	1.771	2.187
2	30.063	22.547	1.676	1.816	2.243
3	29.626	22.219	1.701	1.843	2.276
4	32.425	24.319	1.554	1.683	2.079
5	34.957	26.217	1.441	1.561	1.929
6	37.160	27.870	1.356	1.469	1.814

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF 1.000 ＞ $\Delta B_i \geqq 0.750$ $\Delta B_i = 0.750$

STAGE (i)	ΔB_{1i} (m)	ΔB_{2i} (m)	ΔB_{3i} (m)	ΔB_{1Ei} (m)	ΔB_{2Ei} (m)	ΔB_{3Ei} (m)	ΔB_i (m)
1	1.319	1.429	1.765	1.635	1.771	2.187	0.750
2	1.293	1.401	1.731	1.676	1.816	2.243	0.750
3	1.269	1.375	1.698	1.701	1.843	2.276	0.750
4	1.188	1.287	1.590	1.554	1.683	2.079	0.750
5	1.112	1.205	1.488	1.441	1.561	1.929	0.750
6	1.056	1.145	1.414	1.356	1.469	1.814	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	25.466	19.099	19.099	14.324	30.824	23.118	23.118	17.339
2	0.750	25.973	19.480	19.480	14.610	30.063	22.547	22.547	16.910
3	0.750	26.466	19.849	19.849	14.887	29.626	22.219	22.219	16.664
4	0.750	28.262	21.196	21.196	15.897	32.425	24.319	24.319	18.239
5	0.750	30.195	22.646	22.646	16.985	34.957	26.217	26.217	19.663
6	0.750	31.788	23.841	23.841	17.881	37.160	27.870	27.870	20.902

3.7　STRIP LENGTH

3.7.1　FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^{*} = fo^{*} \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^{*} = \tan\phi 1$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^{*} = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^{*}	$\tan\phi 1$	fi^{*}
1	2.075	1.500	0.727	1.2325
2	2.825	1.500	0.727	1.1358
3	3.575	1.500	0.727	1.0391
4	4.325	1.500	0.727	0.9425
5	5.075	1.500	0.727	0.8458
6	5.825	1.500	0.727	0.7491

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq Ha/2 = 3.100$ (m)

$$Loi = 0.3 \cdot Ha$$

IF $zi > Ha/2 = 3.100$ (m)

$$Loi = 0.6 \cdot (Ha - zi)$$

Loi : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Ha : VIRTUAL WALL HEIGHT = 6.200 (m)
zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq Ha/2 = 3.100$ (m)

$$L_{OE}i = (0.6 + kh1) \cdot (Ha/2)$$

IF $zi > Ha/2 = 3.100$ (m)

$$L_{OE}i = (0.6 + kh1) \cdot (Ha - zi)$$

$L_{OE}i$: LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)
Ha : VIRTUAL WALL HEIGHT = 6.200 (m)
zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
kh1 : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	zi (m)	Loi (m)	$L_{OE}i$ (m)
1	2.075	1.860	2.418
2	2.825	1.860	2.418
3	3.575	1.575	2.048
4	4.325	1.125	1.463
5	5.075	0.675	0.878
6	5.825	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqi} = \frac{F_s \cdot T_i}{2 \cdot f_i^* \cdot (\sigma_{vi} - V_i) \cdot b}$$

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

F_s : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 2.00

T_i : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{vi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_i (kN/PIECE)	f_i^*	$\sigma_{vi}-V_i$ (kN/m^2)	b (m)	L_{reqi} (m)
1	19.099	1.2325	49.466	0.0800	3.916
2	19.480	1.1358	63.366	0.0800	3.384
3	19.849	1.0391	77.290	0.0800	3.090
4	21.196	0.9425	91.236	0.0800	3.082
5	22.646	0.8458	105.200	0.0800	3.182
6	23.841	0.7491	119.182	0.0800	3.339

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^* \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^*	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	23.118	1.2325	39.425	0.0800	3.569
2	22.547	1.1358	53.675	0.0800	2.774
3	22.219	1.0391	67.925	0.0800	2.361
4	24.319	0.9425	82.175	0.0800	2.356
5	26.217	0.8458	96.425	0.0800	2.412
6	27.870	0.7491	110.675	0.0800	2.522

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$L_{mini} = L_{oi} + L_{reqi}$$

EARTHQUAKE

$$L_{minEi} = L_{oEi} + L_{reqEi}$$

L_{mini} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)

L_{oi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

L_{minEi} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{oEi} : LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	L_{oi} (m)	L_{reqi} (m)	L_{mini} (m)	L_{oEi} (m)	L_{reqEi} (m)	L_{minEi} (m)
1	1.860	3.916	5.776	2.418	3.569	5.987
2	1.860	3.384	5.244	2.418	2.774	5.192
3	1.575	3.090	4.665	2.048	2.361	4.409
4	1.125	3.082	4.207	1.463	2.356	3.819
5	0.675	3.182	3.857	0.878	2.412	3.290
6	0.225	3.339	3.564	0.293	2.522	2.815

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

WHEN SURCHERGE BACKFILL HEIGHT IS UNDER 2M ($0\text{m} \leqq H1 = 1.700 < 2\text{m}$)

0.5 Ha OR MORE FROM UPPER EDGE OF Ha

$L_{sd} = 0.7 \cdot Ha = 4.340$ (m)

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$L_{sd} = 0.4 \cdot Ha = 2.480$ (m)

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)
Ha : VIRTUAL WALL HEIGHT = 6.200 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)
L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)
L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)
L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	5.776	5.987	4.340	6.000
2	5.244	5.192	4.340	6.000
3	4.665	4.409	—	5.000
4	4.207	3.819	—	5.000
5	3.857	3.290	4.000	5.000
6	3.564	2.815	4.000	4.000

3.8 CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$Fsi = \frac{Si}{Ti} \geqq Fs = 2.00$$

Fsi : SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

Fs : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 2.0

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Si : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$Si = 2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σvi' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

Lei : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$Lei = Li - Loi$$

Li : DETERMINED STRIP LENGTH AT iTH STAGE (m)

Loi : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	Loi (m)	Lei (m)	fi^{*}	σvi' (kN/m^2)	b (m)	Si (kN/PIECE)	Ti (kN/PIECE)	Fsi	JUDGE
1	6.000	1.860	4.140	1.2325	49.466	0.0800	40.385	19.099	2.114	○
2	6.000	1.860	4.140	1.1358	63.366	0.0800	47.675	19.480	2.447	○
3	5.000	1.575	3.425	1.0391	77.290	0.0800	44.013	19.849	2.217	○
4	5.000	1.125	3.875	0.9425	91.236	0.0800	53.312	21.196	2.515	○
5	5.000	0.675	4.325	0.8458	105.200	0.0800	61.572	22.646	2.718	○
6	4.000	0.225	3.775	0.7491	119.182	0.0800	53.925	23.841	2.261	○

(2) EARTHQUAKE

$$F_{sEi} = \frac{S_{Ei}}{T_{Ei}} \geqq F_{sE} = 1.20$$

F_{siEi} : SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP

F_{sE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

S_{Ei} : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$S_{Ei} = 2 \cdot fi^{*} \cdot \sigma_{VEi} \cdot b \cdot L_{eEi}$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m²)

b : STRIP WIDTH AT iTH STAGE (m)

L_{eEi} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$L_{eEi} = Li - L_{OEi}$

Li : DETERMINED STRIP LENGTH AT iTH STAGE (m)

L_{OEi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	L_{OEi} (m)	L_{eEi} (m)	$f1^{*}$	σ_{VEi}' (kN/m²)	b (m)	S_{Ei} (kN/PIECE)	T_{Ei} (kN/PIECE)	F_{sEi}	JUDGE
1	6.000	2.418	3.582	1.2325	39.425	0.0800	27.849	23.118	1.204	○
2	6.000	2.418	3.582	1.1358	53.675	0.0800	34.941	22.547	1.549	○
3	5.000	2.048	2.953	1.0391	67.925	0.0800	33.344	22.219	1.500	○
4	5.000	1.463	3.538	0.9425	82.175	0.0800	43.835	24.319	1.802	○
5	5.000	0.878	4.123	0.8458	96.425	0.0800	53.793	26.217	2.051	○
6	4.000	0.293	3.708	0.7491	110.675	0.0800	49.180	27.870	1.764	○

3.9 CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \ (N/mm^2)$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	19.099	14.324	79.6	73.5	85.0	○
2	19.480	14.610	81.2	74.9	86.7	○
3	19.849	14.887	82.7	76.3	88.3	○
4	21.196	15.897	88.3	81.5	94.3	○
5	22.646	16.985	94.4	87.1	100.7	○
6	23.841	17.881	99.3	91.7	106.1	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma_{tEi} = \frac{T_{Ei}}{Ag} \leqq \sigma_{aE} = 210.0 \ (\mathrm{N/mm^2})$$

$$\sigma_{otEi} = \frac{T_{fEi}}{An} \leqq \sigma_{aE} = 210.0 \ (\mathrm{N/mm^2})$$

$$\tau_{oEi} = \frac{T_{fEi}}{A\tau} \leqq \tau_{aE} = 300.0 \ (\mathrm{N/mm^2})$$

σ_{tEi} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
σ_{otEi} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
τ_{oEi} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
σ_{aE} : ALLOWABLE TENSILE STRESS OF STRIP ($\mathrm{N/mm^2}$)
τ_{aE} : ALLOWABLE SHEARING STRESS OF BOLT ($\mathrm{N/mm^2}$)
T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
T_{fEi} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 ($\mathrm{mm^2}$)
An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 ($\mathrm{mm^2}$)
$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 ($\mathrm{mm^2}$)

STAGE (i)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)	σ_{tEi} ($\mathrm{N/mm^2}$)	σ_{otEi} ($\mathrm{N/mm^2}$)	τ_{oEi} ($\mathrm{N/mm^2}$)	JUDGE
1	23.118	17.339	96.3	88.9	102.8	○
2	22.547	16.910	93.9	86.7	100.3	○
3	22.219	16.664	92.6	85.5	98.8	○
4	24.319	18.239	101.3	93.5	108.2	○
5	26.217	19.663	109.2	100.8	116.6	○
6	27.870	20.902	116.1	107.2	124.0	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2
CASE 1-1	ORDINARY	○		○	
CASE 1-2	ORDINARY	○		○	
CASE 2-3	EARTHQUAKE	○	○		○

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

CASE 1-1 : OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)

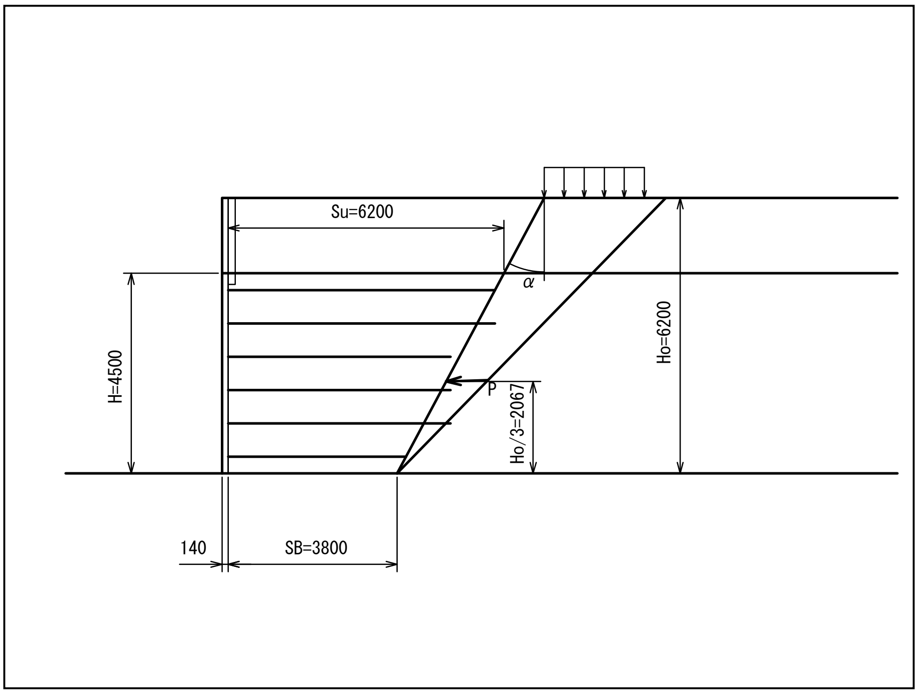
CASE 1-2 : OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 : ORDINARY LOAD

LOAD2 : EARTHQUAKELOAD

4.2 EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1 CASE 1 CALCULATION OF LOAD



Ho : WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 6.200 (m)

H : R/E WALL HEIGHT＝ 4.500 (m)

ho : CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

Su : REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 6.200 (m)

S_B : REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.800 (m)

B : R/E BOTTOM WIDTH ＝ 3.940 (m)

α : VIRTUAL BACK SLOPE INCLINE ＝ -28.072 (°)

$$\alpha = -\tan^{-1}\frac{Su - S_B}{H} = -28.072\ (^\circ)$$

W1 : OWN WEIGHT OF R/E ＝ 658.895 (kN/m)

Wq : LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2\cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = (2\cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = 0.000$ (m)

δ1 : FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

ω1 : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 45.7 (°)

$W2$: OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) = 187.398 (kN/m)

$L1$: LENGTH OF SLIP SURFACE = 8.659 (m)

P_A : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 52.306$$

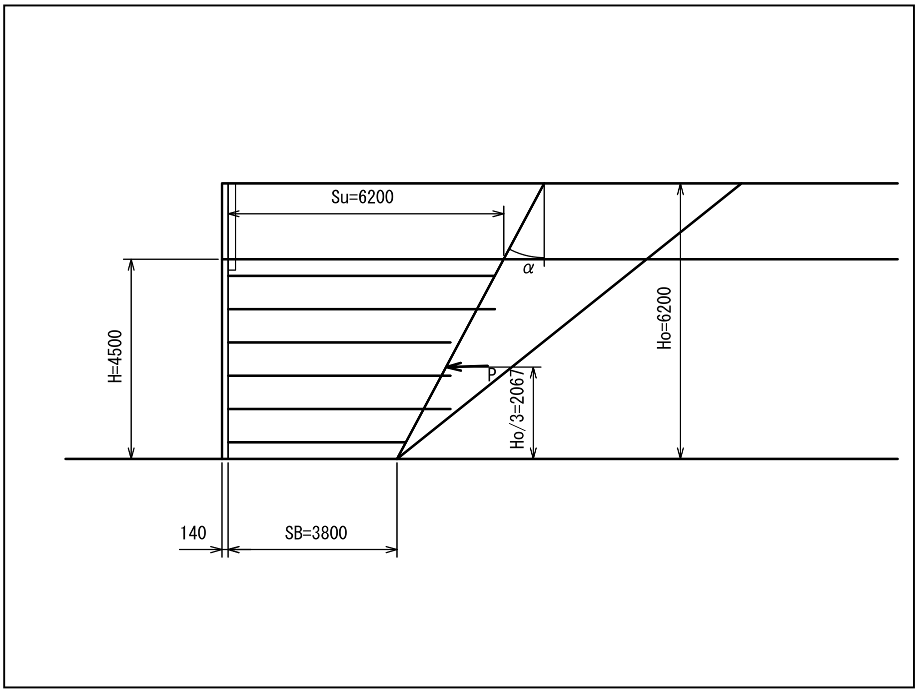
P_{Ah} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 52.277$ (kN/m)

P_{AV} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = 1.759$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK ＝ -28.072 (°)

W1 : OWN WEIGHT OF R/E ＝ 658.895 (kN/m)

Wq : LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45° + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT ＝ 0.13 (QUOTED FROM "DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY")

θ : EARTHQUAKE COMPOUND ANGLE ＝ $\tan^{-1} k_{h2}$ ＝ 7.407 (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE ＝ 38.7 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE ＝ 261.260 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE ＝ 9.919 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 73.548$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 73.506$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = 2.474$ (kN/m)

4.3 AGGREGATION OF LOAD

4.3.1 ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	658.895	2.878	1896.38	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	1.759	5.042	8.87	52.277	2.067	108.04
TOTAL	660.654		1905.25	52.277		108.04

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	658.895	2.878	1896.38	0.000	0.000	0.00
LIVE LOAD	78.261	3.873	303.13	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	1.759	5.042	8.87	52.277	2.067	108.04
TOTAL	738.915		2208.38	52.277		108.04

4.3.2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	658.895	2.878	1896.38	85.656	3.405	291.70
REAR EARTH PRESSURE 2	2.474	5.042	12.47	73.506	2.067	151.91
TOTAL	661.368		1908.85	159.162		443.61

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	658.895	2.878	1896.38	85.656	3.405	291.70
REAR EARTH PRESSURE 2	2.474	5.042	12.47	73.506	2.067	151.91
TOTAL	661.368		1908.85	159.162		443.61

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 7.291 \geqq F_{sa} = 1.50 \cdots OK$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.940 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 660.654 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 52.277 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.397 \geqq F_{saE} = 1.20 \cdots OK$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.940 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 661.368 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 159.162 (kN/m)

4.6　STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.720 \text{ (m)}$$

d　：R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

ΣMr　：R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM
＝ 1905.25 (kN･m/m)

ΣMo　：R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM
＝ 108.04 (kN･m/m)

ΣV　：TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM
＝ 660.654 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d = -0.750 \leqq B/6 = 0.657 \text{ (m)} \quad \cdots \text{ OK}$$

e　：ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B　：R/E BOTTOM WIDTH ＝ 3.940 (m)

4.7　EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.215\ (m)$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM
= 1908.85 (kN･m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM
= 443.61 (kN･m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM
= 661.368 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d3_E = -0.245 \leqq B/3 = 1.313\ (m) \quad \cdots \quad OK$$

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.940 (m)

4.8 STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_s = \Sigma V / B = 187.542$ (kN/m^2)

q_s : VERTICAL SUBGRADE REACTION ON R/E BOTTOM (kN/m^2)

ΣV : TOTAL VERTICAL LOAD ON R/E BOTTOM = 738.915 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B : R/E BOTTOM WIDTH = 3.940 (m)

4.9　STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1　VERTICAL SUBGRADE REACTION ON R/E BOTTOM

q_{SE} ＝ ΣV_E / B ＝ 167.860 (kN/m^2)

q_{SE} ：VERTICAL SUBGRADE REACTION ON R/E BOTTOM
DURING EARTHQUAKE (kN/m^2)

ΣV_E ：TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE
＝ 661.368 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B ：R/E BOTTOM WIDTH ＝ 3.940 (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma (To \cdot \tan\delta) = 45.901 \text{ (kN/m)}$$

To : TENSILE FORCE AGAINST WALL (kN/m)

δ : ANGLE OF WALL FRICTION (°) = 2/3ϕ

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	ϕ (°)	tanδ	Vci (kN/m)
1	19.099	30.0	0.364	6.952
2	19.480	30.0	0.364	7.090
3	19.849	30.0	0.364	7.224
4	21.196	30.0	0.364	7.715
5	22.646	30.0	0.364	8.243
6	23.841	30.0	0.364	8.678
TOTAL				45.901

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 172.518\ (kN/m^2)$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 21.266\ (kN/m)$

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL $= 24.5\ (kN/m^3)$

Hta : TOTAL HEIGHT OF R/E $= 6.200$ (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL $= 0.140$ (m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE $= hf \cdot b \cdot \gamma c2 = 1.840\ (kN/m)$

b : WIDTH OF CONCRETE FOUNDATION $= 0.400$ (m)

hf : HEIGHT OF CONCRETE FOUNDATION $= 0.200$ (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION $= 23.0\ (kN/m^3)$

Vc : VERTICAL COMPONENT AGAINST WALL $= 45.901\ (kN/m)$

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$V_{cE} = \Sigma (T_{oE} \cdot \tan\delta_E) = 39.198 \text{ (kN/m)}$$

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	23.118	30.0	0.268	6.195
2	22.547	30.0	0.268	6.041
3	22.219	30.0	0.268	5.954
4	24.319	30.0	0.268	6.516
5	26.217	30.0	0.268	7.025
6	27.870	30.0	0.268	7.468
TOTAL				39.198

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 155.761 \ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL = 21.266 (kN/m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE = 1.840 (kN/m)

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE = 39.198 (kN/m)

5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

5.1 ORDINARY

5.1.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{Ai} = \frac{\sigma a \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σa : ALLOWABLE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{Ai} (kN/m)
1	140.0	80.0	4.0	1.0	240.0	0.750	44.800
2	140.0	80.0	4.0	1.0	240.0	0.750	44.800
3	140.0	80.0	4.0	1.0	240.0	0.750	44.800
4	140.0	80.0	4.0	1.0	240.0	0.750	44.800
5	140.0	80.0	4.0	1.0	240.0	0.750	44.800
6	140.0	80.0	4.0	1.0	240.0	0.750	44.800

5.1.2 FORMULA FOR SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

$$F_s = \frac{R\Sigma\{c_{es}\cdot l + (W'\cdot\cos\alpha + T_{avail}\cdot\sin\theta)\tan\phi + T_{avail}\cdot\cos\theta\}}{R\Sigma(W\cdot\sin\alpha)}$$

F_s : SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

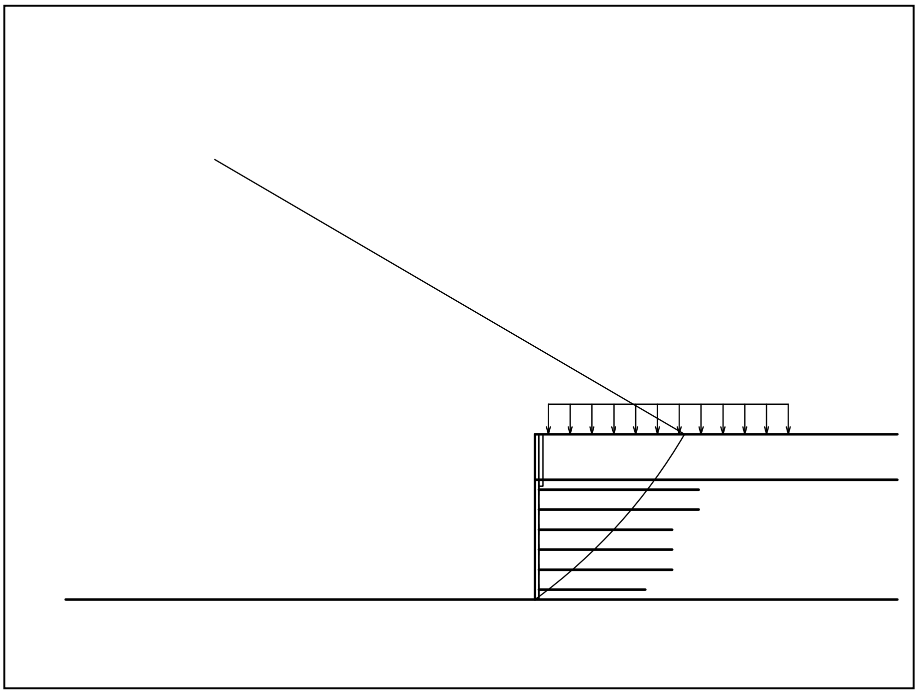
Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

R : RADIUS OF SLIP CIRCULAR ARC (m)

5.1.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [ORDINARY-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$\mathrm{Tavaili} = \min(\mathrm{Tpi},\ T_{Ai})$$

$$\mathrm{Tpi} = \frac{2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei}{Fs} \times \frac{1}{⊿B}$$

Tavaili : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

Tpi : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP (kN/m)

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

fi* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σvi : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot zi$ (kN/m^2)

zi : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

Lei : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

⊿B : HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

Fs : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{vi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pi} (kN/m)	T_{Ai} (kN/m)	T_{availi} (kN/m)
1	1.2325	19.0 19.0	1.700 0.375	39.425	0.0800	1.919	0.750	9.946	44.800	9.946
2	1.1358	19.0 19.0	1.700 1.125	53.675	0.0800	2.520	0.750	16.388	44.800	16.388
3	1.0391	19.0 19.0	1.700 1.875	67.925	0.0800	2.182	0.750	16.428	44.800	16.428
4	0.9425	19.0 19.0	1.700 2.625	82.175	0.0800	2.914	0.750	24.073	44.800	24.073
5	0.8458	19.0 19.0	1.700 3.375	96.425	0.0800	3.728	0.750	32.431	44.800	32.431
6	0.7491	19.0 19.0	1.700 4.125	110.675	0.0800	3.640	0.750	32.190	44.800	32.190
								ΣT_{avail} = 131.455		

(3) LIST OF SAFETY FACTOR

Fs		COORDINATE OF CIRCULAR CENTER X						
		-18.0	-16.0	-14.0	-12.0	-10.0	-8.0	-6.0
Y	22.5	1.362	1.367	1.380	1.404	1.446	1.519	*****
	20.5	1.365	1.361	1.367	1.383	1.412	1.468	1.586
	18.5	1.378	1.365	1.361	1.367	1.387	1.424	1.503
	16.5	1.407	1.382	1.366	1.360	1.369	1.394	1.448
	14.5	1.459	1.420	1.389	1.368	1.361	1.372	1.408
	12.5	1.550	1.491	1.440	1.399	1.372	1.364	1.381
	10.5	1.707	1.621	1.542	1.473	1.418	1.382	1.372

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	ORDINARY-1	JUDGE
MINIMUM SAFETY FACTOR	Fsmin Fs		1.360 (1.200)	○
Σ Tavail	kN/m		131.455	—
RESISTER MOMENT	$[M_{RC}]$ $[M_{RF}]$	kN･m/m	[1718.3] [3663.4]	—
	M_R	kN･m/m	5381.6	—
	M_T	kN･m/m	2999.8	—
ACTIVE MOMENT	M_D	kN･m/m	6163.4	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-12.000 16.500	—
RADIUS	R	m	20.402	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN･m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN･m/m)

M_R : RESISTER MOMENT OF SOIL $= M_{RC}+M_{RF}$ (kN･m/m)

M_T : RESISTER MOMENT OF STRIP (kN･m/m)

Fsmin : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP

$= (M_R+M_T) / M_D$

5.2 EARTHQUAKE

5.2.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{AE}i = \frac{\sigma aE \cdot Ag}{\Delta B} \times 10^{-3}$$

$T_{AE}i$: ALLOWABLE EARTHQUAKE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σaE : ALLOWABLE EARTHQUAKE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σaE (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	$T_{AE}i$ (kN/m)
1	210.0	80.0	4.0	1.0	240.0	0.750	67.200
2	210.0	80.0	4.0	1.0	240.0	0.750	67.200
3	210.0	80.0	4.0	1.0	240.0	0.750	67.200
4	210.0	80.0	4.0	1.0	240.0	0.750	67.200
5	210.0	80.0	4.0	1.0	240.0	0.750	67.200
6	210.0	80.0	4.0	1.0	240.0	0.750	67.200

5.2.2 FORMULA FOR SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

$$F_{SE} = \frac{R\Sigma\{c_{es}\cdot l + (W'\cdot\cos\alpha - k_{h3}\cdot W\cdot\sin\alpha)\tan\phi\} + R\cdot\Sigma\,Tavail(\cos\theta + \sin\theta\cdot\tan\phi)}{\Sigma(R\cdot W\cdot\sin\alpha\ +\ k_{h3}\cdot W\cdot y_G)}$$

F_{SE} : SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

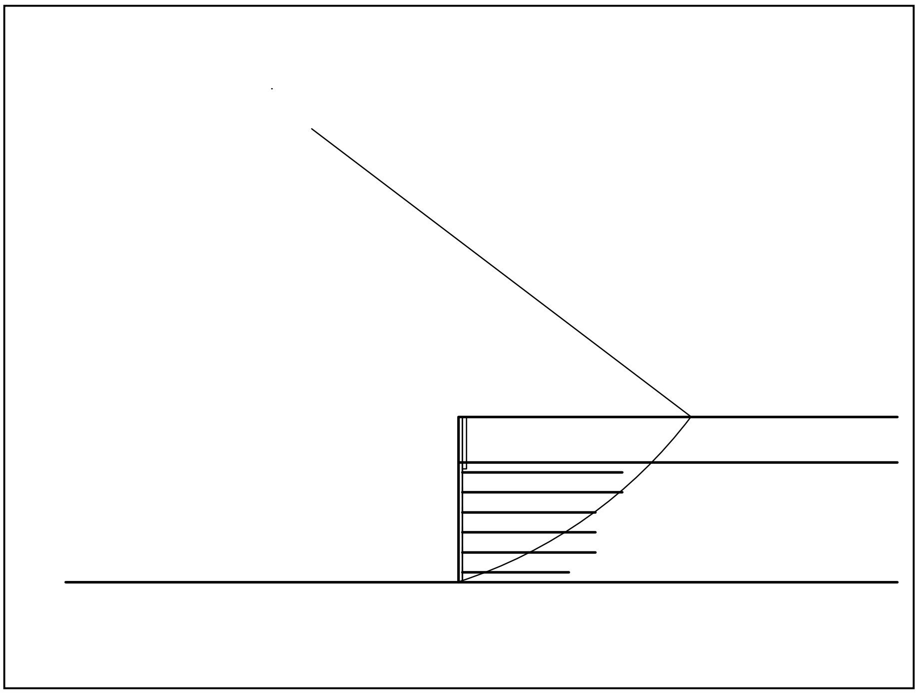
R : RADIUS OF SLIP CIRCULAR ARC (m)

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT = 0.12 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

y_G : VERTICAL DISTANCE FROM CENTER OF CIRCULAR ARC TO CENTER OF GRAVITY OF SPLIT PIECE (m)

5.2.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [EARTHQUAKE-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pEi},\ T_{AEi})$$

$$T_{pEi} = \frac{2 \cdot f_i{*} \cdot \sigma_{VEi} \cdot b \cdot L_{ei}}{F_{SE}} \times \frac{1}{\angle B}$$

Tavaili : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pEi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

T_{AEi} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

fi* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot z_i$ (kN/m^2)

z i : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

Lei : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

⊿B : HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ON THE EARTHQUAKE = 1.20

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{VEi} (kN/m^2)	b (m)	Lei (m)	⊿B (m)	T_{pEi} (kN/m)	T_{AEi} (kN/m)	Tavaili (kN/m)
1	1.2325			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
2	1.1358	19.0 19.0	1.700 1.125	53.675	0.0800	0.081	0.750	0.878	67.200	0.878
3	1.0391	19.0 19.0	1.700 1.875	67.925	0.0800	0.028	0.750	0.351	67.200	0.351
4	0.9425	19.0 19.0	1.700 2.625	82.175	0.0800	1.127	0.750	15.517	67.200	15.517
5	0.8458	19.0 19.0	1.700 3.375	96.425	0.0800	2.440	0.750	35.377	67.200	35.377
6	0.7491	19.0 19.0	1.700 4.125	110.675	0.0800	3.093	0.750	45.588	67.200	45.588
								Σ Tavail = 97.711		

(3) LIST OF SAFETY FACTOR

F_{SE}		COORDINATE OF CIRCULAR CENTER X						
		-7.0	-6.5	-6.0	-5.5	-5.0	-4.5	-4.0
Y	18.5	1.432	1.424	1.423	1.434	1.445	*****	*****
	18.0	1.436	1.428	1.420	1.428	1.439	*****	*****
	17.5	1.441	1.433	1.424	1.422	1.433	*****	*****
	17.0	1.447	1.438	1.429	1.420	1.427	*****	*****
	16.5	1.458	1.443	1.434	1.425	1.421	1.432	*****
	16.0	1.470	1.452	1.439	1.430	1.420	1.426	*****
	15.5	1.482	1.464	1.446	1.436	1.426	1.420	*****

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	EARTHQUAKE-1	JUDGE
MINIMUM SAFETY FACTOR	F_{smin} F_{SE}		1.420 (1.000)	○
HORIZONTAL SEISMIC COEFFICIENT	k_{h3}		0.12	—
ΣT_{avail}	kN/m		97.711	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN·m/m	[1943.7] [5191.3]	—
	M_R	kN·m/m	7135.0	—
	M_T	kN·m/m	2004.2	—
ACTIVE MOMENT	M_D	kN·m/m	6438.0	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-5.500 17.000	—
RADIUS	R	m	17.868	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN·m/m)

M_R : RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN·m/m)

M_T : RESISTER MOMENT OF STRIP (kN·m/m)

F_{smin} : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP
$= (M_R+M_T) / M_D$

Mechanically-Stabilized Earth Wall Type-2

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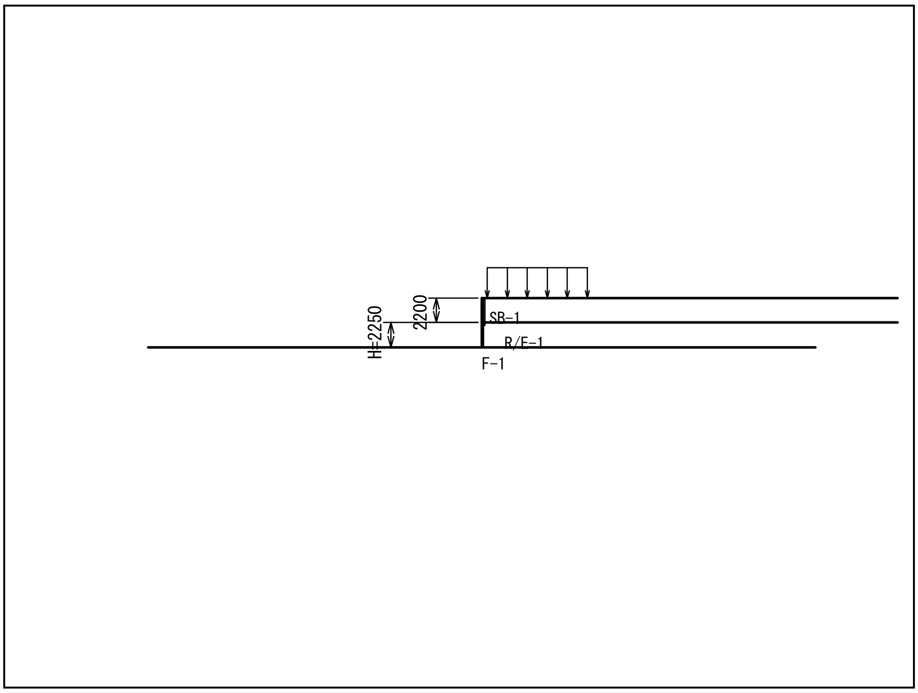
1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H = 2.250 (m)

HEIGHT OF COPING : H4 = 2.200 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	2.250	0.000	4.500

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	4.500
	2	0.000	6.700

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	2.250
	2	30.000	2.250

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE：RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 ：MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION

PERFORMANCE 2 ：KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL

PERFORMANCE 3 ：KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	ces (kN/m^2)	fo*	tanϕ1
SB－1	2.200	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	2.250	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB：SURCHARGE BACKFILL, R/E：R/E, BE：BACKSIDE EMBANKMENT, F：FOUNDATION

Hs ：LAYER THICKNESS (m)

γ ：UNIT WEIGHT OF SOIL (kN/m^3)

γ' ：UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)

ϕ ：ANGLE OF SHEARING RESISTANCE (°)

c ：COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)

ces ：COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL：ces＝10kN/m^2)

fo* ：APPARENT COEFFICIENT OF FRICTION

tanϕ_1 ：APPARENT COEFFICIENT OF FRICTION

1.5 SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP $L+B_G$(m)	WIDTH OF LOAD B_L(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	LIVE LOAD1	0.500	9.000	11.60	—

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6 HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1}= c_z \cdot k_{ho} = 0.18$

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012″

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2}= c_z \cdot k_{ho} \cdot \nu = 0.13$

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM ″ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

ν : CORRECTION COEFFICIENT = 0.70

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT hc (m)	UNIT WEIGHT γc (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σ_a=140.0	σ_{aE}=210.0
BOLT SHEAR STRESS (N/mm^2)	τ_a=200.00	τ_{aE}=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$

1.9 MATERIAL

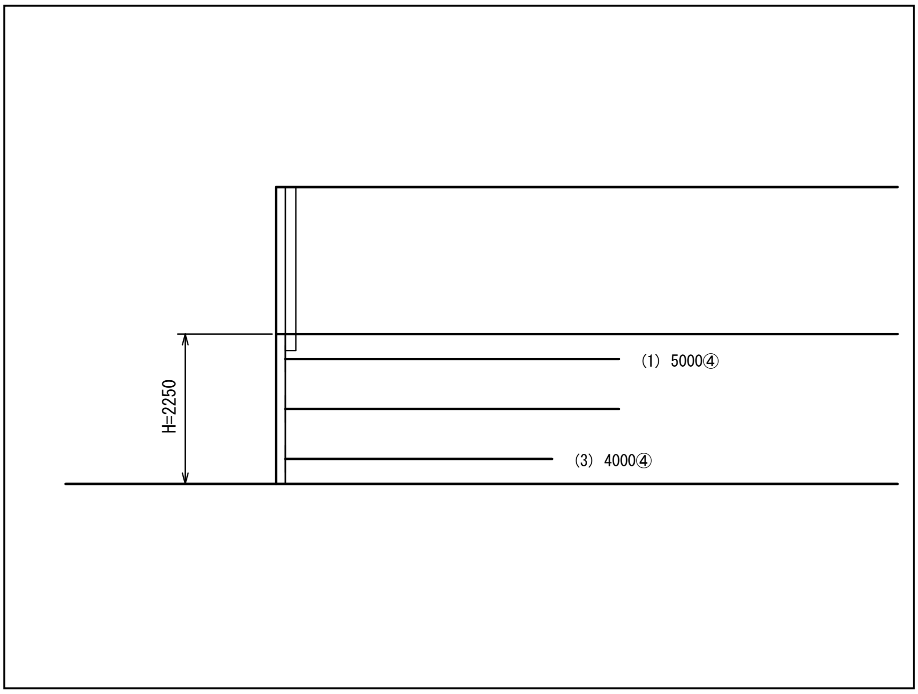
(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	A_e	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n'	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	C_m	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z i (m)	VERTICAL ΔH (m)	HORIZONTAL ΔB (m)	DESIGN LENGTH L (m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.575	0.750	0.750	5.000	4.933	4.845	2.035	1.255	○
2	3.325	0.750	0.750	5.000	4.045	4.000	2.567	1.808	○
3	4.075	0.750	0.750	4.000	4.000	4.000	2.435	1.902	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σ_{ti} (N/mm^2)	σ_{oti} (N/mm^2)	τ_{oi} (N/mm^2)	σ_{tEi} (N/mm^2)	σ_{otEi} (N/mm^2)	τ_{oEi} (N/mm^2)	
1	87.1	80.4	93.0	107.4	99.1	114.6	○
2	87.4	80.7	93.3	102.8	94.9	109.8	○
3	87.2	80.5	93.1	98.1	90.5	104.7	○

σ_{ti} , σ_{tEi} : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σ_{oti}, σ_{otEi} : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τ_{oi} , τ_{oEi} : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3　EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

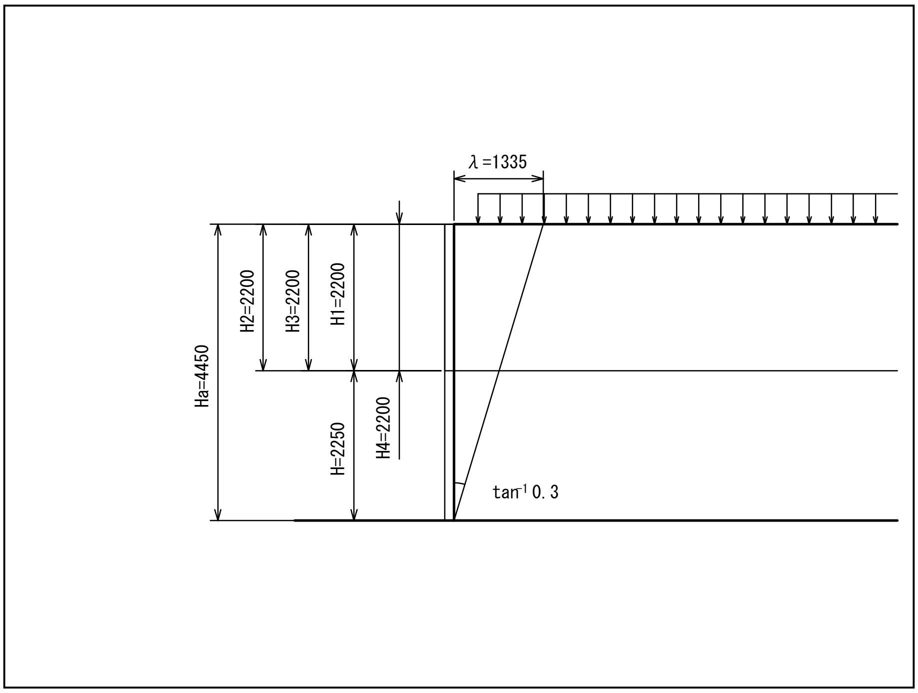
( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	11.391 (1.500)	○	2.876 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.728 (0.648)	○	-0.379 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	135.366	—	116.266	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	99.908	—	92.318	—

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 2.250 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : H4 = 2.200 (m)

HEIGHT OF SURCHARGE BACKFILL : H1 = 2.200 (m)

SLOPE INCLINE (1：n) : n = 0.000

3.1.2 VIRTUAL WALL HEIGHT

$Ha = H + H2 = 4.450$ (m)

Ha　：VIRTUAL WALL HEIGHT (m)

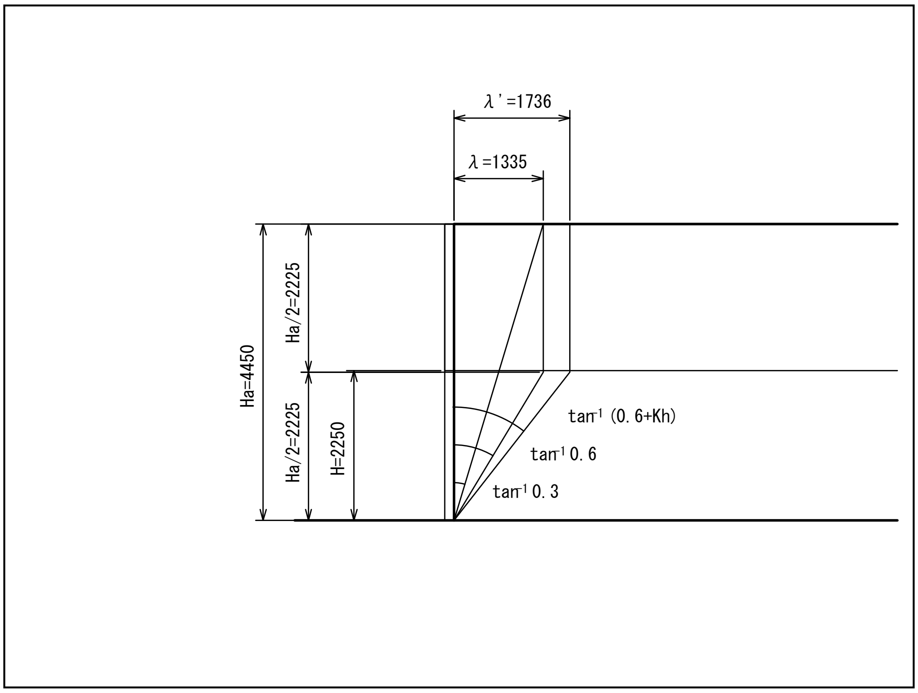
H　：HEIGHT OF R/E ＝ 2.250 (m)

H2　：DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 2.200$

H4　：HEIGHT OF COPING ＝ 2.200 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$z_i = \Delta H \cdot (i-1/2) + H2$$

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

ΔH : VERTICAL SPACING OF STRIP $= 0.750$ (m)

H2 : $H2 + Ha = 0.3(H+H4) - Bb/n - 0.3 + H4$ (m) $= 2.200$ (m)

STAGE (i)	z_i (m)
1	2.575
2	3.325
3	4.075

3.2 LOAD

3.2.1 SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

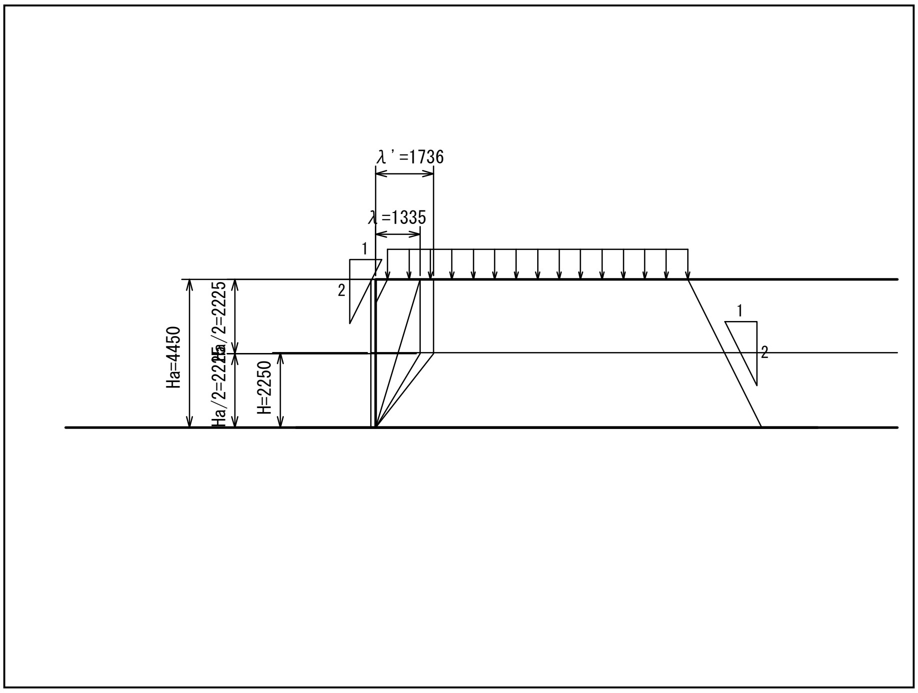
$$q1 = \gamma 2 \cdot H3 = 41.800 \ (kN/m^2)$$

$q1$: SURCHARGE SOIL LOAD (kN/m^2)

$\gamma 2$: UNIT WEIGHT OF SURCHARGE SOIL = 19.0 (kN/m^3)

$H3$: ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 2.200 (m)

3.2.2 CALCULATION OF LIVE LOAD APPLIED TO EACH STAGE



THE LIVE LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA. WHEN THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE LIVE LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{Li} = q_L \cdot \frac{B_L}{B_{Li}}$$

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_L : LIVE LOAD (kN/m^2)

B_L : LIVE LOAD WIDTH (m)

B_{Li} : DISPERSIVE WIDTH OF THE LIVE LOAD AT iTH STAGE STRIP (m)

(1) LIVE LOAD-1

q_L = 11.600 (kN/m^2)　　B_L = 9.000 (m)

STAGE (i)	z_i (m)	B_{Li} (m)	q_{Li} (kN/m^2)
1	2.575	10.648	9.805
2	3.325	11.023	9.472
3	4.075	11.398	9.160

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY (V_i) AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE (V_{Ei}).

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.575	9.599	12.981
2	3.325	7.641	10.180
3	4.075	6.352	8.368

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$$q_i = q_1 + q_{Li} + V_i$$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 41.800 (kN/m^2)

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{Li} (kN/m^2)	V_i (kN/m^2)	q_i (kN/m^2)
1	2.575	9.805	9.599	61.204
2	3.325	9.472	7.641	58.913
3	4.075	9.160	6.352	57.312

(2) EARTHQUAKE

$$q_{Ei} = q_1 + V_{Ei}$$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 41.800 (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	V_{Ei} (kN/m^2)	q_{Ei} (kN/m^2)
1	2.575	12.981	54.781
2	3.325	10.180	51.980
3	4.075	8.368	50.168

3.3　EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（H_i）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_{Ei}）.

STAGE (i)	z_i (m)	ORDINARY H_i (kN/m)	EARTH QUAKE H_{Ei} (kN/m)
1	2.575	5.903	9.436
2	3.325	3.428	4.918
3	4.075	0.945	0.387

3.4 EARTH PRESSURE

3.4.1 COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq zo = 6.0$ (m)

$$Ki = Ko \cdot \left(1 - \frac{zi}{zo}\right) + K_A \cdot \frac{zi}{zo}$$

IF $zi > zo = 6.0$ (m)

$$Ki = K_A$$

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

Ko : COEFFICIENT OF EARTH PRESSURE AT REST

$$Ko = 1 - \sin\phi$$

K_A : COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	ϕ (°)	Ko	K_A	Ki
1	2.575	30.0	0.5000	0.3333	0.4285
2	3.325	30.0	0.5000	0.3333	0.4076
3	4.075	30.0	0.5000	0.3333	0.3868

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_i = K_i \cdot \sigma_{vi} \cdot \Delta H + H_i$$

$$T_o = 0.75 \cdot P_i$$

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

H_i : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σ_{vi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{vi} = \gamma_1 \cdot x_i + q_i$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : $x_i = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	x_i (m)	K_i	q_i (kN/m^2)	σ_{vi} (kN/m^2)	H_i (kN/m)	P_i (kN/m)	T_o (kN/m)
1	0.375	0.428	61.204	68.329	5.903	27.861	20.896
2	1.125	0.408	58.913	80.288	3.428	27.974	20.981
3	1.875	0.387	57.312	92.937	0.945	27.906	20.930

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = P_i + \Delta P_i$$

$$\Delta P_i = \frac{1}{2} \cdot (1 + \frac{z_i}{H_a}) \cdot \alpha \cdot k_h \cdot P_n$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$$P_i = K_i \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$$

ΔP_i : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_n : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 25.276 (kN/m)

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

H_a : VIRTUAL WALL HEIGHT = 4.450 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$$\sigma_{VEi} = \gamma_1 \cdot x_i + q_{Ei}$$

γ_1 : UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.428	54.781	61.906	9.436	29.330
2	1.125	0.408	51.980	73.355	4.918	27.345
3	1.875	0.387	50.168	85.793	0.387	25.276

STAGE (i)	z_i (m)	ΔP_i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.575	5.028	34.357	25.768
2	3.325	5.564	32.909	24.682
3	4.075	6.101	31.377	23.533

3.5 HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP (GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS = 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT = 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP = 80.0 (mm)

t : THE THICKNESS OF STRIP = 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP = 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP = 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER = 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

j　　：THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM ＝ 2

n　　：THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS ＝ 1 (PIECE)

Ae　　：SCREW EFFECTIVE AREA OF MOUNTING BOLTS＝ 84.3 (mm^2)

STAGE (i)	Pi (kN/m)	To (kN/m)	ΔB1i (m)	ΔB2i (m)	ΔB3i (m)
1	27.861	20.896	1.205	1.306	1.613
2	27.974	20.981	1.201	1.301	1.607
3	27.906	20.930	1.204	1.304	1.611

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

・FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

・FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)

τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)

P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	34.357	25.768	1.466	1.589	1.962
2	32.909	24.682	1.531	1.659	2.049
3	31.377	23.533	1.606	1.740	2.149

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF 1.000 $>$ $\Delta Bi \geqq 0.750$　　$\Delta Bi = 0.750$

STAGE (i)	$\Delta B1i$ (m)	$\Delta B2i$ (m)	$\Delta B3i$ (m)	$\Delta B1_Ei$ (m)	$\Delta B2_Ei$ (m)	$\Delta B3_Ei$ (m)	ΔBi (m)
1	1.205	1.306	1.613	1.466	1.589	1.962	0.750
2	1.201	1.301	1.607	1.531	1.659	2.049	0.750
3	1.204	1.304	1.611	1.606	1.740	2.149	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	27.861	20.896	20.896	15.672	34.357	25.768	25.768	19.326
2	0.750	27.974	20.981	20.981	15.736	32.909	24.682	24.682	18.511
3	0.750	27.906	20.930	20.930	15.697	31.377	23.533	23.533	17.650

3.7　STRIP LENGTH

3.7.1　FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^{*} = fo^{*} \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^{*} = \tan\phi 1$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^{*} = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^{*}	$\tan\phi 1$	fi^{*}
1	2.575	1.500	0.727	1.1681
2	3.325	1.500	0.727	1.0714
3	4.075	1.500	0.727	0.9747

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 2.225$ (m)

$$L_{oi} = 0.3 \cdot Ha$$

IF $z_i > Ha/2 = 2.225$ (m)

$$L_{oi} = 0.6 \cdot (Ha - z_i)$$

L_{oi} : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
Ha : VIRTUAL WALL HEIGHT = 4.450 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq Ha/2 = 2.225$ (m)

$$L_{oEi} = (0.6 + kh1) \cdot (Ha/2)$$

IF $z_i > Ha/2 = 2.225$ (m)

$$L_{oEi} = (0.6 + kh1) \cdot (Ha - z_i)$$

L_{oEi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)
Ha : VIRTUAL WALL HEIGHT = 4.450 (m)
z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)
$kh1$: HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	z_i (m)	L_{oi} (m)	L_{oEi} (m)
1	2.575	1.125	1.463
2	3.325	0.675	0.878
3	4.075	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqi} = \frac{Fs \cdot Ti}{2 \cdot fi^* \cdot (\sigma vi - Vi) \cdot b}$$

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)
Fs : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00
Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
fi^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
b : STRIP WIDTH (m)
σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)
Vi : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	Ti (kN/PIECE)	fi^*	$\sigma vi - Vi$ (kN/m^2)	b (m)	L_{reqi} (m)
1	20.896	1.1681	58.730	0.0800	3.808
2	20.981	1.0714	72.647	0.0800	3.370
3	20.930	0.9747	86.585	0.0800	3.101

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^{*} \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^{*}	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	25.768	1.1681	48.925	0.0800	3.382
2	24.682	1.0714	63.175	0.0800	2.735
3	23.533	0.9747	77.425	0.0800	2.339

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$L_{mini} = L_{oi} + L_{reqi}$$

EARTHQUAKE

$$L_{minEi} = L_{oEi} + L_{reqEi}$$

L_{mini} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)
L_{oi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)
L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)
L_{minEi} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)
L_{oEi} : LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)
L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	L_{oi} (m)	L_{reqi} (m)	L_{mini} (m)	L_{oEi} (m)	L_{reqEi} (m)	L_{minEi} (m)
1	1.125	3.808	4.933	1.463	3.382	4.845
2	0.675	3.370	4.045	0.878	2.735	3.613
3	0.225	3.101	3.326	0.293	2.339	2.632

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

IF SURCHARGE BACKFILL REACHES 2 m OR HIGHER ($H1 = 2.200 \geqq 2\,m$)

0.6 Ha OR MORE FROM UPPER EDGE OF Ha

$$L_{sd} = 0.7 \cdot Ha = 3.115 \text{ (m)}$$

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$$L_{sd} = 0.4 \cdot Ha = 1.780 \text{ (m)}$$

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)
Ha : VIRTUAL WALL HEIGHT = 4.450 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)

L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)

L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)

L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	4.933	4.845	3.115	5.000
2	4.045	3.613	4.000	5.000
3	3.326	2.632	4.000	4.000

3.8　CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$F_{si} = \frac{S_i}{T_i} \geqq F_s = 2.00$$

F_{si} : SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

F_s : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 2.0

T_i : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

S_i : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$S_i = 2 \cdot f_i^* \cdot \sigma_{vi} \cdot b \cdot L_{ei}$$

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{vi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

L_{ei} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$L_{ei} = L_i - L_{oi}$$

L_i : DETERMINED STRIP LENGTH AT iTH STAGE (m)

L_{oi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	L_i (m)	L_{oi} (m)	L_{ei} (m)	f_i^*	σ_{vi}' (kN/m^2)	b (m)	S_i (kN/PIECE)	T_i (kN/PIECE)	F_{si}	JUDGE
1	5.000	1.125	3.875	1.1681	58.730	0.0800	42.532	20.896	2.035	○
2	5.000	0.675	4.325	1.0714	72.647	0.0800	53.860	20.981	2.567	○
3	4.000	0.225	3.775	0.9747	86.585	0.0800	50.974	20.930	2.435	○

(2) EARTHQUAKE

$$F_{SEi} = \frac{S_{Ei}}{T_{Ei}} \geqq F_{SE} = 1.20$$

F_{siEi} : SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP

F_{SE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

S_{Ei} : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$S_{Ei} = 2 \cdot f_i^* \cdot \sigma_{VEi} \cdot b \cdot L_{eEi}$$

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

L_{eEi} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$L_{eEi} = L_i - L_{OEi}$$

L_i : DETERMINED STRIP LENGTH AT iTH STAGE (m)

L_{OEi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	L_i (m)	L_{OEi} (m)	L_{eEi} (m)	f_1^*	σ_{VEi}' (kN/m^2)	b (m)	S_{Ei} (kN/PIECE)	T_{Ei} (kN/PIECE)	F_{SEi}	JUDGE
1	5.000	1.463	3.538	1.1681	48.925	0.0800	32.345	25.768	1.255	○
2	5.000	0.878	4.123	1.0714	63.175	0.0800	44.644	24.682	1.808	○
3	4.000	0.293	3.708	0.9747	77.425	0.0800	44.766	23.533	1.902	○

3.9　CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \ (N/mm^2)$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	20.896	15.672	87.1	80.4	93.0	○
2	20.981	15.736	87.4	80.7	93.3	○
3	20.930	15.697	87.2	80.5	93.1	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma t_{Ei} = \frac{T_{Ei}}{Ag} \leqq \sigma a_{E} = 210.0 \ (\mathrm{N/mm^2})$$

$$\sigma ot_{Ei} = \frac{Tf_{Ei}}{An} \leqq \sigma a_{E} = 210.0 \ (\mathrm{N/mm^2})$$

$$\tau o_{Ei} = \frac{Tf_{Ei}}{A\tau} \leqq \tau a_{E} = 300.0 \ (\mathrm{N/mm^2})$$

σt_{Ei} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

σot_{Ei} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

τo_{Ei} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)

σa_{E} : ALLOWABLE TENSILE STRESS OF STRIP ($\mathrm{N/mm^2}$)

τa_{E} : ALLOWABLE SHEARING STRESS OF BOLT ($\mathrm{N/mm^2}$)

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tf_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 ($\mathrm{mm^2}$)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 ($\mathrm{mm^2}$)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 ($\mathrm{mm^2}$)

STAGE (i)	T_{Ei} (kN/PIECE)	Tf_{Ei} (kN/PIECE)	σt_{Ei} ($\mathrm{N/mm^2}$)	σot_{Ei} ($\mathrm{N/mm^2}$)	τo_{Ei} ($\mathrm{N/mm^2}$)	JUDGE
1	25.768	19.326	107.4	99.1	114.6	○
2	24.682	18.511	102.8	94.9	109.8	○
3	23.533	17.650	98.1	90.5	104.7	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2
CASE 1-1	ORDINARY	○		○	
CASE 1-2	ORDINARY	○		○	
CASE 2-3	EARTHQUAKE	○	○		○

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

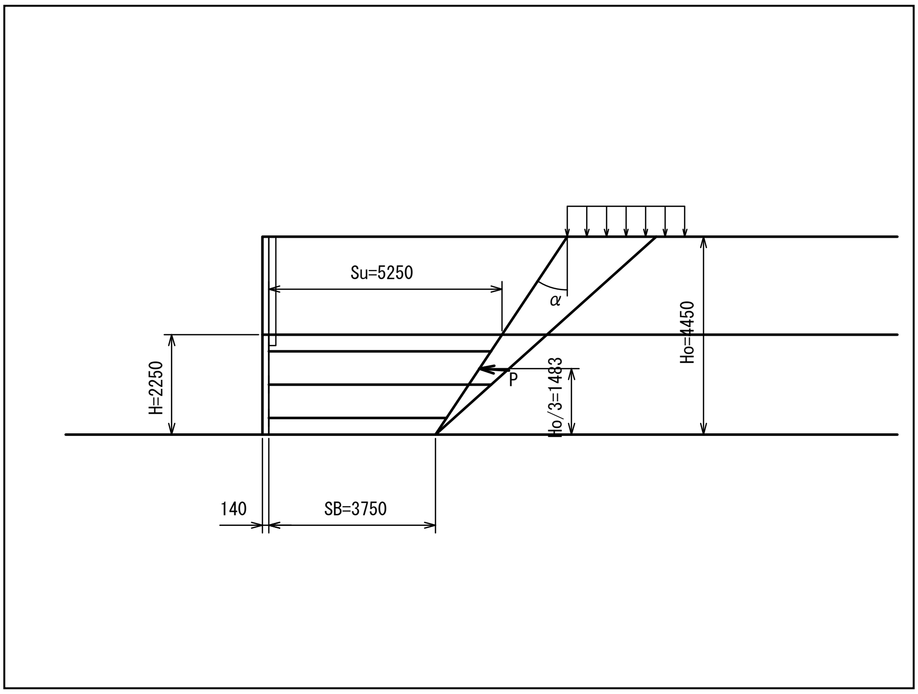
CASE 1-1	：OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)
CASE 1-2	：OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 ：ORDINARY LOAD

LOAD2 ：EARTHQUAKELOAD

4.2 EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1 CASE 1 CALCULATION OF LOAD



Ho : WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 4.450 (m)

H : R/E WALL HEIGHT＝ 2.250 (m)

ho : CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

Su : REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 5.250 (m)

S_B : REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.750 (m)

B : R/E BOTTOM WIDTH ＝ 3.890 (m)

α : VIRTUAL BACK SLOPE INCLINE ＝ -33.690 (°)

$$\alpha = -\tan^{-1}\frac{Su - S_B}{H} = -33.690\ (°)$$

W1 : OWN WEIGHT OF R/E ＝ 454.315 (kN/m)

Wq : LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c : COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma)\cdot\tan(45° + \phi/2) = (2 \cdot c/\gamma)\cdot\tan(45° + \phi/2) = 0.000$ (m)

$\delta 1$: FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

$\omega 1$: ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 41.9 (°)

W2　: OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) ＝ 107.652 (kN/m)

L1　: LENGTH OF SLIP SURFACE ＝ 6.667 (m)

P_A　: ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 22.985$$

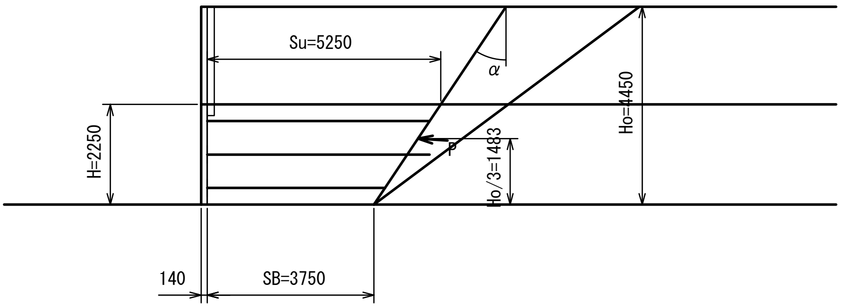
P_{Ah}　: HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 22.938$ (kN/m)

P_{AV}　: VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = -1.479$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK = -33.690 (°)

W1 : OWN WEIGHT OF R/E = 454.315 (kN/m)

Wq : LOADED WEIGHT ON R/E = 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL = 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL = 30.0 (°)

c : COHESION OF BACK FILL = 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45° + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK = 30.000 (°)

$kh2$: HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY")

θ : EARTHQUAKE COMPOUND ANGLE = $\tan^{-1} kh2$ = 7.407 (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE = 36.7 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE = 126.880 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE = 7.444 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 31.728$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 31.662$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = -2.042$ (kN/m)

4.3 AGGREGATION OF LOAD

4.3.1 ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	454.315	2.755	1251.60	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-1.479	4.879	-7.22	22.938	1.483	34.02
TOTAL	452.836		1244.38	22.938		34.02

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	454.315	2.755	1251.60	0.000	0.000	0.00
LIVE LOAD	73.737	3.678	271.23	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-1.479	4.879	-7.22	22.938	1.483	34.02
TOTAL	526.573		1515.61	22.938		34.02

4.3.2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	454.315	2.755	1251.60	59.061	2.430	143.50
REAR EARTH PRESSURE 2	-2.042	4.879	-9.96	31.662	1.483	46.97
TOTAL	452.273		1241.64	90.723		190.47

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	454.315	2.755	1251.60	59.061	2.430	143.50
REAR EARTH PRESSURE 2	-2.042	4.879	-9.96	31.662	1.483	46.97
TOTAL	452.273		1241.64	90.723		190.47

k h2 : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 11.391 \geqq F_{sa} = 1.50 \cdot\cdot\cdot \text{OK}$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 452.836 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 22.938 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.876 \geqq F_{saE} = 1.20 \cdot\cdot\cdot \text{OK}$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 452.273 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 90.723 (kN/m)

4.6 STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.673 \ (m)$$

d : R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

ΣMr : R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1244.38 (kN·m/m)

ΣMo : R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 34.02 (kN·m/m)

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 452.836 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d = -0.728 \leqq B/6 = 0.648 \ (m) \quad \cdots \ OK$$

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.7 EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.324 \text{ (m)}$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1241.64 (kN･m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 190.47 (kN･m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 452.273 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d3_E = -0.379 \leqq B/3 = 1.297 \text{ (m)} \quad \cdots \text{ OK}$$

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.8　STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1　VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$$qs = \Sigma V / B = 135.366 \ (kN/m^2)$$

qs　：VERTICAL SUBGRADE REACTION ON R/E BOTTOM (kN/m^2)

ΣV　：TOTAL VERTICAL LOAD ON R/E BOTTOM = 526.573 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B　：R/E BOTTOM WIDTH = 3.890 (m)

4.9　STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1　VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$$q_{SE} = \Sigma V_E / B = 116.266 \ (kN/m^2)$$

q_{SE} : VERTICAL SUBGRADE REACTION ON R/E BOTTOM
DURING EARTHQUAKE (kN/m^2)

ΣV_E : TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE
= 452.273 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma (To \cdot \tan\delta) = 22.860 \text{ (kN/m)}$$

To : TENSILE FORCE AGAINST WALL (kN/m)
δ : ANGLE OF WALL FRICTION (°) = 2/3φ
φ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	φ (°)	tanδ	Vci (kN/m)
1	20.896	30.0	0.364	7.605
2	20.981	30.0	0.364	7.636
3	20.930	30.0	0.364	7.618
TOTAL				22.860

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 99.908\ (kN/m^2)$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

Wc1 : THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 15.264\ (kN/m)$

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL = 24.5 (kN/m^3)

Hta : TOTAL HEIGHT OF R/E = 4.450 (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL = 0.140 (m)

Wc2 : WEIGHT OF FOUNDATION CONCRETE = $hf \cdot b \cdot \gamma c2$ = 1.840 (kN/m)

b : WIDTH OF CONCRETE FOUNDATION = 0.400 (m)

hf : HEIGHT OF CONCRETE FOUNDATION = 0.200 (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION = 23.0 (kN/m^3)

Vc : VERTICAL COMPONENT AGAINST WALL = 22.860 (kN/m)

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$V_{cE} = \Sigma \left(T_{oE} \cdot \tan\delta_E \right) = 19.824 \text{ (kN/m)}$$

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	25.768	30.0	0.268	6.905
2	24.682	30.0	0.268	6.613
3	23.533	30.0	0.268	6.306
TOTAL				19.824

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 92.318 \ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL = 15.264 (kN/m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE = 1.840 (kN/m)

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE = 19.824 (kN/m)

Mechanically-Stabilized Earth Wall Type-3

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5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

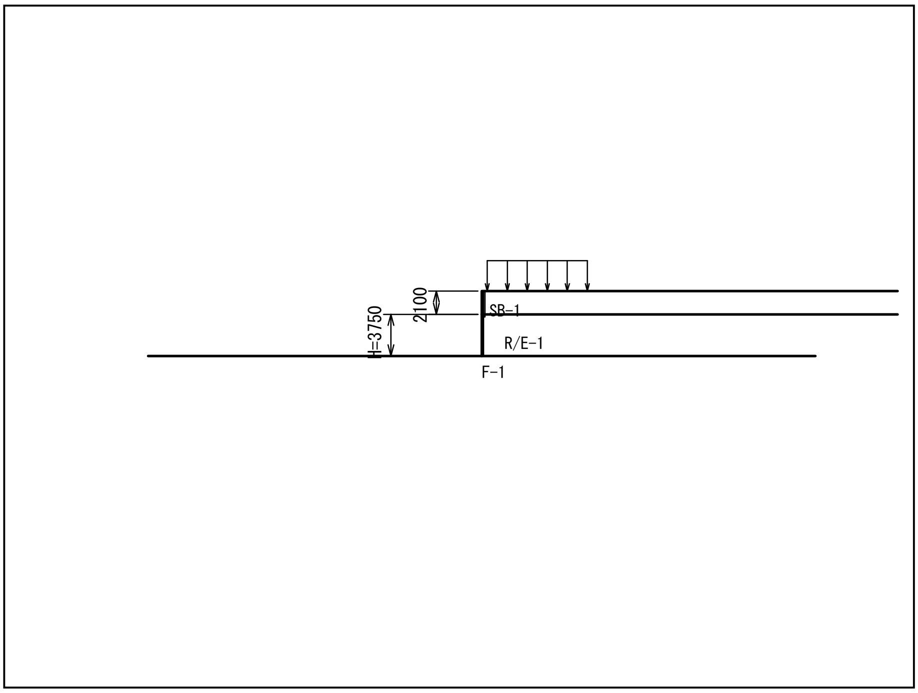
5.1 ORDINARY

5.2 EARTHQUAKE

1. DESIGN CONDITION

1.1 DIMENSION OF R/E AND FOUNDATION

(1) R/E DIMENSION



TOTAL HEIGHT OF THE WALL PANEL OF R/E : H = 3.750 (m)

HEIGHT OF COPING : H4 = 2.100 (m)

(2) COORDINATE OF R/E

R/E NO.	BOOTOM COORDINATE OF R/E		TOP COORDINATE OF R/E	
	XL (m)	YL (m)	XU (m)	YU (m)
R/E－1	0.000	0.750	0.000	4.500

(3) COORDINATES OF THE EMBANKMENT SHAPE

NO.	COORDINATE NO.	X (m)	Y (m)
SB－1	1	0.000	4.500
	2	0.000	6.600

(4) COORDINATE OF FOUNDATION

FOUNDATION NO.	COORDINATE NO.	X (m)	Y (m)
F－1	1	-30.000	0.750
	2	30.000	0.750

1.2 THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE

THE CORRESPONDING IMPORTANCE CLASSIFICATION OF THE STRUCTURE : RANK 2

1.3 PERFORMANCE REQUIRED FOR THE CONSTRUCTION

IMPORTANCE CLASSIFICATION EXPECTED ACTION		RANK 2
ACTION - ORDINARY		PERFORMANCE 1
ACTION - RAIN		PERFORMANCE 1
ACTION - EARTHQUAKE MOTION	LEVEL 1 EARTHQUAKE MOTION	PERFORMANCE 2
	LEVEL 2 EARTHQUAKE MOTION	PERFORMANCE 3

WHERE, PERFORMANCE 1 : MAINTAINS INTEGRITY OF RETAINING WALL INTACT IF SUBJECTED TO EXPECTED ACTION

PERFORMANCE 2 : KEEPS DAMAGE BY EXPECTED ACTION LIMITED, AND ENABLES PROMPT RECOVERY OF FUNCTIONS OF RETAINING WALL

PERFORMANCE 3 : KEEPS EXPECTED ACTION FROM INFLICTING CRITICAL DAMAGE ON RETAINING WALL

1.4 SOIL CHARACTER FOR BACKFILL MATERIAL AND FOUNDATION

LAYER NO	Hs (m)	γ (kN/m^3)	γ' (kN/m^3)	ϕ (°)	c (kN/m^2)	c es (kN/m^2)	fo^*	$\tan\phi 1$
SB－1	2.100	19.0	10.0	30.0	0.0	10.0	—	—
R/E－1	3.750	19.0	10.0	30.0	0.0	10.0	1.50	0.727
F－1	—	19.0	10.0	30.0	0.0	—	—	—

SB : SURCHARGE BACKFILL, R/E : R/E, BM : BACKSIDE EMBANKMENT, F : FOUNDATION

Hs : LAYER THICKNESS (m)

γ : UNIT WEIGHT OF SOIL (kN/m^3)

γ' : UNIT WEIGHT OF SOIL UNDER WATER (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE (°)

c : COHESION OF SOIL DURING EXAMINATION OF INTERNAL OR EXTERNAL STABILITY (kN/m^2)

c es : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)

(FOR SANDY SOIL : $c_{es} = 10kN/m^2$)

fo^* : APPARENT COEFFICIENT OF FRICTION

$\tan\phi_1$: APPARENT COEFFICIENT OF FRICTION

1.5　SURCHARGE

LOAD NO.	LOAD TYPE	DISTANCE FROM WALL TOP L+B$_G$(m)	WIDTH OF LOAD B$_L$(m)	LOAD q (kN/m^2)	
				ORDINARY	EARTHQUAKE
L－1	LIVE LOAD1	0.500	9.000	11.60	—

REFER TO THE DETAIL CALCULATION, AS REGARDING VERTICAL LOAD AND HORIZONTAL LOAD WHICH AEFFECTS EACH STRIPS

1.6　HORIZONTAL SEISMIC COEFFICIENT

(1) EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

$k_{h1} = c_z \cdot k_{ho} = 0.18$

k_{h1} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF MATERIAL

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

(2) EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

$k_{h2} = c_z \cdot k_{ho} \cdot \nu = 0.13$

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT IN EXAMINATION OF STABILITY OF R/E ITSELF

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.18

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR RETAINING WALL CONSTRUCTION 2012

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

ν : CORRECTION COEFFICIENT = 0.70

(3) OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

$k_{h3} = c_z \cdot k_{ho} = 0.12$

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT

k_{ho} : STANDARD HORIZONTAL SEISMIC COEFFICIENT = 0.12

[CLASS OF FOUNDATION : LEVEL 1 EARTHQUAKE MOTIONⅢ]

※ QUOTED FROM "ROAD EARTHWORK: GUIDELINES FOR EMBANKMENT CONSTRUCTION 2012"

c_z : AREA CORRECTION COEFFICIENT = 1.00 [AREA : A]

1.7 CONCRETE FOUNDATION

WIDTH b (m)	HEIGHT h_c (m)	UNIT WEIGHT γ_c (kN/m^3)
0.400	0.200	23.0

1.8 ALLOWABLE STRESS AND DESIGN SAFETY FACTOR

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS (N/mm^2)	σ_a=140.0	σ_{aE}=210.0
BOLT SHEAR STRESS (N/mm^2)	τ_a=200.00	τ_{aE}=300.00
STRIP PULL OUT	$F_s \geqq 2.00$	$F_{sE} \geqq 1.20$
SLIDING	$F_s \geqq 1.50$	$F_{sE} \geqq 1.20$
OVER-TURNING	$e \leqq L/6$	$e \leqq L/3$
BEARING CAPACITY	$F_s \geqq 3.00$	$F_{sE} \geqq 2.00$
CIRCULAR SLIP	$F_s \geqq 1.20$	$F_{sE} \geqq 1.00$

1.9 MATERIAL

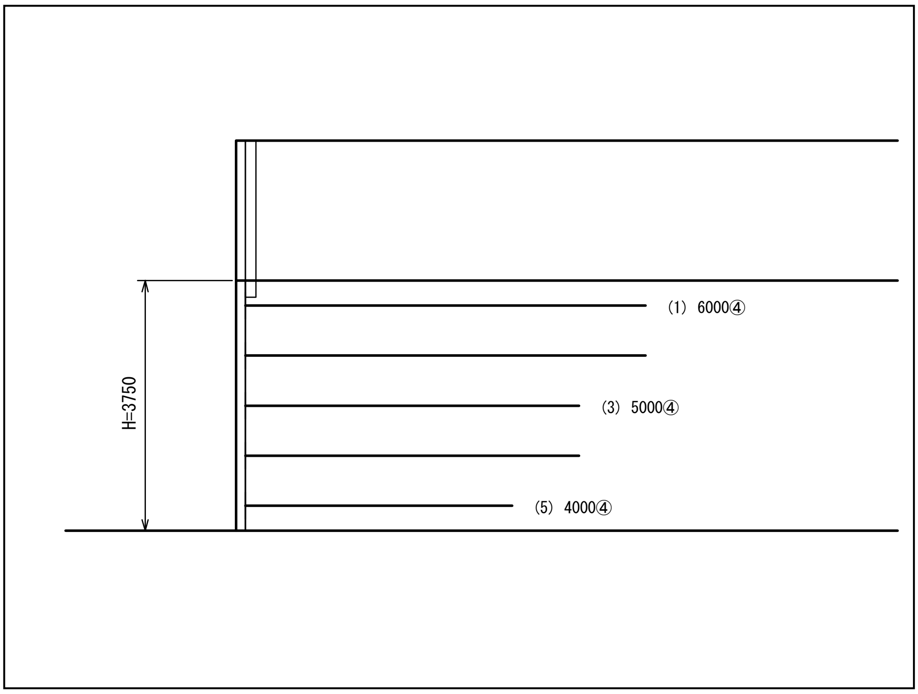
(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	A_e	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n'	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	C_m	mm	1.0 (1.5)

2. OVERVIEW OF CALCULATION

2.1 STRIP ARRANGEMENT



2.2 EXAMINATION OF STABILITY OF MATERIAL (EXAMINATION OF INTERNAL STABILITY)

(1) STRIP LENGTH AND SAFETY FACTOR OF PULLOUT

STAGE (i)	DEPTH z_i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB (m)	DESIGN LENGTH L (m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY L_r(m)	EARTH QUAKE L_{rE}(m)	ORDINARY F_s	EARTH QUAKE F_{SE}	
1	2.475	0.750	0.750	6.000	5.294	5.473	2.398	1.398	○
2	3.225	0.750	0.750	6.000	4.762	4.655	2.777	1.819	○
3	3.975	0.750	0.750	5.000	4.095	3.712	2.610	1.888	○
4	4.725	0.750	0.750	5.000	4.000	4.000	2.900	2.178	○
5	5.475	0.750	0.750	4.000	4.000	4.000	2.405	1.874	○

(2) DEGREE OF MATERIAL STRESS

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σ_{ti} (N/mm^2)	σ_{oti} (N/mm^2)	τ_{oi} (N/mm^2)	σ_{tEi} (N/mm^2)	σ_{otEi} (N/mm^2)	τ_{oEi} (N/mm^2)	
1	79.2	73.1	84.6	98.4	90.9	105.1	○
2	81.5	75.3	87.0	96.2	88.8	102.7	○
3	82.8	76.4	88.4	93.2	86.0	99.5	○
4	87.4	80.7	93.3	100.9	93.1	107.7	○
5	93.6	86.4	99.9	108.9	100.6	116.3	○

σ_{ti} , σ_{tEi} : TENSILE STRESS OF GENERAL SECTION OF STRIP (N/mm^2)

σ_{oti}, σ_{otEi} : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

τ_{oi} , τ_{oEi} : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT (N/mm^2)

2.3 EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	10.495 (1.500)	○	2.677 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-0.969 (0.648)	○	-0.482 (1.297)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m^2)	188.017	—	165.961	—
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m^2)	147.481	—	134.767	—

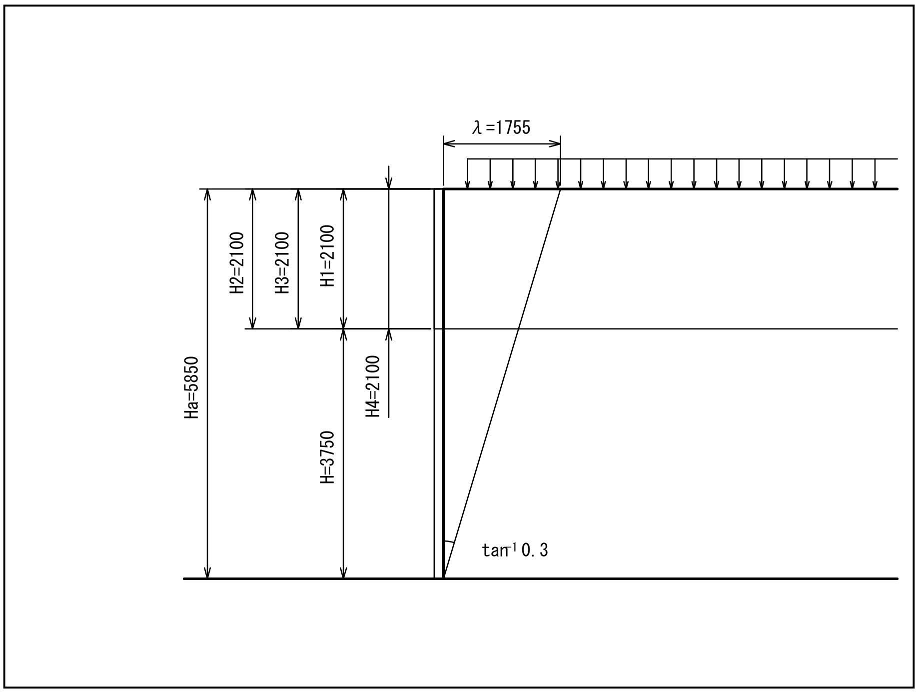
2.4 OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R(m)	Fsmin	Fs	JUDGE
		X(m)	Y(m)				
WITHIN REINFORCED AREA	ORDINARY-1	-13.000	17.000	20.810	1.403	1.200	○
	EARTHQUAKE-1	-4.500	17.000	16.862	1.516	1.000	○

3. INTERNAL STABILITY

3.1 SIZE AND DIMENSION

3.1.1 DESIGN CROSS SECTION



HEIGHT OF R/E : H = 3.750 (m)

VERTICAL INTERVAL OF STRIP : ΔH = 0.750 (m)

HEIGHT OF COPING : H4 = 2.100 (m)

HEIGHT OF SURCHARGE BACKFILL : H1 = 2.100 (m)

SLOPE INCLINE (1：n) : n = 0.000

3.1.2 VIRTUAL WALL HEIGHT

$Ha = H + H2 = 5.850$ (m)

Ha : VIRTUAL WALL HEIGHT (m)

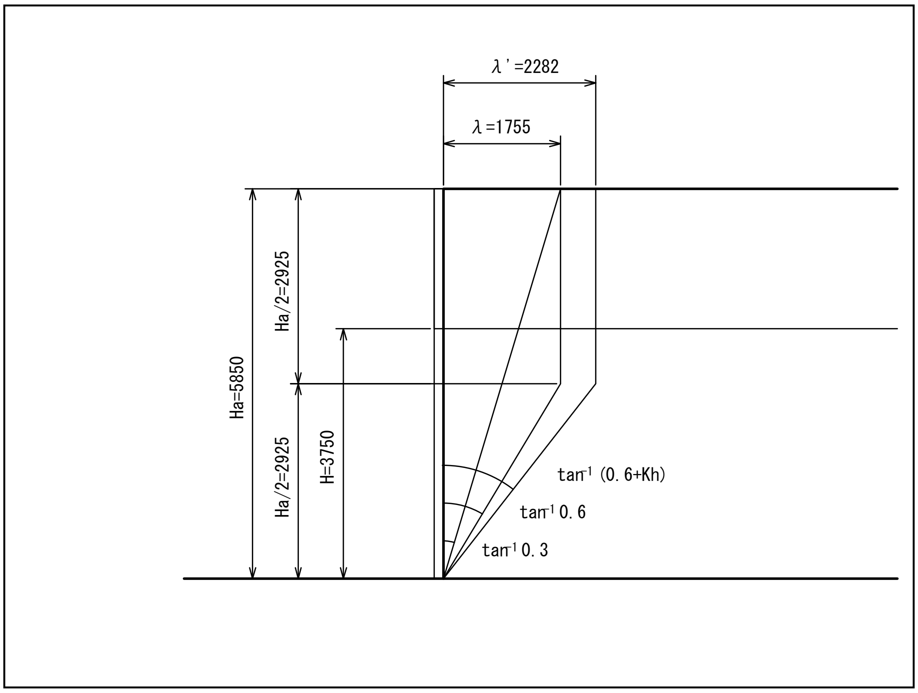
H : HEIGHT OF R/E = 3.750 (m)

H2 : DIFFERENCE BETWEEN VIRTUAL WALL HEIGHT AND HEIGHT OF R/E (m)

$H2 = H4 = 2.100$

H4 : HEIGHT OF COPING = 2.100 (m)

3.1.3 DETERMINATION OF ACTIVE AREA



3.1.4 DEPTH OF EACH STRIP FROM THE TOP OF VIRTUAL WALL HEIGHT

$$z_i = \Delta H \cdot (i-1/2) + H2$$

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

H2 : $H2 + Ha = 0.3(H+H4) - Bb/n - 0.3 + H4$ (m) = 2.100 (m)

STAGE (i)	z_i (m)
1	2.475
2	3.225
3	3.975
4	4.725
5	5.475

3.2　LOAD

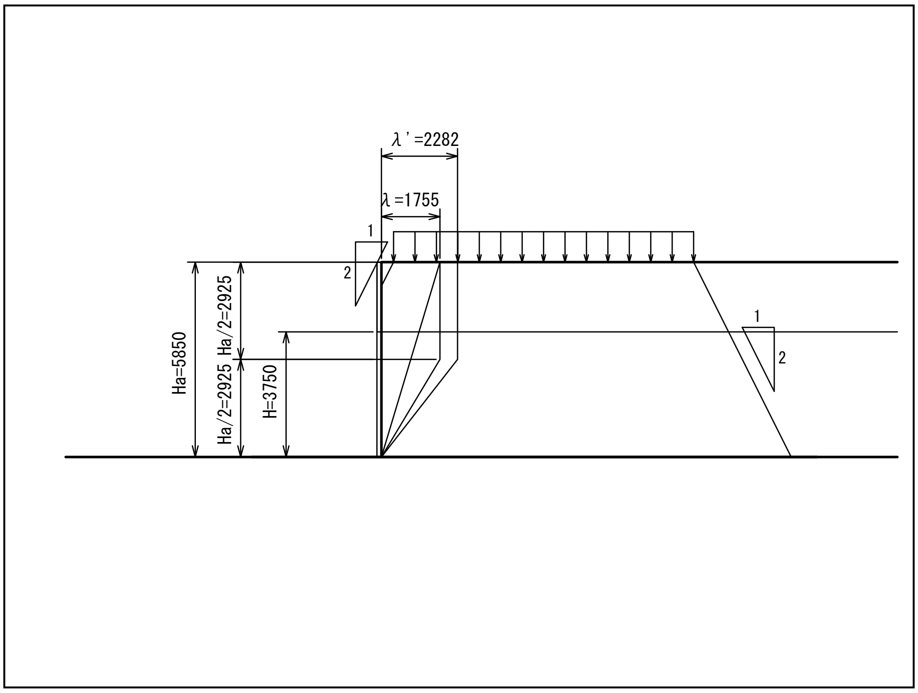
3.2.1　SURCHARGE SOIL LOAD

THE LOAD OF THE SURCHARGE SOIL IS THE EFFECTIVE SURCHARGE LOAD WHICH HAS CONVERTED THE SURCHARGE SOIL TO A UNIFORM LOAD, BUT IS TREATED AS A LOAD THAT UNIFORMLY DISTRIBUTES IN THE DIRECTION OF THE CROSS-SECTION OF THE TOP EDGE OF THE WALL.

$q1 = \gamma 2 \cdot H3 = 39.900$ (kN/m^2)

$q1$　: SURCHARGE SOIL LOAD (kN/m^2)
$\gamma 2$　: UNIT WEIGHT OF SURCHARGE SOIL = 19.0 (kN/m^3)
$H3$　: ROAD CONVERSION HEIGHT OF SURCHARGE SOIL = 2.100 (m)

3.2.2 CALCULATION OF LIVE LOAD APPLIED TO EACH STAGE



THE LIVE LOAD IS THE DISTRIBUTION THROUGH THE SOIL AT WHICH THE RATIO OF THE VERTICAL TO THE HORIZONTAL IS 2:1 SLOPE IN THE TRANSVERSE DIRECTION OF THE ROAD. HOWEVER, CONSIDER THE CALCULATING PULL FORCE AFFECTING THE STRIPS ONLY IN SITUATIONS WHERE THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA. WHEN THE LIVE LOAD AREA OF DISTRIBUTION INTRUDES INTO THE ACTIVE AREA, THE SIZE OF THE LIVE LOAD AT THE POSITION OF EACH STRIP IS CALCULATED BY THE FOLLOWING EQUATION.

$$q_{Li} = q_L \cdot \frac{B_L}{B_{Li}}$$

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_L : LIVE LOAD (kN/m^2)

B_L : LIVE LOAD WIDTH (m)

B_{Li} : DISPERSIVE WIDTH OF THE LIVE LOAD AT iTH STAGE STRIP (m)

(1) LIVE LOAD-1

q_L = 11.600 (kN/m^2)　　B_L = 9.000 (m)

STAGE (i)	z_i (m)	B_{Li} (m)	q_{Li} (kN/m^2)
1	2.475	10.598	9.851
2	3.225	10.973	9.515
3	3.975	11.348	9.200
4	4.725	11.723	8.906
5	5.475	12.098	8.630

3.2.3 EXTERNAL VERTICAL FORCE

CONSIDER EXTERNAL VERTICAL FORCE IN ORDINARY (V_i) AND VERTICAL EXTERNAL FORCE IN EARTHQUAKE (V_{Ei}) .

STAGE (i)	z_i (m)	ORDINARY V_i (kN/m^2)	EARTH QUAKE V_{Ei} (kN/m^2)
1	2.475	4.296	7.069
2	3.225	3.425	5.555
3	3.975	2.848	4.570
4	4.725	2.436	3.880
5	5.475	2.127	3.378

3.2.4 SUMMARY OF VERTICAL SURCHARGE LOAD

(1) ORDINARY

$$q_i = q_1 + q_{Li} + V_i$$

q_i : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 39.900 (kN/m^2)

q_{Li} : LIVE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

V_i : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	q_{Li} (kN/m^2)	V_i (kN/m^2)	q_i (kN/m^2)
1	2.475	9.851	4.296	54.047
2	3.225	9.515	3.425	52.840
3	3.975	9.200	2.848	51.948
4	4.725	8.906	2.436	51.242
5	5.475	8.630	2.127	50.657

(2) EARTHQUAKE

$$q_{Ei} = q_1 + V_{Ei}$$

q_{Ei} : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

q_1 : SURCHARGE SOIL LOAD = 39.900 (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	z_i (m)	V_{Ei} (kN/m^2)	q_{Ei} (kN/m^2)
1	2.475	7.069	46.969
2	3.225	5.555	45.455
3	3.975	4.570	44.470
4	4.725	3.880	43.780
5	5.475	3.378	43.278

3.3　EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP

CONSIDER EXTERNAL HORIZONTAL FORCE IN ORDINARY（H_i）AND CONSIDER EXTERNAL HORIZONTAL FORCE IN EARTHQUAKE（H_{Ei}）.

STAGE (i)	z_i (m)	ORDINARY H_i (kN/m)	EARTH QUAKE H_{Ei} (kN/m)
1	2.475	5.570	8.979
2	3.225	3.245	4.734
3	3.975	0.912	0.477
4	4.725	0.000	0.000
5	5.475	0.000	0.000

3.4 EARTH PRESSURE

3.4.1 COEFFICIENT OF EARTH PRESSURE

COEFFICIENT OF EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

IF $z_i \leqq z_o = 6.0$ (m)

$$K_i = K_o \cdot \left(1 - \frac{z_i}{z_o}\right) + K_A \cdot \frac{z_i}{z_o}$$

IF $z_i > z_o = 6.0$ (m)

$$K_i = K_A$$

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

K_o : COEFFICIENT OF EARTH PRESSURE AT REST

$$K_o = 1 - \sin\phi$$

K_A : COEFFICIENT OF ACTIVE EARTH PRESSURE

$$K_A = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

z_o : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF CONVERSION OF COEFFICIENT OF EARTH PRESSURE = 6.000 (m)

STAGE (i)	z_i (m)	ϕ (°)	K_o	K_A	K_i
1	2.475	30.0	0.5000	0.3333	0.4313
2	3.225	30.0	0.5000	0.3333	0.4104
3	3.975	30.0	0.5000	0.3333	0.3896
4	4.725	30.0	0.5000	0.3333	0.3687
5	5.475	30.0	0.5000	0.3333	0.3479

3.4.2 CALCULATION OF EARTH PRESSURE

(1) ORDINARY EARTH PRESSURE

ORDINARY EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$Pi = Ki \cdot \sigma vi \cdot \Delta H + Hi$$

$$To = 0.75 \cdot Pi$$

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ki : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

ΔH : VERTICAL SPACING OF STRIP = 0.750 (m)

Hi : EXTERNAL HORIZONTAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma vi = \gamma 1 \cdot xi + qi$

$\gamma 1$: UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

xi : $xi = \Delta H \cdot (i - 1/2)$

i : STAGE OF STRIP

qi : VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	xi (m)	Ki	qi (kN/m^2)	σvi (kN/m^2)	Hi (kN/m)	Pi (kN/m)	To (kN/m)
1	0.375	0.431	54.047	61.172	5.570	25.355	19.017
2	1.125	0.410	52.840	74.215	3.245	26.089	19.567
3	1.875	0.390	51.948	87.573	0.912	26.500	19.875
4	2.625	0.369	51.242	101.117	0.000	27.965	20.974
5	3.375	0.348	50.657	114.782	0.000	29.951	22.463

(2) EARTHQUAKE EARTH PRESSURE

EARTHQUAKE EARTH PRESSURE AFFECTING EACH STAGE OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$P_{Ei} = P_i + \Delta P_i$$

$$\Delta P_i = \frac{1}{2} \cdot (1+\frac{z_i}{H_a}) \cdot \alpha \cdot kh \cdot P_n$$

$$T_{oE} = 0.75 \cdot P_{Ei}$$

P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (NO LIVE LOAD) (kN/m)

$P_i = K_i \cdot \sigma_{VEi} \cdot \Delta H + H_{Ei}$

ΔP_i : INCREASED EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

P_n : ORDINARY EARTH PRESSURE AFFECTING BOTTOM-MOST STRIP (NO LIVE LOAD) = 28.025 (kN/m)

z_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

H_a : VIRTUAL WALL HEIGHT = 5.850 (m)

α : EARTHQUAKE INCREASE COEFFICIENT = 1.4

$kh1$: HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

σ_{VEi} : EARTHQUAKE VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

$\sigma_{VEi} = \gamma 1 \cdot x_i + q_{Ei}$

$\gamma 1$: UNIT WEIGHT OF EMBANKMENT MATERIALS = 19.0 (kN/m^3)

x_i : STRIP DEPTH OF iTH STAGE FROM THE TOP OF R/E WALL HEIGHT (m)

K_i : COEFFICIENT OF EARTH PRESSURE AFFECTING iTH STAGE STRIP

q_{Ei} : EARTHQUAKE VERTICAL SURCHARGE LOAD AFFECTING iTH STAGE STRIP (kN/m^2)

H_{Ei} : EARTHQUAKE HORIZONTAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m)

STAGE (i)	x_i (m)	K_i	q_{Ei} (kN/m^2)	σ_{VEi} (kN/m^2)	H_{Ei} (kN/m)	P_i (kN/m)
1	0.375	0.431	46.969	54.094	8.979	26.475
2	1.125	0.410	45.455	66.830	4.734	25.305
3	1.875	0.390	44.470	80.095	0.477	23.880
4	2.625	0.369	43.780	93.655	0.000	25.901
5	3.375	0.348	43.278	107.403	0.000	28.025

STAGE (i)	z_i (m)	ΔP_i (kN/m)	P_{Ei} (kN/m)	T_{OE} (kN/m)
1	2.475	5.025	31.500	23.625
2	3.225	5.478	30.783	23.087
3	3.975	5.931	29.810	22.358
4	4.725	6.383	32.285	24.214
5	5.475	6.836	34.862	26.146

3.5 HORIZONTAL SPACING OF STRIP

(1) ORDINARY

ORDINARY HORIZONTAL SPACING OF STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1i = \frac{\sigma a \cdot Ag}{Pi} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2i = \frac{\sigma a \cdot An}{0.75 \cdot Pi} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3i = \frac{\tau a \cdot A\tau}{0.75 \cdot Pi} \times 10^{-3}$$

$\Delta B1i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3i$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa : ALLOWABLE TENSILE STRESS OF STRIPS = 140.0 (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT = 200.0 (N/mm^2)

Pi : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

To : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP (mm^2)

$Ag = b \cdot (t - Cm) = 240.0$ (mm^2)

b : WIDTH OF STRIP = 80.0 (mm)

t : THE THICKNESS OF STRIP = 4.0 (mm)

Cm : CORROSION ALLOWANCE OF STRIP = 1.0 (mm)

An : NET CROSS-SECTIONAL AREA OF STRIPS (mm^2)

$An = \{b - n' \cdot (3.0 + d)\} \cdot (t - Cm) = 195.0$ (mm^2)

n' : WIDTH DIRECTION OF THE BOLT NUMBER OF STRIP = 1 (PIECE)

d : BOLTS OF NOMINAL DIAMETER = 12.0 (mm)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT (mm^2)

$A\tau = j \cdot n \cdot Ae = 168.6$ (mm^2)

j ：THE NUMBER OF SHEAR PLANE BY THE CONNECTING SYSTEM ＝ 2

n ：THE NUMBER OF THE CONNECTING PORTIONS ONE LOCATION PER MOUNTING BOLTS ＝ 1 (PIECE)

Ae ：SCREW EFFECTIVE AREA OF MOUNTING BOLTS＝ 84.3 (mm^2)

STAGE (i)	Pi (kN/m)	To (kN/m)	ΔB1i (m)	ΔB2i (m)	ΔB3i (m)
1	25.355	19.017	1.325	1.435	1.773
2	26.089	19.567	1.287	1.395	1.723
3	26.500	19.875	1.267	1.373	1.696
4	27.965	20.974	1.201	1.301	1.607
5	29.951	22.463	1.121	1.215	1.501

(2) EARTHQUAKE

HORIZONTAL SPACING OF STRIPS DURING EARTHQUAKE ARE CALCULATED USING FOLLOWING EQUATION.

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS CORROSION ALLOWANCE)

$$\Delta B1_{Ei} = \frac{\sigma a_E \cdot Ag}{P_{Ei}} \times 10^{-3}$$

- FROM TENSILE STRESS OF STRIP(GROSS AREA MINUS THE CORROSION ALLOWANCE AND BOLT HOLE DEFECT)

$$\Delta B2_{Ei} = \frac{\sigma a_E \cdot An}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

- FROM SHEAR STRENGTH OF BOLTS

$$\Delta B3_{Ei} = \frac{\tau a_E \cdot A\tau}{0.75 \cdot P_{Ei}} \times 10^{-3}$$

$\Delta B1_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM MAXIMUM TENSILE STRESS (m)

$\Delta B2_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM TENSILE STRESS NEAR BACK SURFACE OF WALL (m)

$\Delta B3_{Ei}$: HORIZONTAL SPACING OF STRIP AT iTH STAGE STRIP CALCULATED FROM SHEAR STRENGTH OF BOLTS (m)

σa_E : ALLOWABLE TENSILE STRESS OF STRIPS = 210.0 (N/mm^2)

τa_E : ALLOWABLE SHEARING STRESS OF BOLT = 300.0 (N/mm^2)

P_{iE} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)

T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	P_{Ei} (kN/m)	T_{oE} (kN/m)	$\Delta B1_{Ei}$ (m)	$\Delta B2_{Ei}$ (m)	$\Delta B3_{Ei}$ (m)
1	31.500	23.625	1.599	1.733	2.140
2	30.783	23.087	1.637	1.773	2.190
3	29.810	22.358	1.690	1.831	2.262
4	32.285	24.214	1.561	1.691	2.088
5	34.862	26.146	1.445	1.566	1.934

(3) DESIGN HORIZONTAL SPACING OF STRIPS

DESIGN HORIZONTAL SPACING OF STRIPS IS DETERMINED WITH UPPER LIMIT BEING THE SMALLEST OF THE ORDINARY AND EARTHQUAKE VALUES CALCULATED FOR EACH STAGE, ROUNDED TO THE FOLLOWING VALUES.

IN CASE OF 1.000 $>$ $\Delta B i$ $\geqq$ 0.750 $\Delta B i$ = 0.750

STAGE (i)	$\Delta B1i$ (m)	$\Delta B2i$ (m)	$\Delta B3i$ (m)	$\Delta B1_{E}i$ (m)	$\Delta B2_{E}i$ (m)	$\Delta B3_{E}i$ (m)	ΔBi (m)
1	1.325	1.435	1.773	1.599	1.733	2.140	0.750
2	1.287	1.395	1.723	1.637	1.773	2.190	0.750
3	1.267	1.373	1.696	1.690	1.831	2.262	0.750
4	1.201	1.301	1.607	1.561	1.691	2.088	0.750
5	1.121	1.215	1.501	1.445	1.566	1.934	0.750

3.6 PULL FORCE ACTING AFFECTING ON STRIP

PULL FORCE ACTING ON STRIPS ARE CALCULATED USING FOLLOWING EQUATION.

$$T_i = P_i \cdot \Delta B_i$$

$$T_{fi} = T_o \cdot \Delta B_i$$

$$T_{Ei} = P_{Ei} \cdot \Delta B_i$$

$$T_{fEi} = T_{oEi} \cdot \Delta B_i$$

T_i : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
T_{fi} : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
T_{Ei} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
T_{fEi} : EARTHQUAKE PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
P_i : ORDINARY EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)
T_o : ORDINARY TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)
P_{Ei} : EARTHQUAKE EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m)
T_{oE} : EARTHQUAKE TENSILE FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/m)
ΔB_i : HORIZONTAL SPACING OF STRIPS (m)

STAGE (i)	ΔB_i (m)	P_i (kN/m)	T_o (kN/m)	T_i (kN/PIECE)	T_{fi} (kN/PIECE)	P_{Ei} (kN/m)	T_{oE} (kN/m)	T_{Ei} (kN/PIECE)	T_{fEi} (kN/PIECE)
1	0.750	25.355	19.017	19.017	14.262	31.500	23.625	23.625	17.719
2	0.750	26.089	19.567	19.567	14.675	30.783	23.087	23.087	17.315
3	0.750	26.500	19.875	19.875	14.906	29.810	22.358	22.358	16.768
4	0.750	27.965	20.974	20.974	15.730	32.285	24.214	24.214	18.160
5	0.750	29.951	22.463	22.463	16.847	34.862	26.146	26.146	19.610

3.7 STRIP LENGTH

3.7.1 FRICTION COEFFICIENT

APPARENT COEFFICIENT OF FRICTION IS CALCULATED USING FOLLOWING EQUATION.

IN CASE OF $zi \leqq zo = 6.0$ (m)

$$fi^* = fo^* \cdot \left(1 - \frac{zi}{zo}\right) + \tan\phi 1 \cdot \frac{zi}{zo}$$

IN CASE OF $zi > zo = 6.0$ (m)

$$fi^* = \tan\phi 1$$

fi^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

$fo^* = 1.50$

$\phi 1 = 36.0$ (°)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

zo : DEPTH FROM TOP EDGE OF VIRTUAL WALL HEIGHT TO POINT OF COEFFICIENT EARTH PRESSURE = 6.000 (m)

STAGE (i)	zi (m)	fo^*	$\tan\phi 1$	fi^*
1	2.475	1.500	0.727	1.1809
2	3.225	1.500	0.727	1.0843
3	3.975	1.500	0.727	0.9876
4	4.725	1.500	0.727	0.8909
5	5.475	1.500	0.727	0.7942

3.7.2 LENGTH OF ACTIVE AREA

(1) ORDINARY

LENGTH OF ORDINARY ACTIVE AREA IS CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq Ha/2 = 2.925$ (m)

$$Loi = 0.3 \cdot Ha$$

IF $zi > Ha/2 = 2.925$ (m)

$$Loi = 0.6 \cdot (Ha - zi)$$

Loi : LENGTH OF ORDINARY ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

Ha : VIRTUAL WALL HEIGHT = 5.850 (m)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

(2) EARTHQUAKE

LENGTH OF ACTIVE AREA DURING EARTHQUAKE IS CALCULATED USING FOLLOWING EQUATION.

IF $zi \leqq Ha/2 = 2.925$ (m)

$$L_{OE}i = (0.6 + kh1) \cdot (Ha/2)$$

IF $zi > Ha/2 = 2.925$ (m)

$$L_{OE}i = (0.6 + kh1) \cdot (Ha - zi)$$

$L_{OE}i$: LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (m)

Ha : VIRTUAL WALL HEIGHT = 5.850 (m)

zi : STRIP DEPTH OF iTH STAGE FROM THE TOP OF VIRTUAL WALL HEIGHT (m)

$kh1$: HORIZONTAL SEISMIC COEFFICIENT = 0.18 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

STAGE (i)	zi (m)	Loi (m)	$L_{OE}i$ (m)
1	2.475	1.755	2.282
2	3.225	1.575	2.048
3	3.975	1.125	1.463
4	4.725	0.675	0.878
5	5.475	0.225	0.293

3.7.3 LENGTH OF STRIP INSIDE THE RESISTANCE AREA

(1) ORDINARY

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$Lreqi = \frac{Fs \cdot Ti}{2 \cdot fi^{*} \cdot (\sigma vi - Vi) \cdot b}$$

$Lreqi$: LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

Fs : SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σvi : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

Vi : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	Ti (kN/PIECE)	fi^{*}	$\sigma vi-Vi$ (kN/m^2)	b (m)	Lreqi (m)
1	19.017	1.1809	56.876	0.0800	3.539
2	19.567	1.0843	70.790	0.0800	3.187
3	19.875	0.9876	84.725	0.0800	2.970
4	20.974	0.8909	98.681	0.0800	2.983
5	22.463	0.7942	112.655	0.0800	3.139

(2) EARTHQUAKE

LENGTH OF STRIP INSIDE THE RESISTANCE AREA IS CALCULATED USING FOLLOWING EQUATION.

$$L_{reqEi} = \frac{F_{SE} \cdot T_{Ei}}{2 \cdot f_i^* \cdot (\sigma_{VEi} - V_{iE}) \cdot b}$$

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ＝ 1.20

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

b : STRIP WIDTH (m)

σ_{VEi} : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

V_{Ei} : VERTICAL EXTERNAL FORCE AFFECTING iTH STAGE STRIP (kN/m^2)

STAGE (i)	T_{Ei} (kN/PIECE)	f_i^*	$\sigma_{VEi}-V_{Ei}$ (kN/m^2)	b (m)	L_{reqEi} (m)
1	23.625	1.1809	47.025	0.0800	3.191
2	23.087	1.0843	61.275	0.0800	2.607
3	22.358	0.9876	75.525	0.0800	2.249
4	24.214	0.8909	89.775	0.0800	2.271
5	26.146	0.7942	104.025	0.0800	2.374

3.7.4 MINIMUM LENGTH OF STRIP

MINIMUM LENGTH OF STRIP IS CALCULATED USING FOLLOWING EQUATION.

ORDINARY

$$L_{mini} = L_{oi} + L_{reqi}$$

EARTHQUAKE

$$L_{minEi} = L_{oEi} + L_{reqEi}$$

L_{mini} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP (m)

L_{oi} : LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

L_{reqi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP (m)

L_{minEi} : MINIMUM LENGTH OF STRIP AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{oEi} : LENGTH OF ACTIVE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

L_{reqEi} : LENGTH OF STRIP INSIDE RESISTANCE AREA AT iTH STAGE STRIP DURING EARTHQUAKE (m)

STAGE (i)	L_{oi} (m)	L_{reqi} (m)	L_{mini} (m)	L_{oEi} (m)	L_{reqEi} (m)	L_{minEi} (m)
1	1.755	3.539	5.294	2.282	3.191	5.473
2	1.575	3.187	4.762	2.048	2.607	4.655
3	1.125	2.970	4.095	1.463	2.249	3.712
4	0.675	2.983	3.658	0.878	2.271	3.149
5	0.225	3.139	3.364	0.293	2.374	2.667

3.7.5 MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS

MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS IS CALCULATED USING FOLLOWING EQUATION.

IF SURCHARGE BACKFILL REACHES 2 m OR HIGHER ($H1 = 2.100 \geqq 2m$)

0.6 Ha OR MORE FROM UPPER EDGE OF Ha

$L_{sd} = 0.7 \cdot Ha = 4.095$ (m)

0.3 Ha OR LESS FROM LOWER EDGE OF Ha

$L_{sd} = 0.4 \cdot Ha = 2.340$ (m)

AND $L_{min} \geqq 4.0$ (m)

L_{sd} : MINIMUM LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)
Ha : VIRTUAL WALL HEIGHT = 5.850 (m)

3.7.6 DETERMINATION OF STRIP LENGTH

THE MINIMUM STRIP LENGTH OF STRIP SHALL NOT BE LESS THAN THE VALUES SHOWN IN STRUCTURAL DETAIL SPECIFICS, EVEN WHEN A STABILITY MARGIN HAS BEEN DETERMINED TO EXIST THROUGH DESIGN CALCULATIONS.

L_i : ACTUAL LENGTH OF THE iTH STAGE OF THE STRIP (m)
L_{mini} : THE MINIMUM REQUIRED LENGTH OF THE iTH STAGE OF THE REINFORCEMENT OBTAINED BY THE CALCULATION (m)
L_{minEi} : MINIMUM REQUIRED LENGTH OF TIME OF AN EARTHQUAKE OF iTH STAGE OF REINFORCING MEMBER THAT HAS BEEN DETERMINED BY THE CALCULATION (m)
L_{sdi} : LENGTH OF STRIP BASED ON STRUCTURAL DETAIL SPECIFICS (m)

STAGE (i)	L_{mini} (m)	L_{minEi} (m)	L_{sdi} (m)	L_i (m)
1	5.294	5.473	4.095	6.000
2	4.762	4.655	4.095	6.000
3	4.095	3.712	—	5.000
4	3.658	3.149	4.000	5.000
5	3.364	2.667	4.000	4.000

3.8 CHECK OF SAFETY FACTOR AGAINST STRIP PULLOUT

(1) ORDINARY

SAFETY FACTOR AGAINST STRIP PULL OUT IS CALCULATED USING FOLLOWING EQUATION.

$$Fsi = \frac{Si}{Ti} \geqq Fs = 2.00$$

Fsi : SAFETY FACTOR AGAINST STRIPPULLOUT AT iTH STAGE STRIP

Fs : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 2.0

Ti : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Si : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$Si = 2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei$$

fi^{*} : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σvi’ : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

Lei : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$Lei = Li - Loi$$

Li : DETERMINED STRIP LENGTH AT iTH STAGE (m)

Loi : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	Li (m)	Loi (m)	Lei (m)	fi^{*}	σvi’ (kN/m^2)	b (m)	Si (kN/PIECE)	Ti (kN/PIECE)	Fsi	JUDGE
1	6.000	1.755	4.245	1.1809	56.876	0.0800	45.621	19.017	2.398	○
2	6.000	1.575	4.425	1.0843	70.790	0.0800	54.342	19.567	2.777	○
3	5.000	1.125	3.875	0.9876	84.725	0.0800	51.877	19.875	2.610	○
4	5.000	0.675	4.325	0.8909	98.681	0.0800	60.837	20.974	2.900	○
5	4.000	0.225	3.775	0.7942	112.655	0.0800	54.042	22.463	2.405	○

(2) EARTHQUAKE

$$F_{SEi} = \frac{S_{Ei}}{T_{Ei}} \geqq F_{SE} = 1.20$$

F_{siEi} : SAFETY FACTOR AGAINST STRIP PULLOUT AT iTH STAGE STRIP

F_{SE} : DESIGN SAFETY FACTOR AGAINST STRIP PULLOUT = 1.2

T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

S_{Ei} : FRICTION RESISTANCE AT iTH STAGE STRIP (kN/PIECE)

$$S_{Ei} = 2 \cdot f_i^* \cdot \sigma_{VEi} \cdot b \cdot L_{eEi}$$

f_i^* : APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi}' : VERTICAL EARTH PRESSURE AFFECTING iTH STAGE STRIP (kN/m^2)

b : STRIP WIDTH AT iTH STAGE (m)

L_{eEi} : EFFECTIVE STRIP LENGTH OF AT iTH STAGE (m)

$$L_{eEi} = L_i - L_{OEi}$$

L_i : DETERMINED STRIP LENGTH AT iTH STAGE (m)

L_{OEi} : ORDINARY LENGTH OF ACTIVE AREA AFFECTING iTH STAGE STRIP (m)

STAGE (i)	L_i (m)	L_{OEi} (m)	L_{eEi} (m)	f_1^*	σ_{VEi}' (kN/m^2)	b (m)	S_{Ei} (kN/PIECE)	T_{Ei} (kN/PIECE)	F_{SEi}	JUDGE
1	6.000	2.282	3.719	1.1809	47.025	0.0800	33.041	23.625	1.398	○
2	6.000	2.048	3.953	1.0843	61.275	0.0800	42.016	23.087	1.819	○
3	5.000	1.463	3.538	0.9876	75.525	0.0800	42.216	22.358	1.888	○
4	5.000	0.878	4.123	0.8909	89.775	0.0800	52.755	24.214	2.178	○
5	4.000	0.293	3.708	0.7942	104.025	0.0800	49.009	26.146	1.874	○

3.9 CHECK OF STRESS DEGREE OF ITEM

(1) ORDINARY

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma ti = \frac{Ti}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\sigma oti = \frac{Tfi}{Ag} \leqq \sigma a = 140.0 \ (N/mm^2)$$

$$\tau oi = \frac{Tfi}{A\tau} \leqq \tau a = 200.0 \ (N/mm^2)$$

σti : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP (N/mm^2)

σoti : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP (N/mm^2)

τoi : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP (N/mm^2)

σa : ALLOWABLE TENSILE STRESS OF STRIP (N/mm^2)

τa : ALLOWABLE SHEARING STRESS OF BOLT (N/mm^2)

Ti : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)

Tfi : ORDINARY PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 (mm^2)

An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 (mm^2)

$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 (mm^2)

STAGE (i)	Ti (kN/PIECE)	Tfi (kN/PIECE)	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	JUDGE
1	19.017	14.262	79.2	73.1	84.6	○
2	19.567	14.675	81.5	75.3	87.0	○
3	19.875	14.906	82.8	76.4	88.4	○
4	20.974	15.730	87.4	80.7	93.3	○
5	22.463	16.847	93.6	86.4	99.9	○

(2) EARTHQUAKE

STRESS DEGREE OF ITEM IS CALCULATED USING FOLLOWING EQUATION.

$$\sigma t_{Ei} = \frac{T_{Ei}}{Ag} \leqq \sigma a_E = 210.0\ (\mathrm{N/mm^2})$$

$$\sigma ot_{Ei} = \frac{Tf_{Ei}}{An} \leqq \sigma a_E = 210.0\ (\mathrm{N/mm^2})$$

$$\tau o_{Ei} = \frac{Tf_{Ei}}{A\tau} \leqq \tau a_E = 300.0\ (\mathrm{N/mm^2})$$

σt_{Ei} : TENSILE STRESS OF GROSS STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
σot_{Ei} : TENSILE STRESS OF NET STRIP AREA AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
τo_{Ei} : SHEAR STRESS OF BOLTS AT iTH STAGE STRIP ($\mathrm{N/mm^2}$)
σa_E : ALLOWABLE TENSILE STRESS OF STRIP ($\mathrm{N/mm^2}$)
τa_E : ALLOWABLE SHEARING STRESS OF BOLT ($\mathrm{N/mm^2}$)
T_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP (kN/PIECE)
Tf_{Ei} : PULL FORCE AFFECTING iTH STAGE STRIP AT THE WALL (kN/PIECE)
Ag : GROSS CROSS-SECTIONAL AREA OF STRIPS = 240.0 ($\mathrm{mm^2}$)
An : NET CROSS-SECTIONAL AREA OF STRIP = 195.0 ($\mathrm{mm^2}$)
$A\tau$: EFFECTIVE SHEAR AREA OF BOLT = 168.6 ($\mathrm{mm^2}$)

STAGE (i)	T_{Ei} (kN/PIECE)	Tf_{Ei} (kN/PIECE)	σt_{Ei} ($\mathrm{N/mm^2}$)	σot_{Ei} ($\mathrm{N/mm^2}$)	τo_{Ei} ($\mathrm{N/mm^2}$)	JUDGE
1	23.625	17.719	98.4	90.9	105.1	○
2	23.087	17.315	96.2	88.8	102.7	○
3	22.358	16.768	93.2	86.0	99.5	○
4	24.214	18.160	100.9	93.1	107.7	○
5	26.146	19.610	108.9	100.6	116.3	○

4. STUDY OF EXTERNAL STABILITY

4.1 CASE

CASE		OWN WEIGHT OF RETAINING WALL	INERTIA FORCE OF OWN WEIGHT	LOAD1-1	LOAD2
CASE 1-1	ORDINARY	○		○	
CASE 1-2	ORDINARY	○		○	
CASE 2-3	EARTHQUAKE	○	○		○

EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA IS CALCULATED USING TRIAL WEDGE METHOD.

CASE 1-1 : OWN WEIGHT OF RETAINING WALL NOT TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING SLIDING OR OVER-TURNING)

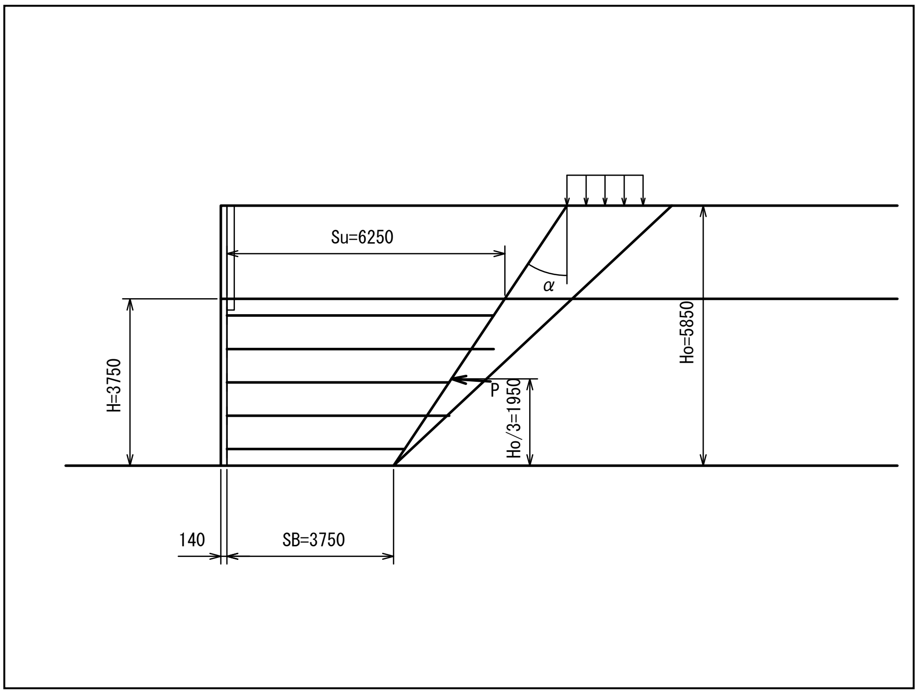
CASE 1-2 : OWN WEIGHT OF RETAINING WALL TAKING INTO ACCOUNT LIVE LOAD ON RETAINING WALL(WHEN EXAMINING BEARING CAPACITY)

LOAD1-1 : ORDINARY LOAD

LOAD2 : EARTHQUAKELOAD

4.2　EARTH PRESSURE AFFECTING BACK OF REINFORCED AREA

4.2.1　CASE 1 CALCULATION OF LOAD



Ho　: WORKING HEIGHT OF EARTH PRESSURE(REINFORCED AREA PORTION + SURCHARGE BACKFILL PORTION)＝ 5.850 (m)

H　: R/E WALL HEIGHT＝ 3.750 (m)

ho　: CRITERION HEIGHT OF SLIDING ＝ 0.000 (m)

Su　: REINFORCED AREA WIDTH AT R/E LEVEE CROWN ＝ 6.250 (m)

S_B　: REINFORCED AREA WIDTH AT R/E BOTTOM END ＝ 3.750 (m)

B　: R/E BOTTOM WIDTH ＝ 3.890 (m)

α　: VIRTUAL BACK SLOPE INCLINE ＝ -33.690 (°)

$$\alpha = -\tan^{-1}\frac{Su - S_B}{H} = -33.690\ (^\circ)$$

W1　: OWN WEIGHT OF R/E ＝ 649.116 (kN/m)

Wq　: LOADED WEIGHT ON R/E ＝ 0.000 (kN/m)

γ　: UNIT WEIGHT OF BACKFILL SOIL ＝ 19.0 (kN/m^3)

ϕ　: ANGLE OF SHEARING RESISTANCE FOR BACKFILL ＝ 30.0 (°)

c　: COHESION OF BACK FILL ＝ 0.0 (kN/m^2)

z　: $z = (2 \cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = (2 \cdot c/\gamma)\cdot\tan(45^\circ + \phi/2) = 0.000$ (m)

$\delta 1$　: FRICTION ANGLE AGAINST VIRTUAL WALL BACK ＝ 30.000 (°)

$\omega 1$　: ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE ＝ 43.1 (°)

$W2$: OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) = 150.883 (kN/m)

$L1$: LENGTH OF SLIP SURFACE = 8.567 (m)

P_A : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE (kN/m)

$$P_A = \frac{W2 \cdot \sin(\omega 1 - \phi) - c \cdot L1 \cdot \cos\phi}{\cos(\omega 1 - \phi - \alpha - \delta 1)} = 35.634$$

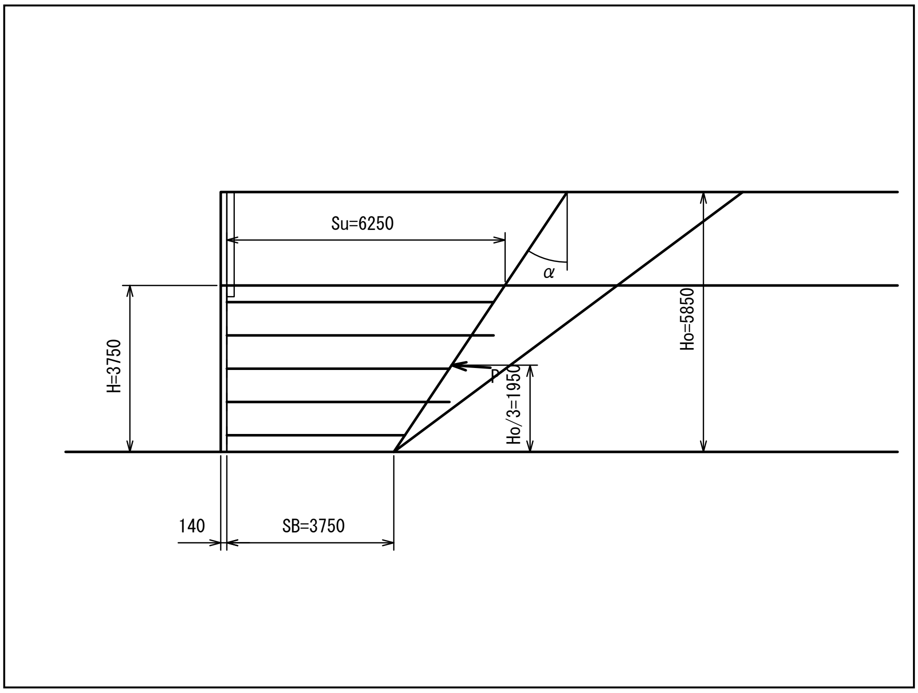
P_{Ah} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \cos(\delta 1 + \alpha) = 35.561$ (kN/m)

P_{AV} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_A \cdot \sin(\delta 1 + \alpha) = -2.293$ (kN/m)

4.2.2 CASE 2-3 CALCULATION OF LOAD



α : SLOPE INCLINE OF VIRTUAL WALL BACK = -33.690 (°)

W1 : OWN WEIGHT OF R/E = 649.116 (kN/m)

Wq : LOADED WEIGHT ON R/E = 0.000 (kN/m)

γ : UNIT WEIGHT OF BACKFILL SOIL = 19.0 (kN/m^3)

ϕ : ANGLE OF SHEARING RESISTANCE FOR BACKFILL = 30.0 (°)

c : COHESION OF BACK FILL = 0.0 (kN/m^2)

z : $z = (2 \cdot c/\gamma) \cdot \tan(45° + \phi/2) = 0.000$ (m)

δ_E : EARTHQUAKE FRICTION ANGLE AGAINST VIRTUAL WALL BACK = 30.000 (°)

k_{h2} : HORIZONTAL SEISMIC COEFFICIENT = 0.13 (QUOTED FROM "DESIGN CONDITION)38 / DESIGN HORIZONTAL SEISMIC INTENSITY")

θ : EARTHQUAKE COMPOUND ANGLE = $\tan^{-1} k_{h2}$ = 7.407 (°)

ω_E : ANGLE BETWEEN HYPOTHETICAL VIRTUAL SLIP SURFACE AND HORIZONTAL SURFACE DURING EARTHQUAKE = 36.7 (°)

W_E : OWN WEIGHT OF EARTH WEDGE (INCLUDING LOADED WEIGHT) DURING EARTHQUAKE = 219.273 (kN/m)

L_E : LENGTH OF SLIP SURFACE DURING EARTHQUAKE = 9.786 (m)

P_{AE} : ACTIVE EARTH RESULTANT FORCE AFFECTING VIRTUAL BACK SURFACE DURING EARTHQUAKE (kN/m)

$$P_{AE} = \frac{W_E \cdot \sec\theta \cdot \sin(\omega_E - \phi + \theta) - c \cdot L_E \cdot \cos\phi}{\cos(\omega_E - \phi - \alpha - \delta_E)} = 54.832$$

P_{AEh} : HORIZONTAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \cos(\delta_E + \alpha) = 54.718$ (kN/m)

P_{AEv} : VERTICAL COMPONENT OF ACTIVE EARTH RESULTANT FORCE

$= P_{AE} \cdot \sin(\delta_E + \alpha) = -3.529$ (kN/m)

4.3 AGGREGATION OF LOAD

4.3.1 ORDINARY

(1) CASE 1-1 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	649.116	3.029	1965.86	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-2.293	5.190	-11.90	35.561	1.950	69.34
TOTAL	646.823		1953.96	35.561		69.34

(2) CASE 1-2 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	649.116	3.029	1965.86	0.000	0.000	0.00
LIVE LOAD	84.564	4.145	350.52	0.000	0.000	0.00
REAR EARTH PRESSURE 1-1	-2.293	5.190	-11.90	35.561	1.950	69.34
TOTAL	731.387		2304.48	35.561		69.34

4. 3. 2 EARTHQUAKE

(1) CASE 2-3 (WHEN EXAMINING SLIDING OR OVER-TURNING)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	649. 116	3. 029	1965. 86	84. 385	3. 251	274. 30
REAR EARTH PRESSURE 2	-3. 529	5. 190	-18. 32	54. 718	1. 950	106. 70
TOTAL	645. 587		1947. 54	139. 103		381. 00

k h2 : HORIZONTAL SEISMIC COEFFICIENT ＝ 0. 13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

(2) CASE 2-3 (WHEN EXAMINING BEARING CAPACITY)

LOAD ITEM	VERTICAL FORCE V (kN/m)	ACTION POSITION X (m)	POSITION MOMENT Mx (kNm/m)	HORIZONTAL FORCE H (kN/m)	ACTION POSITION Y (m)	POSITION MOMENT My (kNm/m)
VIRTUAL WALL OF WEIGHT	649. 116	3. 029	1965. 86	84. 385	3. 251	274. 30
REAR EARTH PRESSURE 2	-3. 529	5. 190	-18. 32	54. 718	1. 950	106. 70
TOTAL	645. 587		1947. 54	139. 103		381. 00

k h2 : HORIZONTAL SEISMIC COEFFICIENT ＝ 0. 13 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

4.4 STUDY OF ORDINARY SLIDING

(1) CASE 1-1

$$F_s = \frac{c_B \cdot B + \mu \cdot \Sigma V}{\Sigma H} = 10.495 \geqq F_{sa} = 1.50 \quad \cdots \quad OK$$

F_s : SAFETY FACTOR OF SLIDING

F_{sa} : DESIGN SAFETY FACTOR OF SLIDING = 1.50

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

ΣV : TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 646.823 (kN/m)

ΣH : TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 35.561 (kN/m)

4.5 STUDY OF SLIDING DURING EARTHQUAKE

(1) CASE 2-3

$$F_{sE} = \frac{c_B \cdot B + \mu \cdot \Sigma V3_E}{\Sigma H3_E} = 2.677 \geqq F_{saE} = 1.20 \quad \cdots \quad OK$$

F_{sE} : SAFETY FACTOR OF SLIDING

F_{saE} : DESIGN SAFETY FACTOR OF SLIDING = 1.20

B : BOTTOM WIDTH R/E = 3.890 (m)

c_B : ADHESION BETWEEN BASE OF REINFORCED AREA AND SOIL = 0.0 (kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL AGAINST BASE OF REINFORCED AREA = 30.0 (°)

μ : COEFFICIENT OF FRICTION BETWEEN BASE OF REINFORCED AREA AND SOIL = $\tan\phi$ = 0.577

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 645.587 (kN/m)

$\Sigma H3_E$: TOTAL HORIZONTAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 139.103 (kN/m)

4.6 STUDY OF ORDINARY OVERTURNING

(1) CASE 1-1

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d = \frac{\Sigma Mr - \Sigma Mo}{\Sigma V} = 2.914 \text{ (m)}$$

d	: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)
ΣMr	: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1953.96 (kN·m/m)
ΣMo	: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 69.34 (kN·m/m)
ΣV	: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 646.823 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$$e = B/2 - d = -0.969 \leqq B/6 = 0.648 \text{ (m)} \quad \cdot\cdot\cdot \quad \text{OK}$$

e	: ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)
B	: R/E BOTTOM WIDTH = 3.890 (m)

4.7 EARTHQUAKE STUDY OF OVERTURNING DURING EARTHQUAKE

(1) CASE 2-3

DETERMINE R/E DISTANCE FROM LOWER EDGE OF WALL TO ACTION POSITION OF RESULTANT FORCE

$$d3_E = \frac{\Sigma Mr3_E - \Sigma Mo3_E}{\Sigma V3_E} = 2.427 \text{ (m)}$$

$d3_E$: R/E DISTANCE FROM LOWER EDGE OF WALL OF BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

$\Sigma Mr3_E$: R/E RESISTER MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 1947.54 (kN·m/m)

$\Sigma Mo3_E$: R/E OVER-TURNING MOMENT AROUND LOWER EDGE OF WALL OF BOTTOM = 381.00 (kN·m/m)

$\Sigma V3_E$: TOTAL VERTICAL LOAD PER 1 m DEPTH ON R/E BOTTOM = 645.587 (kN/m)

ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTING POINT OF RESULTANT FORCE

$e = B/2 - d3_E = -0.482 \leqq B/3 = 1.297$ (m) ・・・ OK

e : ECCENTRICITY FROM CENTER OF R/E BOTTOM TO ACTION POSITION OF RESULTANT FORCE (m)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.8 STUDY OF ORDINARY BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM)

4.8.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$$q_s = \Sigma V / B = 188.017 \ (\mathrm{kN/m^2})$$

q_s : VERTICAL SUBGRADE REACTION ON R/E BOTTOM ($\mathrm{kN/m^2}$)

ΣV : TOTAL VERTICAL LOAD ON R/E BOTTOM = 731.387 (kN/m)
(AGGREGATION OF LOAD - SEE CASE 1-2)

B : R/E BOTTOM WIDTH = 3.890 (m)

4.9 STUDY OF BEARING CAPACITY STABILITY UNDER SURCHARGE BACKFILL (R/E BOTTOM) DURING EARTHQUAKE

4.9.1 VERTICAL SUBGRADE REACTION ON R/E BOTTOM

$q_{SE} = \Sigma V_E / B = 165.961$ (kN/m^2)

q_{SE} : VERTICAL SUBGRADE REACTION ON R/E BOTTOM DURING EARTHQUAKE (kN/m^2)

ΣV_E : TOTAL VERTICAL LOAD ON R/E BOTTOM DURING EARTHQUAKE $= 645.587$ (kN/m)
(AGGREGATION OF LOAD - SEE CASE 2-3)

B : R/E BOTTOM WIDTH $= 3.890$ (m)

4.10 STUDY OF ORDINARY BEARING CAPACITY UNDER CONCRETE FOUNDATION

4.10.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$$Vc = \Sigma \left(To \cdot \tan\delta \right) = 37.087 \text{ (kN/m)}$$

To : TENSILE FORCE AGAINST WALL (kN/m)

δ : ANGLE OF WALL FRICTION (°) = $2/3\phi$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	To (kN/m)	ϕ (°)	tan δ	Vci (kN/m)
1	19.017	30.0	0.364	6.921
2	19.567	30.0	0.364	7.122
3	19.875	30.0	0.364	7.234
4	20.974	30.0	0.364	7.634
5	22.463	30.0	0.364	8.176
TOTAL				37.087

4.10.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$qw = \frac{Wc1+Wc2+Vc}{b} = 147.481\ (\mathrm{kN/m^2})$$

qw : ORDINARY LOAD UNDER WALL (kN/m^2)

Wc1 : THE WEIGHT OF THE CONCRETE WALL MATERIAL

$= \gamma c1 \cdot Hta \cdot t = 20.066$ (kN/m)

$\gamma c1$: UNIT WEIGHT OF THE CONCRETE WALL MATERIAL = 24.5 (kN/m^3)

Hta : TOTAL HEIGHT OF R/E = 5.850 (m)

t : THE THICKNESS OF THE CONCRETE WALL MATERIAL = 0.140 (m)

Wc2 : WEIGHT OF FOUNDATION CONCRETE = $hf \cdot b \cdot \gamma c2$ = 1.840 (kN/m)

b : WIDTH OF CONCRETE FOUNDATION = 0.400 (m)

hf : HEIGHT OF CONCRETE FOUNDATION = 0.200 (m)

$\gamma c2$: UNIT WEIGHT OF CONCRETE FOUNDATION = 23.0 (kN/m^3)

Vc : VERTICAL COMPONENT AGAINST WALL = 37.087 (kN/m)

4.11 STUDY OF BEARING CAPACITY UNDER CONCRETE FOUNDATION DURING EARTHQUAKE

4.11.1 VERTICAL COMPONENT AGAINST WALL BEING CAUSED BY LOAD

$V_{cE} = \Sigma (T_{oE} \cdot \tan\delta_E) = 32.001$ (kN/m)

T_{oE} : THE TENSILE FORCE OF THE WALL AT THE TIME OF EARTHQUAKE (kN/m)

δ_E : WALL FRICTION ANGLE AT THE TIME OF EARTHQUAKE (°) = $\phi/2$

ϕ : ANGLE OF SHEARING RESISTANCE (°)

STAGE (i)	T_{oE} (kN/m)	ϕ (°)	$\tan\delta_E$	V_{cEi} (kN/m)
1	23.625	30.0	0.268	6.330
2	23.087	30.0	0.268	6.186
3	22.358	30.0	0.268	5.991
4	24.214	30.0	0.268	6.488
5	26.146	30.0	0.268	7.006
TOTAL				32.001

4.11.2 VERTICAL GROUND SUPPORT FORCE IN BASIC BOTTOM OF THE WALL

$$q_{WE} = \frac{Wc1 + Wc2 + V_{cE}}{b} = 134.767 \ (kN/m^2)$$

q_{WE} : THE VERTICAL GROUND REACTION FORCE AT THE TIME OF THE EARTHQUAKE IN BASIC BOTTOM OF THE WALL (kN/m^2)

$Wc1$: THE WEIGHT OF THE CONCRETE WALL MATERIAL = 20.066 (kN/m)

$Wc2$: WEIGHT OF FOUNDATION CONCRETE = 1.840 (kN/m)

V_{cE} : THE VERTICAL COMPONENT FORCE OF THE WALL DURING AN EARTHQUAKE = 32.001 (kN/m)

5. OVERALL STABILITY EXAMINATION INCLUDING R/E (EXAMINATION OF STABILITY CONCERNING CIRCULAR SLIP)

5.1 ORDINARY

5.1.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{Ai} = \frac{\sigma a \cdot Ag}{\Delta B} \times 10^{-3}$$

T_{Ai} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σa : ALLOWABLE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	T_{Ai} (kN/m)
1	140.0	80.0	4.0	1.0	240.0	0.750	44.800
2	140.0	80.0	4.0	1.0	240.0	0.750	44.800
3	140.0	80.0	4.0	1.0	240.0	0.750	44.800
4	140.0	80.0	4.0	1.0	240.0	0.750	44.800
5	140.0	80.0	4.0	1.0	240.0	0.750	44.800

5.1.2 FORMULA FOR SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

$$Fs = \frac{R\Sigma\{ces\cdot l + (W'\cdot\cos\alpha + Tavail\cdot\sin\theta)\tan\phi + Tavail\cdot\cos\theta\}}{R\Sigma(W\cdot\sin\alpha)}$$

Fs : SAFETY FACTOR OF ORDINARY CIRCULAR SLIP

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

ces : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : ces = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

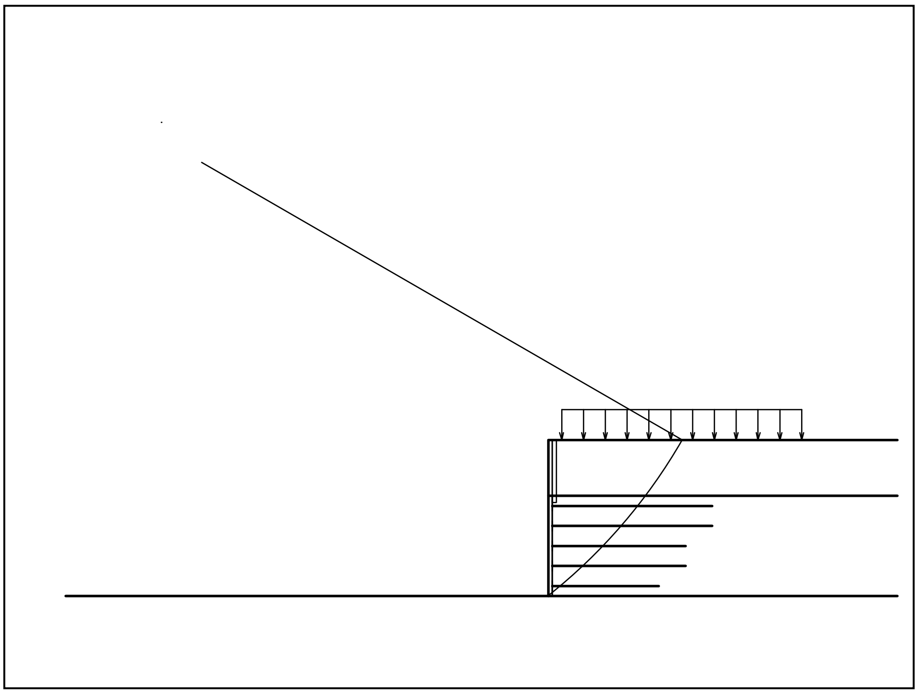
Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(Tp), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

R : RADIUS OF SLIP CIRCULAR ARC (m)

5.1.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [ORDINARY-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$Tavaili = \min(Tpi,\ T_{A}i)$$

$$Tpi = \frac{2 \cdot fi^{*} \cdot \sigma vi \cdot b \cdot Lei}{Fs} \times \frac{1}{\triangle B}$$

Tavaili	: AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
Tpi	: DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP (kN/m)
$T_{A}i$	: ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)
fi*	: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP
σvi	: VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE = $\gamma \cdot zi$ (kN/m^2)
zi	: DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)
b	: WIDTH OF STRIP (m)
Lei	: ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)
⊿B	: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)
Fs	: SAFETY FACTOR AGAINST STRIP PULLOUT = 2.00

STAGE (i)	fi*	γ (kN/m^3)	zi (m)	σvi (kN/m^2)	b (m)	Lei (m)	⊿B (m)	Tpi (kN/m)	T$_{Ai}$ (kN/m)	Tavaili (kN/m)
1	1.1809	19.0 19.0	2.100 0.375	47.025	0.0800	2.790	0.750	16.527	44.800	16.527
2	1.0843	19.0 19.0	2.100 1.125	61.275	0.0800	3.410	0.750	24.166	44.800	24.166
3	0.9876	19.0 19.0	2.100 1.875	75.525	0.0800	3.092	0.750	24.600	44.800	24.600
4	0.8909	19.0 19.0	2.100 2.625	89.775	0.0800	3.846	0.750	32.811	44.800	32.811
5	0.7942	19.0 19.0	2.100 3.375	104.025	0.0800	3.684	0.750	32.466	44.800	32.466
								Σ Tavail = 130.570		

(3) LIST OF SAFETY FACTOR

Fs		COORDINATE OF CIRCULAR CENTER X						
		-19.0	-17.0	-15.0	-13.0	-11.0	-9.0	-7.0
Y	23.0	1.405	1.410	1.424	1.450	1.489	1.546	1.638
	21.0	1.407	1.404	1.410	1.426	1.457	1.505	1.577
	19.0	1.421	1.408	1.403	1.410	1.430	1.467	1.528
	17.0	1.452	1.427	1.410	1.403	1.410	1.435	1.483
	15.0	1.509	1.468	1.435	1.413	1.404	1.412	1.445
	13.0	1.607	1.546	1.493	1.450	1.419	1.406	1.418
	11.0	1.781	1.691	1.608	1.535	1.475	1.432	1.414

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	ORDINARY-1	JUDGE
MINIMUM SAFETY FACTOR	Fsmin Fs		1.403 (1.200)	○
ΣTavail	kN/m		130.570	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN·m/m	[1614.2] [3047.4]	—
	M_R	kN·m/m	4661.6	—
	M_T	kN·m/m	3019.5	—
ACTIVE MOMENT	M_D	kN·m/m	5476.5	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-13.000 17.000	—
RADIUS	R	m	20.810	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (φ) (kN·m/m)

M_R : RESISTER MOMENT OF SOIL = $M_{RC}+M_{RF}$ (kN·m/m)

M_T : RESISTER MOMENT OF STRIP (kN·m/m)

Fsmin : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP
= (M_R+M_T) / M_D

5.2 EARTHQUAKE

5.2.1 THE TENSILE STRENGTH OF THE STRIP (FOR LONG-TERM LOAD)

$$T_{AE}i = \frac{\sigma a_E \cdot Ag}{\Delta B} \times 10^{-3}$$

$T_{AE}i$: ALLOWABLE EARTHQUAKE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

σa_E : ALLOWABLE EARTHQUAKE TENSILE STRENGTH OF STRIP (N/mm^2)

Ag : GROSS CROSS-SECTIONAL AREA OF STRIP, WITH CORROSION ALLOWANCE SUBTRACTED (mm^2)

$$Ag = b \cdot (t - Cm)$$

b : WIDTH OF STRIP (mm)

t : THE THICKNESS OF THE STRIP (mm)

Cm : CORROSION ALLOWANCE OF STEEL STRIP (mm)

ΔB : HORIZONTAL INSTALLATION INTERVAL OF STRIP (m)

STAGE (i)	σa_E (N/mm^2)	b (mm)	t (mm)	Cm (mm)	Ag (mm^2)	ΔB (m)	$T_{AE}i$ (kN/m)
1	210.0	80.0	4.0	1.0	240.0	0.750	67.200
2	210.0	80.0	4.0	1.0	240.0	0.750	67.200
3	210.0	80.0	4.0	1.0	240.0	0.750	67.200
4	210.0	80.0	4.0	1.0	240.0	0.750	67.200
5	210.0	80.0	4.0	1.0	240.0	0.750	67.200

5.2.2 FORMULA FOR SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

$$F_{SE} = \frac{R\,\Sigma\{c_{es}\cdot l+(W'\cdot\cos\alpha-k_{h3}\cdot W\cdot\sin\alpha)\tan\phi\}+R\cdot\Sigma\,T_{avail}(\cos\theta+\sin\theta\cdot\tan\phi)}{\Sigma(R\cdot W\cdot\sin\alpha\;+\;k_{h3}\cdot W\cdot y_G)}$$

F_{SE} : SAFETY FACTOR OF CIRCULAR SLIP DURING EARTHQUAKE

l : LENGTH OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES (m)

W : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) (kN/m)

W' : TOTAL WEIGHT OF SPLIT PIECES (INCLUDING LOADED WEIGHT) TAKING BUOYANCY INTO ACCOUNT (kN/m)

α : ANGLE BETWEEN LINE THAT CONNECTS MIDPOINT OF SLIP SURFACE PARTITIONED WITH SPLIT PIECES WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

c_{es} : COHESION OF SOIL DURING EXAMINATION OF CIRCULAR SLIP (kN/m^2)
(FOR SANDY SOIL : c_{es} = 10kN/m^2)

ϕ : ANGLE OF SHEARING RESISTANCE OF SOIL (°)

Tavail : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP(ALLOWABLE TENSILE STRENGTH AT A STRIP ON DEEPER SIDE OF EXPECTED SLIP CIRCLE (T_A) OR DRAWING RESISTANCE(T_p), WHICHEVER SMALLER) (kN/m)

θ : ANGLE BETWEEN LINE THAT CONNECTS INTERSECTION POINT OF SLIP SURFACE AT A STRIP(ALLOWABLE TENSILE WITH CENTER OF SLIP CIRCLE, AND VERTICAL LINE (°)

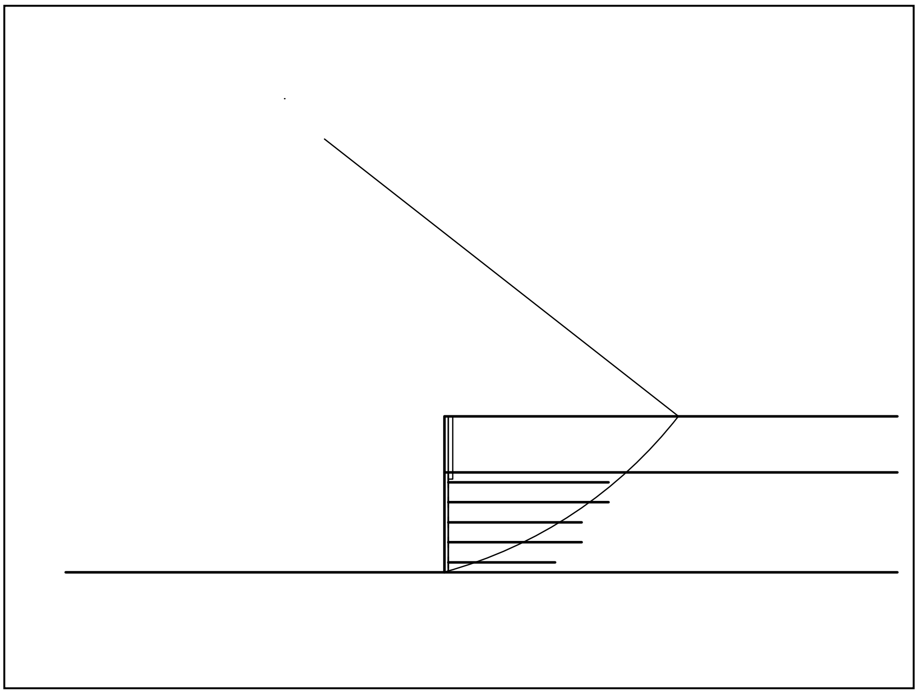
R : RADIUS OF SLIP CIRCULAR ARC (m)

k_{h3} : HORIZONTAL SEISMIC COEFFICIENT = 0.12 (QUOTED FROM "DESIGN CONDITION / DESIGN HORIZONTAL SEISMIC INTENSITY")

y_G : VERTICAL DISTANCE FROM CENTER OF CIRCULAR ARC TO CENTER OF GRAVITY OF SPLIT PIECE (m)

5.2.3 CALCULATION OF SAFETY FACTOR OF CIRCULAR SLIP THAT TRAVERSES REINFORCED AREA [EARTHQUAKE-1]

(1) SHAPE OF SLIP SLIDING



(2) AVAILABLE TENSILE STRENGTH

$$T_{availi} = \min(T_{pEi},\ T_{AEi})$$

$$T_{pEi} = \frac{2 \cdot f_i{*} \cdot \sigma_{VEi} \cdot b \cdot L_{ei}}{F_{SE}} \times \frac{1}{\triangle B}$$

T_{availi} : AVAILABLE TENSILE STRENGTH AT iTH STAGE STRIP (kN/m)

T_{pEi} : DRAWING RESISTANCE IN ANCHORAGE ZONE AFFECTING iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

T_{AEi} : ALLOWABLE TENSILE STRENGTH AT iTH STAGE STRIP DURING EARTHQUAKE (kN/m)

$f_i{*}$: APPARENT COEFFICIENT OF FRICTION AFFECTING iTH STAGE STRIP

σ_{VEi} : VERTICAL EARTH PRESSURE AT CENTER OF ANCHORAGE ZONE AT iTH STAGE $= \gamma \cdot z_i$ (kN/m^2)

z_i : DISTANCE FROM CENTER OF ANCHORAGE ZONE AT iTH STAGE TO LEVEE CROWN OF SURCHARGE BACKFILL (m)

b : WIDTH OF STRIP (m)

L_{ei} : ANCHORAGE LENGTH ON DEEPER SIDE OF STRIP LINE (m)

$\triangle B$: HORIZONTAL INSTALLATION INTERVAL OF THE STRIP (m)

F_{SE} : SAFETY FACTOR AGAINST STRIP PULLOUT ON THE EARTHQUAKE = 1.20

STAGE (i)	f_i^*	γ (kN/m^3)	z_i (m)	σ_{VEi} (kN/m^2)	b (m)	L_{ei} (m)	⊿B (m)	T_{pEi} (kN/m)	T_{AEi} (kN/m)	T_{availi} (kN/m)
1	1.1809			0.000	0.0800	0.000	0.750	0.000	67.200	0.000
2	1.0843	19.0 19.0	2.100 1.125	61.275	0.0800	0.706	0.750	8.339	67.200	8.339
3	0.9876	19.0 19.0	2.100 1.875	75.525	0.0800	0.826	0.750	10.953	67.200	10.953
4	0.8909	19.0 19.0	2.100 2.625	89.775	0.0800	2.187	0.750	31.097	67.200	31.097
5	0.7942	19.0 19.0	2.100 3.375	104.025	0.0800	2.956	0.750	43.417	67.200	43.417
								$\Sigma T_{avail} = 93.805$		

(3) LIST OF SAFETY FACTOR

F_{SE}		COORDINATE OF CIRCULAR CENTER X						
		-10.5	-8.5	-6.5	-4.5	-2.5	-0.5	1.5
Y	23.0	1.562	1.530	1.527	*****	*****	*****	*****
	21.0	1.586	1.547	1.522	*****	*****	*****	*****
	19.0	1.620	1.572	1.529	*****	*****	*****	*****
	17.0	1.667	1.607	1.554	1.516	*****	*****	*****
	15.0	1.735	1.659	1.592	1.535	*****	*****	*****
	13.0	1.839	1.739	1.651	1.577	*****	*****	*****
	11.0	2.003	1.868	1.748	1.646	*****	*****	*****

(4) CALCULATION RESULTS FOR CIRCULAR SLIP SAFETY WITH REINFORCEMENT

( )ALLOWABLE VALUE

ITEM	CODE	UNIT	EARTHQUAKE-1	JUDGE
MINIMUM SAFETY FACTOR	F_{smin} F_{SE}		1.516 (1.000)	○
HORIZONTAL SEISMIC COEFFICIENT	k_{h3}		0.12	—
ΣT_{avail}	kN/m		93.805	—
RESISTER MOMENT	[M_{RC}] [M_{RF}]	kN·m/m	[1808.2] [4804.5]	—
	M_R	kN·m/m	6612.7	—
	M_T	kN·m/m	1810.5	—
ACTIVE MOMENT	M_D	kN·m/m	5556.9	—
THE CENTER OF THE SLIP CIRCLE X COORDINATE Y COORDINATE	X Y	m	-4.500 17.000	—
RADIUS	R	m	16.862	—

M_{RC} : RESISTER MOMENT OF SOIL COHESION (c) (kN·m/m)

M_{RF} : RESISTER MOMENT OF ANGLE OF SHEARING RESISTANCE (ϕ) (kN·m/m)

M_R : RESISTER MOMENT OF SOIL $= M_{RC} + M_{RF}$ (kN·m/m)

M_T : RESISTER MOMENT OF STRIP (kN·m/m)

F_{smin} : MINIMUM SAFETY FACTOR OF STABILITY CONCERNING CIRCULAR SLIP $= (M_R + M_T) / M_D$

Mechanically-Stabilized Earth Wall Type-1

(H=2150)

1 Design Conditons

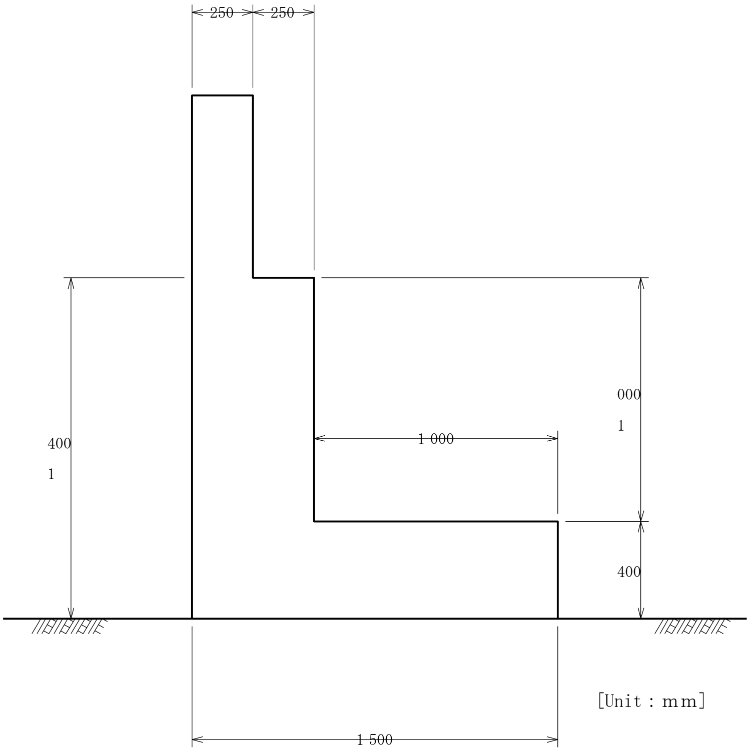
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 8300(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)
RC Bottom Slab: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil

Bearing/Supporting ground: Sand

【Angle of Internal Friction】Soil at Rear Side: 30°

【Unit Weight】

(kN/m^3)

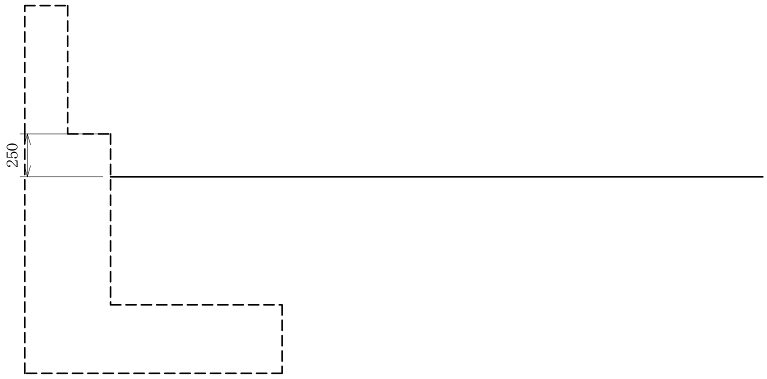
Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】

Frame: Kh = 0.18
Soil (Front side): Kh = 0.18
Soil (Rear side): Kh = 0.18

1.6 Soil

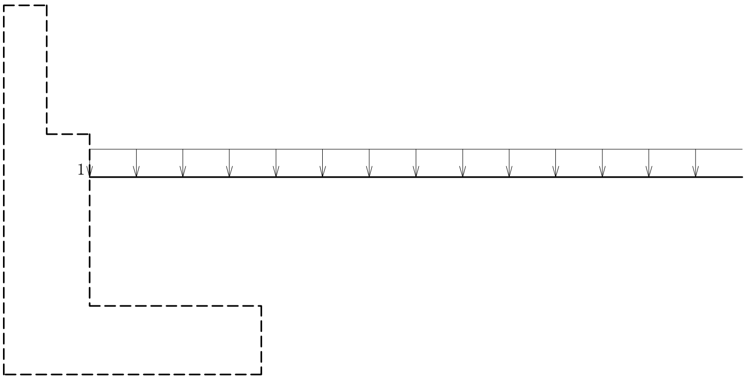
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr (m)	0.000

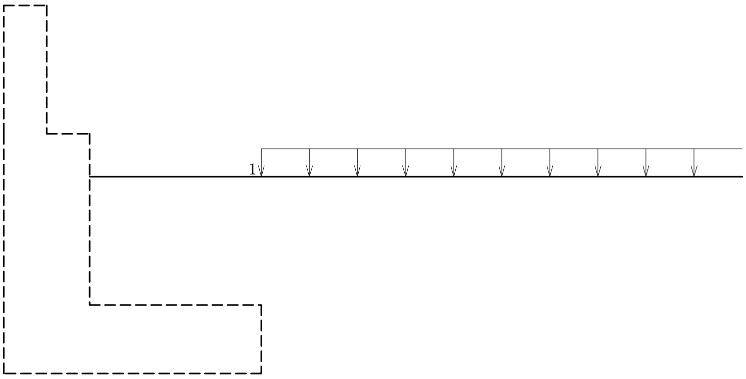
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m²)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

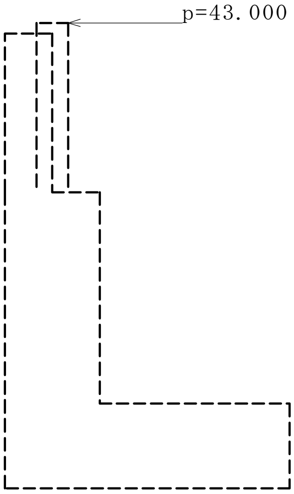


Number	Load Location (m)	Load Width (m)	Load (kN/m²)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.900	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.800	1.000	43.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method
- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

- In the stability analysis, the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).

- In the stability analysis, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	――
Earthquake	0.000	0.000	――

- Earth Pressure with Direction Other Than the Direction of Earthquake

	Treatment	Effectiveness Ratio
Stability Analysis	Ordinary Earth Pressure	0.500
Vertical Wall Design		0.500

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	――	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	——	——	——	——
	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, $\tan\varphi_B$	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)

e_B : Load Eccentricity (m), wherein: $e_B = M_B / V$

M_B : Moment in the base of foundation (kN.m)

V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

2) Bottom Slab (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress	
				τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

τ_{a1} : Shear stress when shear force is bored by concrete

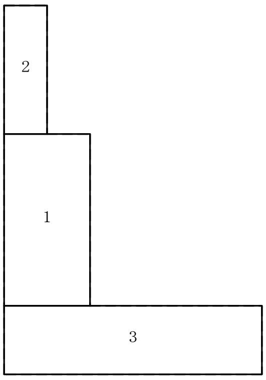
τ_{a2} : Shear stress when shared with diagonal tension bars

2 Stability Analysis

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



2) Volume・Center of Gravity

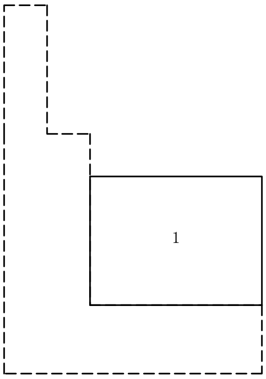
Classification	Width x Height x Depth	Vol. Vi (m³)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2 3	0.500× 1.000× 1.000 0.250× 0.750× 1.000 1.500× 0.400× 1.000	0.500 0.188 0.600	0.250 0.125 0.750	0.900 1.775 0.200	0.125 0.023 0.450	0.450 0.333 0.120	
Σ		1.288	——	——	0.598	0.903	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.598／ 1.288 ＝ 0.465 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.903／ 1.288 ＝ 0.701 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m³)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	1.000× 0.750× 1.000	0.750	1.000	0.775	0.750	0.581	
Σ		0.750	——	——	0.750	0.581	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.750／ 0.750 ＝ 1.000 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.581／ 0.750 ＝ 0.775 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.288 = 31.544	0.465
Soil (Rear Side)	19.000 × 0.750 = 14.250	1.000
Total	45.794	0.631

[2] Earthquake

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.288 = 31.544	0.465
Soil (Rear Side)	19.000 × 0.750 = 14.250	1.000
Total	45.794	0.631

Location	Horizontal Force $H = W \cdot kh$ (kN)	Location Y (m)
Frame (Rebar)	31.544 × 0.18 = 5.678	0.701
Soil (Rear Side)	14.250 × 0.18 = 2.565	0.775
Total	8.243	0.724

2.3 Ground Surface Loading, Snow Load

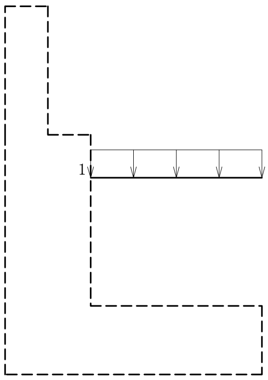
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

q : Load
L : Length of load
X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.000	11.600	1.000

2.4 Collision Load

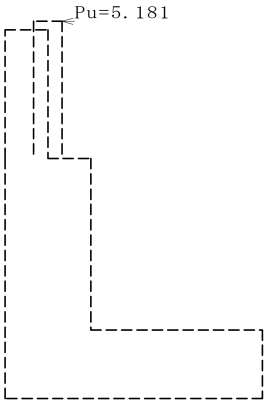
Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

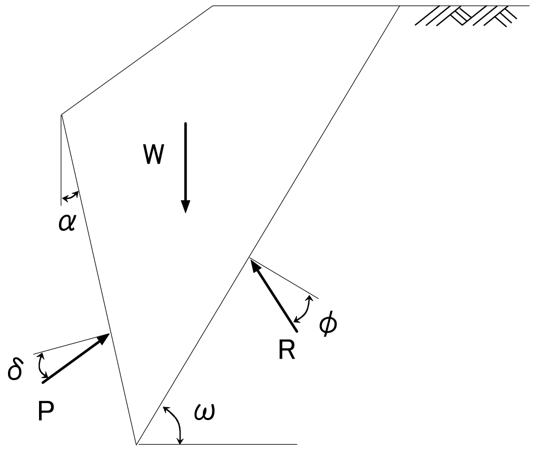
Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 8.300(m)

[1] Collision



p (kN)	Pu (kN/m)	Location Y (m)
43.000	5.181	2.200

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	8.015	15.564	8.627
60.00	7.254	0.000	7.702	14.956	8.635
61.00	6.965	0.000	7.394	14.359	8.628

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 8.635 kN

Earth Pressure, P

$$P = \frac{W\cdot\sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{14.956\times\sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

＝ 8.635 kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

Ph ＝ P・cos(α＋δ) ＝ 8.635×cos(0.000°＋ 0.000°) ＝ 8.635 kN

Vertical Component

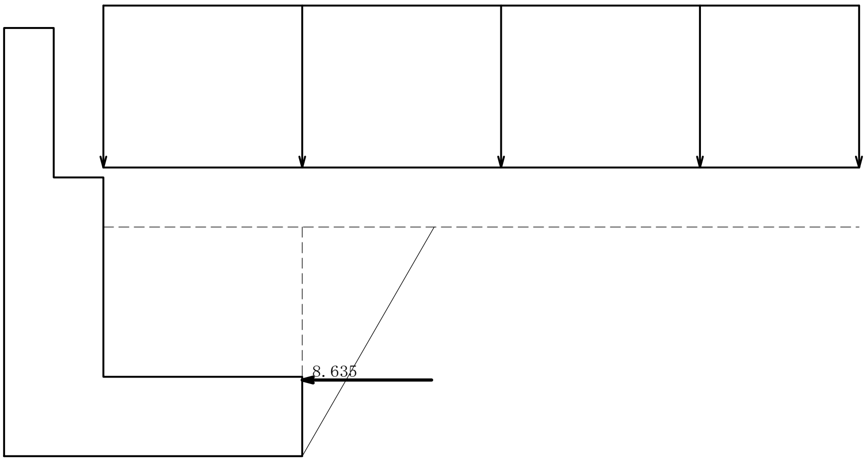
Pv ＝ P・sin(α＋δ) ＝ 8.635×sin(0.000°＋ 0.000°) ＝ 0.000 kN

Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383\ \mathrm{m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m
$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	xp =	1.500 m
	yp =	0.000 m
Height of Assumed Rear Surface	H =	1.150 m
Angle to the Vertical of Earth Pressure Surface	α =	0.000 °
Unit Weight of Soil at Rear Side	γs =	19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ =	30.000 °
Angle of Ground Surface from the Horizontal	β =	0.000 °
Composite angle during earthquake	θ =	$\tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω = 44.00° P = 5.508 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{13.011/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

= 5.508 kN

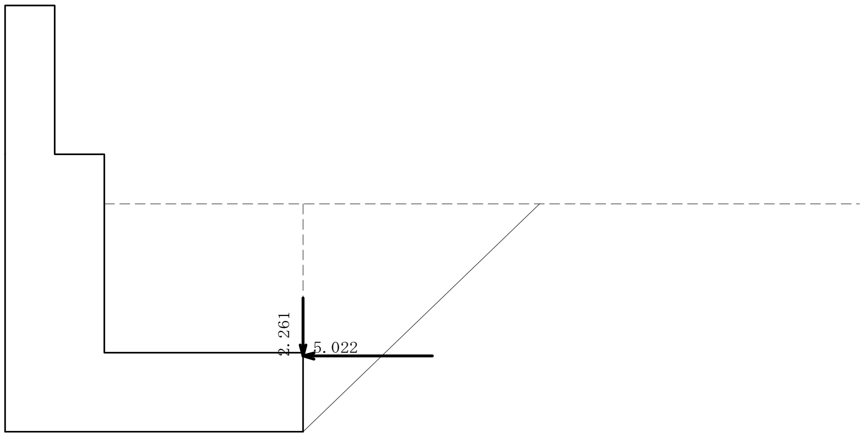
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 5.508 \times \cos(0.000° + 24.239°) = 5.022$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 5.508 \times \sin(0.000° + 24.239°) = 2.261$ kN
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m
$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

· Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance for. toe) xp ＝ 1.500 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical α ＝ 0.000 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal β ＝ 0.000 °
Wall Friction Angle δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	0.000	7.549	4.184
60.00	7.254	0.000	0.000	7.254	4.188
61.00	6.965	0.000	0.000	6.965	4.185

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 4.188 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.254 \times \sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

＝ 4.188 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha+\delta) = 4.188 \times \cos(0.000° + 0.000°) = 4.188$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha+\delta) = 4.188 \times \sin(0.000° + 0.000°) = 0.000$ kN

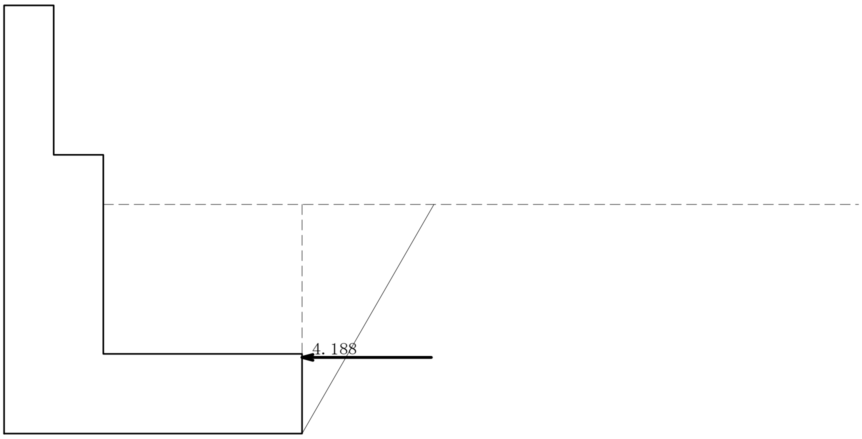
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m

$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

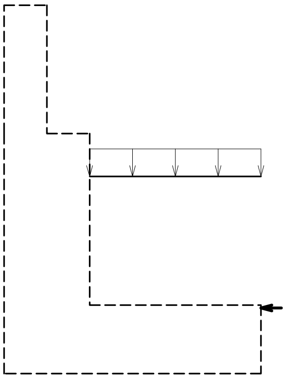
- Earth Pressure Diagram



2.6 Total Acting Force

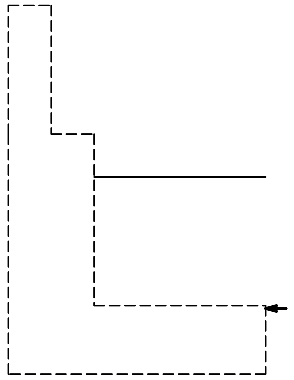
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



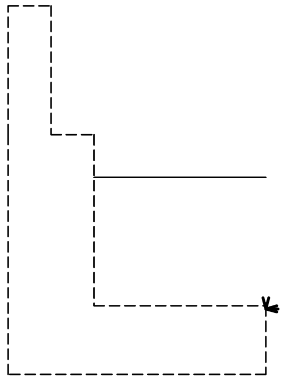
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Loading, Snow	11.600	0.000	1.000	0.000	11.600	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	57.394	8.635	———	———	40.512	3.307

[2] Ordinary Condition (Rear Side)



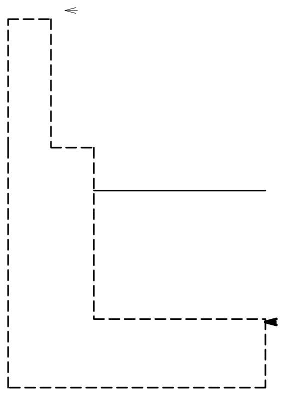
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	45.794	8.635	———	———	28.912	3.307

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	45.794	8.243	0.631	0.724	28.912	5.969
Earth Pressure	2.261	5.022	1.500	0.383	3.391	1.923
Total	48.055	13.265	———	———	32.303	7.893

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Loading, Snow	0.000	4.188	1.500	0.383	0.000	1.604
Collision Load	0.000	5.181	0.000	2.200	0.000	11.398
Total	45.794	9.369	———	———	28.912	13.002

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	57.394	8.635	37.204
Ordinary (Rear Side)	45.794	8.635	25.604
Earthquake	48.055	13.265	24.410
Collision	45.794	9.369	15.910

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)

Horizontal Force : $H_c = H_o$ (kN)

Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,

Footing Earth Pressure Width : $B_j = 1.500$ (m)

- ■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	57.394	8.635	5.841
Ordinary (Rear Side)	45.794	8.635	8.741
Earthquake	48.055	13.265	11.631
Collision	45.794	9.369	18.435

■ Full Width per 8,300m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	476.369	71.671	48.479
Ordinary (Rear Side)	380.089	71.671	72.549
Earthquake	398.855	110.099	96.534
Collision	380.089	77.760	153.013

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B ＝ 1.500

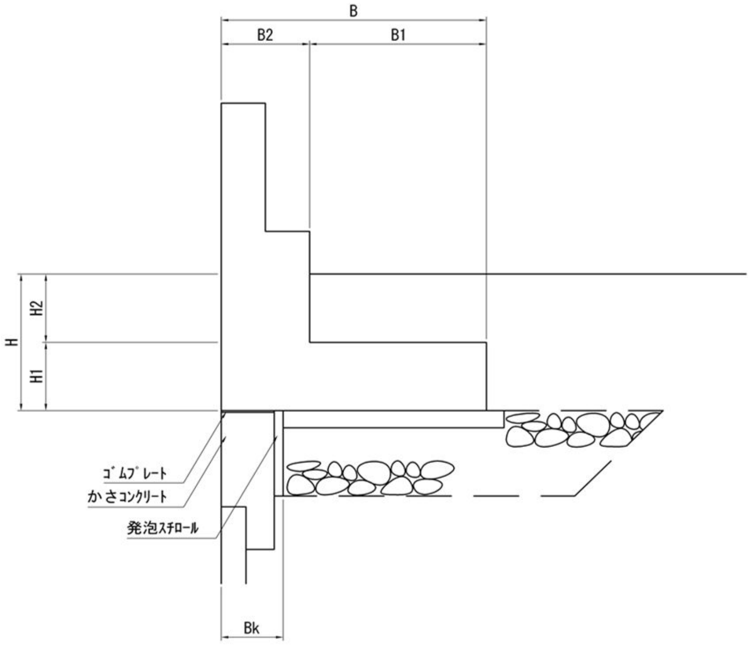
$$e_a = B / n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	40.512	3.307	57.394	0.648	0.102	≦	0.250	○
Ordinary (Rear Side)	28.912	3.307	45.794	0.559	0.191	≦	0.250	○
Earthquake	32.303	7.893	48.055	0.508	0.242	≦	0.500	○
Collision	28.912	13.002	45.794	0.347	0.403	≦	0.500	○

2.7.2 Stability from Sliding



形状寸法

Ｈ	(m)	1.150
Ｈ１	(m)	0.400
Ｈ２	(m)	0.750
Ｂ	(m)	1.500
Ｂ１	(m)	1.000
Ｂ２	(m)	0.500
Ｂｋ	(m)	0.350

1) Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m^2)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.191	1.500	7.220	53.838	53.838	42.96
Earthquake	0.242	1.500	1.022	63.052	63.052	48.57
Collision	0.403	1.041	0.000	87.980	87.980	58.40

2) Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Ｎｎａ (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN’=ΣN-Ｎｎａ	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	45.794	16.940	28.854	8.635
Earthquake	48.055	19.535	28.520	13.265
Collision	45.794	25.617	20.177	9.369

3) Factor of safety for sliding, Fs:

- $F_s = \Sigma N' \cdot \mu / \Sigma H$

Load Condition	Factor of Safety F s	Required Factor of Safety F s a
Ordinary (Rear Side)	2.005	≧1.5
Earthquake	1.290	≧1.2
Collision	1.292	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

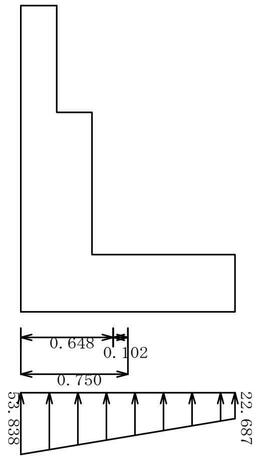
1) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

Where,

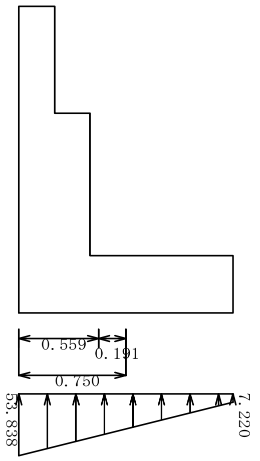
ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)
B : Bottom plate width (m), B = 1.500
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



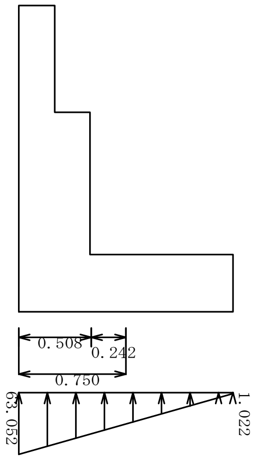
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.500	Trapezoid	22.687	53.838	≦	200.000	○

[2] Ordinary Condition (Rear Side)



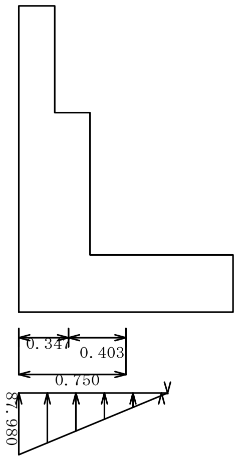
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.500	Trapezoid	27.220	53.838 ≦	200.000	○

[3] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.500	Trapezoid	1.022	63.052 ≦	300.000	○

[4] Collision



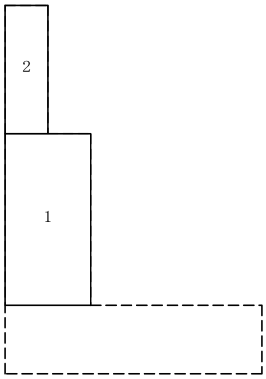
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.041	Triangle	0.000	87.980 ≦	300.000	○

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.500× 1.000× 1.000 0.250× 0.750× 1.000	0.500 0.188	0.250 0.125	0.500 1.375	0.125 0.023	0.250 0.258	
Σ		0.688	—	—	0.148	0.508	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.148／ 0.688 ＝ 0.216 (m)

YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.508／ 0.688 ＝ 0.739 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.688 ＝ 16.844	0.034

Force Location

X ＝ Xc－XG ＝ 0.250－0.216

＝ 0.034 m

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.688 ＝ 16.844	0.034

Location	H ＝ W ・ kh (kN)	Location Y (m)
Frame(Rebar)	16.844 × 0.180 ＝ 3.032	0.739

Force Location

作用位置

$$X = Xc - XG = 0.250 - 0.216$$

$$= 0.034 \text{ m}$$

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

Horizontal Force

$$Pu = \frac{p}{\lambda + h + y}$$

Where,

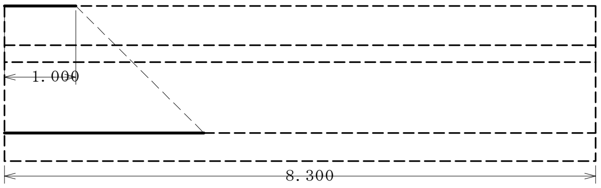
Pu : Collision load of the effective width (kN/m)

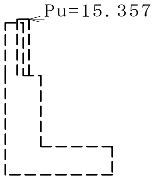
p : Collision load per block (kN)

λ : Load range (m)

y : Distance from the tip of section being checked (m), y ＝ 1.000(m)

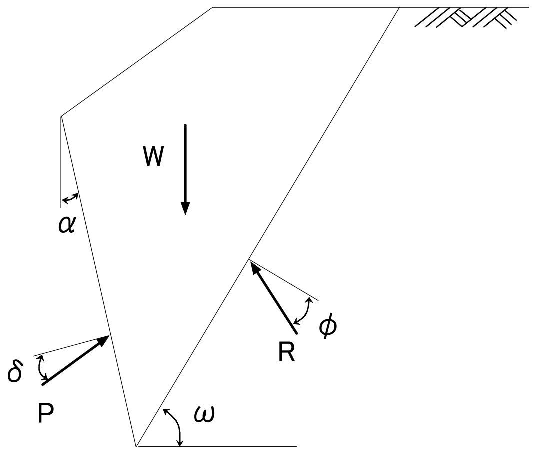
[1] Collision





p (kN)	λ (m)	h (m)	Pu (kN/m)	Location Y (m)
43.000	1.000	0.800	15.357	1.000

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.250 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.750 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	6.092	9.834	4.172
56.00	3.605	0.000	5.868	9.473	4.176
57.00	3.471	0.000	5.650	9.121	4.172

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 4.176 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{9.473 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

＝ 4.176 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha+\delta) = 4.176\times\cos(0.000°+20.000°) = 3.924$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha+\delta) = 4.176\times\sin(0.000°+20.000°) = 1.428$ kN

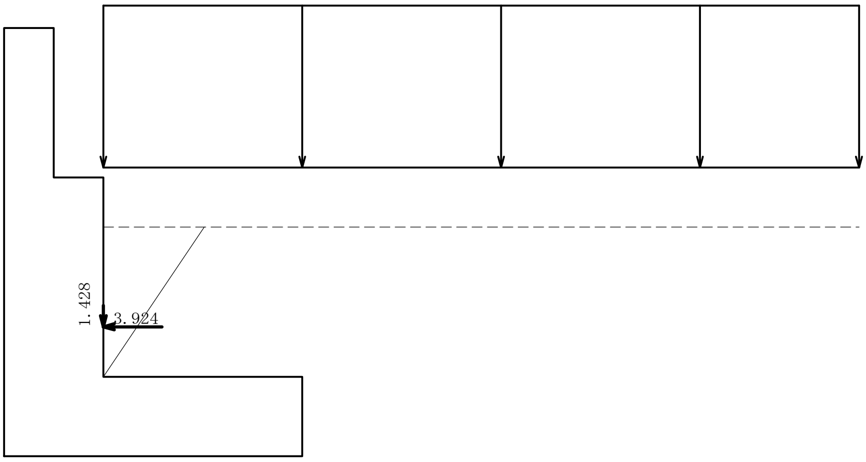
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250\times\tan0.000° - 0.250 = -0.250$ m

$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

- Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.250$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.750$ m
Angle of Earth Pressure Acting Surface to the Vertical $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 2/3\varphi = 20.000$ °
Rate of Change of Slip Angle $\omega i = 10.00$ °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw = 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	0.000	3.742	1.587
56.00	3.605	0.000	0.000	3.605	1.589
57.00	3.471	0.000	0.000	3.471	1.588

Earth pressure becomes maximum when:
$\omega = 56.00°$ $P = 1.589$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{3.605 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

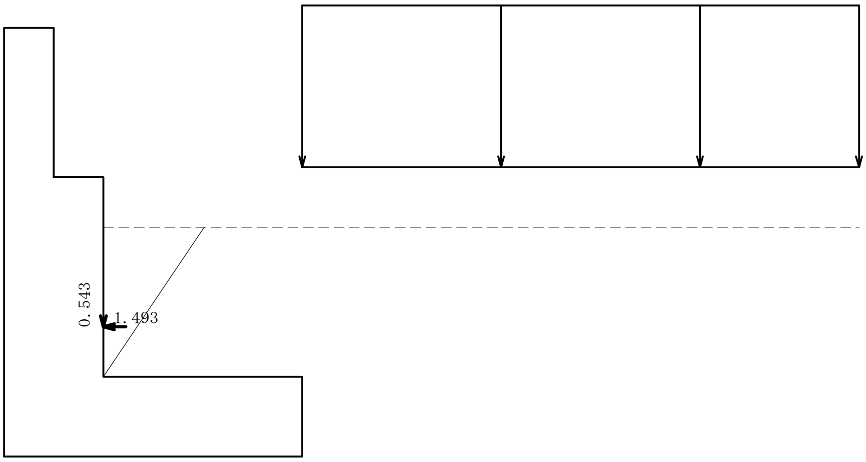
$= 1.589$ kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 1.589 \times \cos(0.000° + 20.000°) = 1.493$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 1.589 \times \sin(0.000° + 20.000°) = 0.543$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.300 = -0.300$ m
$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.250$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.750$ m
Angle to the Vertical of Earth Pressure Surface $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 1/2\varphi = 15.000$ °
Composite angle during earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$
Rate of Change of Slip Angle $\omega i = 10.00$ °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
45.00	5.344	0.000	0.000	5.344	2.312
46.00	5.161	0.000	0.000	5.161	2.316
47.00	4.983	0.000	0.000	4.983	2.316

Earth pressure becomes maximum when:
$\omega = 46.00°$ $P = 2.316$ kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.161/\cos 10.204° \times \sin(46.00° - 30.00° + 10.204°)}{\cos(46.00° - 30.00° - 0.000° - 15.000°)}$$

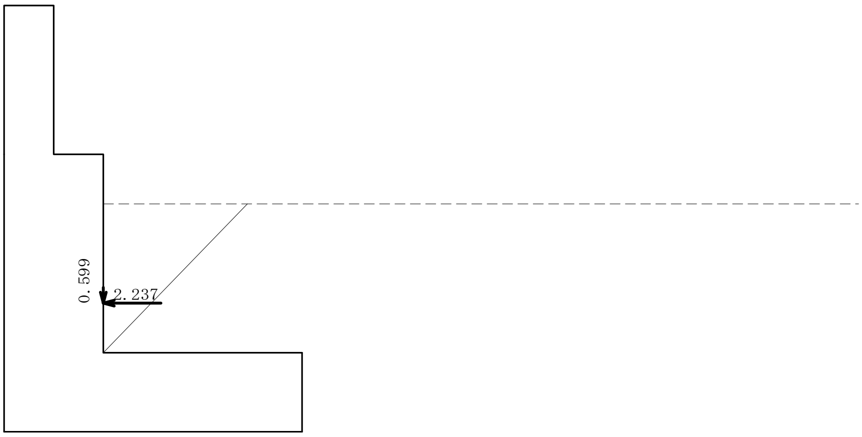
＝ 2.316 kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 2.316 \times \cos(0.000° + 15.000°) = 2.237$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 2.316 \times \sin(0.000° + 15.000°) = 0.599$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.250 = -0.250$ m
$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

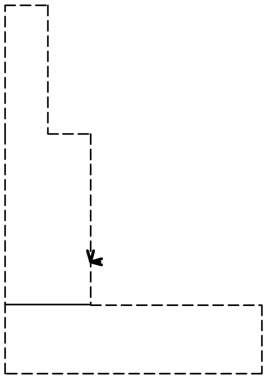
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

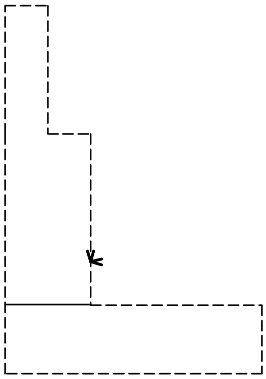
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	1.428	3.924	-0.250	0.250	0.981
Total	0.000	3.924	———	———	0.981

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

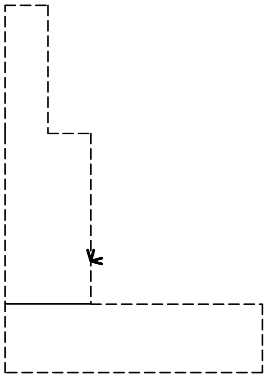
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	0.543	1.493	-0.250	0.250	0.373
Total	0.000	1.493	———	———	0.373

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

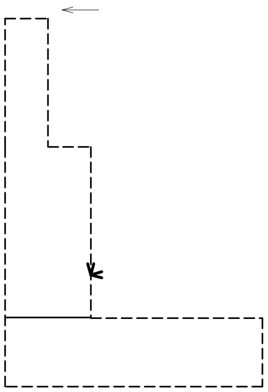
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	16.844	3.032	0.034	0.739	2.239
Earth Pressure	0.599	2.237	-0.250	0.250	0.559
Total	0.000	5.269	———	———	2.799

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

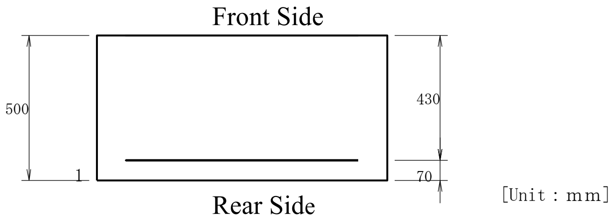


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	0.543	1.493	-0.250	0.250	0.373
Collision Load	0.000	15.357	0.000	1.800	27.643
Total	0.000	16.850	———	———	28.016

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	7.0	D13	1.267	4.000	5.068
	2	—	–	—	—	—

Required rebar amount at tension side: 2.261 (cm2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 41666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 500000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 500.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	1.668	≦	79.737	0.000	5.000	○
Ordinary (Rear Side)	5.068	0.635	≦	79.737	0.000	5.000	○
Earthquake	5.068	4.758	≦	79.737	0.000	5.000	○
Collision	5.068	47.627	≦	79.737	0.000	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu}+\varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu}+\varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 （$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2\right)$$

Concrete compressive force at section y2 （$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars （$d_i \leqq x$）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars （$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 430.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	137.877	○
Ordinary (Rear Side)	5.068	≦	137.877	○
Earthquake	5.068	≦	137.877	○
Collision	5.068	≦	137.877	○

(4) Check of Bending Stress

(Reference)

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2 \cdot n}{b} \left\{ As \cdot (x-d) \right\} = 0.0$$

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2} \cdot \left(\frac{h}{2} - \frac{x}{3} \right) + n \cdot As \cdot \frac{(x-d) \cdot (h/2-d)}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h = 500.000
b : Member cross-section width (mm), b = 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n = 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
				Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	0.000	7.361	0.066	≦	8.000	4.774	≦	180.000	○
Ordinary (Rear Side)	0.373	0.000	7.361	0.025	≦	8.000	1.816	≦	180.000	○
Earthquake	2.799	0.000	7.361	0.188	≦	12.000	13.620	≦	300.000	○
Collision	28.016	0.000	7.361	1.877	≦	12.000	136.340	≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member’s effective height (mm)

b : Sectional member’s effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member’s effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

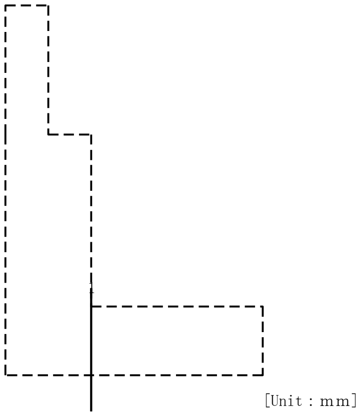
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judge ment
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	3.924	43.000	0.009	≦	0.224	1.700	1.33	0.74	1.00	○
Ordinary(Rear Side)	1.493	43.000	0.003	≦	0.224	1.700	1.33	0.74	1.00	○
Earthquake	5.269	43.000	0.012	≦	0.341	2.550	1.33	0.74	1.00	○
Collision	16.850	43.000	0.039	≦	0.336	2.550	1.33	0.74	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

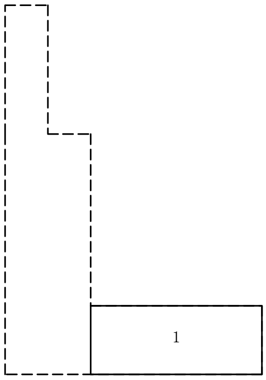
Distance from joint: 0.000 m



4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



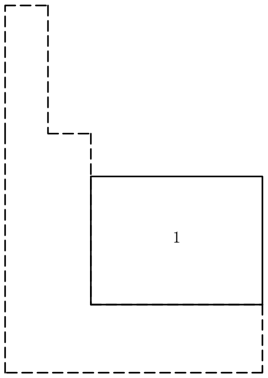
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	1.000× 0.400× 1.000	0.400	0.500	0.200	
Σ		0.400	——	0.200	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.200／ 0.400 ＝ 0.500 (m)

(2) Soil at Rear Side

1) Block Division



3) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	1.000× 0.750× 1.000	0.750	0.500	0.375	
Σ		0.750	——	0.375	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.375／ 0.750 ＝ 0.500 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.400 ＝ 9.800	0.500
Soil(Rear Side)	19.000 × 0.750 ＝ 14.250	0.500

4.1.3 Ground Surface Load・Snow Load

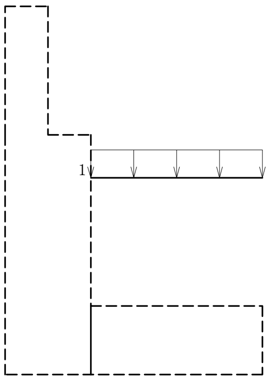
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

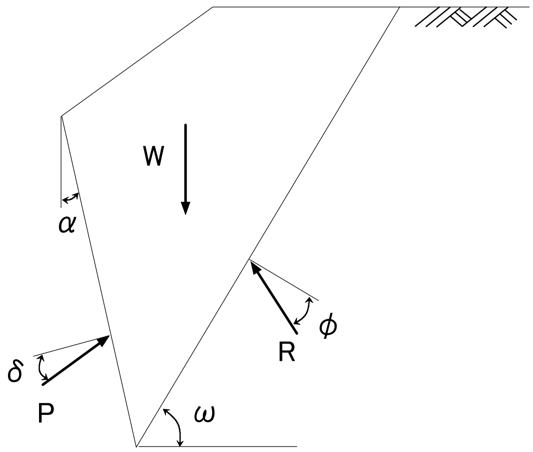
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.000	11.600	0.500

4.1.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) $xp = 1.500$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 1.150$ m
Angle to the vertical of the Earth Pressure Surface $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Angle to the Horizontal of the Ground Surface $\beta = 0.000$ °
Composite Angle during Earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta+\Delta-\beta)}{1-\sin\phi \cdot \cos(\theta+\Delta-\beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1-\sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta+\theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
$\omega = 44.00°$ $P = 5.508$ kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega-\phi+\theta)}{\cos(\omega-\phi-\alpha-\delta)}$$

$$= \frac{13.011/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$$= 5.508 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component
$P_v = P \cdot \sin(\alpha+\delta) = 5.508 \times \sin(0.000° + 24.239°) = 2.261$ kN

The ff. equivalent triangular distributed load of the vertical component will be used:

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 2.261}{1.000} = 4.522 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.000 m
Width of distributed load perpendicular to the design section L2 = 1.000 m

Distributed Load at the Design Section, pd

$$pd = \frac{pv}{L} \cdot L1 = \frac{4.522}{1.000} \times 0.000 = 0.000 \text{ kN/m}$$

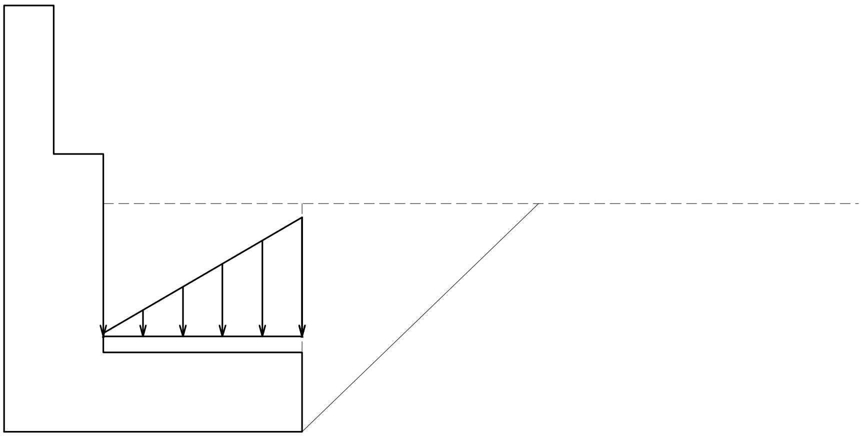
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (0.000 + 4.522) \times 1.000 = 2.261 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{0.000 + 2 \times 4.522}{0.000 + 4.522} \times \frac{1.000}{3} = 0.667 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

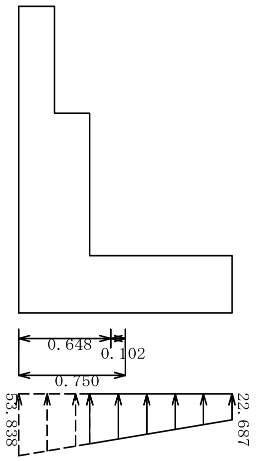
Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

Where,

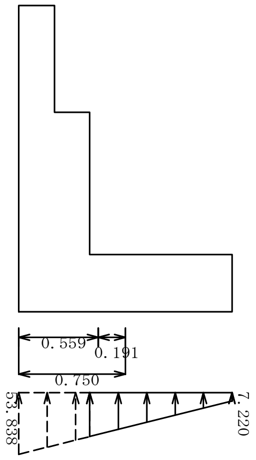
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



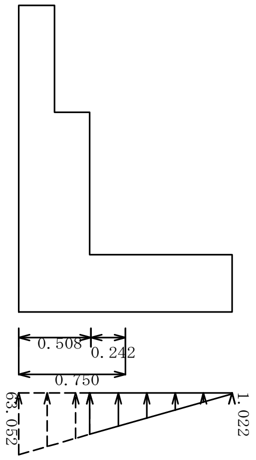
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.687	43.454	1.000	33.071	0.448

[2] Ordinary Condition (Rear Side)



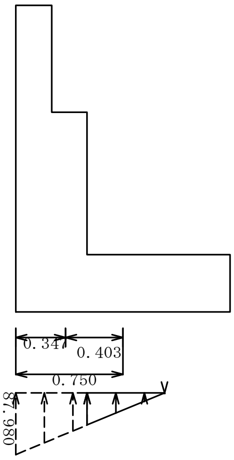
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.220	38.299	1.000	22.759	0.386

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
1.022	42.375	1.000	21.699	0.341

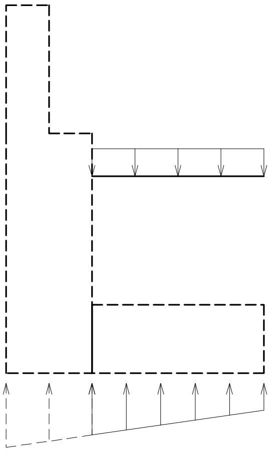
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	45.723	0.541	12.368	0.180

4.1.6 Total Sectional Forces

[1] Ordinary Condition (Surcharge)



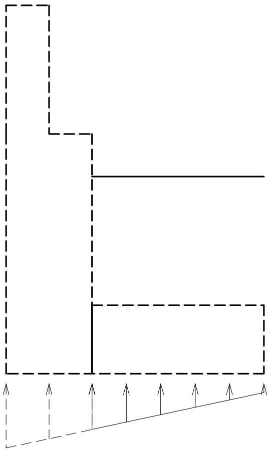
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Loading, Snow Load	11.600	0.500	5.800
Ground Reaction	-33.071	0.448	-14.805
Total	2.579	———	3.020

Sectional force of vertical wall foundation, M1 = 0.981 kN.m

Sectional force of the base of heel, M3 = 2.125 kN.m

Since M3 > M1, M1 is applied as the sectional force of the joint

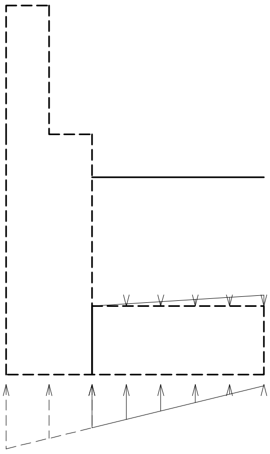
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Ground Reaction	-22.759	———	-8.790
Total	1.291	0.500	3.235

Sectional force of vertical wall foundation, M1 ＝ 0.373 kN.m
Sectional force of the base of heel, M3 ＝ 3.235 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

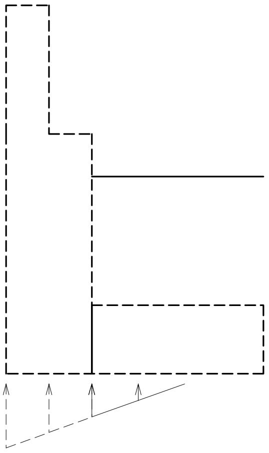
[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Loading, Snow Load	2.261	0.667	1.507
Ground Reaction	-21.699	0.341	-7.403
Total	4.612	———	6.129

Sectional force of vertical wall foundation, M1 ＝ 2.799 kN.m
Sectional force of the base of heel, M3 ＝ 6.129 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

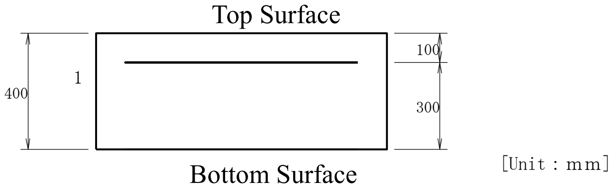
[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Ground Reaction	-12.368	0.180	-2.230
Total	11.682	———	9.795

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.0	D13	1.267	4.000	5.068
	2	——	–	———	——	———
Bottom Surface	1'	——	–	———	——	———
	2'	——	–	———	——	———

Required tension side rebar amount 1.127 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 400.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m) ×1.7		M_c (kN.m)	Min. Rebar Amount (cm^2)	Judge ment
Ordinary (Surcharge)	5.068	1.668	≦	51.031	5.000	○
Ordinary (Rear Side)	5.068	0.635	≦	51.031	5.000	○
Earthquake	5.068	4.758	≦	51.031	5.000	○
Collision	5.068	16.651	≦	51.031	5.000	○

If the 1.7 x actual bending moment is:

・less than Mc, Min. rebar amount is 5.0

・greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（0.002≦ε(y)≦εcu）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars（d_i≦x）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars（d_j＞x）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

Stress Calculation

$$\sigma_c=\frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s=n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h ＝ 400.000

b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^2)

n : Rebar and concrete’s coefficient of Young’s modulus, n ＝ 15.00

e : Distance from the sectional member’s centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^2)

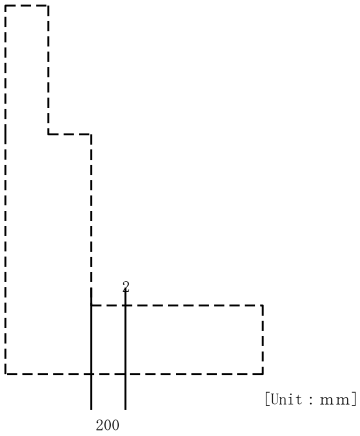
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	6.035	0.116	≦	8.000	6.916	≦	180.000	○
Ordinary (Rear Side)	0.373	6.035	0.044	≦	8.000	2.632	≦	180.000	○
Earthquake	2.799	6.035	0.331	≦	12.000	19.732	≦	300.000	○
Collision	9.795	6.035	1.159	≦	12.000	69.055	≦	300.000	○

4.2 Design of Section [2]

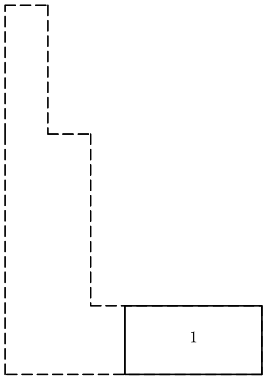
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



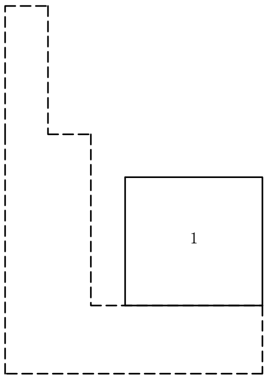
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.800× 0.400× 1.000	0.320	0.400	0.128	
Σ		0.320	——	0.128	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.128／ 0.320 ＝ 0.400 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.800× 0.750× 1.000	0.600	0.400	0.240	
Σ		0.600	——	0.240	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.240／ 0.600 ＝ 0.400 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force W ＝ γ ・ V (kN)	Location X (m)
Frame	24.500 × 0.320 ＝ 7.840	0.400
Soil(Rear Side)	19.000 × 0.600 ＝ 11.400	0.400

4.2.3 Ground Surface Load・Snow Load

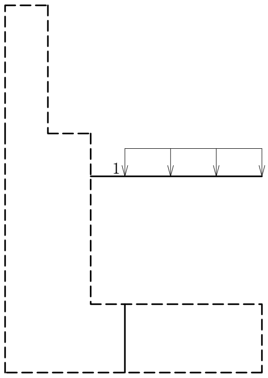
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

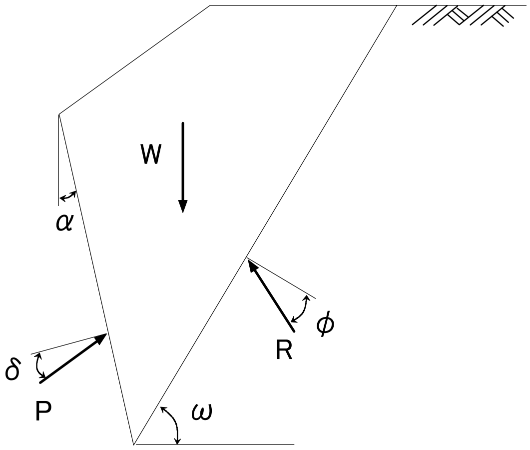
q : Ground surface loading
L : Length of ground surface load
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11. 600	11. 600	0. 800	9. 280	0. 400

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle to the vertical of the Earth Pressure Surface	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle to the Horizontal of the Ground Surface	β ＝ 0.000 °
Composite Angle during Earthquake	θ ＝ $\tan^{-1}$kh＝ $\tan^{-1}$0.18 ＝10.204°

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00^\circ \times \sin(10.204^\circ + 20.751^\circ - 0.000^\circ)}{1 - \sin 30.00^\circ \times \cos(10.204^\circ + 20.751^\circ - 0.000^\circ)}$$

$$= 24.239^\circ$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000^\circ + 10.204^\circ)}{\sin 30.00^\circ} = 20.751^\circ$$

Rate of Change of Slip Angle $\omega i = 10.00°\sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:

$\omega = 44.00^\circ \quad P = 5.508$ kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{13.011/\cos 10.204^\circ \times \sin(44.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(44.00^\circ - 30.00^\circ - 0.000^\circ - 24.239^\circ)}$$

＝ 5.508 kN

Vertical component of Earth Pressure:

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 5.508 \times \sin(0.000^\circ + 24.239^\circ) = 2.261$ kN

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2 \cdot Pv}{L} = \frac{2 \times 2.261}{1.000} = 4.522 \text{ kN/m}$$

Where,

pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.200 m
Width of distributed load perpendicular to the design section L2 = 0.800 m

$$pd = \frac{pv}{L} \cdot L1 = \frac{4.522}{1.000} \times 0.200 = 0.904 \text{ kN/m}$$

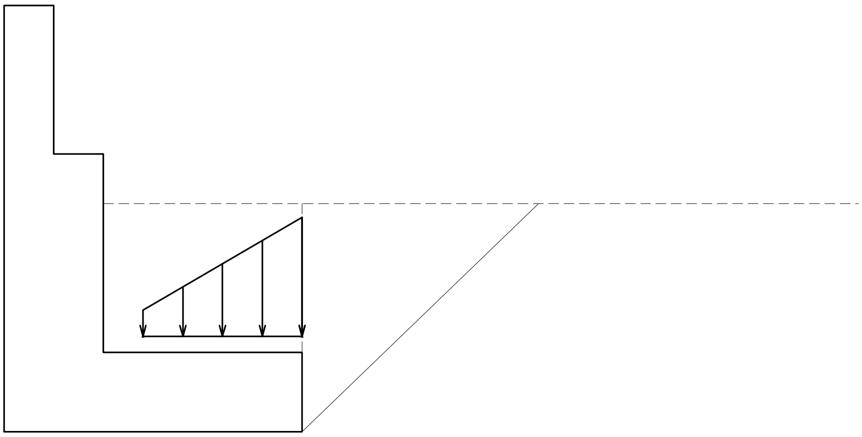
Vertical Force

$$N = \frac{1}{2} \cdot (pd+pv) \cdot L2 = \frac{1}{2} \times (0.904+4.522) \times 0.800 = 2.171 \text{ kN}$$

Location

$$x = \frac{pd+2 \cdot pv}{pd+pv} \cdot \frac{L2}{3} = \frac{0.904+2\times4.522}{0.904+4.522} \times \frac{0.800}{3} = 0.489 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2) \cdot L$$

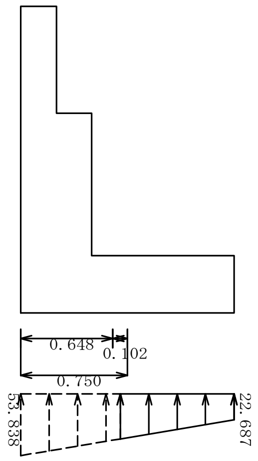
Location

$$X = \frac{2 \cdot q1+q2}{3 \cdot (q1+q2)} \cdot L$$

Where,

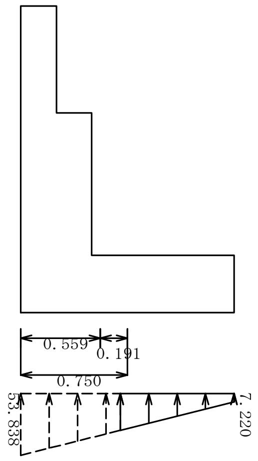
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



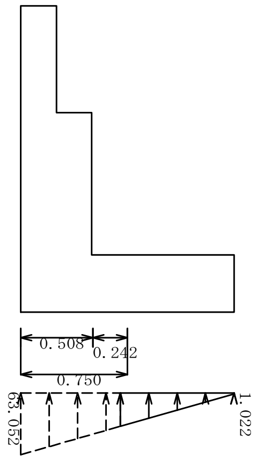
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.687	39.301	0.800	24.795	0.364

[2] Ordinary Condition (Rear Side)



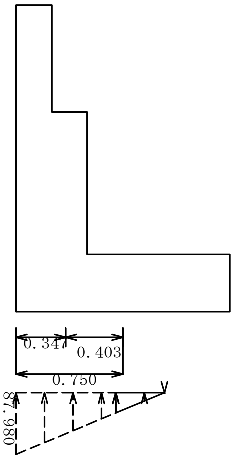
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.220	32.083	0.800	15.721	0.316

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
1.022	34.105	0.800	14.051	0.274

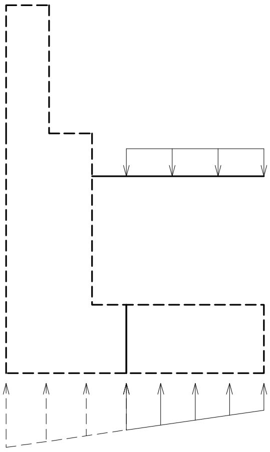
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	28.820	0.341	4.914	0.114

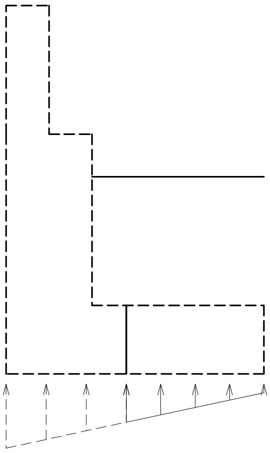
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



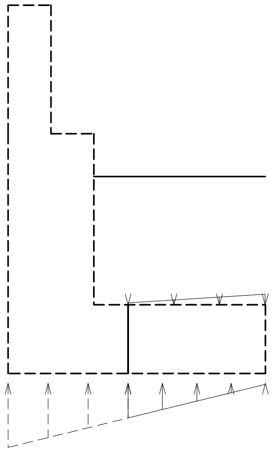
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Loading, Snow Load	9.280	0.400	3.712
Ground Reaction	-24.795	0.364	-9.032
Total	3.725	———	2.376

[2] Ordinary Condition (Rear Side)



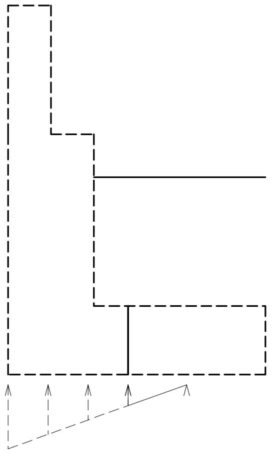
Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Ground Reaction	-15.721	0.316	-4.962
Total	3.519	———	2.734

[3] Earthquake



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Loading, Snow Load	2.171	0.489	1.061
Ground Reaction	-14.051	0.274	-3.856
Total	7.360	———	4.901

[4] Collision



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Ground Reaction	-4.914	0.114	-0.559
Total	14.326	———	7.137

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ $a > 2.5d$,

when $a \leqq 2.5d$, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
M : Bending moment acting on the sectional member (N.mm)
θ : Angle of the top surface footing to the horizontal, $\theta = 0.000$, $\tan\theta = 0.000$
a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span (cm) 2.5・d		a	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000	≦	125.000	3.725	2.376	0.000	3.725
Ordinary (Rear Side)	30.000	75.000	≦	125.000	3.519	2.734	0.000	3.519
Earthquake	30.000	75.000	≦	125.000	7.360	4.901	0.000	7.360
Collision	30.000	75.000	≦	108.840	14.326	7.137	0.000	14.326

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm : Average shear stress of concrete (N/mm2)
Sh : Actual Shear Force (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
τa1 : Allowable shear stress (N/mm2)

$$\tau a1 = Ce \cdot Cpt \cdot Cdc \cdot \tau a1'$$

Where,

τa1' : Allowable shear stress when shear force acts in concrete (N/mm2)
Ce : Correction factor for the sectional member's effective height
Cpt : Correction factor for the tensile main reinforcement ratio, Pt
Cdc : Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	3.725	30.000	0.012	≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	3.519	30.000	0.012	≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	7.360	30.000	0.025	≦	0.411	2.550	1.40	0.84	1.00	○
Collision	14.326	30.000	0.048	≦	0.405	2.550	1.40	0.84	1.00	○

Mechanically-Stabilized Earth Wall Type-2

(H=2450)

1 Design Conditons

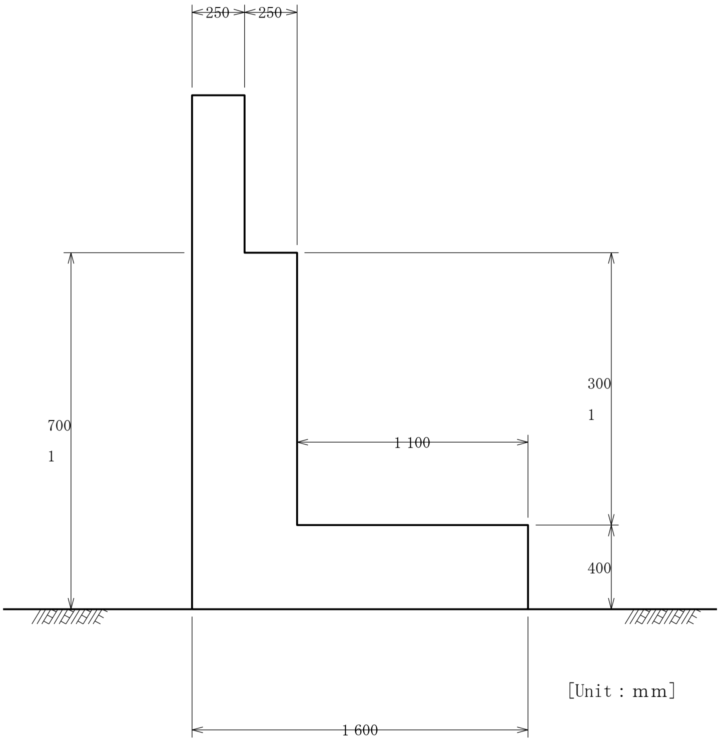
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 10000(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)
RC Bottom Slab: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】 Soil at Rear Side: 30°

【Unit Weight】

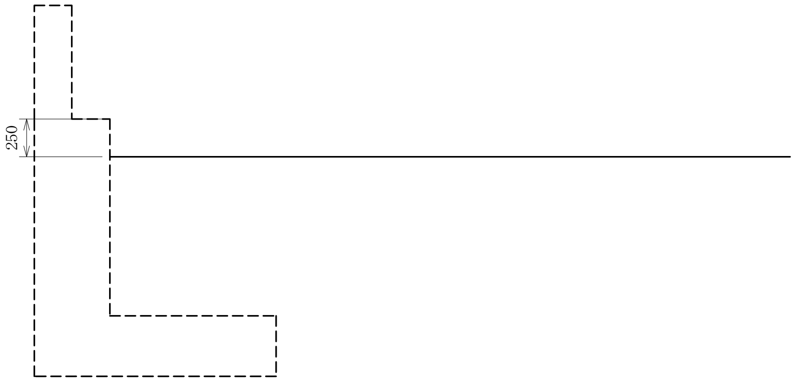
(kN/m^3)

Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】
Frame: Kh ＝ 0.18
Soil (Front side): Kh ＝ 0.18
Soil (Rear side): Kh ＝ 0.18

1.6 Soil

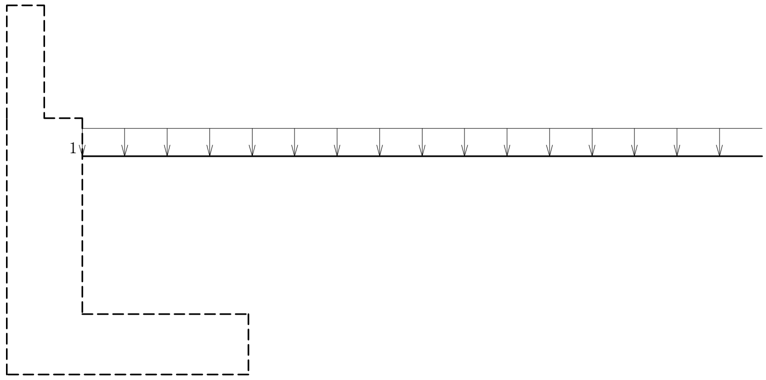
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr (m)	0.000

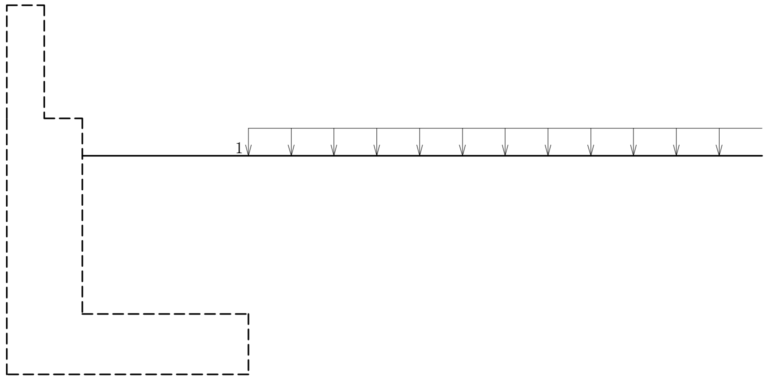
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

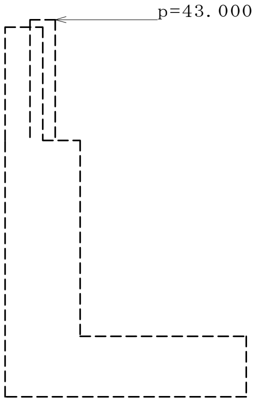


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	1.100	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.800	1.000	43.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method
- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

- In the stability analysis, the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).
- In the stability analysis, the angle formed by the earth pressure's acting surface with the

vertical is 0.000 degrees.

- In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	———
Earthquake	0.000	0.000	———

- Earth Pressure with Direction Other Than the Direction of Earthquake

	Treatment	Effectiveness Ratio
Stability Analysis	Ordinary Earth Pressure	0.500
Vertical Wall Design		0.500

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	——	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability・Section	Stability・Section	Stability・Section	Stability・Section
Limit State Design	Checking Efficiency	——	——	——	——

Method	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, tanφ_B	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B ／ V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

2) Bottom Slab (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}

Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

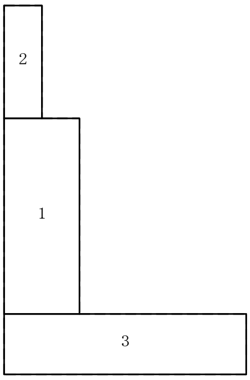
τ_{a1} : Shear stress when shear force is bored by concrete
τ_{a2} : Shear stress when shared with diagonal tension bars

2 **Stability Analysis**

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



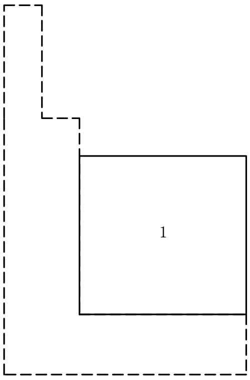
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2 3	0.500× 1.300× 1.000 0.250× 0.750× 1.000 1.600× 0.400× 1.000	0.650 0.188 0.640	0.250 0.125 0.800	1.050 2.075 0.200	0.162 0.023 0.512	0.682 0.389 0.128	
Σ		1.477	——	——	0.698	1.200	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.698／ 1.477 ＝ 0.472 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 1.200／ 1.477 ＝ 0.812 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	1.100× 1.050× 1.000	1.155	1.050	0.925	1.213	1.068	
Σ		1.155	——	——	1.213	1.068	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 1.213／ 1.155 ＝ 1.050 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 1.068／ 1.155 ＝ 0.925 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.477 = 36.199	0.472
Soil (Rear Side)	19.000 × 1.155 = 21.945	1.050
Total	58.144	0.690

[2] Earthquake

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.477 = 36.199	0.472
Soil (Rear Side)	19.000 × 1.155 = 21.945	1.050
Total	58.144	0.690

Location	Horizontal Force $H = W \cdot kh$ (kN)	Location Y (m)
Frame (Rebar)	36.199 × 0.18 = 6.516	0.812
Soil (Rear Side)	21.945 × 0.18 = 3.950	0.925
Total	10.466	0.855

2.3 Ground Surface Loading, Snow Load

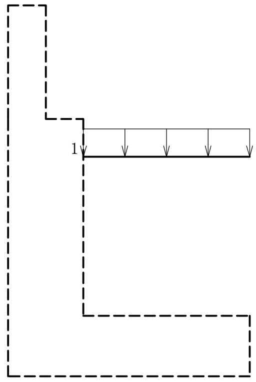
Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

Where,

q : Load
L : Length of load
X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	1.050

2.4 Collision Load

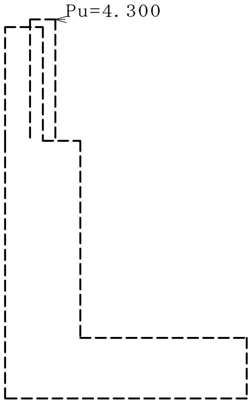
Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

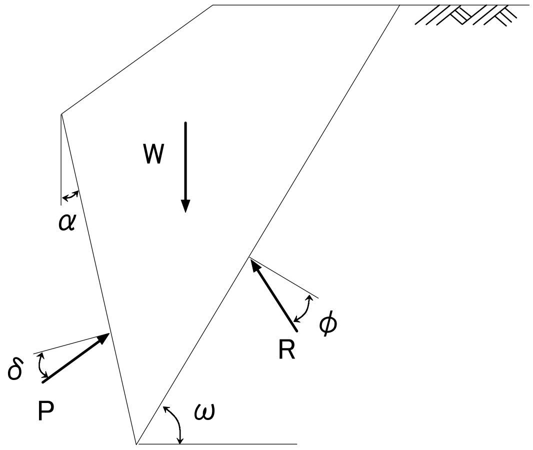
Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 10.000(m)

[1] Collision



p (kN)	Pu (kN/m)	Location Y (m)
43.000	4.300	2.500

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.60m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.450 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	12.001	0.000	10.106	22.107	12.254
60.00	11.532	0.000	9.711	21.243	12.265
61.00	11.071	0.000	9.323	20.394	12.254

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 12.265 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{21.243 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

$= 12.265$ kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 12.265 \times \cos(0.000° + 0.000°) = 12.265$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 12.265 \times \sin(0.000° + 0.000°) = 0.000$ kN

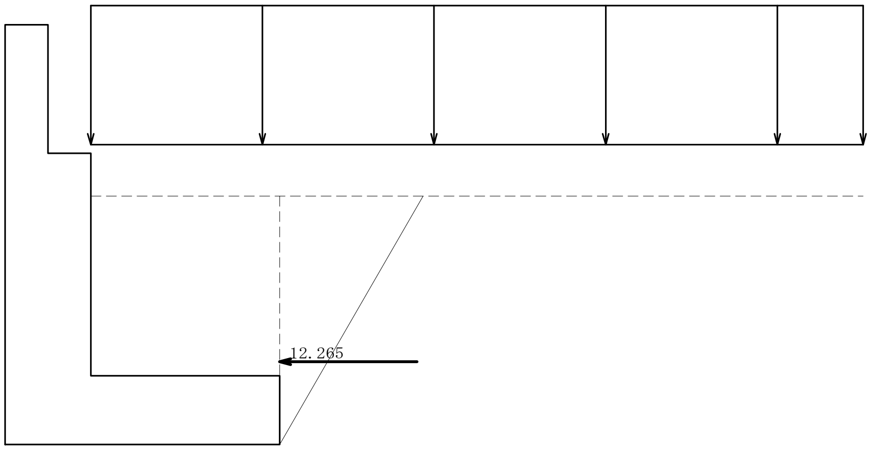
Location

$$Ho = \frac{H}{3} = \frac{1.450}{3} = 0.483 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.600 - 0.483 \times \tan 0.000° = 1.600$ m

$y = yp + Ho = 0.000 + 0.483 = 0.483$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	$xp = 1.600$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 1.450$ m
Angle to the Vertical of Earth Pressure Surface	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Angle of Ground Surface from the Horizontal	$\beta = 0.000$ °
Composite angle during earthquake	$\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	20.683	0.000	0.000	20.683	8.755
44.00	19.974	0.000	0.000	19.974	8.756
45.00	19.288	0.000	0.000	19.288	8.744

Earth pressure becomes maximum when:
ω = 45.00°P = 8.756 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{19.974/\cos 10.204° \times \sin(45.00° - 30.00° + 10.204°)}{\cos(45.00° - 30.00° - 0.000° - 24.239°)}$$

= 8.756 kN

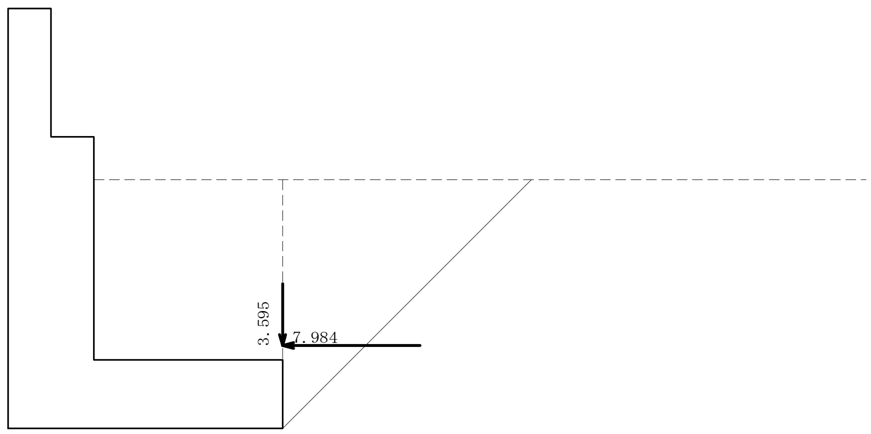
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
Ph = P・cos(α+δ) = 8.756×cos(0.000°+24.239°) = 7.984 kN
Vertical Component
Pv = P・sin(α+δ) = 8.756×sin(0.000°+24.239°) = 3.595 kN
Location

$$Ho = \frac{H}{3} = \frac{1.450}{3} = 0.483 \text{ m}$$

x = xp－Ho・tanα = 1.600－0.483×tan0.000°= 1.600 m
y = yp＋Ho = 0.000＋0.483 = 0.483 m

· Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	$xp = 1.600$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 1.450$ m
Angle of Earth Pressure Acting Surface to the Vertical	$\alpha = 0.000°$
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000°$
Angle of Ground Surface from the Horizontal	$\beta = 0.000°$
Wall Friction Angle	$\delta = \beta = 0.000°$
Rate of Change of Slip Angle	$\omega i = 10.00° \sim 80.00°$

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	12.001	0.000	0.000	12.001	6.652
60.00	11.532	0.000	0.000	11.532	6.658
61.00	11.071	0.000	0.000	11.071	6.652

Earth pressure becomes maximum when:
$\omega = 60.00°$ $P = 6.658$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{11.532 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

$= 6.658$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 6.658 \times \cos(0.000° + 0.000°) = 6.658$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 6.658 \times \sin(0.000° + 0.000°) = 0.000$ kN

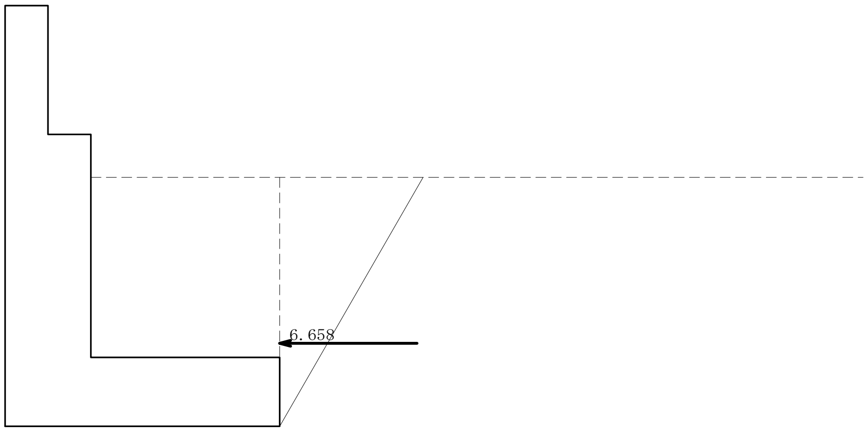
Location

$$Ho = \frac{H}{3} = \frac{1.450}{3} = 0.483 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.600 - 0.483 \times \tan 0.000° = 1.600$ m

$y = yp + Ho = 0.000 + 0.483 = 0.483$ m

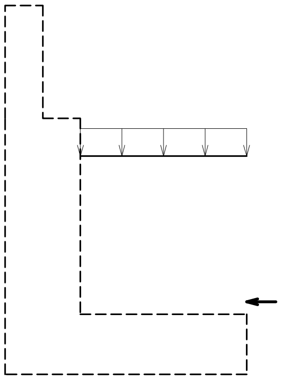
- Earth Pressure Diagram



2.6 Total Acting Force

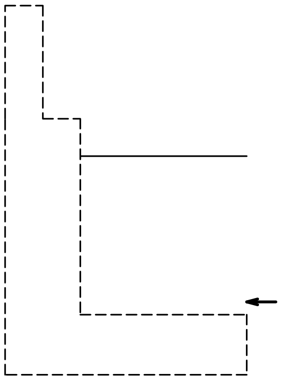
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



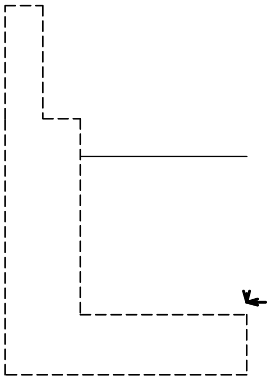
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	58.144	0.000	0.690	0.000	40.142	0.000
Loading, Snow	12.760	0.000	1.050	0.000	13.398	0.000
Earth Pressure	0.000	12.265	1.600	0.483	0.000	5.924
Total	70.904	12.265	———	———	53.540	5.924

[2] Ordinary Condition (Rear Side)



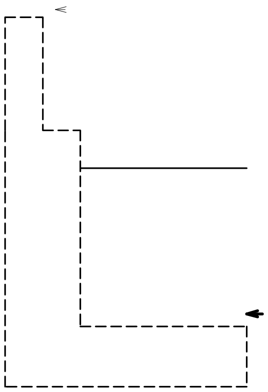
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	58.144	0.000	0.690	0.000	40.142	0.000
Earth Pressure	0.000	12.265	1.600	0.483	0.000	5.924
Total	58.144	12.265	―――	―――	40.142	5.924

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	58.144	10.466	0.690	0.855	40.142	8.944
Earth Pressure	3.595	7.984	1.600	0.483	5.752	3.856
Total	61.739	18.450	―――	―――	45.894	12.800

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	58.144	0.000	0.690	0.000	40.142	0.000
Loading, Snow	0.000	6.658	1.600	0.483	0.000	3.216
Collision Load	0.000	4.300	0.000	2.500	0.000	10.750
Total	58.144	10.958	———	———	40.142	13.966

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	70.904	12.265	47.616
Ordinary (Rear Side)	58.144	12.265	34.218
Earthquake	61.739	18.450	33.094
Collision	58.144	10.958	26.176

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,
Footing Earth Pressure Width : $B_j = 1.600$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	70.904	12.265	9.107
Ordinary (Rear Side)	58.144	12.265	12.297
Earthquake	61.739	18.450	16.298
Collision	58.144	10.958	20.339

■ Full Width per 10.000m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	709.038	122.650	91.073

Ordinary (Rear Side)	581.438	122.650	122.973
Earthquake	617.388	184.499	162.975
Collision	581.438	109.580	203.392

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B = 1.600

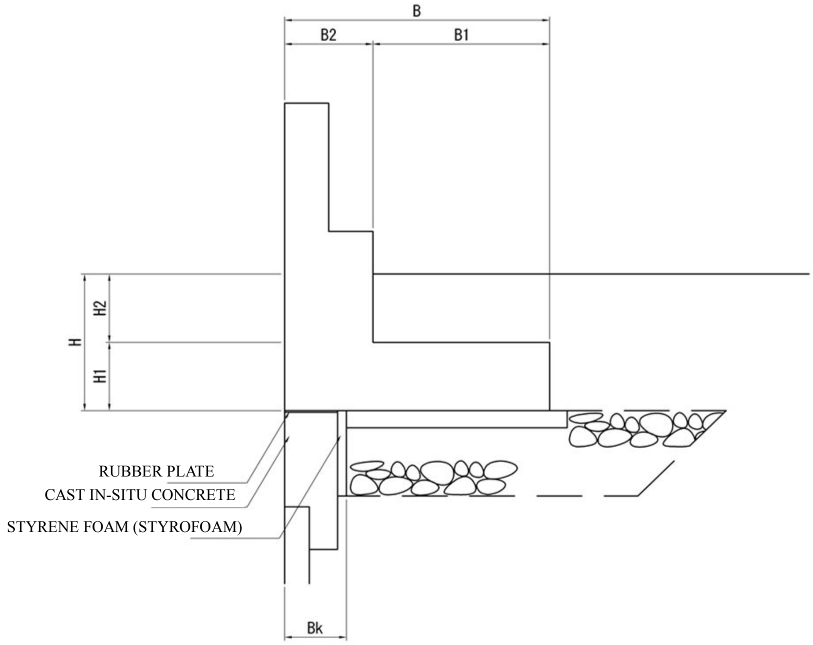
$$e_a = B / n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	53.540	5.924	70.904	0.672	0.128	≦	0.267	○
Ordinary (Rear Side)	40.142	5.924	58.144	0.589	0.211	≦	0.267	○
Earthquake	45.894	12.800	61.739	0.536	0.264	≦	0.533	○
Collision	40.142	13.966	58.144	0.450	0.350	≦	0.533	○

2.7.2 Stability from Sliding



H	(m)	1. 450
H 1	(m)	0. 400
H 2	(m)	1. 050
B	(m)	1. 600
B 1	(m)	1. 100
B 2	(m)	0. 500
B k	(m)	0. 350

Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m²)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.211	1.600	7.518	65.162	65.162	52.55
Earthquake	0.264	1.600	0.389	76.784	76.784	60.07
Collision	0.350	1.350	0.000	86.139	86.139	63.80

Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Nｎａ (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN’=ΣN-Nｎａ	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	58.144	20.600	37.544	12.265
Earthquake	61.739	23.950	37.789	18.450
Collision	58.144	26.241	31.903	10.958

Factor of safety for sliding, Fs:

- $Fs = \Sigma N' \cdot \mu / \Sigma H$

Load Condition	Factor of Safety F s	Required Factor of Safety F s a
Ordinary (Rear Side)	1.837	≧1.5
Earthquake	1.229	≧1.2
Collision	1.747	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

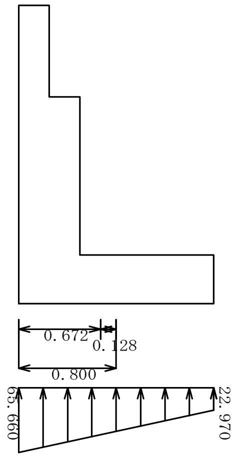
2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

Where,

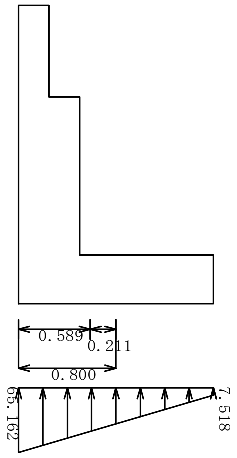
ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)
B : Bottom plate width (m), B = 1.600
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



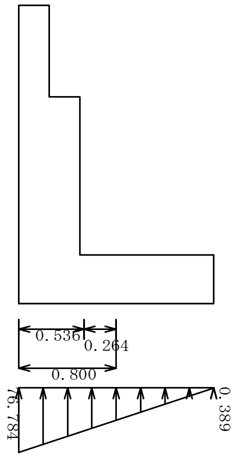
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)				Judgement
		qmin	qmax		Allow.	
1.600	Trapezoid	22.970	65.660	≦	200.000	○

[2] Ordinary Condition (Rear Side)



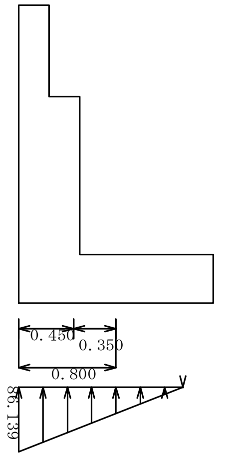
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.600	Trapezoid	7.518	65.162 ≦	200.000	○

[3] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.600	Trapezoid	0.389	76.784 ≦	300.000	○

[4]衝突時



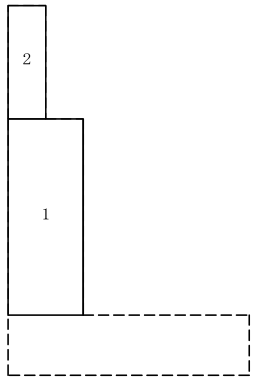
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.350	Triangle	0.000	86.139 ≦	300.000	○

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classific ation	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.500× 1.300× 1.000 0.250× 0.750× 1.000	0.650 0.188	0.250 0.125	0.650 1.675	0.162 0.023	0.422 0.314	
Σ		0.837	—	—	0.186	0.737	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.186／ 0.837 ＝ 0.222 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.737／ 0.837 ＝ 0.879 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.837 ＝ 20.519	0.028

Force Location

X ＝ Xc－XG ＝ 0.250－0.222
＝ 0.028 m

Where,
Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W ＝ γ ・ V (kN)	Location X (m)

Frame(Rebar)	24.500 × 0.837 = 20.519	0.028

Location	H = W ・ kh (kN)	Location Y (m)
Frame(Rebar)	20.519 × 0.180 = 3.693	0.879

Force Location

$X = Xc - XG = 0.250 - 0.222$

$= 0.028$ m

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

Horizontal Force

$$Pu = \frac{p}{\lambda + y}$$

Where,

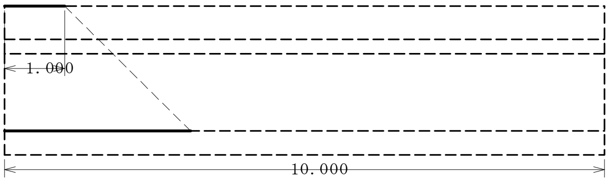
Pu : Collision load of the effective width (kN/m)

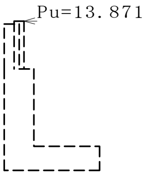
p : Collision load per block (kN)

λ : Load range (m)

y : Distance from the tip of section being checked (m), y = 1.300(m)

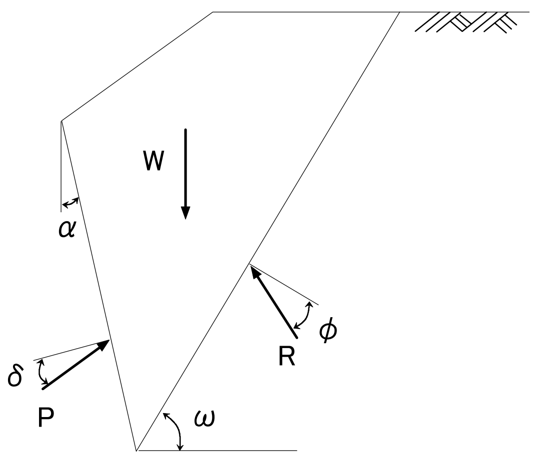
[1] Collision





p (kN)	λ (m)	h (m)	Pu (kN/m)	Location Y (m)
43.000	1.000	0.800	13.871	2.100

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.250 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.050 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	7.334	0.000	8.529	15.863	6.730
56.00	7.064	0.000	8.216	15.280	6.735
57.00	6.802	0.000	7.910	14.712	6.729

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 6.735 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{15.280 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

＝ 6.735 kN

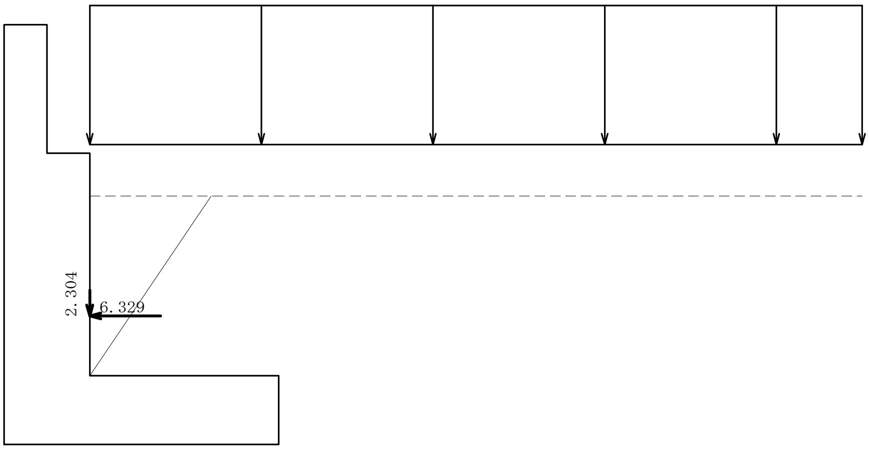
Earth Pressure's location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 6.735 \times \cos(0.000° + 20.000°) = 6.329$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 6.735 \times \sin(0.000° + 20.000°) = 2.304$ kN
Location

$$Ho = \frac{H}{3} = \frac{1.050}{3} = 0.350 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.350 \times \tan 0.000° - 0.250 = -0.250$ m
$y = yp + Ho = 0.000 + 0.350 = 0.350$ m

・ Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) xp ＝ 0.250 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 1.050 m
Angle of Earth Pressure Acting Surface to the Vertical α ＝ 0.000 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	7.334	0.000	0.000	7.334	3.111
56.00	7.064	0.000	0.000	7.064	3.114
57.00	6.802	0.000	0.000	6.802	3.111

Earth pressure becomes maximum when:
ω = 56.00° P = 3.114 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.064 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

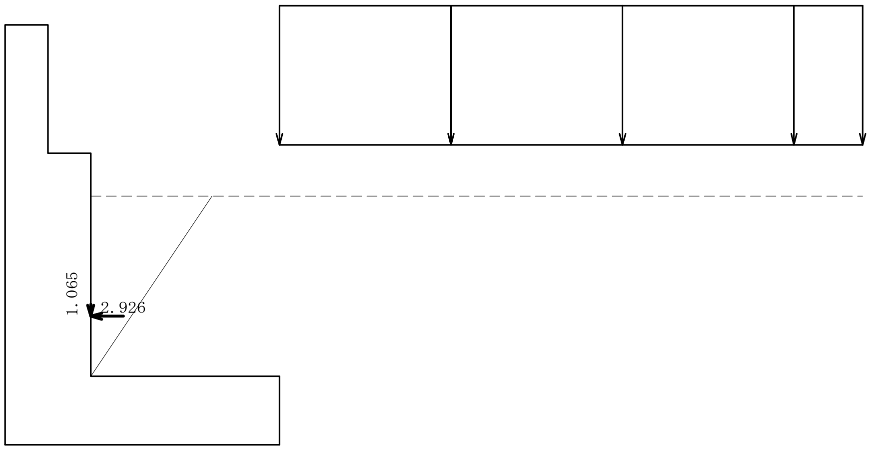
= 3.114 kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
Ph = P・cos(α+δ) = 3.114×cos(0.000°+20.000°) = 2.926 kN
Vertical Component
Pv = P・sin(α+δ) = 3.114×sin(0.000°+20.000°) = 1.065 kN
Location

$$Ho = \frac{H}{3} = \frac{1.050}{3} = 0.350 \text{ m}$$

x = Ho・tanα−xp = 0.350×tan0.000°−0.250 = -0.250 m
y = yp+Ho = 0.000+0.350 = 0.350 m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL)	xp = 0.250 m
	yp = 0.000 m
Height of Assumed Rear Surface	H = 1.050 m
Angle to the Vertical of Earth Pressure Surface	α = 0.000 °
Unit Weight of Soil at Rear Side	γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ = 30.000 °
Wall Friction Angle	δ = 1／2φ = 15.000 °
Composite angle during earthquake	θ = $\tan^{-1}$kh = $\tan^{-1}$0.18 = 10.204°
Rate of Change of Slip Angle	ωi = 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)

Water Level, $hw = 0.000$ m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
46.00	10.115	0.000	0.000	10.115	4.539
47.00	9.767	0.000	0.000	9.767	4.540
48.00	9.430	0.000	0.000	9.430	4.535

Earth pressure becomes maximum when:

$\omega = 47.00°$ $P = 4.540$ kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{9.767/\cos 10.204° \times \sin(47.00° - 30.00° + 10.204°)}{\cos(47.00° - 30.00° - 0.000° - 15.000°)}$$

$= 4.540$ kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 4.540 \times \cos(0.000° + 15.000°) = 4.385$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 4.540 \times \sin(0.000° + 15.000°) = 1.175$ kN

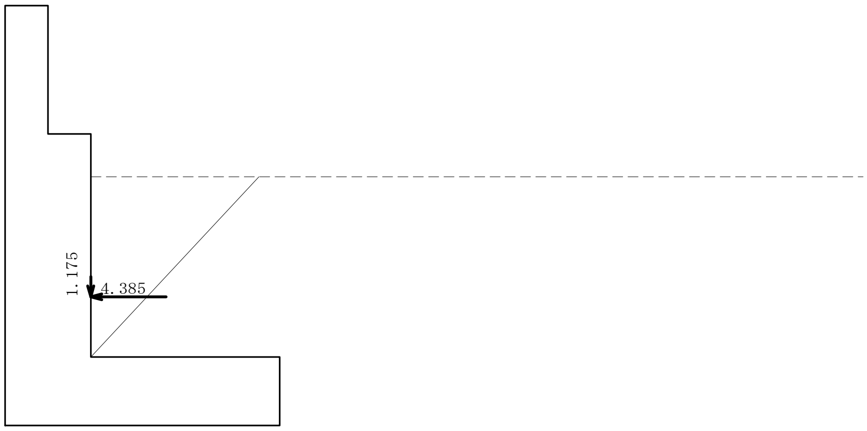
Location

$$Ho = \frac{H}{3} = \frac{1.050}{3} = 0.350 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.350 \times \tan 0.000° - 0.250 = -0.250$ m

$y = yp + Ho = 0.000 + 0.350 = 0.350$ m

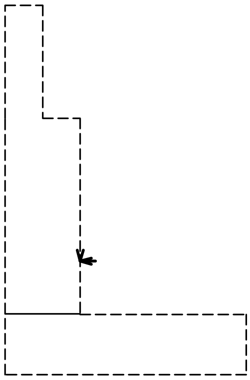
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

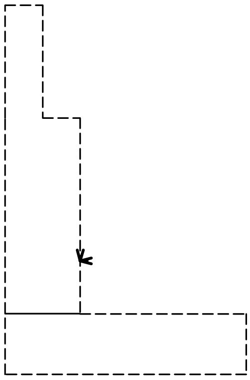
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	20.519	0.000	0.028	0.000	0.000
Earth Pressure	2.304	6.329	-0.250	0.350	2.215
Total	0.000	6.329	———	———	2.215

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

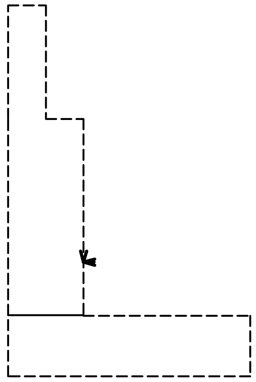
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	20.519	0.000	0.028	0.000	0.000
Earth Pressure	1.065	2.926	-0.250	0.350	1.024
Total	0.000	2.926	———	———	1.024

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

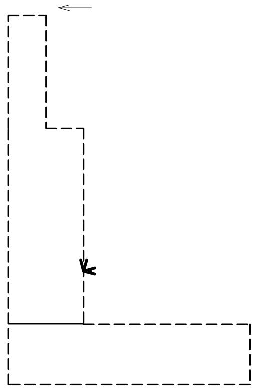
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M $=M_{xi}+M_{yi}$ (kN.m)
Self-weight	20.519	3.693	0.028	0.879	3.248
Earth Pressure	1.175	4.385	-0.250	0.350	1.535
Total	0.000	8.078	———	———	4.783

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

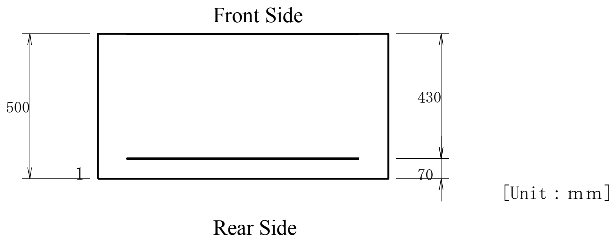


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M $=M_{xi}+M_{yi}$ (kN.m)
Self-weight	20.519	0.000	0.028	0.000	0.000
Earth Pressure	1.065	2.926	-0.250	0.350	1.024
Collision Load	0.000	13.871	0.000	2.100	29.129
Total	0.000	16.797	———	———	30.153

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	7.0	D13	1.267	4.000	5.068
	2	—	–	—	—	—

Required rebar amount at tension side: 2.437 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 41666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 500000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 500.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	3.766	≦	79.737	0.000	5.000	○
Ordinary (Rear Side)	5.068	1.741	≦	79.737	0.000	5.000	○
Earthquake	5.068	8.131	≦	79.737	0.000	5.000	○
Collision	5.068	3.766	≦	79.737	0.000	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2 $(0.002 \leqq \varepsilon(y) \leqq \varepsilon cu)$

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars $(d_i \leqq x)$

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars $(d_j > x)$

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 430.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	137.877	○
Ordinary (Rear Side)	5.068	≦	137.877	○
Earthquake	5.068	≦	137.877	○
Collision	5.068	≦	137.877	○

(4) Check of Bending Stress

(Reference)

Calculation of neutral axis

Find x from:

$$x^2 + \frac{2 \cdot n}{b}\left\{As \cdot (x-d)\right\} = 0.0$$

Stress Calculation

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2} \cdot \left(\frac{h}{2} - \frac{x}{3}\right) + n \cdot As \cdot \frac{(x-d) \cdot (h/2-d)}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h = 500.000
b : Member cross-section width (mm), b = 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n = 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
				Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	2.215	0.000	7.361	0.148	≦	8.000	10.780	≦	180.000	○
Ordinary (Rear Side)	1.024	0.000	7.361	0.069	≦	8.000	4.984	≦	180.000	○
Earthquake	4.783	0.000	7.361	0.320	≦	12.000	23.276	≦	300.000	○
Collision	30.153	0.000	7.361	2.020	≦	12.000	146.740	≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member’s effective height (mm)

b : Sectional member’s effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member’s effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

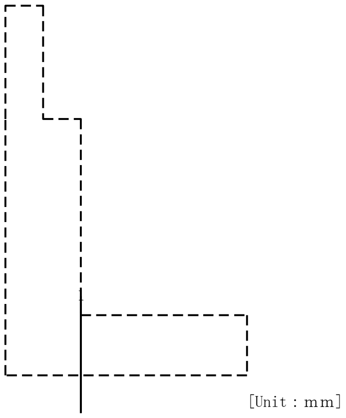
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	6.329	43.000	0.015	≦	0.224	1.700	1.33	0.74	1.00	○
Ordinary(Rear Side)	2.926	43.000	0.007	≦	0.224	1.700	1.33	0.74	1.00	○
Earthquake	8.078	43.000	0.019	≦	0.341	2.550	1.33	0.74	1.00	○
Collision	16.797	43.000	0.039	≦	0.336	2.550	1.33	0.74	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

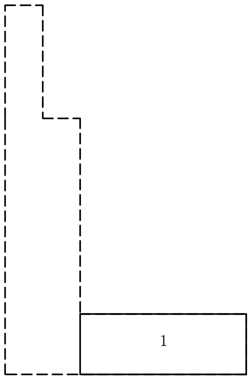
Distance from joint: 0.000 m



4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



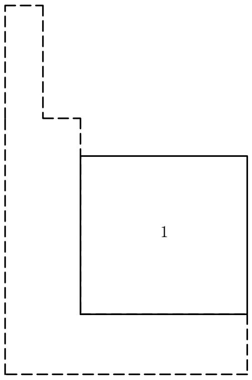
1) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	1.100× 0.400× 1.000	0.440	0.550	0.242	
Σ		0.440	——	0.242	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.242／ 0.440 ＝ 0.550 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	1.100× 1.050× 1.000	1.155	0.550	0.635	
Σ		1.155	——	0.635	

Center of Gravity XG = Σ（Vi・Xi）／ΣVi = 0.635／ 1.155 = 0.550 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force W = γ ・ V (kN)	Location X (m)
Frame	24.500 × 0.440 = 10.780	0.550
Soil(Rear Side)	19.000 × 1.155 = 21.945	0.550

4.1.3 Ground Surface Load・Snow Load

Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

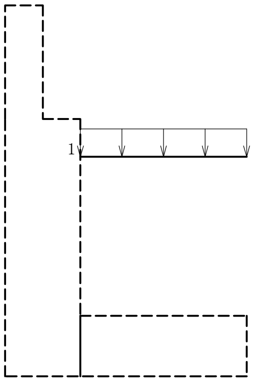
Where,

q : Ground surface loading

L : Length of ground surface load

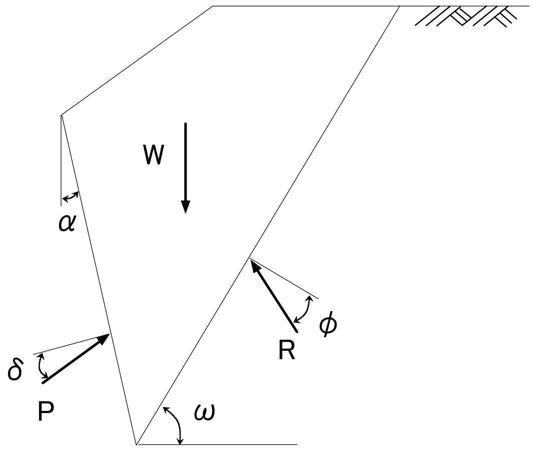
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m²)	q2 (kN/m²)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.100	12.760	0.550

4.1.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) xp = 1.600 m

yp = 0.000 m

Height of Assumed Rear Surface H = 1.450 m

Angle to the vertical of the Earth Pressure Surface α = 0.000 °

Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3

Angle of Internal Friction of Soil at Rear Side φ = 30.000 °

Angle to the Horizontal of the Ground Surface β = 0.000 °

Composite Angle during Earthquake θ = $\tan^{-1}$kh= $\tan^{-1}$0.18 =10.204°

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00^\circ \times \sin(10.204^\circ + 20.751^\circ - 0.000^\circ)}{1 - \sin 30.00^\circ \times \cos(10.204^\circ + 20.751^\circ - 0.000^\circ)}$$

$$= 24.239^\circ$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000^\circ + 10.204^\circ)}{\sin 30.00^\circ} = 20.751^\circ$$

Rate of Change of Slip Angle $\omega i = 10.00^\circ \sim 80.00^\circ$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
44.00	20.683	0.000	0.000	20.683	8.755
45.00	19.974	0.000	0.000	19.974	8.756
46.00	19.288	0.000	0.000	19.288	8.744

Earth pressure becomes maximum when:
$\omega = 45.00^\circ$ $P = 8.756$ kN

Earth Pressure

$$P = \frac{W / \cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{19.974 / \cos 10.204^\circ \times \sin(45.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(45.00^\circ - 30.00^\circ - 0.000^\circ - 24.239^\circ)}$$

$$= 8.756 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component

$$P_v = P \cdot \sin(\alpha + \delta) = 8.756 \times \sin(0.000^\circ + 24.239^\circ) = 3.595 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 3.595}{1.100} = 6.536 \text{ kN/m}$$

Where,

pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.000 m

Width of distributed load perpendicular to the design section L2 = 1.100 m

Distributed Load at the Design Section, pd

$$pd = \frac{pv}{L} \cdot L1 = \frac{6.536}{1.100} \times 0.000 = 0.000 \text{ kN/m}$$

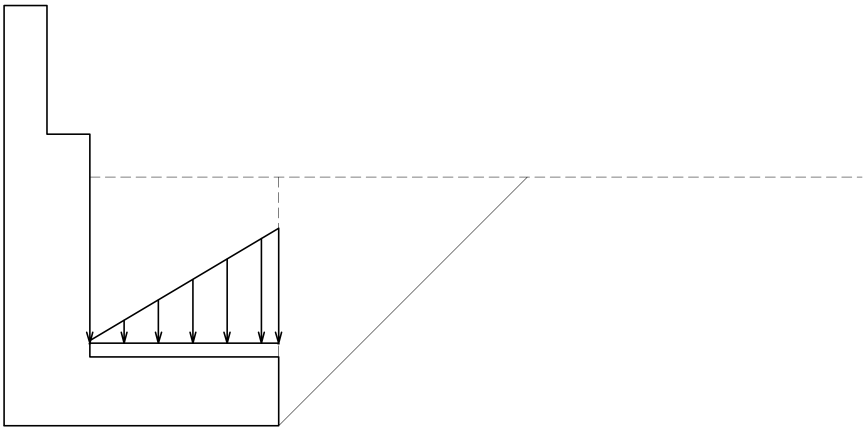
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (0.000 + 6.536) \times 1.100 = 3.595 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{0.000 + 2 \times 6.536}{0.000 + 6.536} \times \frac{1.100}{3} = 0.733 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

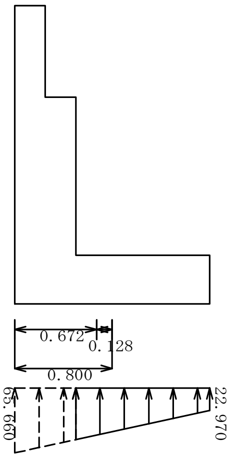
Where,

q1 : Ground reaction pressure at the front surface of heel (kN/m^2)

q2 : Ground reaction pressure at the design section of heel (kN/m^2)

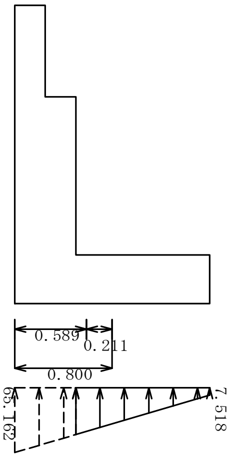
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



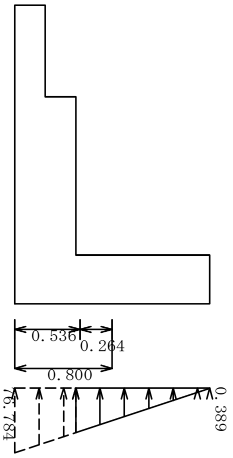
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.970	52.319	1.100	41.409	0.479

[2] Ordinary Condition (Rear Side)



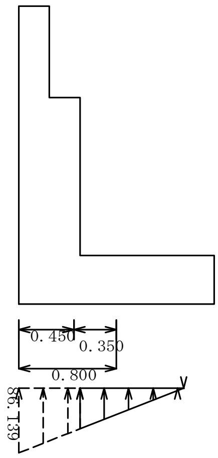
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.518	47.148	1.100	30.066	0.417

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.389	52.911	1.100	29.315	0.369

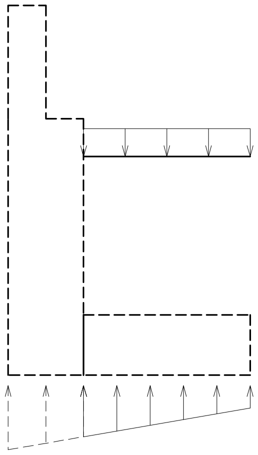
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	54.236	0.850	23.050	0.283

4.1.6 Total Sectional Forces

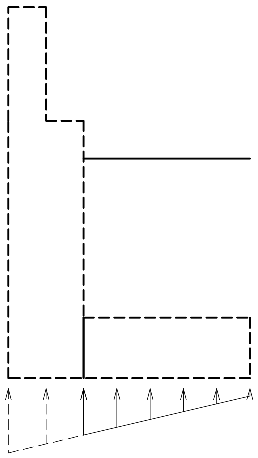
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	32.725	0.550	17.999
Loading, Snow Load	12.760	0.550	7.018
Ground Reaction	-41.409	0.479	-19.816
Total	4.076	——	5.201

Sectional force of vertical wall foundation, M1 ＝ 2.215 kN.m
Sectional force of the base of heel, M3 ＝ 5.201 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

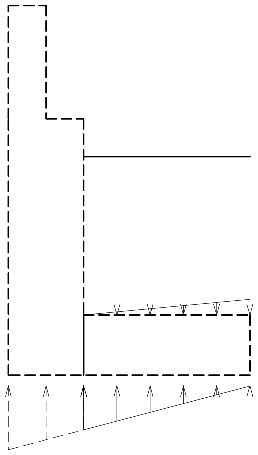
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	32.725	0.550	17.999
Ground Reaction	-30.066	0.417	-12.540
Total	2.659	———	5.458

Sectional force of vertical wall foundation, M1 ＝ 1.024 kN.m
Sectional force of the base of heel, M3 ＝ 5.458 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

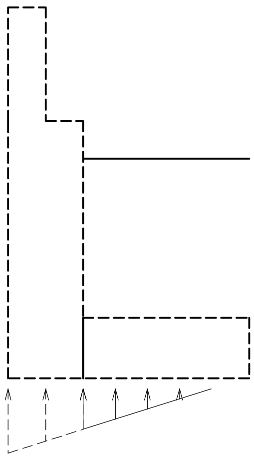
[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	32.725	0.550	17.999
Loading, Snow Load	3.595	0.733	2.636
Ground Reaction	-29.315	0.369	-10.827
Total	7.005	———	9.808

Sectional force of vertical wall foundation, M1 ＝ 4.783 kN.m
Sectional force of the base of heel, M3 ＝ 9.808 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

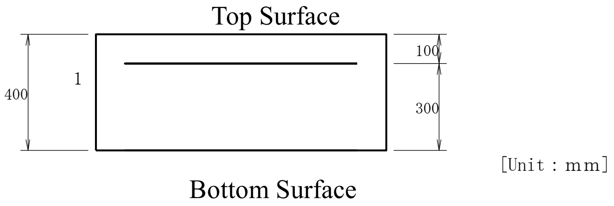
[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	32.725	0.550	17.999
Ground Reaction	-23.050	0.283	-6.531
Total	9.675	———	11.468

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.0	D13	1.267	4.000	5.068
	2	——	–	———	——	———
Bottom Surface	1'	——	–	———	——	———
	2'	——	–	———	——	———

Required tension side rebar amount 1.323 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 400.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m) ×1.7 M_c (kN.m)	Min. Rebar Amount (cm^2)	Judge ment
Ordinary (Surcharge)	5.068	3.766 ≦ 51.031	5.000	○
Ordinary (Rear Side)	5.068	1.741 ≦ 51.031	5.000	○
Earthquake	5.068	8.131 ≦ 51.031	5.000	○
Collision	5.068	19.495 ≦ 51.031	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$$\varepsilon_{cu} = 0.0035$$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars（$d_i \leqq x$)）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars（$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)

σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$

E_s : Rebar Young's Modulus (N/mm^2), Es = 200000

d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0

b : Width of member (mm), b = 1000.0

σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$

d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)

A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)

σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)

d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)

A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)

σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^{2}+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

Stress Calculation

$$\sigma_{c}=\frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_{s}=n\cdot\sigma_{c}\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h ＝ 400.000

b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^{2})

n : Rebar and concrete’s coefficient of Young’s modulus, n ＝ 15.00

e : Distance from the sectional member’s centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^{2})

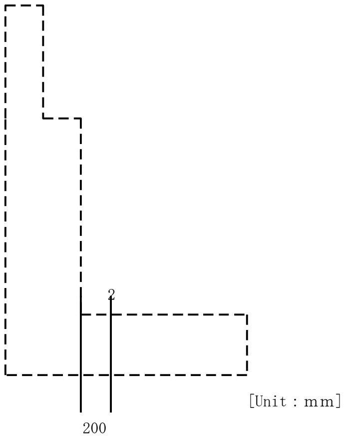
σs : Rebar tensile stress (N/mm^{2})

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^{2})			Tensile Stress (N/mm^{2})			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	2.215	6.035	0.262	≦	8.000	15.618	≦	180.000	○
Ordinary (Rear Side)	1.024	6.035	0.121	≦	8.000	7.220	≦	180.000	○
Earthquake	4.783	6.035	0.566	≦	12.000	33.722	≦	300.000	○
Collision	11.468	6.035	1.357	≦	12.000	80.852	≦	300.000	○

4.2 Design of Section [2]

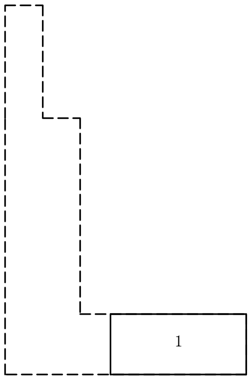
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



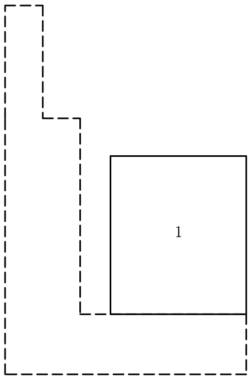
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	0.900× 0.400× 1.000	0.360	0.450	0.162	
Σ		0.360	——	0.162	

Center of Gravity XG＝ Σ（Vi・Xi）／ΣVi ＝ 0.162／ 0.360 ＝ 0.450 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. V_i (m^3)	Center of Gravity X_i (m)	Vi・Xi	Remarks
1	0.900× 1.050× 1.000	0.945	0.450	0.425	
Σ		0.945	——	0.425	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.425／ 0.945 ＝ 0.450 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.360 ＝ 8.820	0.450
Soil(Rear Side)	19.000 × 0.945 ＝ 17.955	0.450

4.2.3 Ground Surface Load・Snow Load

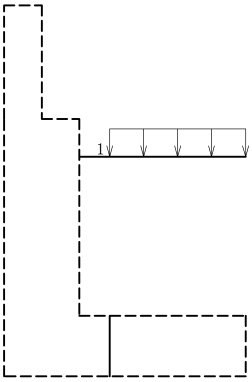
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

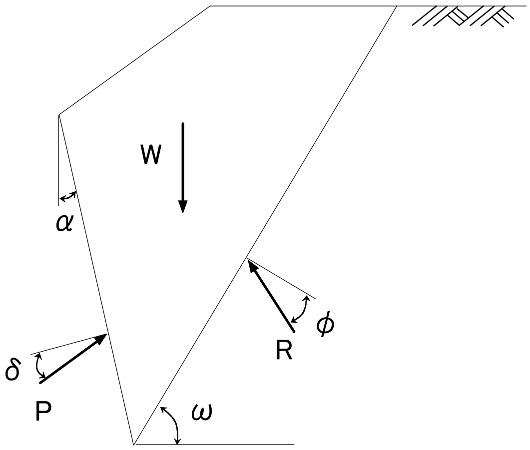
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.900	10.440	0.450

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe) xp = 1.600 m
yp = 0.000 m
Height of Assumed Rear Surface H = 1.450 m
Angle to the vertical of the Earth Pressure Surface α = 0.000 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °
Angle to the Horizontal of the Ground Surface β = 0.000 °
Composite Angle during Earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00^\circ \times \sin(10.204^\circ + 20.751^\circ - 0.000^\circ)}{1 - \sin 30.00^\circ \times \cos(10.204^\circ + 20.751^\circ - 0.000^\circ)}$$

$$= 24.239^\circ$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000^\circ + 10.204^\circ)}{\sin 30.00^\circ} = 20.751^\circ$$

Rate of Change of Slip Angle $\omega i = 10.00^\circ \sim 80.00^\circ$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, $hw = 0.000$ m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
44.00	20.683	0.000	0.000	20.683	8.755
45.00	19.974	0.000	0.000	19.974	8.756
46.00	19.288	0.000	0.000	19.288	8.744

Earth pressure becomes maximum when:
$\omega = 45.00^\circ \quad P = 8.756$ kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{19.974/\cos 10.204^\circ \times \sin(45.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(45.00^\circ - 30.00^\circ - 0.000^\circ - 24.239^\circ)}$$

$$= 8.756 \text{ kN}$$

Vertical component of Earth Pressure:
Vertical Component

$$Pv = P \cdot \sin(\alpha + \delta) = 8.756 \times \sin(0.000^\circ + 24.239^\circ) = 3.595 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2 \cdot Pv}{L} = \frac{2 \times 3.595}{1.100} = 6.536 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.200 m

Width of distributed load perpendicular to the design section L2 = 0.900 m

Distributed Load at the Design Section, pd

$$pd = \frac{pv}{L} \cdot L1 = \frac{6.536}{1.100} \times 0.200 = 1.188 \text{ kN/m}$$

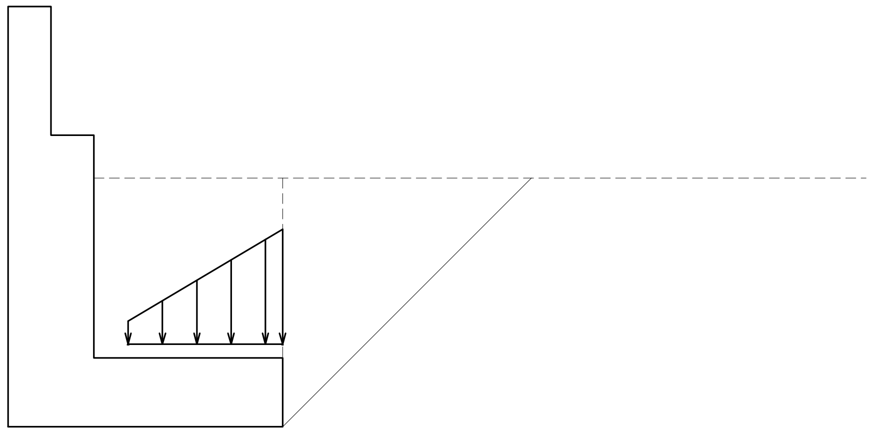
Vertical Force

$$N = \frac{1}{2} \cdot (pd+pv) \cdot L2 = \frac{1}{2} \times (1.188+6.536) \times 0.900 = 3.476 \text{ kN}$$

Location

$$x = \frac{pd+2 \cdot pv}{pd+pv} \cdot \frac{L2}{3} = \frac{1.188+2\times 6.536}{1.188+6.536} \times \frac{0.900}{3} = 0.554 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2) \cdot L$$

Location

$$X = \frac{2 \cdot q1+q2}{3 \cdot (q1+q2)} \cdot L$$

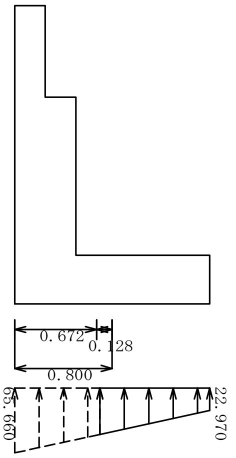
Where,

q1 : Ground reaction pressure at the front surface of heel (kN/m^2)

q2 : Ground reaction pressure at the design section of heel (kN/m^2)

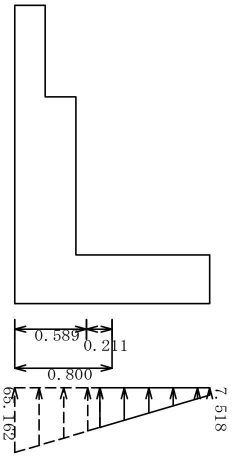
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



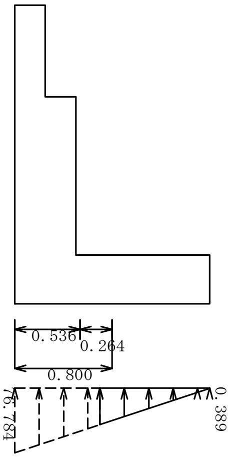
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.970	46.983	0.900	31.479	0.399

[2] Ordinary Condition (Rear Side)



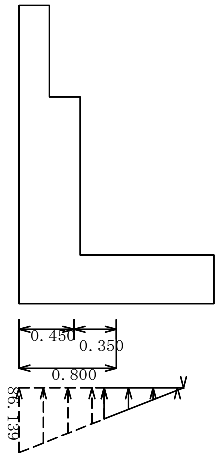
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.518	39.943	0.900	21.357	0.348

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.389	43.361	0.900	19.688	0.303

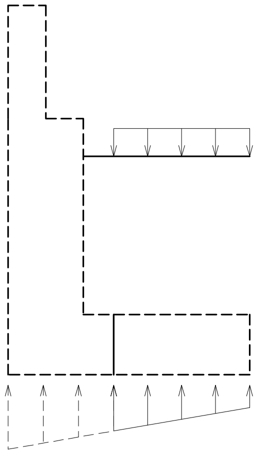
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	41.474	0.650	13.479	0.217

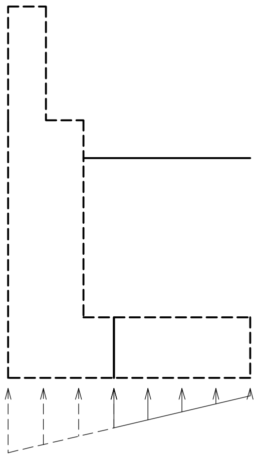
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



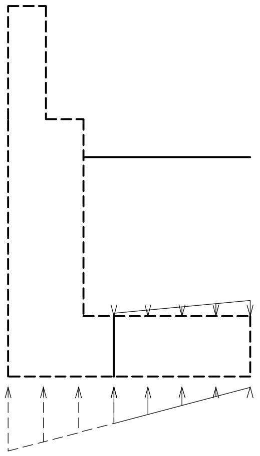
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	26.775	0.450	12.049
Loading, Snow Load	10.440	0.450	4.698
Ground Reaction	-31.479	0.399	-12.545
Total	5.736	———	4.202

[2] Ordinary Condition (Rear Side)



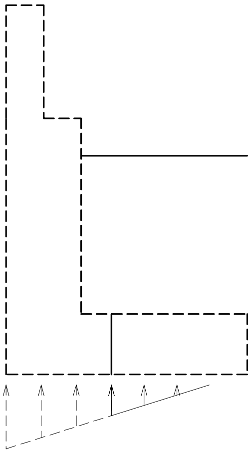
Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	26.775	0.450	12.049
Ground Reaction	-21.357	0.348	-7.422
Total	5.418	———	4.627

[3] Earthquake



Item	N_i (kN)	X_i (m)	M $=N_i \cdot X_i$ (kN.m)
Self-weight	26.775	0.450	12.049
Loading, Snow Load	3.476	0.554	1.925
Ground Reaction	-19.688	0.303	-5.959
Total	10.564	———	8.015

[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	26.775	0.450	12.049
Ground Reaction	-13.479	0.217	-2.920
Total	13.296	———	9.128

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ $a > 2.5d$,

when $a \leqq 2.5d$, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)
d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
M : Bending moment acting on the sectional member (N.mm)
θ : Angle of the top surface footing to the horizontal, $\theta = 0.000$, $\tan\theta = 0.000$
a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span 2.5・d (cm)		a	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000	≦	135.000	5.736	4.202	0.000	5.736
Ordinary (Rear Side)	30.000	75.000	≦	135.000	5.418	4.627	0.000	5.418
Earthquake	30.000	75.000	≦	135.000	10.564	8.015	0.000	10.564
Collision	30.000	75.000	≦	135.000	13.296	9.128	0.000	13.296

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm : Average shear stress of concrete (N/mm2)
Sh : Actual Shear Force (N)

d : Effective height of footing calculated from the column, front wall or rear wall (mm)
d' : Effective height of sectional member (mm)
b : Effective width of sectional member (mm)
S : Shear force acting on the sectional member (N)
τa1 : Allowable shear stress (N/mm2)

$$\tau a1 = Ce \cdot Cpt \cdot Cdc \cdot \tau a1'$$

Where,

τa1' : Allowable shear stress when shear force acts in concrete (N/mm2)
Ce : Correction factor for the sectional member’s effective height
Cpt : Correction factor for the tensile main reinforcement ratio, Pt
Cdc : Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	5.736	30.000	0.019	≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	5.418	30.000	0.018	≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	10.564	30.000	0.035	≦	0.411	2.550	1.40	0.84	1.00	○
Collision	13.296	30.000	0.044	≦	0.405	2.550	1.40	0.84	1.00	○

Mechanically-Stabilized Earth Wall Type-3

(H=2650)

1 Design Conditons

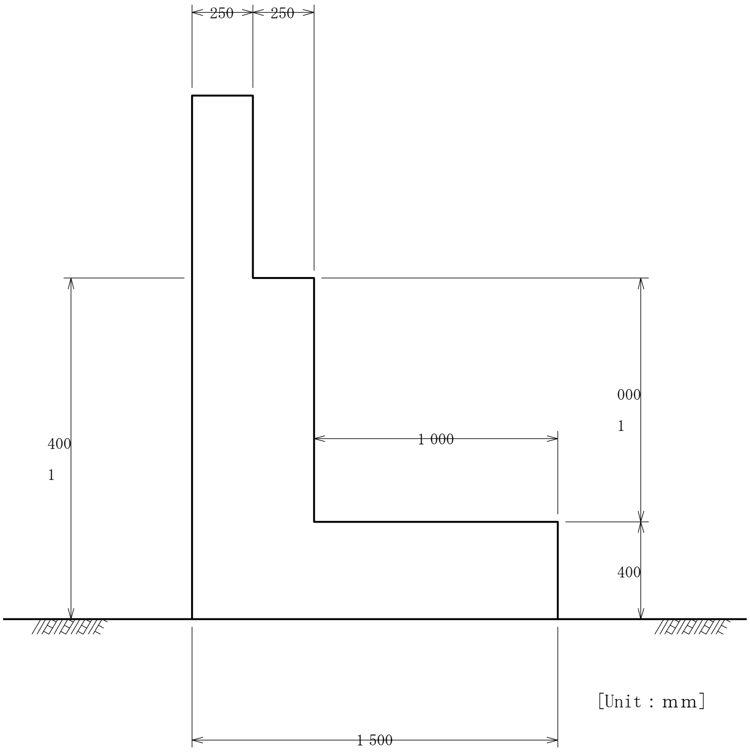
1.1 Applicable Standards

Japan Road Association Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

L-Shaped －A（Direct-foundation）

1.3 Configuration



Block Length, B ＝ 8300(mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)

RC Bottom Slab: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil

Embankment: Sandy soil

Bearing/Supporting ground: Sand

【Angle of Internal Friction】Soil at Rear Side: 30°

【Unit Weight】

(kN/m^3)

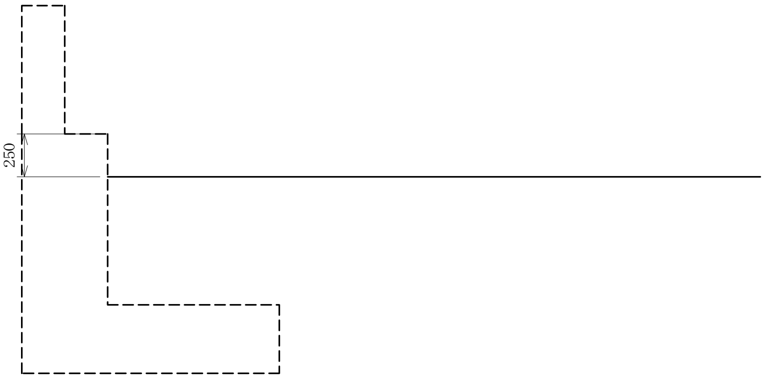
Framework	RC Concrete	24.500	
Water	For Frame Buoyancy	10.000	
	For Soil Buoyancy	9.000	
Soil		Damp Weight	Saturated Weight
Rear		19.000	20.000
Front		19.000	20.000

【Design Horizontal Seismic Coefficient】

Frame: Kh = 0.18
Soil (Front side): Kh = 0.18
Soil (Rear side): Kh = 0.18

1.6 Soil

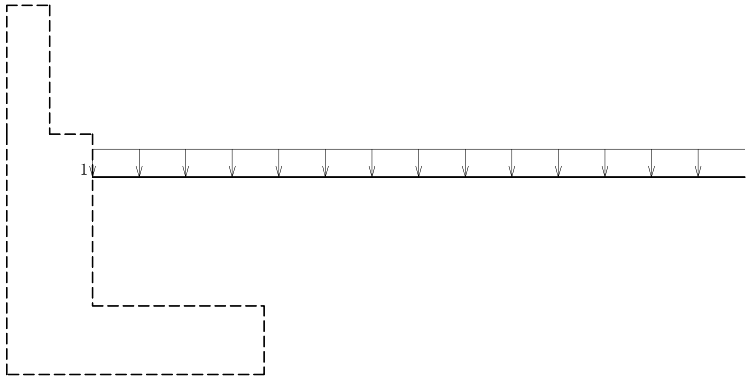
(1) Figure of Soil at Rear Side



Difference in elevation between the tip of retaining wall and ground surface (m)	0.250
Height not considering earth pressure, Hr (m)	0.000

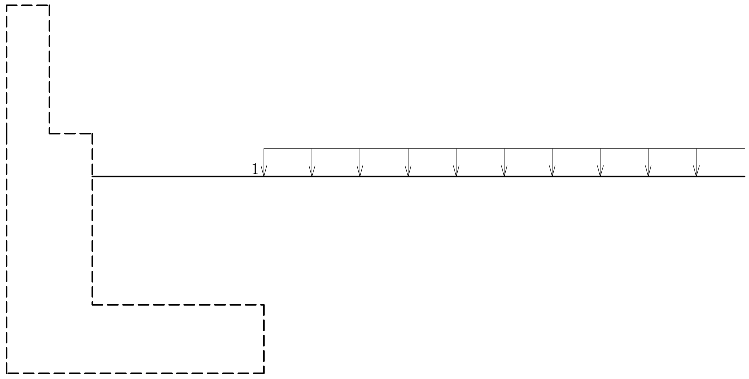
1.7 Loading

[1] At Ordinary Condition (Surcharge)



Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	○

[2] At Ordinary Condition (Rear side)

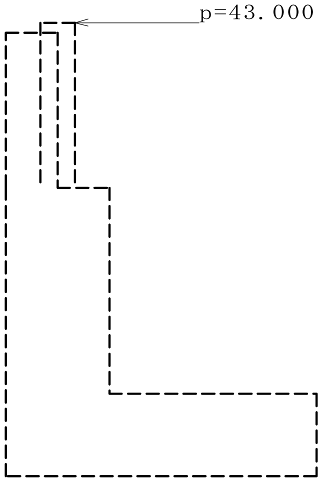


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical wall	Bottom Slab
1	1.000	∞	11.600	11.600	○	○	○

1.8 Collision Load

Fence Type: Flexible

[1] At Collision



Horizontal Force

Loading Location h (m)	Load Range λ (m)	Collision Load P (kN)
0.800	1.000	43.000

1.9 Arbitrary Load

Neglected

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method

- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Calculation	Cut	
Ordinary	0.000	20.000	———	———
Earthquake	24.239	15.000	———	———

- In the stability analysis, the assumed rear surface of earth pressure is the edge of the heel (line extending vertically from the heel).

- In the stability analysis, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- In the vertical wall design, the angle formed by the earth pressure's acting surface with the vertical is 0.000 degrees.

- Adhesive Force (kN/m^2)

Load Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	———
Earthquake	0.000	0.000	———

- Earth Pressure with Direction Other Than the Direction of Earthquake

	Treatment	Effectiveness Ratio
Stability Analysis	Ordinary Earth Pressure	0.500
Vertical Wall Design		0.500

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary (Surcharge)	Ordinary
2	Ordinary (Rear Side)	Ordinary
3	Earthquake	kh=0.18
4	Collision	Type A

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
3	Earthquake	Level 1	←Towards left	———	Neglected	Considered

	Load Name	1	2	3	4
Soil	Soil 1				
Loading	Surcharge	○			
	Assumed rear surface		○		
Collision・Wind	Collision Load				○
Active Earth Pressure	Neglect				
	Ordinary Earth Pressure	○	○		○
	Seismic Earth Pressure			○	

Check Items		1	2	3	4
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	——	——	——	——
	Rigid Body Stability	——	——	——	——
	Cross-Section Disruption	——	——	——	——

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, $\tan\varphi_B$	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary (Surcharge)	1/6	1.500	200.000
Ordinary (Rear Side)	1/6	1.500	200.000
Earthquake	1/3	1.200	300.000
Collision	1/3	1.200	300.000

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B ／ V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress	
				τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

2) Bottom Slab (General Members)

$(\mathrm{N/mm^2})$

Load Condition	Multiplication Factor	Compressive Stress Of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress	
				τ_{a1}	τ_{a2}
Ordinary (Surcharge)	1.00	8.000	180.000	0.230	1.700
Ordinary (Rear Side)	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

τ_{a1} : Shear stress when shear force is bored by concrete

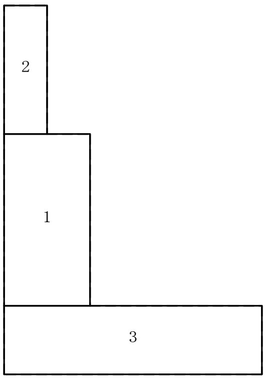
τ_{a2} : Shear stress when shared with diagonal tension bars

2 Stability Analysis

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



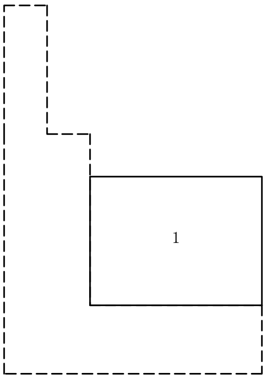
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2 3	0.500× 1.000× 1.000 0.250× 0.750× 1.000 1.500× 0.400× 1.000	0.500 0.188 0.600	0.250 0.125 0.750	0.900 1.775 0.200	0.125 0.023 0.450	0.450 0.333 0.120	
Σ		1.288	——	——	0.598	0.903	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.598／ 1.288 ＝ 0.465 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.903／ 1.288 ＝ 0.701 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1	1.000× 0.750× 1.000	0.750	1.000	0.775	0.750	0.581	
Σ		0.750	——	——	0.750	0.581	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.750／ 0.750 ＝ 1.000 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.581／ 0.750 ＝ 0.775 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.288 = 31.544	0.465
Soil (Rear Side)	19.000 × 0.750 = 14.250	1.000
Total	45.794	0.631

[2] Earthquake

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame (Rebar)	24.500 × 1.288 = 31.544	0.465
Soil (Rear Side)	19.000 × 0.750 = 14.250	1.000
Total	45.794	0.631

Location	Horizontal Force $H = W \cdot kh$ (kN)	Location Y (m)
Frame (Rebar)	31.544 × 0.18 = 5.678	0.701
Soil (Rear Side)	14.250 × 0.18 = 2.565	0.775
Total	8.243	0.724

2.3 Ground Surface Loading, Snow Load

Vertical Force

$$N = \frac{1}{2} \cdot (q1+q2) \cdot L$$

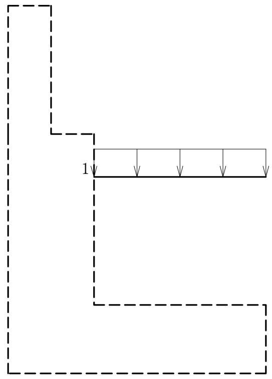
Where,

q : Load

L : Length of load

X : Distance fromt toe to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.000	11.600	1.000

2.4 Collision Load

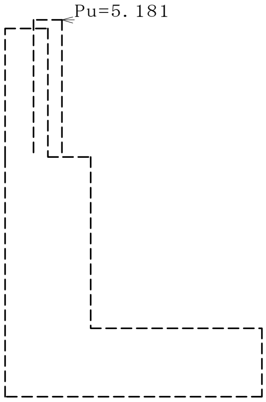
Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 8.300(m)

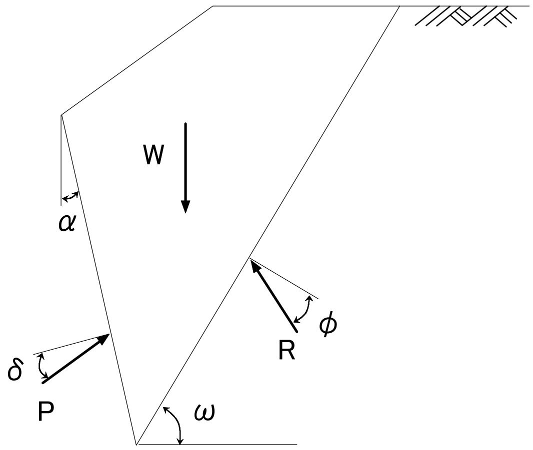
[1] Collision



Horizontal Force

p (kN)	Pu (kN/m)	Location Y (m)
43.000	5.181	2.200

2.5 Earth Pressure・Water Pressure



[1] Ordinary Conditions (Surcharge and Rear Side)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal	β ＝ 0.000 °
Wall Friction Angle	δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	8.015	15.564	8.627
60.00	7.254	0.000	7.702	14.956	8.635
61.00	6.965	0.000	7.394	14.359	8.628

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 8.635 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{14.956 \times \sin(60.00^\circ - 30.00^\circ)}{\cos(60.00^\circ - 30.00^\circ - 0.000^\circ - 0.000^\circ)}$$

$= 8.635$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 8.635 \times \cos(0.000^\circ + 0.000^\circ) = 8.635$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 8.635 \times \sin(0.000^\circ + 0.000^\circ) = 0.000$ kN

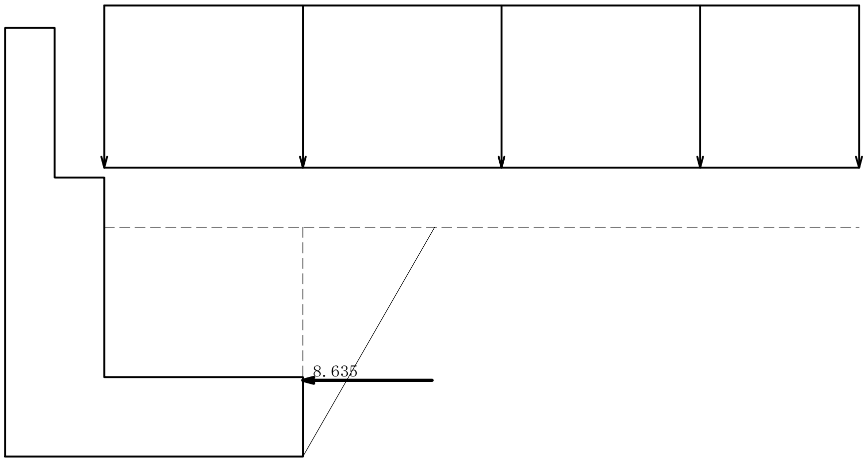
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000^\circ = 1.500$ m

$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	$xp = 1.500$ m
	$yp = 0.000$ m
Height of Assumed Rear Surface	$H = 1.150$ m
Angle to the Vertical of Earth Pressure Surface	$\alpha = 0.000$ °
Unit Weight of Soil at Rear Side	$\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side	$\varphi = 30.000$ °
Angle of Ground Surface from the Horizontal	$\beta = 0.000$ °
Composite angle during earthquake	$\theta = \tan^{-1} kh = \tan^{-1} 0.18 = 10.204^\circ$

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi\cdot\sin(\theta+\Delta-\beta)}{1-\sin\phi\cdot\cos(\theta+\Delta-\beta)}$$

$$= \tan^{-1}\frac{\sin 30.00^\circ\times\sin(10.204^\circ+20.751^\circ-0.000^\circ)}{1-\sin 30.00^\circ\times\cos(10.204^\circ+20.751^\circ-0.000^\circ)}$$

$$= 24.239^\circ$$

$$\Delta = \sin^{-1}\frac{\sin(\beta+\theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000^\circ+10.204^\circ)}{\sin 30.00^\circ} = 20.751^\circ$$

Rate of Change of Slip Angle $\omega i = 10.00°\sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω = 44.00° P = 5.508 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta\cdot\sin(\omega-\phi+\theta)}{\cos(\omega-\phi-\alpha-\delta)}$$

$$= \frac{13.011/\cos 10.204^\circ\times\sin(44.00^\circ-30.00^\circ+10.204^\circ)}{\cos(44.00^\circ-30.00^\circ-0.000^\circ-24.239^\circ)}$$

= 5.508 kN

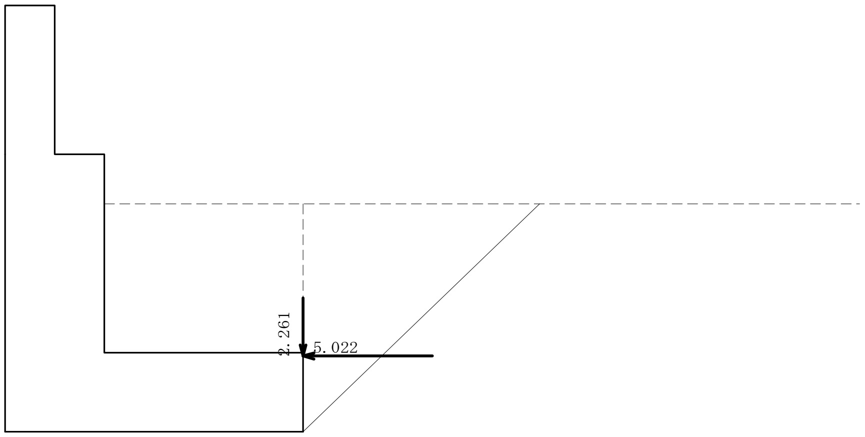
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
$Ph = P\cdot\cos(\alpha+\delta) = 5.508\times\cos(0.000^\circ+24.239^\circ) = 5.022$ kN
Vertical Component
$Pv = P\cdot\sin(\alpha+\delta) = 5.508\times\sin(0.000^\circ+24.239^\circ) = 2.261$ kN
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383\ \text{m}$$

$x = xp-Ho\cdot\tan\alpha = 1.500-0.383\times\tan 0.000^\circ = 1.500$ m
$y = yp+Ho = 0.000+0.383 = 0.383$ m

Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe) xp ＝ 1.500 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 1.150 m
Angle of Earth Pressure Acting Surface to the Vertical α ＝ 0.000 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Angle of Ground Surface from the Horizontal β ＝ 0.000 °
Wall Friction Angle δ ＝ β ＝ 0.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
59.00	7.549	0.000	0.000	7.549	4.184
60.00	7.254	0.000	0.000	7.254	4.188
61.00	6.965	0.000	0.000	6.965	4.185

Earth pressure becomes maximum when:
ω ＝ 60.00° P ＝ 4.188 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{7.254 \times \sin(60.00° - 30.00°)}{\cos(60.00° - 30.00° - 0.000° - 0.000°)}$$

＝ 4.188 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 4.188 \times \cos(0.000° + 0.000°) = 4.188$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 4.188 \times \sin(0.000° + 0.000°) = 0.000$ kN

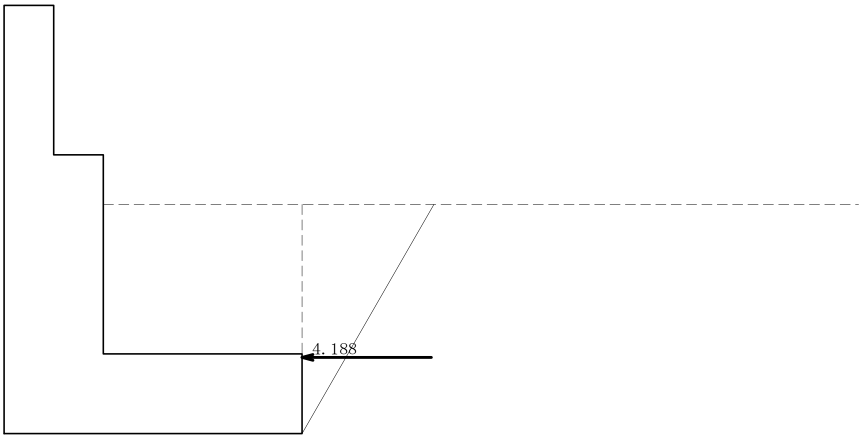
Location

$$Ho = \frac{H}{3} = \frac{1.150}{3} = 0.383 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.500 - 0.383 \times \tan 0.000° = 1.500$ m

$y = yp + Ho = 0.000 + 0.383 = 0.383$ m

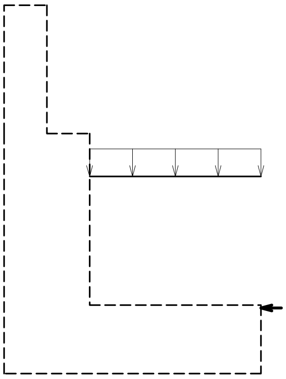
- Earth Pressure Diagram



2.6 Total Acting Force

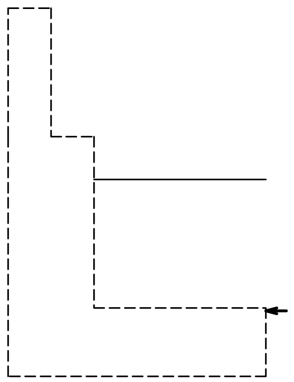
(1) Forces on Front Side of Footing

[1] Ordinary Condition (Surcharge)



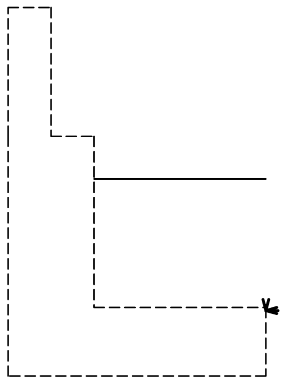
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Loading, Snow	11.600	0.000	1.000	0.000	11.600	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	57.394	8.635	―――	―――	40.512	3.307

[2] Ordinary Condition (Rear Side)



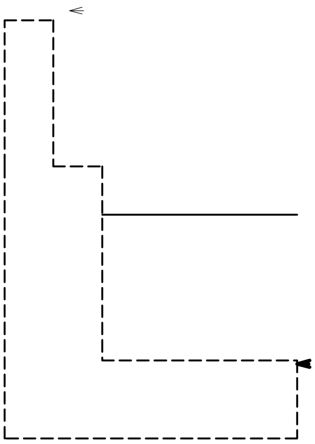
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Earth Pressure	0.000	8.635	1.500	0.383	0.000	3.307
Total	45.794	8.635	———	———	28.912	3.307

[3] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	45.794	8.243	0.631	0.724	28.912	5.969
Earth Pressure	2.261	5.022	1.500	0.383	3.391	1.923
Total	48.055	13.265	———	———	32.303	7.893

[4] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	45.794	0.000	0.631	0.000	28.912	0.000
Loading, Snow	0.000	4.188	1.500	0.383	0.000	1.604
Collision Load	0.000	5.181	0.000	2.200	0.000	11.398
Total	45.794	9.369	———	———	28.912	13.002

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary (Surcharge)	57.394	8.635	37.204
Ordinary (Rear Side)	45.794	8.635	25.604
Earthquake	48.055	13.265	24.410
Collision	45.794	9.369	15.910

(2) Forces at the Center of Footing

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,

Footing Earth Pressure Width : $B_j = 1.500$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	57.394	8.635	5.841
Ordinary (Rear Side)	45.794	8.635	8.741
Earthquake	48.055	13.265	11.631
Collision	45.794	9.369	18.435

- Full Width per 8,300m

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary (Surcharge)	476.369	71.671	48.479
Ordinary (Rear Side)	380.089	71.671	72.549
Earthquake	398.855	110.099	96.534
Collision	380.089	77.760	153.013

2.7 Stability Analysis Result

2.7.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B ＝ 1.500

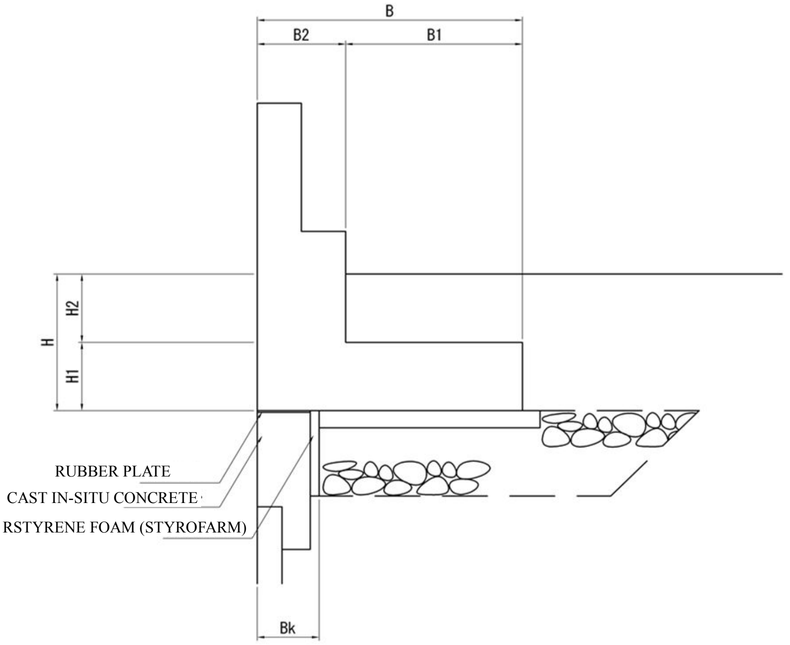
$$e_a = B／n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary (Surcharge)	40.512	3.307	57.394	0.648	0.102	≦	0.250	○
Ordinary (Rear Side)	28.912	3.307	45.794	0.559	0.191	≦	0.250	○
Earthquake	32.303	7.893	48.055	0.508	0.242	≦	0.500	○
Collision	28.912	13.002	45.794	0.347	0.403	≦	0.500	○

2.7.2 Stability from Sliding



Ｈ　(m)	1.250
Ｈ１　(m)	0.400
Ｈ２　(m)	0.850
Ｂ　(m)	1.600
Ｂ１　(m)	1.100
Ｂ２　(m)	0.500
Ｂｋ　(m)	0.350

Ground Reaction Force Reduction

Load Condition	Eccentricity (m)	Ground Reaction Width (m)	Ground Reaction (kN/m^2)		Deducted Reaction (kN/m2)	
			q min	q max	0.000 (m)	0.350 (m)
Ordinary (Rear Side)	0.208	1.600	7.825	62.783	62.783	50.76
Earthquake	0.260	1.600	0.971	73.825	73.825	57.88
Collision	0.349	1.353	0.000	83.498	83.498	61.89

Stability from Sliding

In the stability analysis against sliding, the ground reaction of the reinforced earth bulk concrete does not contribute to the sliding force.

Load Condition	Vertical Forces ΣN (kN)	Deducted Vertical Force Ｎｎａ (kN)	Vertical Load on Earth Wall (Deducted V. Force) ΣN’=ΣN-Ｎｎａ	Total Horizontal Forces ΣH (kN)
Ordinary (Rear Side)	56.486	19.870	36.616	11.620
Earthquake	59.837	23.050	36.787	17.610
Collision	56.486	25.444	31.042	10.507

Factor of safety for sliding, Fs:

$$Fs = \Sigma N' \cdot \mu / \Sigma H$$

Load Condition	Factor of Safety F s	Required Factor of Safety F s a
Ordinary (Rear Side)	1.891	≧1.5
Earthquake	1.253	≧1.2
Collision	1.773	≧1.2

2.7.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

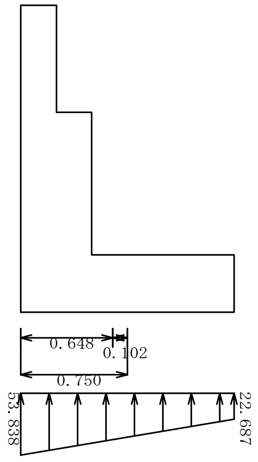
Where,

ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)

B : Bottom plate width (m), B = 1.500

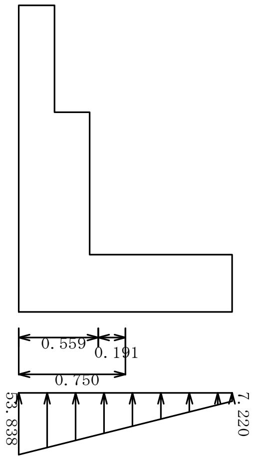
e : Eccentricity (m)

[1] Ordinary Condition (Surcharge)



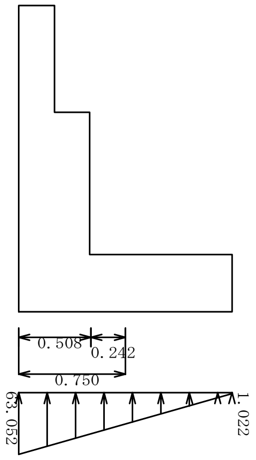
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.500	Trapezoid	22.687	53.838 ≦	200.000	○

[2]常時(背面)



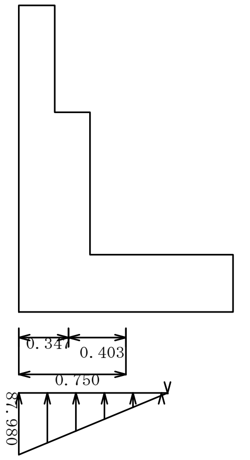
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.500	Trapezoid	7.220	53.838 ≦	200.000	○

[3]地震時



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.500	Trapezoid	1.022	63.052 ≦	300.000	○

[4]衝突時



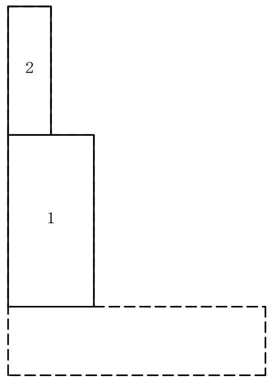
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judge ment
		qmin	qmax	Allow.	
1.041	Triangle	0.000	87.980 ≦	300.000	○

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m)		Vi・Xi	Vi・Yi	Remarks
			Xi	Yi			
1 2	0.500× 1.000× 1.000 0.250× 0.750× 1.000	0.500 0.188	0.250 0.125	0.500 1.375	0.125 0.023	0.250 0.258	
Σ		0.688	—	—	0.148	0.508	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.148／ 0.688 ＝ 0.216 (m)
YG ＝ Σ（Vi・Yi）／ΣVi ＝ 0.508／ 0.688 ＝ 0.739 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Ordinary Conditions (Surcharge and Rear Side), Collision

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.688 ＝ 16.844	0.034

Force Location

X ＝ Xc－XG ＝ 0.250－0.216
＝ 0.034 m

Where,
Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

[2] Earthquake

Location	W ＝ γ ・ V (kN)	Location X (m)
Frame(Rebar)	24.500 × 0.688 ＝ 16.844	0.034

Location	H ＝ W ・ kh (kN)	Location Y (m)
Frame(Rebar)	16.844 × 0.180 ＝ 3.032	0.739

Force Location

$$X = X_c - X_G = 0.250 - 0.216$$

$$= 0.034\ \mathrm{m}$$

Where,

Xc : Horizontal distance from the front of vertical wall to the center of design cross section (m)

3.1.3 Collision Load

Collision load for brace type protection fence is considered.

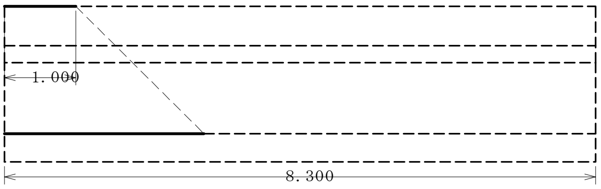
Horizontal Force

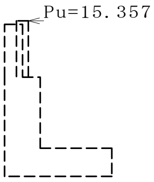
$$P_u = \frac{p}{\lambda + h + y}$$

Where,

Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
y : Distance from the tip of section being checked (m), y ＝ 1.000(m)

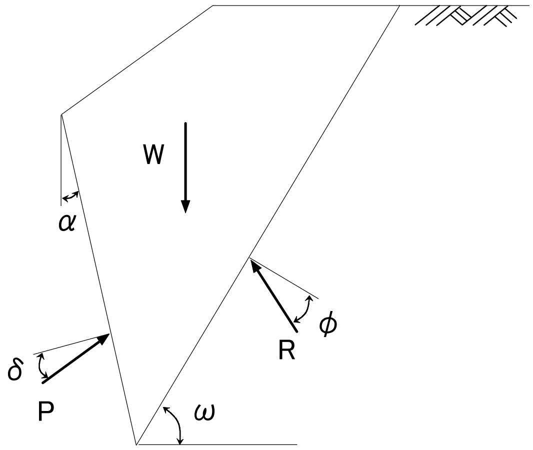
[1] Collision





p (kN)	λ (m)	h (m)	Pu (kN/m)	Location Y (m)
43.000	1.000	0.800	15.357	1.800

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition (Surcharge)

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp ＝ 0.250 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 0.750 m
Angle of Earth Pressure Acting Surface to the Vertical	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	6.092	9.834	4.172
56.00	3.605	0.000	5.868	9.473	4.176
57.00	3.471	0.000	5.650	9.121	4.172

Earth pressure becomes maximum when:
ω ＝ 56.00° P ＝ 4.176 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{9.473 \times \sin(56.00^\circ - 30.00^\circ)}{\cos(56.00^\circ - 30.00^\circ - 0.000^\circ - 20.000^\circ)}$$

＝ 4.176 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 4.176 \times \cos(0.000° + 20.000°) = 3.924$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 4.176 \times \sin(0.000° + 20.000°) = 1.428$ kN

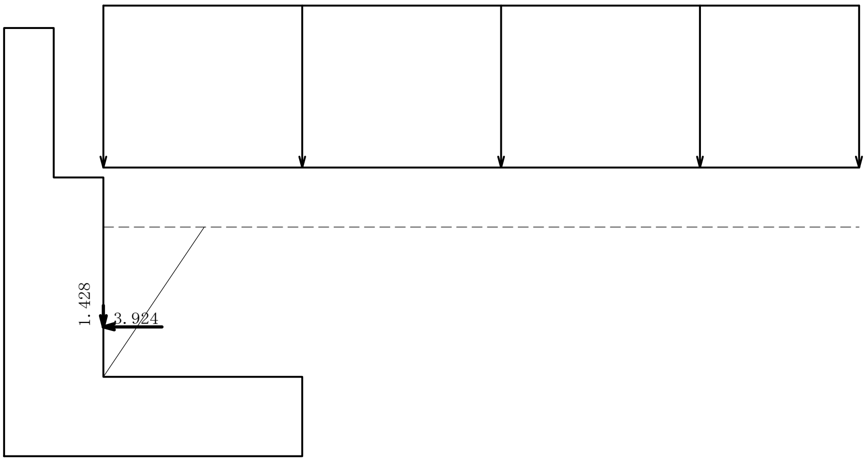
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.250 = -0.250$ m

$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

・ Earth Pressure Diagram



[2] Ordinary Condition (Rear Side), Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.250$ m

$yp = 0.000$ m

Height of Assumed Rear Surface $H = 0.750$ m

Angle of Earth Pressure Acting Surface to the Vertical $\alpha = 0.000$ °

Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3

Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °

Wall Friction Angle $\delta = 2/3\varphi = 20.000$ °

Rate of Change of Slip Angle $\omega i = 10.00$ °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
55.00	3.742	0.000	0.000	3.742	1.587
56.00	3.605	0.000	0.000	3.605	1.589
57.00	3.471	0.000	0.000	3.471	1.588

Earth pressure becomes maximum when:
$\omega = 56.00°$ $P = 1.589$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{3.605 \times \sin(56.00° - 30.00°)}{\cos(56.00° - 30.00° - 0.000° - 20.000°)}$$

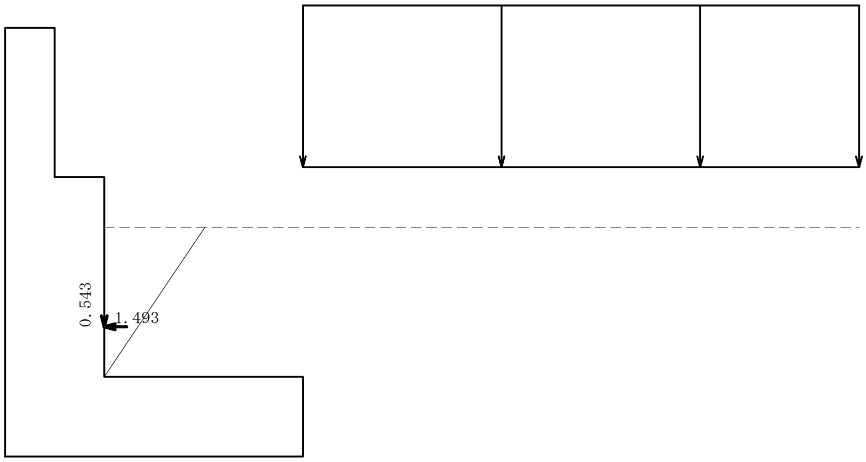
$= 1.589$ kN

Earth Pressure’s location and horizontal and vertical components:
Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 1.589 \times \cos(0.000° + 20.000°) = 1.493$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 1.589 \times \sin(0.000° + 20.000°) = 0.543$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.250 = -0.250$ m
$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

- Earth Pressure Diagram



[3] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.250$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.750$ m
Angle to the Vertical of Earth Pressure Surface $\alpha = 0.000$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 1/2\varphi = 15.000$ °
Composite angle during earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)

Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
45.00	5.344	0.000	0.000	5.344	2.312
46.00	5.161	0.000	0.000	5.161	2.316
47.00	4.983	0.000	0.000	4.983	2.316

Earth pressure becomes maximum when:

$\omega = 46.00° \quad P = 2.316$ kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.161/\cos 10.204° \times \sin(46.00° - 30.00° + 10.204°)}{\cos(46.00° - 30.00° - 0.000° - 15.000°)}$$

$= 2.316$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 2.316 \times \cos(0.000° + 15.000°) = 2.237$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 2.316 \times \sin(0.000° + 15.000°) = 0.599$ kN

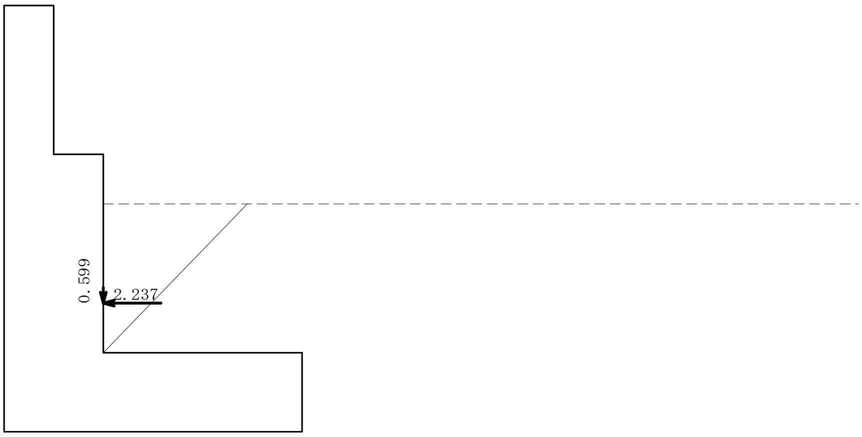
Location

$$Ho = \frac{H}{3} = \frac{0.750}{3} = 0.250 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.250 \times \tan 0.000° - 0.250 = -0.250$ m

$y = yp + Ho = 0.000 + 0.250 = 0.250$ m

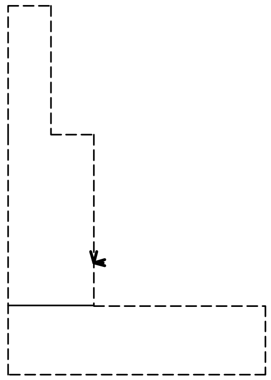
- Earth Pressure Diagram



3.1.5 Total Sectional Force

(To neglect axial forces and eccentric moment, vertical forces are not considered)

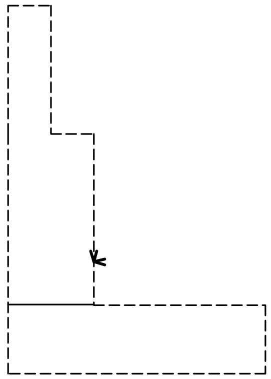
[1] Ordinary Condition (Surcharge)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi} + M_{yi}$ (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	1.428	3.924	-0.250	0.250	0.981
Total	0.000	3.924	―――	―――	0.981

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

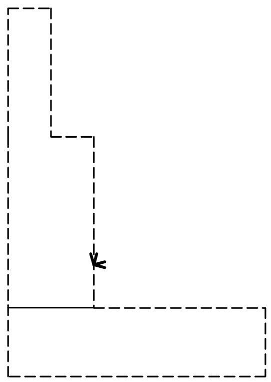
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi} + M_{yi}$ (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	0.543	1.493	-0.250	0.250	0.373
Total	0.000	1.493	―――	―――	0.373

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

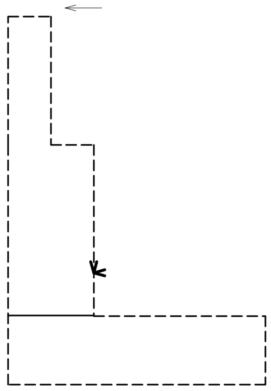
[3] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M =M_{xi}+M_{yi} (kN.m)
Self-weight	16.844	3.032	0.034	0.739	2.239
Earth Pressure	0.599	2.237	-0.250	0.250	0.559
Total	0.000	5.269	———	———	2.799

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[4] Collision

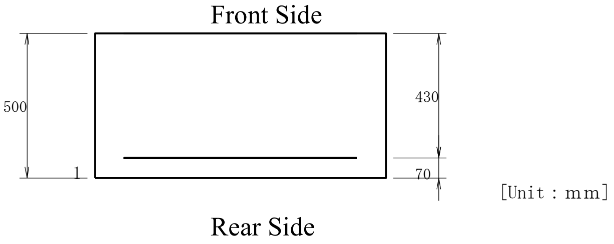


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	M =M_{xi}+M_{yi} (kN.m)
Self-weight	16.844	0.000	0.034	0.000	0.000
Earth Pressure	0.543	1.493	-0.250	0.250	0.373
Collision Load	0.000	15.357	0.000	1.800	27.643
Total	0.000	16.850	———	———	28.016

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	7.0	D13	1.267	4.000	5.068
	2	—	–	—	—	—

Required rebar amount at tension side: 2.261 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 41666.7\times10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 500000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 500.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	1.668	≦	79.737	0.000	5.000	○
Ordinary (Rear Side)	5.068	0.635	≦	79.737	0.000	5.000	○
Earthquake	5.068	4.758	≦	79.737	0.000	5.000	○
Collision	5.068	47.627	≦	79.737	0.000	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2 $(0.002 \leqq \varepsilon(y) \leqq \varepsilon cu)$

$y2 = x - y1$

$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$

Compressive force at the compression side rebars $(d_i \leqq x)$

$\varepsilon_i = A \cdot (x - d_i)$

$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$

$S_{ci} = \sigma si \cdot Asi$

$S1 = \Sigma S_{ci}$

Tensile force at the tension side rebars $(d_j > x)$

$\varepsilon_j = A \cdot (d_j - x)$

$\sigma_{sj} = \varepsilon_j \cdot E_s$

$S_{tj} = \sigma_{sj} \cdot A_{sj}$

$S2 = \Sigma S_{tj}$

From the axial force balance

$N = C1 + C2 + S1 - \alpha \cdot S2$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$A_{sb} = \alpha \cdot \Sigma A_{sj}$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 430.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	137.877	○
Ordinary (Rear Side)	5.068	≦	137.877	○
Earthquake	5.068	≦	137.877	○
Collision	5.068	≦	137.877	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^2+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

Stress Calculation

$$\sigma_c=\frac{M}{\dfrac{b\cdot x}{2}\cdot\left(\dfrac{h}{2}-\dfrac{x}{3}\right)+n\cdot As\cdot\dfrac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s=n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h ＝ 500.000
b : Member cross-section width (mm), b ＝ 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n ＝ 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
				Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	0.000	7.361	0.066	≦	8.000	4.774	≦	180.000	○
Ordinary (Rear Side)	0.373	0.000	7.361	0.025	≦	8.000	1.816	≦	180.000	○
Earthquake	2.799	0.000	7.361	0.188	≦	12.000	13.620	≦	300.000	○
Collision	28.016	0.000	7.361	1.877	≦	12.000	136.340	≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member’s effective height (mm)

b : Sectional member’s effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member’s effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

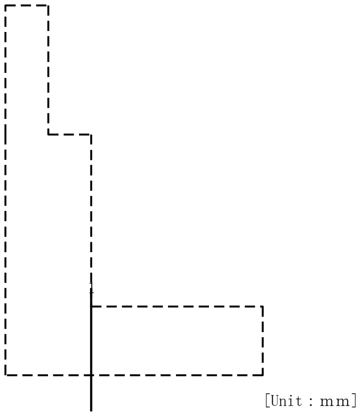
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judge ment
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary(Surcharge)	3.924	43.000	0.009	≦	0.224	1.700	1.33	0.74	1.00	○
Ordinary(Rear Side)	1.493	43.000	0.003	≦	0.224	1.700	1.33	0.74	1.00	○
Earthquake	5.269	43.000	0.012	≦	0.341	2.550	1.33	0.74	1.00	○
Collision	16.850	43.000	0.039	≦	0.336	2.550	1.33	0.74	1.00	○

4 Design of Heel Plate

4.1 Design of Section [1]

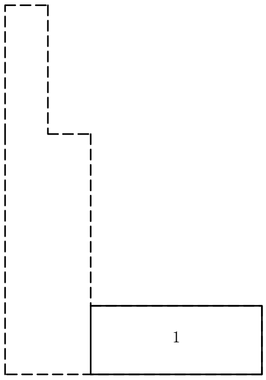
Distance from joint: 0.000 m



4.1.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



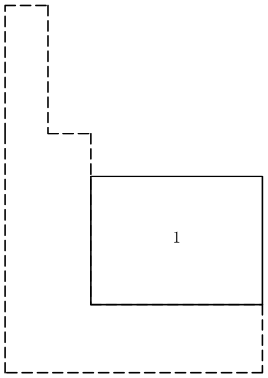
2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity (m) Xi	Vi・Xi	Remarks
1	1.000× 0.400× 1.000	0.400	0.500	0.200	
Σ		0.400	——	0.200	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.200／ 0.400 ＝ 0.500 (m)

(2) Soil at Rear Side

1) Block Division



3) Volume・Center of Gravity

区分	計算式 幅 × 高さ × 奥行	体積 Vi (m³)	重心位置 Xi (m)	Vi・Xi	備考
1	1.000× 0.750× 1.000	0.750	0.500	0.375	
Σ		0.750	———	0.375	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.375／ 0.750 ＝ 0.500 (m)

4.1.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force W ＝ γ ・ V (kN)	Location X (m)
Frame	24.500 × 0.400 ＝ 9.800	0.500
Soil(Rear Side)	19.000 × 0.750 ＝ 14.250	0.500

4.1.3 Ground Surface Load・Snow Load

Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

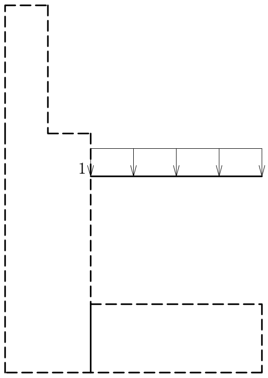
Where,

q : Ground surface loading

L : Length of ground surface load

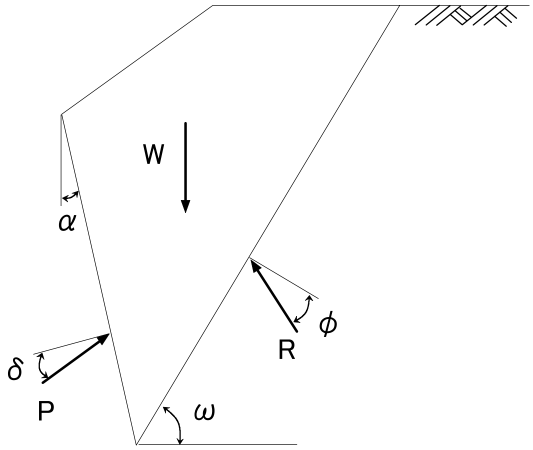
X : Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m^2)	q2 (kN/m^2)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	1.000	11.600	0.500

4.1.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle to the vertical of the Earth Pressure Surface	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle to the Horizontal of the Ground Surface	β ＝ 0.000 °
Composite Angle during Earthquake	θ ＝ $\tan^{-1}$kh＝ $\tan^{-1}$0.18 ＝10.204°

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi = 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω ＝ 44.00° P ＝ 5.508 kN

Earth Pressure

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{13.011/\cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

$$= 5.508 \text{ kN}$$

Vertical component of Earth Pressure:

Vertical Component

$$P_v = P \cdot \sin(\alpha + \delta) = 5.508 \times \sin(0.000° + 24.239°) = 2.261 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:。

$$p_v = \frac{2 \cdot P_v}{L} = \frac{2 \times 2.261}{1.000} = 4.522 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 ＝ 0.000 m
Width of distributed load perpendicular to the design section L2 ＝ 1.000 m

Distributed Load at the Design Section, pd

$$pd = \frac{pv}{L} \cdot L1 = \frac{4.522}{1.000} \times 0.000 = 0.000 \text{ kN/m}$$

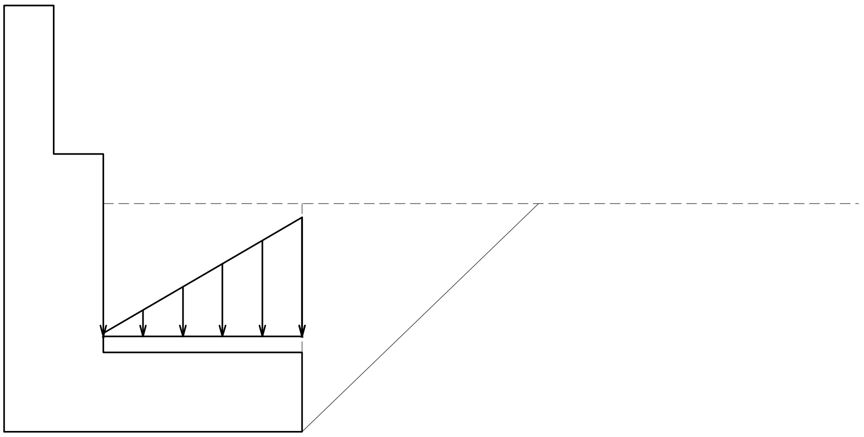
Vertical Force

$$N = \frac{1}{2} \cdot (pd+pv) \cdot L2 = \frac{1}{2} \times (0.000+4.522) \times 1.000 = 2.261 \text{ kN}$$

Location

$$x = \frac{pd+2 \cdot pv}{pd+pv} \cdot \frac{L2}{3} = \frac{0.000+2\times 4.522}{0.000+4.522} \times \frac{1.000}{3} = 0.667 \text{ m}$$

- Earth Pressure Diagram



4.1.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1+q2) \cdot L$$

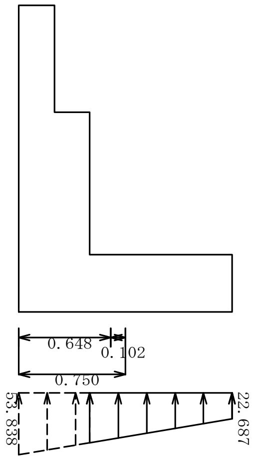
Location

$$X = \frac{2 \cdot q1+q2}{3 \cdot (q1+q2)} \cdot L$$

Where,

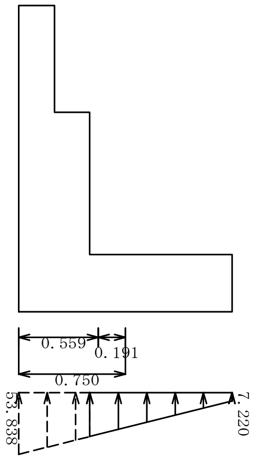
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



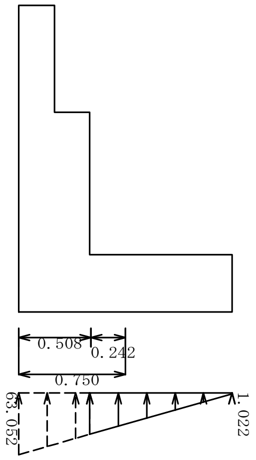
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.687	43.454	1.000	33.071	0.448

[2] Ordinary Condition (Rear Side)



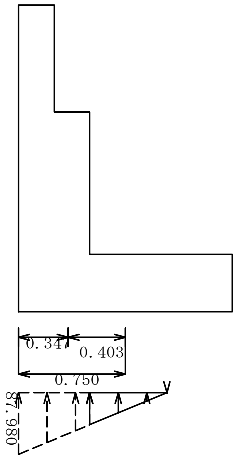
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.220	38.299	1.000	22.759	0.386

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
1.022	42.375	1.000	21.699	0.341

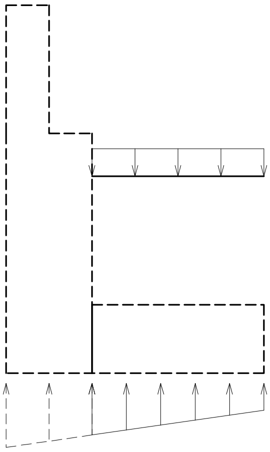
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	45.723	0.541	12.368	0.180

4.1.6 Total Sectional Forces

[1] Ordinary Condition (Surcharge)



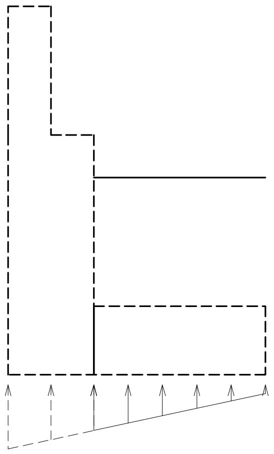
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Loading, Snow Load	11.600	0.500	5.800
Ground Reaction	-33.071	0.448	-14.805
Total	2.579	———	3.020

Sectional force of vertical wall foundation, M1 ＝ 0.981 kN.m

Sectional force of the base of heel, M3 ＝ 3.020 kN.m

Since M3 > M1, M1 is applied as the sectional force of the joint

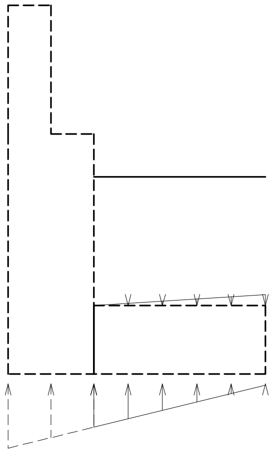
[2] Ordinary Condition (Rear Side)



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Ground Reaction	-22.759	0.386	-8.790
Total	1.291	———	3.235

Sectional force of vertical wall foundation, M1 ＝ 0.373 kN.m
Sectional force of the base of heel, M3 ＝ 3.235 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

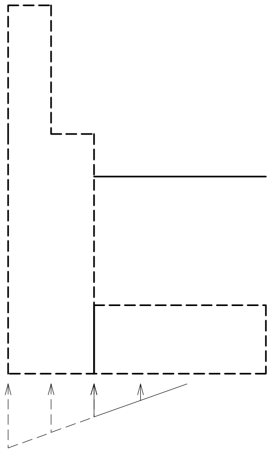
[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Loading, Snow Load	2.261	0.667	1.507
Ground Reaction	-21.699	0.341	-7.403
Total	4.612	———	6.129

Sectional force of vertical wall foundation, M1 ＝ 2.799 kN.m
Sectional force of the base of heel, M3 ＝ 6.129 kN.m
Since M3 > M1, M1 is applied as the sectional force of the joint

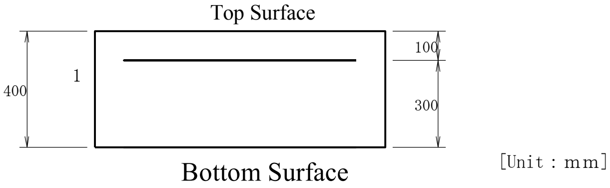
[4] Collision



Item	N_i (kN)	X_i (m)	$M=N_i \cdot X_i$ (kN.m)
Self-weight	24.050	0.500	12.025
Ground Reaction	-12.368	0.180	-2.230
Total	11.682	―――	9.795

4.1.7 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Top Surface	1	10.0	D13	1.267	4.000	5.068
	2	――	–	――	――	――
Bottom Surface	1'	――	–	――	――	――
	2'	――	–	――	――	――

Required tension side rebar amount 1.127 (cm^2)

(2) Check of Minimum Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 26666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 400000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 400.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	Min. Rebar Amount (cm^2)	Judgement
Ordinary (Surcharge)	5.068	1.668	≦	51.031	5.000	○
Ordinary (Rear Side)	5.068	0.635	≦	51.031	5.000	○
Earthquake	5.068	4.758	≦	51.031	5.000	○
Collision	5.068	16.651	≦	51.031	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1 $(0 \leqq \varepsilon(y) \leqq 0.002)$

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2 （$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$y2 = x - y1$

$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$

Compressive force at the compression side rebars（$d_i \leqq x$）

$\varepsilon_i = A \cdot (x - d_i)$

$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$

$S_{ci} = \sigma si \cdot Asi$

$S1 = \Sigma S_{ci}$

Tensile force at the tension side rebars（$d_j > x$）

$\varepsilon_j = A \cdot (d_j - x)$

$\sigma_{sj} = \varepsilon_j \cdot E_s$

$S_{tj} = \sigma_{sj} \cdot A_{sj}$

$S2 = \Sigma S_{tj}$

From the axial force balance

$N = C1 + C2 + S1 - \alpha \cdot S2$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$A_{sb} = \alpha \cdot \Sigma A_{sj}$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young’s Modulus (N/mm^2), $Es = 200000$
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 300.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the “ith” compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the “ith” compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the “ith” compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the “jth” tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the “jth” tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the “jth” tension side rebar (N/mm^2)

Load Conditon (Water Level)	A_s (cm^2)		A_{sb} (cm^2)	Judgement
Ordinary (Surcharge)	5.068	≦	96.193	○
Ordinary (Rear Side)	5.068	≦	96.193	○
Earthquake	5.068	≦	96.193	○
Collision	5.068	≦	96.193	○

(4) Check of Bending Stress

(Reference)

Calculation of neutral axis

Find x from:

$$x^2+\frac{2\cdot n}{b}\left\{As\cdot(x-d)\right\}=0.0$$

Stress Calculation

$$\sigma_c=\frac{M}{\frac{b\cdot x}{2}\cdot\left(\frac{h}{2}-\frac{x}{3}\right)+n\cdot As\cdot\frac{(x-d)\cdot(h/2-d)}{x}}$$

$$\sigma_s=n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)

h : Member cross-section height (mm), h = 400.000

b : Member cross-section width (mm), b = 1000.000

d : Effective height of the member (mm)

As : Total cross-sectional area of the tension side rebar (mm^2)

n : Rebar and concrete's coefficient of Young's modulus, n = 15.00

e : Distance from the sectional member's centroid to the axial force (mm)

σc : Concrete bending compressive stress (N/mm^2)

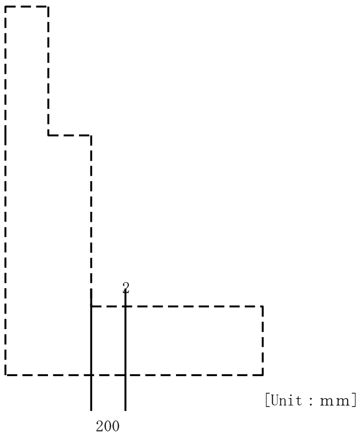
σs : Rebar tensile stress (N/mm^2)

M : Bending moment (N.mm)

Load Conditon (Water Level)	M (kN.m)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
			Calculated		Allowable	Calculated		Allowable	
Ordinary (Surcharge)	0.981	6.035	0.116	≦	8.000	6.916	≦	180.000	○
Ordinary (Rear Side)	0.373	6.035	0.044	≦	8.000	2.632	≦	180.000	○
Earthquake	2.799	6.035	0.331	≦	12.000	19.732	≦	300.000	○
Collision	9.795	6.035	1.159	≦	12.000	69.055	≦	300.000	○

4.2 Design of Section [2]

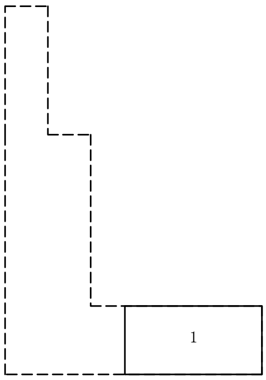
Distance from joint: 0.200 m



4.2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



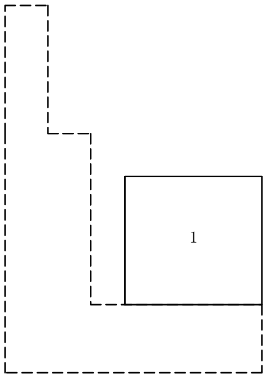
2) Volume・Center of Gravity

	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.800× 0.400× 1.000	0.320	0.400	0.128	
Σ		0.320	———	0.128	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.128／ 0.320 ＝ 0.400 (m)

(2) Soil at Rear Side

1) Block Division



2) Volume・Center of Gravity

Classification	Width x Height x Depth	Vol. Vi (m^3)	Center of Gravity Xi (m)	Vi・Xi	Remarks
1	0.800× 0.750× 1.000	0.600	0.400	0.240	
Σ		0.600	——	0.240	

Center of Gravity XG ＝ Σ（Vi・Xi）／ΣVi ＝ 0.240／ 0.600 ＝ 0.400 (m)

4.2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force

(1) Actual Force due to Self-weight

[1] Ordinary Conditions (Surcharge and Rear Side), Earthquake, Collision

Location	Vertical Force $W = \gamma \cdot V$ (kN)	Location X (m)
Frame	24.500 × 0.320 ＝ 7.840	0.400
Soil(Rear Side)	19.000 × 0.600 ＝ 11.400	0.400

4.2.3 Ground Surface Load・Snow Load

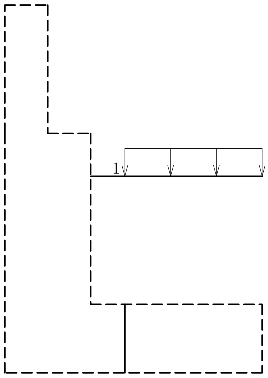
Vertical Force

$$N = \frac{1}{2} \cdot (q1 + q2) \cdot L$$

Where,

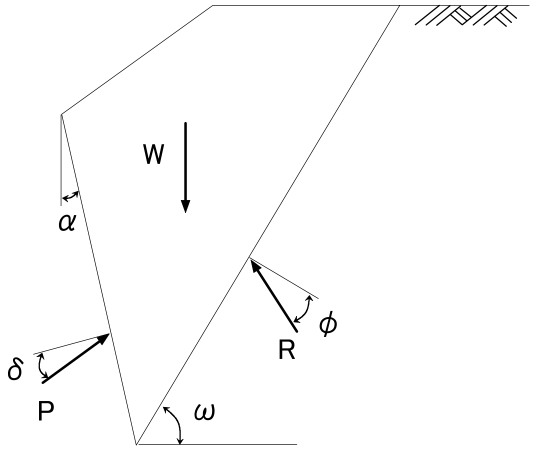
q ：Ground surface loading
L ：Length of ground surface load
X ：Distance from the design section to the resultant force

[1] Ordinary Condition (Surcharge)



No.	q1 (kN/m²)	q2 (kN/m²)	L (m)	Vertical Force N (kN)	Location X (m)
1	11.600	11.600	0.800	9.280	0.400

4.2.4 Earth Pressure



[1] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance from toe)	xp ＝ 1.500 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 1.150 m
Angle to the vertical of the Earth Pressure Surface	α ＝ 0.000 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m³
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Angle to the Horizontal of the Ground Surface	β ＝ 0.000 °
Composite Angle during Earthquake	θ ＝ $\tan^{-1}kh$＝ $\tan^{-1}0.18$ ＝10.204°

Wall Friction Angle

$$\delta = \tan^{-1}\frac{\sin\phi \cdot \sin(\theta + \Delta - \beta)}{1 - \sin\phi \cdot \cos(\theta + \Delta - \beta)}$$

$$= \tan^{-1}\frac{\sin 30.00° \times \sin(10.204° + 20.751° - 0.000°)}{1 - \sin 30.00° \times \cos(10.204° + 20.751° - 0.000°)}$$

$$= 24.239°$$

$$\Delta = \sin^{-1}\frac{\sin(\beta + \theta)}{\sin\phi} = \sin^{-1}\frac{\sin(0.000° + 10.204°)}{\sin 30.00°} = 20.751°$$

Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
43.00	13.473	0.000	0.000	13.473	5.499
44.00	13.011	0.000	0.000	13.011	5.508
45.00	12.564	0.000	0.000	12.564	5.508

Earth pressure becomes maximum when:
ω ＝ 44.00° P ＝ 5.508 kN

Earth Pressure

$$P = \frac{W / \cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{13.011 / \cos 10.204° \times \sin(44.00° - 30.00° + 10.204°)}{\cos(44.00° - 30.00° - 0.000° - 24.239°)}$$

＝ 5.508 kN

Vertical component of Earth Pressure:
Vertical Component

$$Pv = P \cdot \sin(\alpha + \delta) = 5.508 \times \sin(0.000° + 24.239°) = 2.261 \text{ kN}$$

The ff. equivalent triangular distributed load of the vertical component will be used:

$$pv = \frac{2 \cdot Pv}{L} = \frac{2 \times 2.261}{1.000} = 4.522 \text{ kN/m}$$

Where,
pv : equivalent triangular distributed load
Pv : vertical component of earth pressure
L : length of heel plate

Distance from the joint to the design section L1 = 0.200 m
Width of distributed load perpendicular to the design section L2 = 0.800 m

Distributed Load at the Design Section, pd

$$pd = \frac{pv}{L} \cdot L1 = \frac{4.522}{1.000} \times 0.200 = 0.904 \text{ kN/m}$$

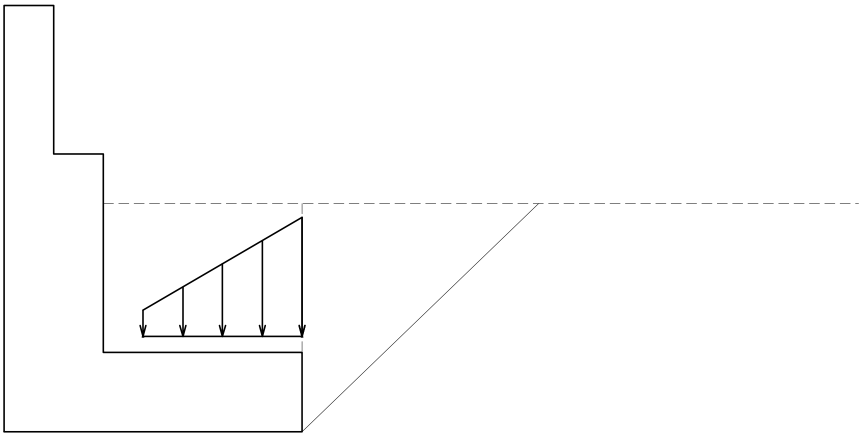
Vertical Force

$$N = \frac{1}{2} \cdot (pd + pv) \cdot L2 = \frac{1}{2} \times (0.904 + 4.522) \times 0.800 = 2.171 \text{ kN}$$

Location

$$x = \frac{pd + 2 \cdot pv}{pd + pv} \cdot \frac{L2}{3} = \frac{0.904 + 2 \times 4.522}{0.904 + 4.522} \times \frac{0.800}{3} = 0.489 \text{ m}$$

- Earth Pressure Diagram



4.2.5 Ground Reaction

Vertical Force

$$N = \frac{1}{2}(q1 + q2) \cdot L$$

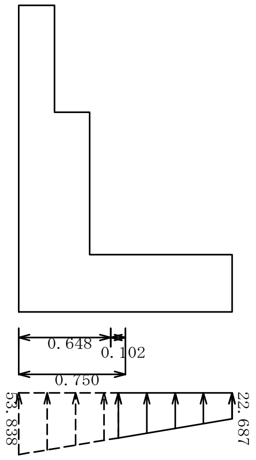
Location

$$X = \frac{2 \cdot q1 + q2}{3 \cdot (q1 + q2)} \cdot L$$

Where,

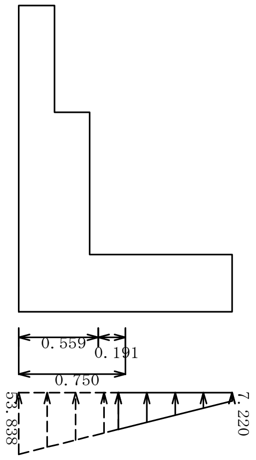
q1 : Ground reaction pressure at the front surface of heel (kN/m^2)
q2 : Ground reaction pressure at the design section of heel (kN/m^2)
L : Width of ground reaction (m)

[1] Ordinary Condition (Surcharge)



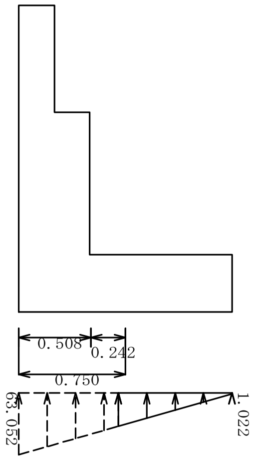
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
22.687	39.301	0.800	24.795	0.364

[2] Ordinary Condition (Rear Side)



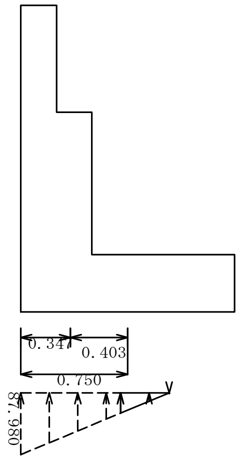
Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
7.220	32.083	0.800	15.721	0.316

[3] Earthquake



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
1.022	34.105	0.800	14.051	0.274

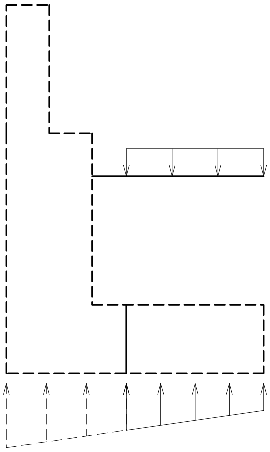
[4] Collision



Ground Reaction (kN/m^2)		Width L (m)	Vertical Force N (kN)	Location X (m)
q1	q2			
0.000	28.820	0.341	4.914	0.114

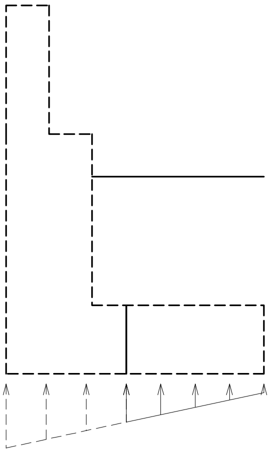
4.2.6 Total Sectional Forces

[1] Ordinary (Surcharge)



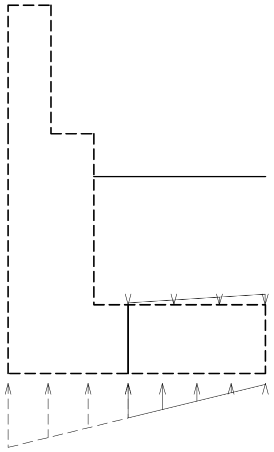
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Loading, Snow Load	9.280	0.400	3.712
Ground Reaction	-24.795	0.364	-9.032
Total	3.725	———	2.376

[2] Ordinary Condition (Rear Side)



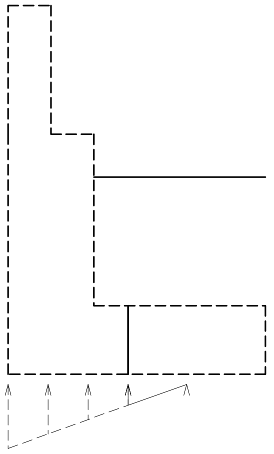
Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Ground Reaction	-15.721	0.316	-4.962
Total	3.519	――	2.734

[3] Earthquake



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Loading, Snow Load	2.171	0.489	1.061
Ground Reaction	-14.051	0.274	-3.856
Total	7.360	――	4.901

[4] Collision



Item	N_i (kN)	X_i (m)	$M = N_i \cdot X_i$ (kN.m)
Self-weight	19.240	0.400	7.696
Ground Reaction	-4.914	0.114	-0.559
Total	14.326	――	7.137

4.2.7 Section Design (Allowable Stress Method)

(1) Check of Shear Stress

when $S_h = S - \frac{M}{d'} \tan\theta$ $a > 2.5d$,

when $a \leqq 2.5d$, $S_h = S$

Where,

S_h : Shear force considering the effect of various effective height of design sections (N)

d : Effective height of footing calculated from the column, front wall or rear wall (mm)

d' : Effective height of sectional member (mm)

b : Effective width of sectional member (mm)

S : Shear force acting on the sectional member (N)

M : Bending moment acting on the sectional member (N.mm)

θ : Angle of the top surface footing to the horizontal, θ = 0.000 , tanθ = 0.000

a : Shear span (mm)

Loading Condition (Water Level)	Eff. Height d' (cm)	Shear Span 2.5・d (cm)		a	S (kN)	M (kN.m)	M/d'・tanθ	S_h (kN)
Ordinary (Surcharge)	30.000	75.000	≦	125.000	3.725	2.376	0.000	3.725
Ordinary (Rear Side)	30.000	75.000	≦	125.000	3.519	2.734	0.000	3.519
Earthquake	30.000	75.000	≦	125.000	7.360	4.901	0.000	7.360
Collision	30.000	75.000	≦	108.840	14.326	7.137	0.000	14.326

$$\tau_m = \frac{S_h}{b \cdot d'} \leqq \tau_{a1}$$

Where,

τm : Average shear stress of concrete (N/mm2)

Sh : Actual Shear Force (N)

d : Effective height of footing calculated from the column, front wall or rear wall (mm)

d' : Effective height of sectional member (mm)

b : Effective width of sectional member (mm)

S : Shear force acting on the sectional member (N)

τa1 : Allowable shear stress (N/mm2)

$$\tau_{a1} = Ce \cdot Cpt \cdot Cdc \cdot \tau_{a1}'$$

Where,

τa1' : Allowable shear stress when shear force acts in concrete (N/mm2)

Ce : Correction factor for the sectional member's effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

Cdc : Correction factor for shear span ratio

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judgement
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	Cdc	
Ordinary(Surcharge)	3.725	30.000	0.012	≦	0.270	1.700	1.40	0.84	1.00	○
Ordinary(Rear Side)	3.519	30.000	0.012	≦	0.270	1.700	1.40	0.84	1.00	○
Earthquake	7.360	30.000	0.025	≦	0.411	2.550	1.40	0.84	1.00	○
Collision	14.326	30.000	0.048	≦	0.405	2.550	1.40	0.84	1.00	○

Gravity Wall

(H=2800)

1 Design Conditions

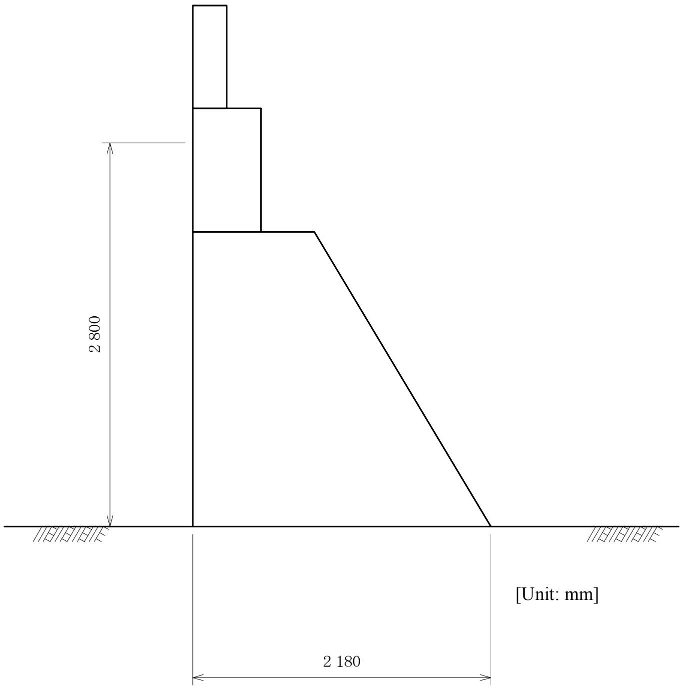
1.1 Applicable Standard

Japan Road Association
Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

『Arbitrary Form – A (Direct Foundation)』

1.3 Configuration



Block Length, B ＝ 8000 (mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】 Soil at Rear Side: 30°

【Unit Weight】

(kN/m^3)

Framework	Vertical Wall	23.000	
	Parapet ①	24.500	
	Parapet ②	24.500	
Water	For Frame Buoyancy Calculation	10.000	
	For Soil Buoyancy Calculation	9.000	
Soil		Damp Weight	Saturated Weight
Rear Side		19.000	20.000
Front Side		19.000	20.000

【Design Horizontal Seismic Coefficient】

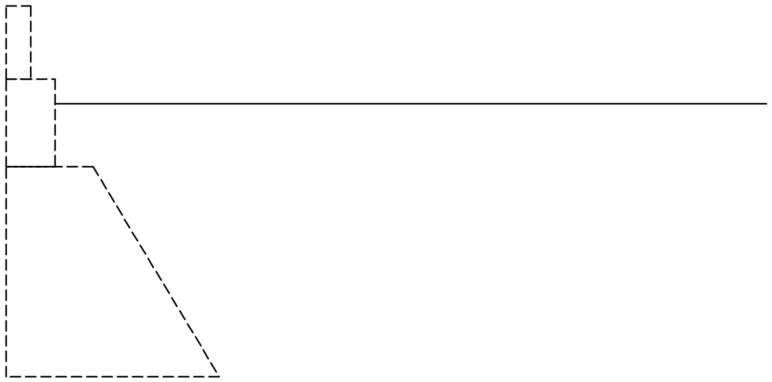
Frame: Kh = 0.18

Soil (Front side): Kh = 0.18

Soil (Rear side): Kh = 0.18

1.6 Soil

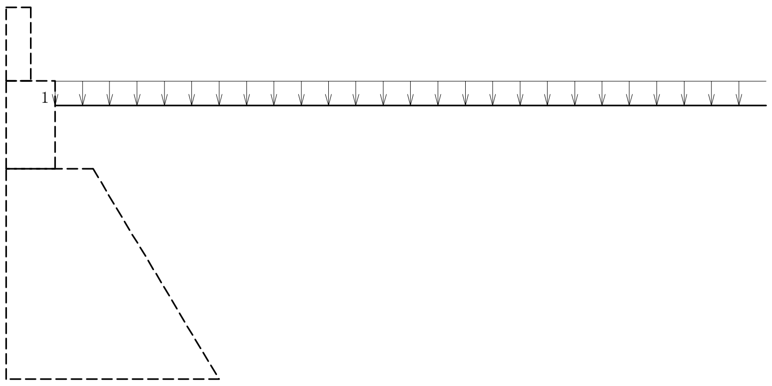
(1) Figure of Soil at Rear Side



Difference in elevation of the tip of retaining wall and ground surface (m)	0.000
Height not considering earth pressure, Hr (m)	0.000

1.7 Loading

[1] Ordinary Condition

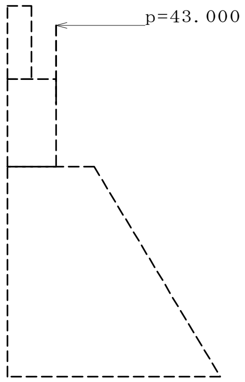


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical Wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	×

1.8 Collision Load

Fence Type: Rigid

[1] Collision



Classification: SC, Load: Straight Wall Type

Horizontal Force

Load Location, h (m)	Load Range, λ (m)	Collision Load, P (kN)
0.800	1.000	43.000

1.9 Arbitrary Load

Not considered.

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method
- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Analysis	Cut	
Ordinary	20.000	20.000	———	———
Earthquake	15.000	15.000	———	———

- In the stability analysis, the assumed rear side is the back of the vertical wall.
- In the stability analysis, the angle formed by the earth pressure's active surface with the vertical is 30.964 degrees.
- In the vertical wall design, the angle formed by the earth pressure's active surface with the vertical is 30.964 degrees.

・Adhesive Force (kN/m^2)

Loading Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	――
Earthquake	0.000	0.000	――

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary	Ordinary Condition
2	Earthquake	kh=0.18
3	Collision	SC Class

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
2	Earthquake	Level 1	←Towards left	――	Neglected	Considered

	Load Condition	1	2	3
Soil	Soil 1			
Loading	Surcharge	○		
Collision・Wind	Collision Load			○
Active Earth Pressure	Not considered			
	Ordinary Earth Pressure	○		○
	Seismic Earth Pressure		○	

Check Items		1	2	3
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	――	――	――
	Rigid Body Stability	――	――	――
	Cross-Section Disruption	――	――	――

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, tanφ_B	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary	1/6	1.500	—
Earthquake	1/3	1.200	—
Collision	1/3	1.200	—

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B ／ V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	τ_{a2}	Rebar Compressive Stress σ_{sna}
Ordinary	1.00	8.000	180.000	0.230	1.700	200.000
Earthquake	1.50	12.000	300.000	0.350	2.550	300.000
Collision	1.50	12.000	300.000	0.345	2.550	300.000

Where,

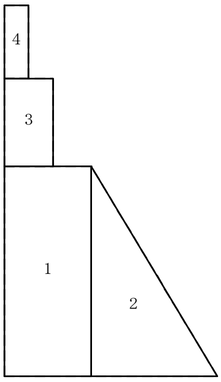
τ_{a1} : Shear stress when shear force acts on concrete
τ_{a2} : Shear stress when shared with diagonal tension bars

2 **Stability Analysis**

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



2) Self-Weight・Center of Gravity

Classification	Width x Height x Depth x Unit Weight	Weight Wi(kN)	Location of Center of Gravity (m)		Wi・Xi	Wi・Yi	Remarks
			Xi	Yi			
1	0.890× 2.150× 1.000×23.000	44.011	0.445	1.075	19.585	47.311	Vertical Wall
2	1/2× 1.290× 2.150× 1.000×23.000	31.896	1.320	0.717	42.103	22.860	Vertical Wall
3	0.500× 0.900× 1.000×24.500	11.025	0.250	2.600	2.756	28.665	Parapet ①
4	0.250× 0.750× 1.000×24.500	4.594	0.125	3.425	0.574	15.734	Parapet ②
Σ		91.526	—	—	65.018	114.570	

Center of Gravity XG ＝ Σ（Wi・Xi）／ΣWi ＝ 65.018／ 91.526 ＝ 0.710 (m)

YG ＝ Σ（Wi・Yi）／ΣWi ＝ 114.570／ 91.526 ＝ 1.252 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Earthquake

Location	Horizontal Force H ＝ W ・ kh (kN)	Location Y (m)
Frame (Rebar)	91.526 × 0.18 ＝ 16.475	1.252
Total	16.475	1.252

2.3 Collision Load

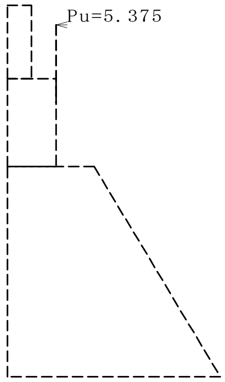
Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

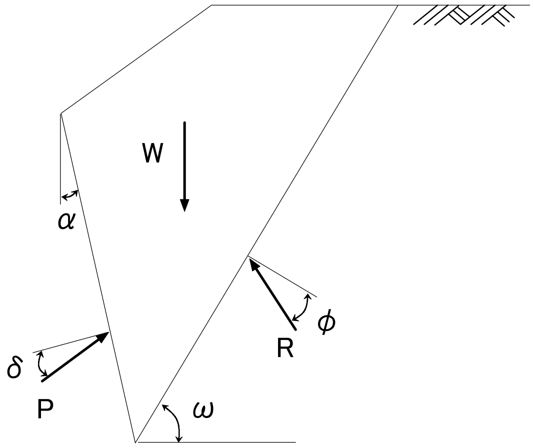
Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 8.000(m)

[1] Collision



p (kN)	Pu (kN/m)	Location Y (m)
43.000	5.375	3.600

2.4 Earth Pressure・Water Pressure



[1] Ordinary Condition

Earth Pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Toe)	xp = 2.180 m
	yp = 0.000 m
Height of Assumed Rear Surface	H = 2.800 m
Angle of Earth Pressure Active Surface to the Vertical	α = 30.964 °
Unit Weight of Soil at Rear Side	γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ = 30.000 °

Wall Friction Angle $\delta = 2/3\varphi = 20.000°$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw = 0.000 m

Slip Angle ω(°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	81.016	0.000	35.330	116.346	68.019
65.00	79.420	0.000	34.634	114.054	68.043
66.00	77.848	0.000	33.949	111.797	68.019

Earth pressure becomes maximum when:

ω = 65.00° P = 68.043 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{114.054 \times \sin(65.00° - 30.00°)}{\cos(65.00° - 30.00° - 30.964° - 20.000°)}$$

$= 68.043$ kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 68.043 \times \cos(30.964° + 20.000°) = 42.854$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 68.043 \times \sin(30.964° + 20.000°) = 52.852$ kN

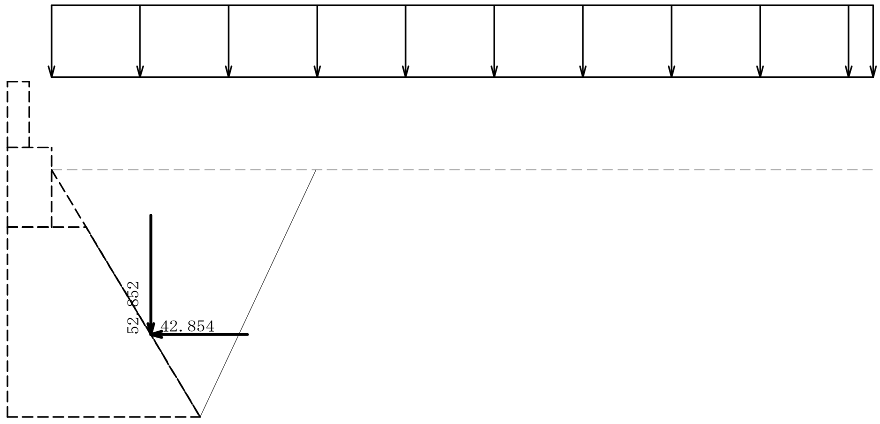
Location

$$Ho = \frac{H}{3} = \frac{2.800}{3} = 0.933 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 2.180 - 0.933 \times \tan 30.964° = 1.620$ m

$y = yp + Ho = 0.000 + 0.933 = 0.933$ m

・ ・Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe) xp = 2.180 m
yp = 0.000 m
Height of Assumed Rear Surface H = 2.800 m
Angle to the Vertical of Earth Pressure Surface α = 30.964 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °
Wall Friction Angle δ = 1／2φ = 15.000 °
Composite angle during earthquake θ = $\tan^{-1}$kh = $\tan^{-1}$0.18 = 10.204°
Rate of Change of Slip Angle ωi = 10.00 °～80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw = 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
50.00	107.185	0.000	0.000	107.185	60.940
51.00	105.001	0.000	0.000	105.001	60.970
52.00	102.878	0.000	0.000	102.878	60.963

Earth pressure becomes maximum when:
ω = 51.00° P = 60.970 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{105.001/\cos 10.204^\circ \times \sin(51.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(51.00^\circ - 30.00^\circ - 30.964^\circ - 15.000^\circ)}$$

= 60.970 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

Ph = P・cos(α+δ) = 60.970×cos(30.964°+15.000°) = 42.381 kN

Vertical Component

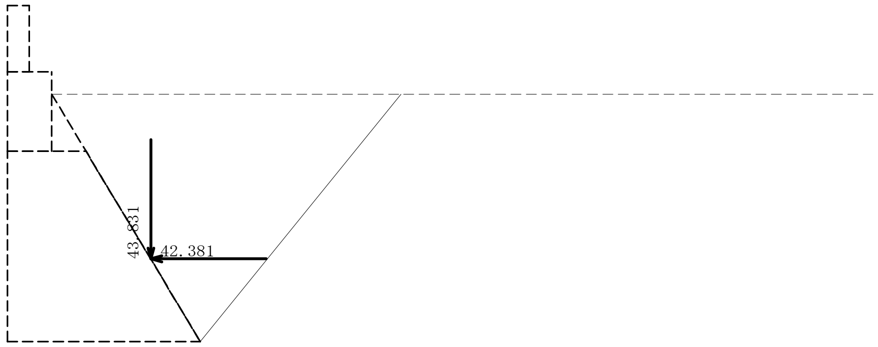
Pv = P・sin(α+δ) = 60.970×sin(30.964°+15.000°) = 43.831 kN

Location

$$Ho = \frac{H}{3} = \frac{2.800}{3} = 0.933 \text{ m}$$

x = xp－Ho・tanα = 2.180－0.933×tan30.964°= 1.620 m
y = yp＋Ho = 0.000＋0.933 = 0.933 m

- Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe) xp ＝ 2.180 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 2.800 m
Angle of Earth Pressure Active Surface to the Vertical α ＝ 30.964 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	81.016	0.000	0.000	81.016	47.364
65.00	79.420	0.000	0.000	79.420	47.381
66.00	77.848	0.000	0.000	77.848	47.364

Earth pressure becomes maximum when:
ω ＝ 65.00° P ＝ 47.381 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{79.420 \times \sin(65.00^\circ - 30.00^\circ)}{\cos(65.00^\circ - 30.00^\circ - 30.964^\circ - 20.000^\circ)}$$

$$= 47.381 \text{ kN}$$

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$$Ph = P \cdot \cos(\alpha + \delta) = 47.381 \times \cos(30.964° + 20.000°) = 29.841 \text{ kN}$$

Vertical Component

$$Pv = P \cdot \sin(\alpha + \delta) = 47.381 \times \sin(30.964° + 20.000°) = 36.803 \text{ kN}$$

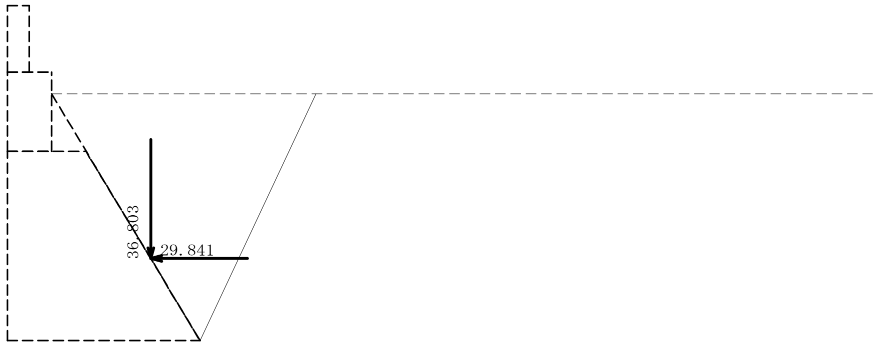
Location

$$Ho = \frac{H}{3} = \frac{2.800}{3} = 0.933 \text{ m}$$

$$x = xp - Ho \cdot \tan\alpha = 2.180 - 0.933 \times \tan 30.964° = 1.620 \text{ m}$$

$$y = yp + Ho = 0.000 + 0.933 = 0.933 \text{ m}$$

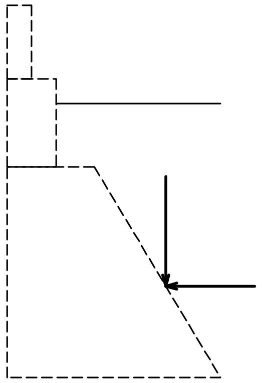
- Earth Pressure Diagram



2.5 Total Acting Force

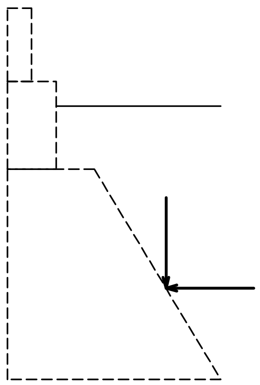
(1) Total Acting Force at the Frame's Front Side

[1] Ordinary Condition



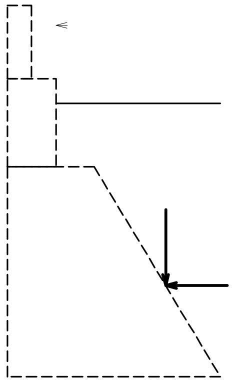
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	91.526	0.000	0.710	0.000	65.018	0.000
Earth Pressure	52.852	42.854	1.620	0.933	85.620	39.983
Total	144.378	42.854	―――	―――	150.639	39.983

[2] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	91.526	16.475	0.710	1.252	65.018	20.623
Earth Pressure	43.831	42.381	1.620	0.933	71.006	39.541
Total	135.357	58.856	――	――	136.025	60.164

[3] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi}= N_i \cdot X_i$	$M_{yi}= H_i \cdot Y_i$
Self Weight	91.526	0.000	0.710	0.000	65.018	0.000
Earth Pressure	36.803	29.841	1.620	0.933	59.621	27.842
Collision Load	0.000	5.375	0.000	3.600	0.000	19.350
Total	128.329	35.216	――	――	124.639	47.192

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary	144.378	42.854	110.656
Earthquake	135.357	58.856	75.860
Collision	128.329	35.216	77.448

(2) Total Acting Force at the Frame's Center

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,
Frame Earth Pressure Width : $B_j = 2.180$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary	144.378	42.854	46.716
Earthquake	135.357	58.856	71.678
Collision	128.329	35.216	62.431

■ Per 8m Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary	1155.022	342.832	373.728
Earthquake	1082.854	470.845	573.427
Collision	1026.630	281.728	499.446

2.6 Stability Analysis Result

2.6.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B = 2.180

$$e_a = B / n$$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary	150.639	39.983	144.378	0.766	0.324	≦	0.363	○
Earthquake	136.025	60.164	135.357	0.560	0.530	≦	0.727	○
Collision	124.639	47.192	128.329	0.604	0.486	≦	0.727	○

2.6.2 Stability in Sliding

$$F_s = \frac{\Sigma V \cdot \mu + C_B \cdot B'}{\Sigma H}$$

Where,

ΣV : Total vertical load at the bottom surface of bottom slab (kN)

ΣH : Total horizontal load at the bottom surface of bottom slab (kN)

μ : Coefficient of friction between the bottom slab and the ground, $\mu = 0.600$

C_B : Adhesive force between the bottom slab and the ground (kN/m^2), $C_B = 0.000$

B' : Effective width of load (m), $B' = B - 2e$

B : Width of bottom slab (m), $B = 2.1800$

e : Eccentricity (m)

Load Condition (Water Level)	Eccentricity, e (m)	Load Effective Width B'(m)
Ordinary	0.324	1.532
Earthquake	0.530	1.120
Collision	0.486	1.208

Load Condition (Water Level)	Total V. Load ΣV(kN)	Total H. Load ΣH(kN)	Factor of Safety F_s		Req'd. F_s F_{sa}	Judgement
Ordinary	144.378	42.854	2.021	≧	1.500	○
Earthquake	135.357	58.856	1.380	≧	1.200	○
Collision	128.329	35.216	2.186	≧	1.200	○

2.6.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

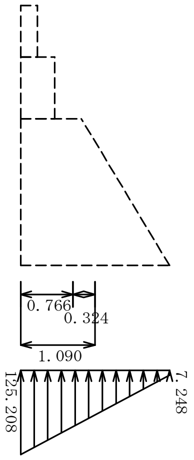
Where,

ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)

B : Bottom plate width (m), $B = 2.180$

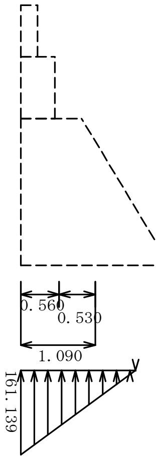
e : Eccentricity (m)

[1] Ordinary Condition



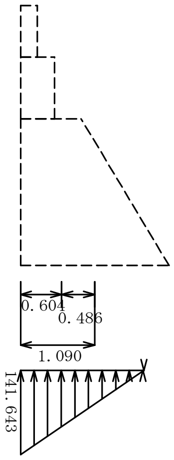
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
2.180	Trapezoid	7.248	125.208 ≧	—	○

[2] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.680	Triangle	0.000	161.139 ≧	—	

[3] Collision



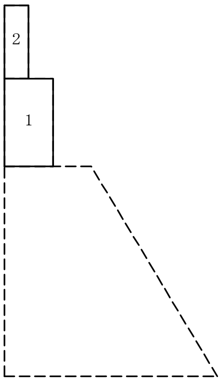
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.812	Triangle	0.000	141.643 ≧	—	

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Self-Weight・Center of Gravity

Classification	Width x Height x Depth x Unit Weight	Weight Wi (kN)	Center of Gravity (m)		Wi・Xi	Wi・Yi	Remarks
			Xi	Yi			
1	0.500× 0.900× 1.000×24.500	11.025	0.25	0.45	2.756	4.961	Parapet①
2	0.250× 0.750× 1.000×24.500	4.594	0.125	1.275	0.574	5.857	Parapet②
Σ		15.619	—	—	3.330	10.818	

Center of Gravity XG = Σ（Wi・Xi）／ΣVi = 3.330／ 15.619 = 0.213 (m)

YG = Σ（Wi・Yi）／ΣVi = 10.818／ 15.619 = 0.693 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Earthquake

Location	H = W ・ kh (kN)	Location, Y (m)
Frame (Rebar)	15.619 × 0.180 = 2.811	0.693

3.1.3 Collision Load

Consider collision load for rigid protection fence.

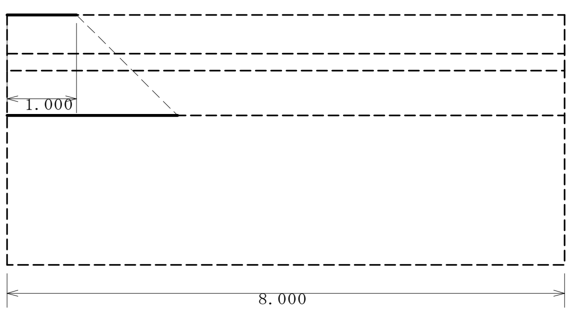
Horizontal Force

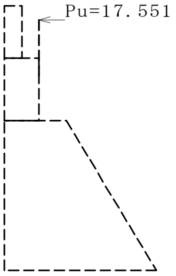
$$Pu = \frac{p}{\lambda + h + y}$$

Where,

Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
h : Vertical distance from the tip (m)
y : Distance from the tip of section being checked (m), y = 0.650(m)

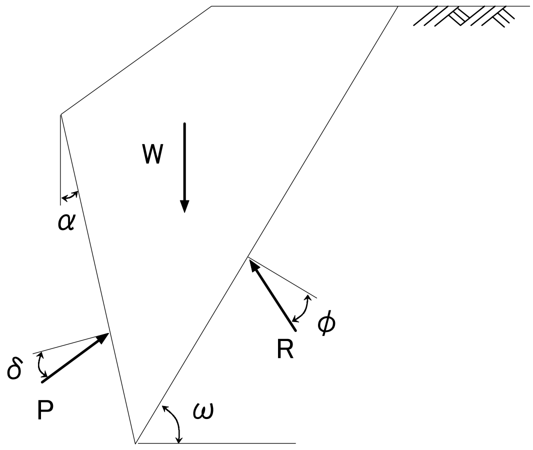
[1] Collision





p (kN)	λ (m)	h (m)	Pu (kN/m)	Location Y (m)
43.000	1.000	0.800	17.551	1.450

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) xp ＝ 0.640 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 0.650 m
Angle of Earth Pressure Active Surface to the Vertical α ＝ 30.964 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	4.366	0.000	8.202	12.568	7.348
65.00	4.280	0.000	8.040	12.320	7.350
66.00	4.195	0.000	7.881	12.076	7.347

Earth pressure becomes maximum when:
ω ＝ 65.00° P ＝ 7.350 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{12.320 \times \sin(65.00° - 30.00°)}{\cos(65.00° - 30.00° - 30.964° - 20.000°)}$$

＝ 7.350 kN

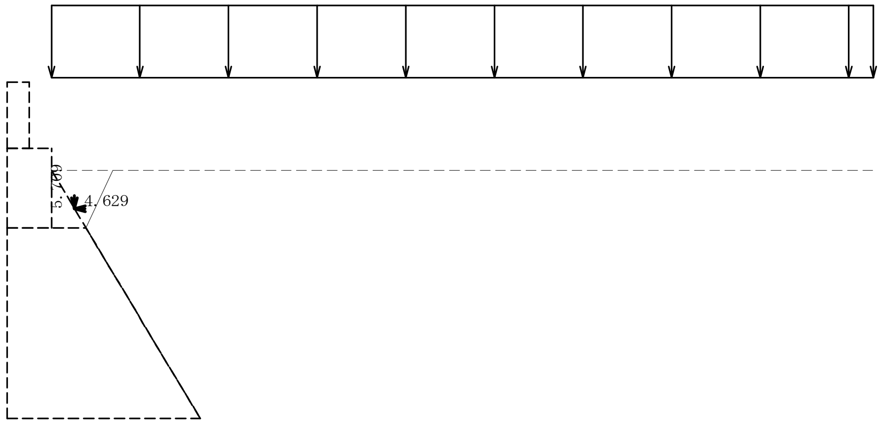
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
$Ph = P \cdot \cos(\alpha + \delta) = 7.350 \times \cos(30.964° + 20.000°) = 4.629$ kN
Vertical Component
$Pv = P \cdot \sin(\alpha + \delta) = 7.350 \times \sin(30.964° + 20.000°) = 5.709$ kN
Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.217 \times \tan 30.964° - 0.640 = -0.510$ m
$y = yp + Ho = 0.000 + 0.217 = 0.217$ m

・Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.640$ m
$yp = 0.000$ m
Height of Assumed Rear Surface $H = 0.650$ m
Angle to the Vertical of Earth Pressure Surface $\alpha = 30.964$ °
Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3
Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °
Wall Friction Angle $\delta = 1／2\varphi = 15.000$ °
Composite angle during earthquake $\theta = \tan^{-1}kh = \tan^{-1}0.18 = 10.204°$
Rate of Change of Slip Angle $\omega i = 10.00$ °～80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, $hw = 0.000$ m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
50.00	5.776	0.000	0.000	5.776	3.284
51.00	5.659	0.000	0.000	5.659	3.286
52.00	5.544	0.000	0.000	5.544	3.285

Earth pressure becomes maximum when:
$\omega = 51.00°$ $P = 3.286$ kN

Earth Pressure, P

$$P = \frac{W／\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.659／\cos 10.204° \times \sin(51.00° - 30.00° + 10.204°)}{\cos(51.00° - 30.00° - 30.964° - 15.000°)}$$

$= 3.286$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 3.286 \times \cos(30.964° + 15.000°) = 2.284$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 3.286 \times \sin(30.964° + 15.000°) = 2.362$ kN

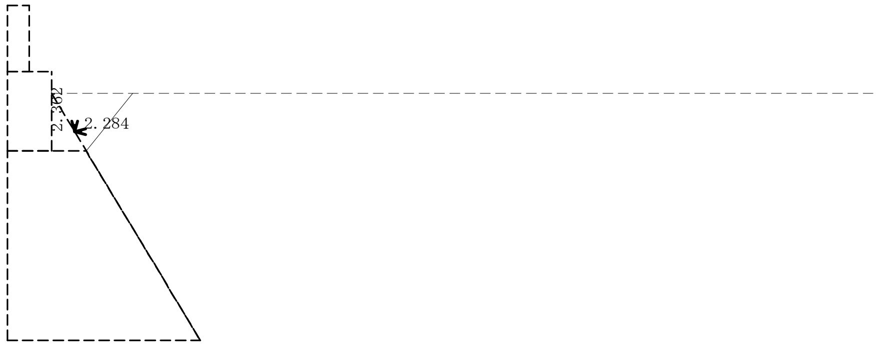
Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.217 \times \tan 30.964° - 0.640 = -0.510$ m

$y = yp + Ho = 0.000 + 0.217 = 0.217$ m

- Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL)	xp = 0.640 m
	yp = 0.000 m
Height of Assumed Rear Surface	H = 0.650 m
Angle of Earth Pressure Active Surface to the Vertical	α = 30.964 °
Unit Weight of Soil at Rear Side	γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ = 30.000 °
Wall Friction Angle	δ = 2／3φ = 20.000 °
Rate of Change of Slip Angle	ωi = 10.00 °～80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw = 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	4.366	0.000	0.000	4.366	2.552
65.00	4.280	0.000	0.000	4.280	2.553
66.00	4.195	0.000	0.000	4.195	2.552

Earth pressure becomes maximum when:

ω = 65.00° P = 2.553 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{4.280 \times \sin(65.00^\circ - 30.00^\circ)}{\cos(65.00^\circ - 30.00^\circ - 30.964^\circ - 20.000^\circ)}$$

$= 2.553$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 2.553 \times \cos(30.964^\circ + 20.000^\circ) = 1.608$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 2.553 \times \sin(30.964^\circ + 20.000^\circ) = 1.983$ kN

Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

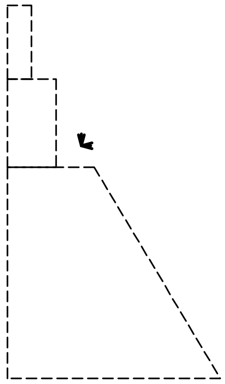
$x = Ho \cdot \tan\alpha - xp = 0.217 \times \tan 30.964^\circ - 0.640 = -0.510$ m

$y = yp + Ho = 0.000 + 0.217 = 0.217$ m

- Earth Pressure Diagram

3.1.5 Total Sectional Force

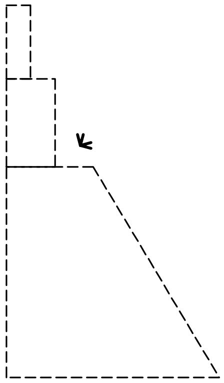
[1] Ordinary Condition



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	0.000	0.037	0.000	0.575
Earth Pressure	5.709	4.629	-0.510	0.217	-1.907
Total	21.328	4.629	———	———	-1.332

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

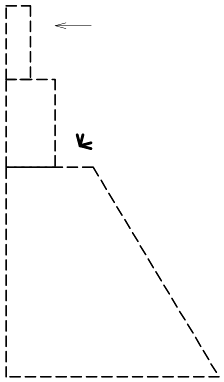
[2] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	2.811	0.037	0.693	2.522
Earth Pressure	2.362	2.284	-0.510	0.217	-0.709
Total	17.981	5.095	———	———	1.813

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[3] Collision

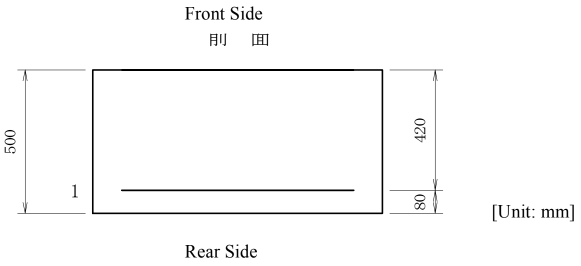


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	0.000	0.037	0.000	0.575
Earth Pressure	1.983	1.608	-0.510	0.217	-0.662
Collision Load	0.000	17.551	0.000	1.450	25.449
Total	17.602	19.159	———	———	25.361

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	8.00	D16	1.986	4.000	7.944
	2	—	–	—	—	—

Required rebar amount at tension side: 1.759 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 41666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23\sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 500000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 500.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary	7.944	2.265	≦	81.514	21.328	5.000	○
Earthquake	7.944	3.082	≦	81.235	17.981	5.000	○
Collision	7.944	43.114	≦	81.203	17.602	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1（$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$y2 = x - y1$

$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$

Compressive force at the compression side rebars（$d_i \leqq x$）

$\varepsilon_i = A \cdot (x - d_i)$

$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$

$S_{ci} = \sigma si \cdot Asi$

$S1 = \Sigma S_{ci}$

Tensile force at the tension side rebars（$d_j > x$）

$\varepsilon_j = A \cdot (d_j - x)$

$\sigma_{sj} = \varepsilon_j \cdot E_s$

$S_{tj} = \sigma_{sj} \cdot A_{sj}$

$S2 = \Sigma S_{tj}$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 80.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	N (kN)	A_s (cm^2)　　A_{sb} (cm^2)	Judgement
Ordinary	21.328	7.944 ≦ 25.033	○
Earthquake	17.981	7.944 ≦ 134.149	○
Collision	17.602	7.944 ≦ 134.160	○

(4) Check of Bending Stress
（Reference）
Calculation of neutral axis
Find x from:

$$x^3 - 3\cdot\left(\frac{h}{2}-e\right)\cdot x^2 - \frac{6n}{b}\left\{As\cdot\left(\frac{h}{2}-d-e\right)\right\}\cdot x + \frac{6n}{b}\left\{As\cdot d\cdot\left(\frac{h}{2}-d-e\right)\right\} = 0.0$$

よりxを求める。

Stress Calculation

$$\sigma_c = \frac{N}{\dfrac{b\cdot x}{2} - n\cdot As\cdot\dfrac{d-x}{x}}$$

$$\sigma_s = n\cdot\sigma_c\cdot\frac{d-x}{x}$$

Where,
x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h = 500.000
b : Member cross-section width (mm), b = 1000.000
d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n = 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
N : Axial Force (N)

Load Condition (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)			Tensile Stress (N/mm^2)			Judgement
				Calculated		Allowable	Calculated		Allowable	
Ordinary	-1.332	21.328	60.504	0.070	≦	8.000	-0.914	≧	-200.000	○
Earthquake	1.813	17.981	44.505	0.081	≦	12.000	-0.068	≧	-300.000	○
Collision	25.361	17.602	10.015	1.465	≦	12.000	70.158	≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,

τ_m : Concrete average shear stress (N/mm^2)

S_h : Shear force (N)

d : Sectional member's effective height (mm)

b : Sectional member's effective width (mm)

τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,

τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)

Ce : Correction factor for the sectional member's effective height

Cpt : Correction factor for the tensile main reinforcement ratio, Pt

CN : Correction factor from axial compression force

M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)				Correction Factor			Judge ment
			Calc. τ		Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary	4.629	8.000	0.058	≦	0.963	1.700	1.40	1.50	2.00	○
Earthquake	5.095	42.000	0.012	≦	0.748	2.550	1.33	0.88	1.83	○
Collision	19.159	42.000	0.046	≦	0.427	2.550	1.33	0.88	1.06	○

Gravity Wall

(H=2100)

1 Design Conditions

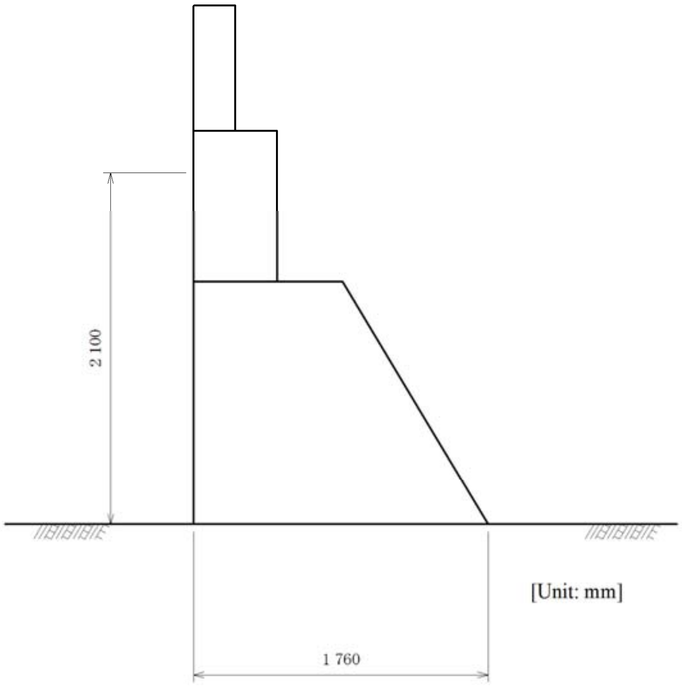
1.1 Applicable Standard

Japan Road Association
Road Earthworks Retaining Wall Construction Guidelines (July 2012 Ed.)

1.2 Structural Figure

『Arbitrary Form – A (Direct Foundation)』

1.3 Configuration



Block Length, B ＝ 8000 (mm)

1.4 Ground Conditions

Seismic Scale:	Level 1
Area Classification:	A
Ground Type:	Type 3

1.5 Materials

【Concrete】 RC Vertical Wall: σck ＝ 24 (N/mm^2)

【Rebar】 Grade: SD345

【Soil Type】 Backfill: Sandy soil
Embankment: Sandy soil
Bearing/Supporting ground: Sand

【Angle of Internal Friction】 Soil at Rear Side: 30°

【Unit Weight】

(kN/m^3)

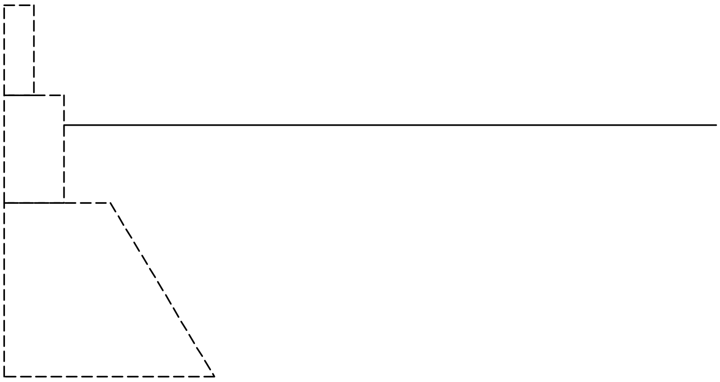
Framework	Vertical Wall	23.000	
	Parapet ①	24.500	
	Parapet ②	24.500	
Water	For Frame Buoyancy Calculation	10.000	
	For Soil Buoyancy Calculation	9.000	
Soil		Damp Weight	Saturated Weight
Rear Side		19.000	20.000
Front Side		19.000	20.000

【Design Horizontal Seismic Coefficient】

Frame: Kh = 0.18
Soil (Front side): Kh = 0.18
Soil (Rear side): Kh = 0.18

1.6 Soil

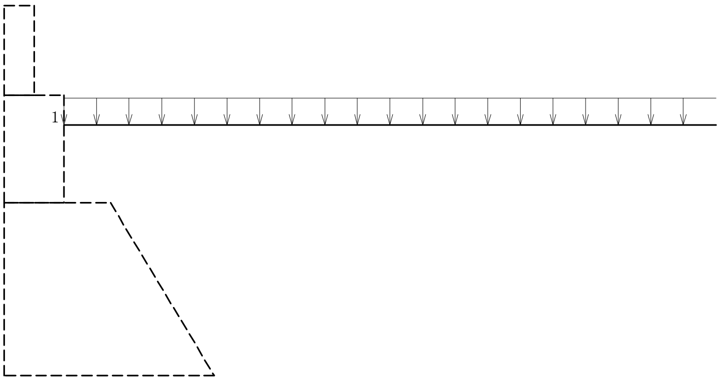
(1) Figure of Soil at Rear Side



Difference in elevation of the tip of retaining wall and ground surface (m)	0.000
Height not considering earth pressure, Hr (m)	0.000

1.7 Loading

[1] Ordinary Condition

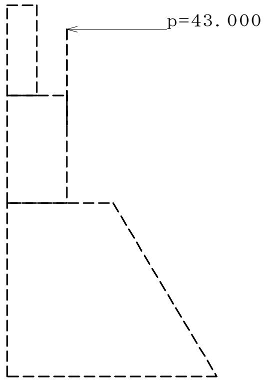


Number	Load Location (m)	Load Width (m)	Load (kN/m^2)		Effectiveness		
			Start Point	End Point	Stability	Vertical Wall	Bottom Slab
1	0.000	∞	11.600	11.600	○	○	×

1.8 Collision Load

Fence Type: Rigid

[1] Collision



Classification: SC, Load: Straight Wall Type
Horizontal Force

Load Location, h (m)	Load Range, λ (m)	Collision Load, P (kN)
0.800	1.000	43.000

1.9 Arbitrary Load

Not considered.

1.10 Earth Pressure

- Earth Pressure Analysis Method: Trial Wedge Method

- Earth Pressure Wall Friction Angle on its Surface (degrees)

Load Condition	Active Earth Pressure			Passive Earth Pressure
	Stability Analysis	Sectional Analysis	Cut	
Ordinary	20.000	20.000	———	———
Earthquake	15.000	15.000	———	———

- In the stability analysis, the assumed rear side is the back of the vertical wall.
- In the stability analysis, the angle formed by the earth pressure's active surface with the vertical is 30.964 degrees.
- In the vertical wall design, the angle formed by the earth pressure's active surface with the vertical is 30.964 degrees.

・Adhesive Force (kN/m^2)

Loading Condition	Sliding Surface	Adhesion Height	Passive Earth Pressure
Ordinary	0.000	0.000	―――
Earthquake	0.000	0.000	―――

・Earth Pressure with Direction Other Than the Direction of Earthquake

	Treatment	Effectiveness Ratio
Stability Analysis	Ordinary Earth Pressure	0.500
Vertical Wall Design		0.500

1.11 Load Combination

No	Load Condition	Comment
1	Ordinary	Ordinary Condition
2	Earthquake	kh=0.18
3	Collision	SC Class

No	Load Condition	Treatment During Earthquake				
		Seismic Scale	Inertial Force Direction		Soil Inertial Force	
			Horizontal	Vertical	Front	Rear
2	Earthquake	Level 1	←Towards left	――	Neglected	Considered

	Load Condition	1	2	3
Soil	Soil 1			
Loading	Surcharge	○		
Collision・Wind	Collision Load			○
Active Earth Pressure	Not considered			
	Ordinary Earth Pressure	○		○
	Seismic Earth Pressure		○	

Check Items		1	2	3
Allowable Stress Method		Stability • Section	Stability • Section	Stability • Section
Limit State Design Method	Checking Efficiency	――	――	――
	Rigid Body Stability	――	――	――
	Cross-Section Disruption	――	――	――

1.12 Foundation

1.12.1 Allowable Shear Resistance Calculation Data

Bottom Slab Inspection Width	Effective Load Width
Adhesive Force between Foundation Base and Ground, CB (kN/m^2)	0.000
Coefficient of Friction between Foundation Base and Ground, tanφ_B	0.600

1.13 Allowable Values for Stability Analysis and Allowable Stresses of Members

1.13.1 Allowable Values for Stability Analysis

Load Condition	Allowable Eccentricity e_B ／ B (m)	Sliding Factor of Safety	Allowable Support Pressure (kN/m^2)
Ordinary	1/6	1.500	—
Earthquake	1/3	1.200	—
Collision	1/3	1.200	—

Where,

B : Foundation Width (m)
e_B : Load Eccentricity (m), wherein: $e_B = M_B / V$
M_B : Moment in the base of foundation (kN.m)
V : Vertical load on the base of foundation (kN)

1.13.2 Allowable Stress of Members

(1) RC Concrete

1) Vertical Wall (General Members)

(N/mm^2)

Load Condition	Multiplication Factor	Compressive Stress of Concrete σ_{ca}	Rebar Tensile Stress σ_{sa}	Shear Stress τ_{a1}	Shear Stress τ_{a2}
Ordinary	1.00	8.000	180.000	0.230	1.700
Earthquake	1.50	12.000	300.000	0.350	2.550
Collision	1.50	12.000	300.000	0.345	2.550

Where,

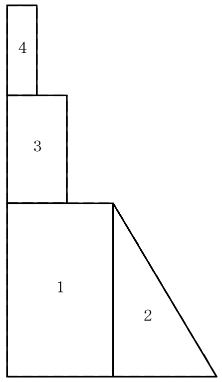
τ_{a1} : Shear stress when shear force acts on concrete
τ_{a2} : Shear stress when shared with diagonal tension bars

2 **Stability Analysis**

2.1 Block Data Without Considering the Water Level

(1) Framework

1) Block Division



2) Self-Weight・Center of Gravity

Classification	Width x Height x Depth x Unit Weight	Weight Wi(kN)	Location of Center of Gravity (m)		Wi・Xi	Wi・Yi	Remarks
			Xi	Yi			
1	0.890× 1.450× 1.000×23.000	29.681	0.445	0.725	13.208	21.519	Vertical Wall
2	1/2× 0.870× 1.450× 1.000×23.000	14.508	1.180	0.483	17.120	7.012	Vertical Wall
3	0.500× 0.900× 1.000×24.500	11.025	0.250	1.900	2.756	20.947	Parapet ①
4	0.250× 0.750× 1.000×24.500	4.594	0.125	2.725	0.574	12.518	Parapet ②
Σ		59.809	—	—	33.659	61.997	

Center of Gravity XG ＝ Σ（Wi・Xi）／ΣWi ＝ 33.659／ 59.809 ＝ 0.563 (m)
YG ＝ Σ（Wi・Yi）／ΣWi ＝ 61.997／ 59.809 ＝ 1.037 (m)

2.2 Frame Weight, Soil Weight, Arbitrary Load, Buoyancy Vertical Force, Horizontal Force

(1) Force Due to Self-Weight

[1] Earthquake

Location	Horizontal Force H ＝ W ・ kh (kN)	Location Y (m)
Frame (Rebar)	59.809 × 0.18 ＝ 10.766	1.037
Total	10.766	1.037

2.3 Collision Load

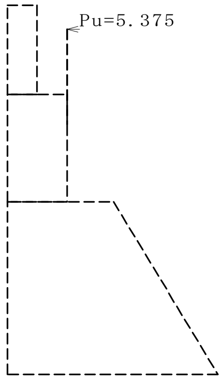
Horizontal Force

$$Pu = \frac{p}{L}$$

Where,

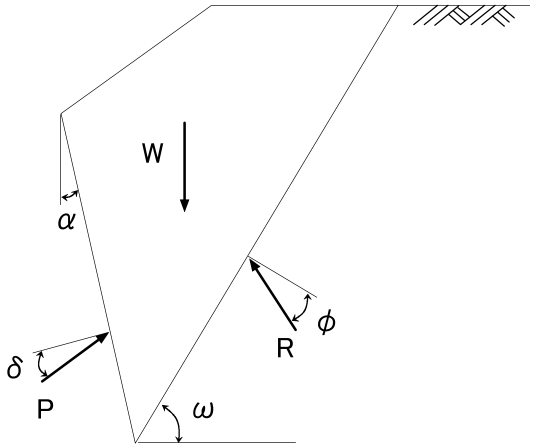
Pu : Collision load per unit width (kN/m)
p : Collision load per block (kN)
L : Block length (m), L = 10.000(m)

[1] Collision



p (kN)	Pu (kN/m)	Location Y (m)
43.000	5.375	2.900

2.4 Earth Pressure・Water Pressure



[1] Ordinary Condition

Earth Pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Toe) xp = 1.760 m
yp = 0.000 m
Height of Assumed Rear Surface H = 2.100 m
Angle of Earth Pressure Active Surface to the Vertical α = 30.964 °
Unit Weight of Soil at Rear Side γs = 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ = 30.000 °

Wall Friction Angle $\delta = 2/3\varphi = 20.000°$

Rate of Change of Slip Angle $\omega i = 10.00° \sim 80.00°$

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level hw = 0.000 m

Slip Angle ω(°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	45.570	0.000	26.497	72.067	42.133
65.00	44.672	0.000	25.975	70.647	42.147
66.00	43.790	0.000	25.462	69.252	42.134

Earth pressure becomes maximum when:

ω = 65.00° P = 42.147 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{70.647 \times \sin(65.00° - 30.00°)}{\cos(65.00° - 30.00° - 30.964° - 20.000°)}$$

= 42.147 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 42.147 \times \cos(30.964° + 20.000°) = 26.545$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 42.147 \times \sin(30.964° + 20.000°) = 32.738$ kN

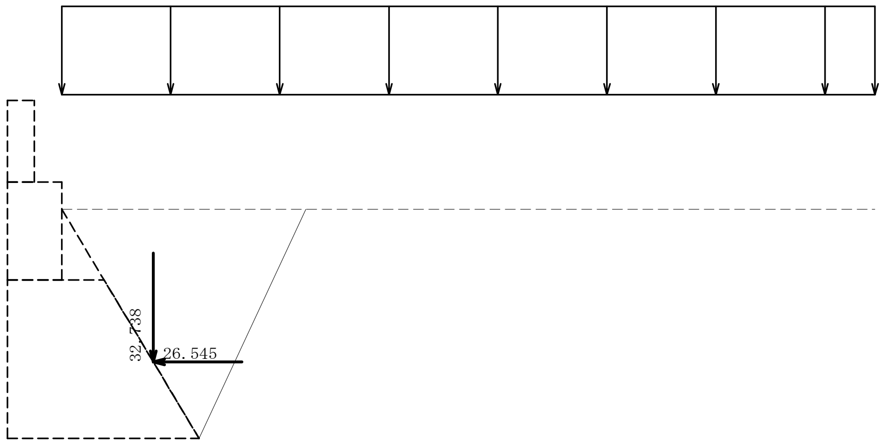
Location

$$Ho = \frac{H}{3} = \frac{2.100}{3} = 0.700 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.760 - 0.700 \times \tan 30.964° = 1.340$ m

$y = yp + Ho = 0.000 + 0.700 = 0.700$ m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.760 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 2.100 m
Angle to the Vertical of Earth Pressure Surface	α ＝ 30.964 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 1／2φ ＝ 15.000 °
Composite angle during earthquake	θ ＝ $\tan^{-1}kh$ ＝ $\tan^{-1}0.18$ ＝ 10.204°
Rate of Change of Slip Angle	ωi ＝ 10.00 °～80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight W(kN)				Earth Pressure, P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
50.00	60.291	0.000	0.000	60.291	34.278
51.00	59.062	0.000	0.000	59.062	34.295
52.00	57.869	0.000	0.000	57.869	34.292

Earth pressure becomes maximum when:
ω ＝ 51.00° P ＝ 34.295 kN

Earth Pressure, P

$$P = \frac{W／\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{59.062／\cos 10.204° \times \sin(51.00° - 30.00° + 10.204°)}{\cos(51.00° - 30.00° - 30.964° - 15.000°)}$$

＝ 34.295 kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 34.295 \times \cos(30.964° + 15.000°) = 23.839$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 34.295 \times \sin(30.964° + 15.000°) = 24.655$ kN

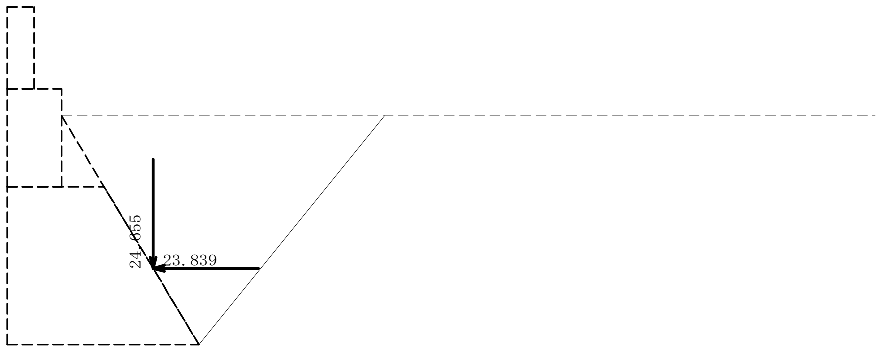
Location

$$Ho = \frac{H}{3} = \frac{2.100}{3} = 0.700 \text{ m}$$

$x = xp - Ho \cdot \tan\alpha = 1.760 - 0.700 \times \tan 30.964° = 1.340$ m

$y = yp + Ho = 0.000 + 0.700 = 0.700$ m

- Earth Pressure Diagram



[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. toe)	xp ＝ 1.760 m
	yp ＝ 0.000 m
Height of Assumed Rear Surface	H ＝ 2.100 m
Angle of Earth Pressure Active Surface to the Vertical	α ＝ 30.964 °
Unit Weight of Soil at Rear Side	γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side	φ ＝ 30.000 °
Wall Friction Angle	δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle	ωi ＝ 10.00 °～80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	45.570	0.000	0.000	45.570	26.642
65.00	44.672	0.000	0.000	44.672	26.651
66.00	43.790	0.000	0.000	43.790	26.643

Earth pressure becomes maximum when:
ω ＝ 65.00° P ＝ 26.651 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{44.672 \times \sin(65.00^\circ - 30.00^\circ)}{\cos(65.00^\circ - 30.00^\circ - 30.964^\circ - 20.000^\circ)}$$

＝ 26.651 kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$$Ph = P \cdot \cos(\alpha + \delta) = 26.651 \times \cos(30.964° + 20.000°) = 16.785\ kN$$

Vertical Component

$$Pv = P \cdot \sin(\alpha + \delta) = 26.651 \times \sin(30.964° + 20.000°) = 20.701\ kN$$

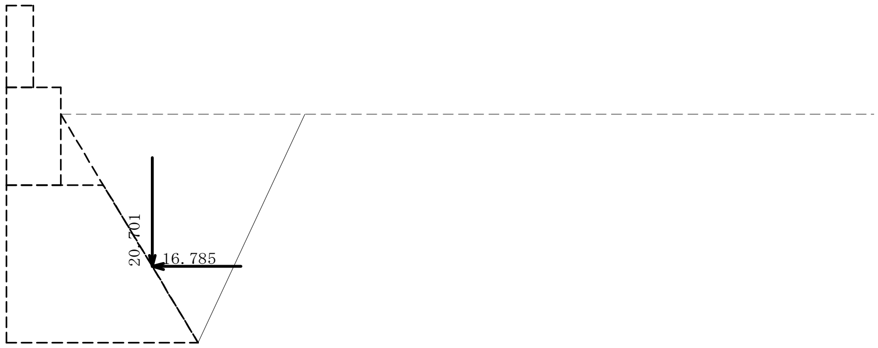
Location

$$Ho = \frac{H}{3} = \frac{2.100}{3} = 0.700\ m$$

$$x = xp - Ho \cdot \tan\alpha = 1.760 - 0.700 \times \tan 30.964° = 1.340\ m$$

$$y = yp + Ho = 0.000 + 0.700 = 0.700\ m$$

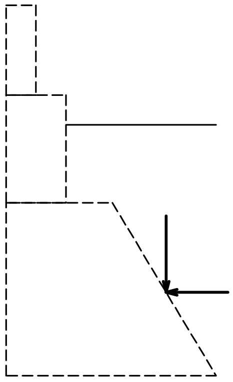
- Earth Pressure Diagram



2.5 Total Acting Force

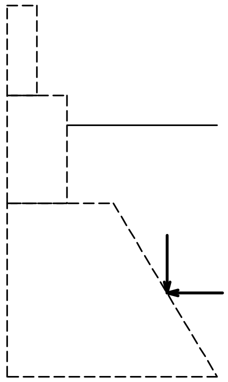
(1) Total Acting Force at the Frame's Front Side

[1] Ordinary Condition



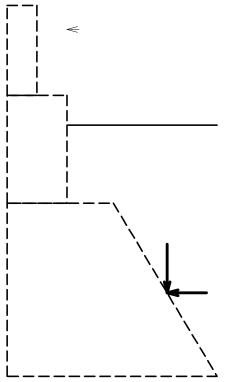
Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	59.809	0.000	0.563	0.000	33.659	0.000
Earth Pressure	32.738	26.545	1.340	0.700	43.869	18.581
Total	92.547	26.545	———	———	77.528	18.581

[2] Earthquake



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	59.809	10.766	0.563	1.037	33.659	11.159
Earth Pressure	24.655	23.839	1.340	0.700	33.038	16.687
Total	84.464	34.605	———	———	66.696	27.847

[3] Collision



Item	Vertical Forces N_i (kN)	Horizontal Forces H_i (kN)	Arm Length		Overturning Moment (kN.m)	
			X_i (m)	Y_i (m)	$M_{xi} = N_i \cdot X_i$	$M_{yi} = H_i \cdot Y_i$
Self Weight	59.809	0.000	0.563	0.000	33.659	0.000
Earth Pressure	20.701	16.785	1.340	0.700	27.739	11.749
Collision Load	0.000	5.375	0.000	2.900	0.000	15.587
Total	80.510	22.160	———	———	61.398	27.337

Load Condition (Water Level)	N_o (kN)	H_o (kN)	M_o (kN.m)
Ordinary	92.547	26.545	58.946
Earthquake	84.464	34.605	38.850
Collision	80.510	22.160	34.061

(2) Total Acting Force at the Frame’s Center

Vertical Force : $N_c = N_o$ (kN)
Horizontal Force : $H_c = H_o$ (kN)
Overturning Moment : $M_c = N_o \cdot B_j / 2.0 - M_o$ (kN.m)

Where,
Frame Earth Pressure Width : $B_j = 1.760$ (m)

■ Per Unit Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary	92.547	26.545	22.495
Earthquake	84.464	34.605	35.478
Collision	80.510	22.160	36.788

■ Per 8m Width

Load Condition (Water Level)	N_c (kN)	H_c (kN)	M_c (kN.m)
Ordinary	740.374	212.360	179.961
Earthquake	675.710	276.837	283.828
Collision	644.078	177.280	294.301

2.6 Stability Analysis Result

2.6.1 Stability in Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Where,

d : Distance from toe to resultant force (m)
ΣMr : Resisting Moment around toe (kN.m)
ΣMt : Overturning Moment around toe (kN.m)
ΣV : Total vertical load on bottom surface of bottom slab (kN)

$$e = \frac{B}{2} - d$$

Where,

e : Eccentricity of resultant force from the center of bottom plate (m)
B : Bottom Plate Width (m), B = 1.760

$e_a = B / n$

Where,

e_a : Allowable Eccentricity (m)
n : Factor of Safety

Load Condition (Water Level)	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Judgement
Ordinary	77.528	18.581	92.547	0.637	0.243	≦	0.293	○
Earthquake	66.696	27.847	84.464	0.460	0.420	≦	0.587	○
Collision	61.398	27.337	80.510	0.423	0.457	≦	0.587	○

2.6.2 Stability in Sliding

$$F_s = \frac{\Sigma V \cdot \mu + C_B \cdot B'}{\Sigma H}$$

Where,

ΣV : Total vertical load at the bottom surface of bottom slab (kN)

ΣH : Total horizontal load at the bottom surface of bottom slab (kN)

μ : Coefficient of friction between the bottom slab and the ground, μ=0.600

C_B : Adhesive force between the bottom slab and the ground (kN/m^2), C_B = 0.000

B' : Effective width of load (m), B' = B −2e

B : Width of bottom slab (m), B = 1.760

e : Eccentricity (m)

Load Condition (Water Level)	Eccentricity, e (m)	Load Effective Width B'(m)
Ordinary	0.243	1.274
Earthquake	0.420	0.920
Collision	0.457	0.846

Load Condition (Water Level)	Total V. Load ΣV(kN)	Total H. Load ΣH(kN)	Factor of Safety F_s		Req'd. F_s F_{sa}	Judgement
Ordinary	92.547	26.545	2.092	≧	1.500	○
Earthquake	84.464	34.605	1.464	≧	1.200	○
Collision	80.510	22.160	2.180	≧	1.200	○

2.6.3 Inspection for Support

1) When the resultant force is acting on the middle third of the width of bottom plate

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2) When the resultant force is acting on the middle two-third of the width of bottom plate

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

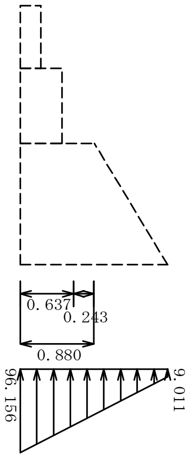
Where,

ΣV : Total vertical load acting on the bottom surface of bottom plate (kN)

B : Bottom plate width (m), B = 1.760

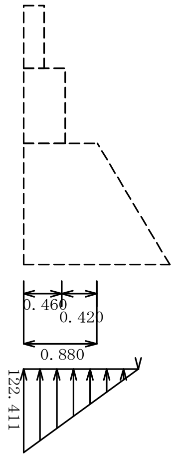
e : Eccentricity (m)

[1] Ordinary Condition



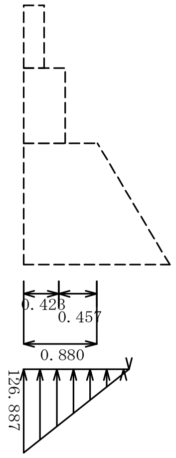
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.760	Trapezoid	9.011	96.156 ≧	—	○

[2] Earthquake



Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.380	Triangle	0.000	122.411 ≧	—	

[3] Collision



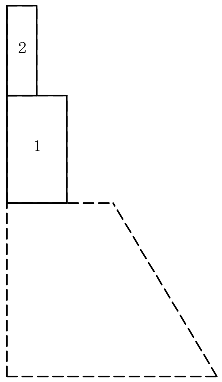
Ground Reaction Width (m)	Ground Reaction Shape	Ground Reaction (kN/m^2)			Judgement
		qmin	qmax	Allow.	
1.269	Triangle	0.000	126.887 ≧	—	

3 Design of Vertical Wall

3.1 Design of Vertical Wall Foundation

3.1.1 Block Data Without Considering the Water Level

(1) Block Division



(2) Self-Weight・Center of Gravity

Classification	Width x Height x Depth x Unit Weight	Weight Wi (kN)	Center of Gravity (m)		Wi・Xi	Wi・Yi	Remarks
			Xi	Yi			
1	0.500× 0.900× 1.000×24.500	11.025	0.25	0.45	2.756	4.961	Parapet①
2	0.250× 0.750× 1.000×24.500	4.594	0.125	1.275	0.574	5.857	Parapet②
Σ		15.619	—	—	3.330	10.818	

Center of Gravity XG ＝ Σ（Wi・Xi）／ΣVi ＝ 3.330／ 15.619 ＝ 0.213 (m)

YG ＝ Σ（Wi・Yi）／ΣVi ＝ 10.818／ 15.619 ＝ 0.693 (m)

3.1.2 Frame Weight, Arbitrary Load

(1) Frame Weight

[1] Earthquake

Location	H ＝ W ・ kh (kN)	Location, Y (m)
Frame (Rebar)	15.619 × 0.180 ＝ 2.811	0.693

3.1.3 Collision Load

Consider collision load for rigid protection fence.

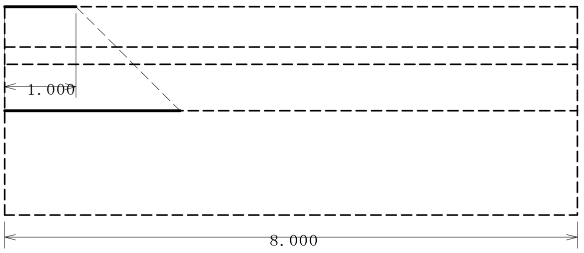
Horizontal Force

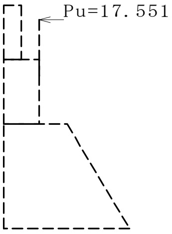
$$Pu = \frac{p}{\lambda + h + y}$$

Where,

Pu : Collision load of the effective width (kN/m)
p : Collision load per block (kN)
λ : Load range (m)
h : Vertical distance from the tip (m)
y : Distance from the tip of section being checked (m), y ＝ 0.650(m)

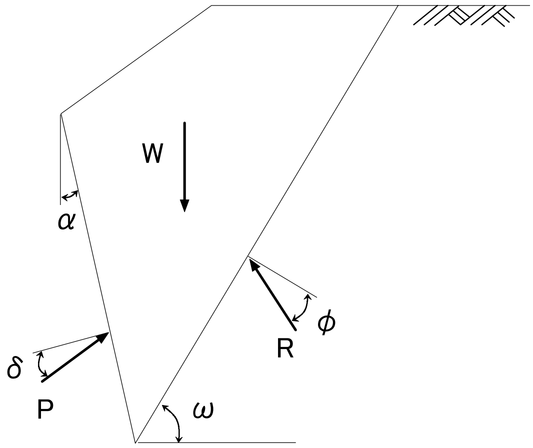
[1] Collision





p (kN)	λ (m)	h (m)	Pu (kN/m)	Location Y (m)
43.000	1.000	0.800	17.551	1.450

3.1.4 Earth Pressure・Water Pressure



[1] Ordinary Condition

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) xp ＝ 0.640 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 0.650 m
Angle of Earth Pressure Active Surface to the Vertical α ＝ 30.964 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 2／3φ ＝ 20.000 °
Rate of Change of Slip Angle ωi ＝ 10.00 °～ 80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)
Water Level hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	4.366	0.000	8.202	12.568	7.348
65.00	4.280	0.000	8.040	12.320	7.350
66.00	4.195	0.000	7.881	12.076	7.347

Earth pressure becomes maximum when:
ω ＝ 65.00° P ＝ 7.350 kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{12.320 \times \sin(65.00^\circ - 30.00^\circ)}{\cos(65.00^\circ - 30.00^\circ - 30.964^\circ - 20.000^\circ)}$$

＝ 7.350 kN

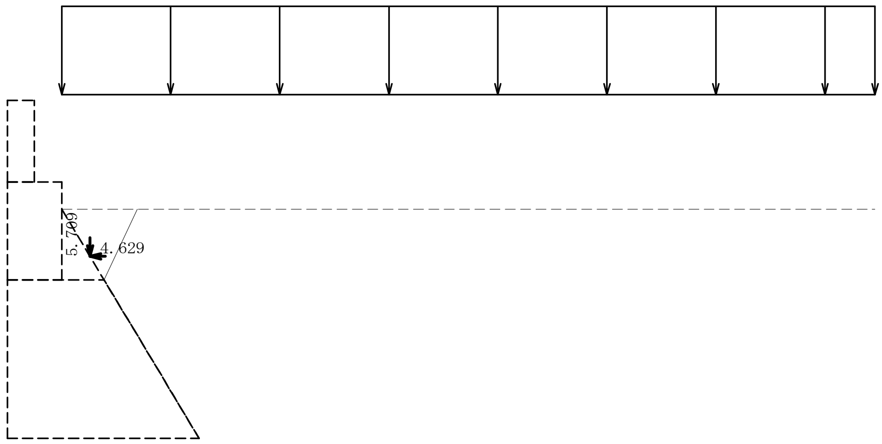
Earth Pressure’s location and horizontal and vertical components:

Horizontal Component
Ph ＝ P・cos(α＋δ) ＝ 7.350×cos(30.964°＋20.000°) ＝ 4.629 kN
Vertical Component
Pv ＝ P・sin(α＋δ) ＝ 7.350×sin(30.964°＋20.000°) ＝ 5.709 kN
Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

x ＝ Ho・tanα－xp ＝ 0.217×tan30.964°－0.640 ＝ -0.510 m
y ＝ yp＋Ho ＝ 0.000＋0.217 ＝ 0.217 m

- Earth Pressure Diagram



[2] Earthquake

During earthquake, earth pressure is obtained by Trial Wedge Method considering inertial forces.

Assumed Rear Surface Location (distance fr. Section CL) xp ＝ 0.640 m
yp ＝ 0.000 m
Height of Assumed Rear Surface H ＝ 0.650 m
Angle to the Vertical of Earth Pressure Surface α ＝ 30.964 °
Unit Weight of Soil at Rear Side γs ＝ 19.000 kN/m^3
Angle of Internal Friction of Soil at Rear Side φ ＝ 30.000 °
Wall Friction Angle δ ＝ 1／2φ ＝ 15.000 °
Composite angle during earthquake θ ＝ $\tan^{-1}$kh ＝ $\tan^{-1}$0.18 ＝ 10.204°
Rate of Change of Slip Angle ωi ＝ 10.00 °～80.00 °

Soil Weight (W) and Earth Pressure (P) with Regards to Slip Angle (ω)
Water Level, hw ＝ 0.000 m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
50.00	5.776	0.000	0.000	5.776	3.284
51.00	5.659	0.000	0.000	5.659	3.286
52.00	5.544	0.000	0.000	5.544	3.285

Earth pressure becomes maximum when:
ω ＝ 51.00° P ＝ 3.286 kN

Earth Pressure, P

$$P = \frac{W/\cos\theta \cdot \sin(\omega - \phi + \theta)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{5.659/\cos 10.204^\circ \times \sin(51.00^\circ - 30.00^\circ + 10.204^\circ)}{\cos(51.00^\circ - 30.00^\circ - 30.964^\circ - 15.000^\circ)}$$

＝ 3.286 kN

Earth Pressure’s location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 3.286 \times \cos(30.964° + 15.000°) = 2.284$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 3.286 \times \sin(30.964° + 15.000°) = 2.362$ kN

Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

$x = Ho \cdot \tan\alpha - xp = 0.217 \times \tan 30.964° - 0.640 = -0.510$ m

$y = yp + Ho = 0.000 + 0.217 = 0.217$ m

- Earth Pressure Diagram

[3] Collision

Earth pressure is obtained by Trial Wedge Method.

Assumed Rear Surface Location (distance fr. Section CL) $xp = 0.640$ m

$yp = 0.000$ m

Height of Assumed Rear Surface $H = 0.650$ m

Angle of Earth Pressure Active Surface to the Vertical $\alpha = 30.964$ °

Unit Weight of Soil at Rear Side $\gamma s = 19.000$ kN/m^3

Angle of Internal Friction of Soil at Rear Side $\varphi = 30.000$ °

Wall Friction Angle $\delta = 2／3\varphi = 20.000$ °

Rate of Change of Slip Angle $\omega i = 10.00$ °～80.00 °

Earth Pressure (P) and Soil Weight (W) with regards to Slip Angle (ω)

Water Level $hw = 0.000$ m

Slip Angle, ω (°)	Soil Weight, W (kN)				Earth Pressure P (kN)
	≧Water Level	≦Water Level	Surcharge	Total	
64.00	4.366	0.000	0.000	4.366	2.552
65.00	4.280	0.000	0.000	4.280	2.553
66.00	4.195	0.000	0.000	4.195	2.552

Earth pressure becomes maximum when:

$\omega = 65.00°$ $P = 2.553$ kN

Earth Pressure, P

$$P = \frac{W \cdot \sin(\omega - \phi)}{\cos(\omega - \phi - \alpha - \delta)}$$

$$= \frac{4.280 \times \sin(65.00^\circ - 30.00^\circ)}{\cos(65.00^\circ - 30.00^\circ - 30.964^\circ - 20.000^\circ)}$$

$= 2.553$ kN

Earth Pressure's location and horizontal and vertical components:

Horizontal Component

$Ph = P \cdot \cos(\alpha + \delta) = 2.553 \times \cos(30.964^\circ + 20.000^\circ) = 1.608$ kN

Vertical Component

$Pv = P \cdot \sin(\alpha + \delta) = 2.553 \times \sin(30.964^\circ + 20.000^\circ) = 1.983$ kN

Location

$$Ho = \frac{H}{3} = \frac{0.650}{3} = 0.217 \text{ m}$$

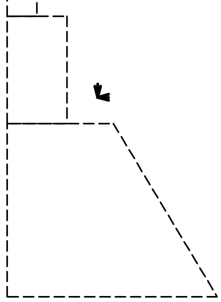
$x = Ho \cdot \tan\alpha - xp = 0.217 \times \tan 30.964^\circ - 0.640 = -0.510$ m

$y = yp + Ho = 0.000 + 0.217 = 0.217$ m

- Earth Pressure Diagram

3.1.5 Total Sectional Force

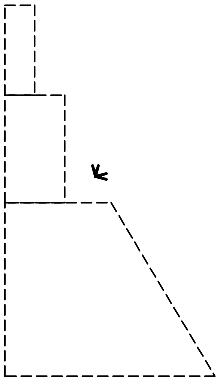
[1] Ordinary Condition



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	0.000	0.037	0.000	0.575
Earth Pressure	5.709	4.629	-0.510	0.217	-1.907
Total	21.328	4.629	────	────	-1.332

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

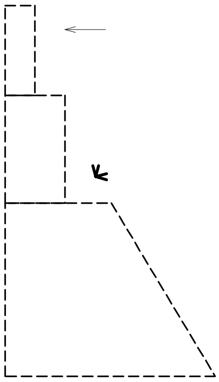
[2] Earthquake



Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	2.811	0.037	0.693	2.522
Earth Pressure	2.362	2.284	-0.510	0.217	-0.709
Total	17.981	5.095	────	────	1.813

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

[3] Collision

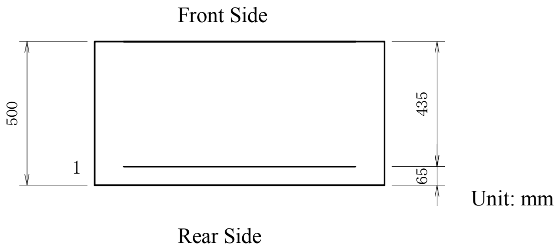


Item	N_i (kN)	H_i (kN)	X_i (m)	Y_i (m)	$M = M_{xi}+M_{yi}$ (kN.m)
Self-weight	15.619	0.000	0.037	0.000	0.575
Earth Pressure	1.983	1.608	-0.510	0.217	-0.662
Collision Load	0.000	17.551	0.000	1.450	25.449
Total	17.602	19.159	────	────	25.361

※ X_i is the hor. distance from the CL of design section (+ towards front side),
Y_i is the vertical distance from the design section

3.1.6 Section Design (Allowable Stress Method)

(1) Rebar Arrangement



One Layer Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm^2/bar)	Qty.	Total Rebar Area (cm^2)
Front Side	1'	—	–	—	—	—
	2'	—	–	—	—	—
Rear Side	1	6.5	D13	1.267	4.000	5.068
	2	—	–	—	—	—

Required rebar amount at tension side: 1.696 (cm^2)

(2) Check of Minimun Rebar Amount

$$M_c = Z_c \cdot \left(\sigma_{bt} + \frac{N}{A_c} \right)$$

Where,

M_c : Crack bending moment (N.mm)

Z_c : Section modulus of concrete member (mm^3), $Z_c = b \cdot h^2 / 6 = 41666.7 \times 10^3$

σ_{bt} : Concrete bending tensile strength (N/mm^2), $\sigma_{bt} = 0.23 \sigma_{ck}^{2/3}$

σ_{ck} : Concrete design standard strength (N/mm^2), $\sigma_{ck} = 24.00$

N : Axial force (N)

A_c : Cross-sectional area of concrete (mm^2), $A_c = b \cdot h = 500000.000$

b : Member cross-section width (mm)

h : Member cross-section height (mm), $h = 500.000$

Load Conditon (Water Level)	Actual Rebar Amount (cm^2)	M (kN.m)	×1.7	M_c (kN.m)	N (kN)	Min. Rebar Amount (cm^2)	Judgement
Ordinary	5.068	2.265	≦	81.514	21.328	5.000	○
Earthquake	5.068	3.082	≦	81.235	17.981	5.000	○
Collision	5.068	43.114	≦	81.203	17.602	5.000	○

If the 1.7 x actual bending moment is:

- less than Mc, Min. rebar amount is 5.0
- greater than Mc, Min. rebar amount is the larger of 5.0 or the rebar amount at the condition of crack bending moment = end bending moment.

(3) Check of Maximum Rebar Amount

Check if the actual rebar amount, As is less than or equal to the balanced rebar amount, Asb

Concrete Ultimate Strain

$\varepsilon_{cu} = 0.0035$

Rebar Yield Strain

$$\varepsilon_{sy} = \frac{\sigma_{sy}}{E_s}$$

Neutral Axis Location

$$x = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}} \cdot d$$

Strain at y-distance from the neutral axis

$$A = \frac{\varepsilon_{cu} + \varepsilon_{sy}}{d}$$

$$\varepsilon(y) = A \cdot y$$

Concrete compressive force at section y1（$0 \leqq \varepsilon(y) \leqq 0.002$）

$$y1 = \frac{0.002}{\varepsilon_{cu}} \cdot x$$

$$C1 = b \cdot \frac{0.85 \cdot \sigma_{ck} \cdot A}{0.002} \cdot \left(-\frac{A}{0.006} \cdot y1^3 + y1^2 \right)$$

Concrete compressive force at section y2（$0.002 \leqq \varepsilon(y) \leqq \varepsilon cu$）

$$y2 = x - y1$$

$$C2 = b \cdot 0.85 \cdot \sigma_{ck} \cdot y2$$

Compressive force at the compression side rebars（$d_i \leqq x$）

$$\varepsilon_i = A \cdot (x - d_i)$$

$$\sigma_{si} = \varepsilon_i \cdot E_s \ (\leqq \sigma_{sy})$$

$$S_{ci} = \sigma si \cdot Asi$$

$$S1 = \Sigma S_{ci}$$

Tensile force at the tension side rebars（$d_j > x$）

$$\varepsilon_j = A \cdot (d_j - x)$$

$$\sigma_{sj} = \varepsilon_j \cdot E_s$$

$$S_{tj} = \sigma_{sj} \cdot A_{sj}$$

$$S2 = \Sigma S_{tj}$$

From the axial force balance

$$N = C1 + C2 + S1 - \alpha \cdot S2$$

$$\alpha = \frac{-N + C1 + C2 + S1}{S2}$$

Balanced rebar amount

$$A_{sb} = \alpha \cdot \Sigma A_{sj}$$

Where,

A_{sb} : Balanced rebar amount (mm^2)
σ_{sy} : Rebar yield stress (N/mm^2), $\sigma_{sy} = 345.00$
E_s : Rebar Young's Modulus (N/mm^2), Es = 200000
d : Distance fr. Compression side edge to the farthest rebar (mm), d = 65.0
b : Width of member (mm), b = 1000.0
σ_{ck} : Concrete standard design strength (N/mm^2), $\sigma_{ck} = 24.00$
d_i : Distance fr. compression side edge to the "ith" compression side rebar (mm)
A_{si} : Area enclosed by the compression side edge to the "ith" compression side rebar (mm^2)
σ_{si} : Compressive stress from compression side edge to the "ith" compression side rebar (N/mm^2)
d_j : Distance fr. compression side edge to the "jth" tension side rebar (mm)
A_{sj} : Area eclosed by the compression side edge to the "jth" tension side rebar (mm^2)
σ_{sj} : Tensile stress from compression side edge to the "jth" tension side rebar (N/mm^2)

Load Conditon (Water Level)	N (kN)	A_s (cm^2) A_{sb} (cm^2)	Judgement
Ordinary	21.328	5.068 ≦ 20.224	○
Earthquake	17.981	5.068 ≦ 138.959	○
Collision	17.602	5.68 ≦ 138.970	○

(4) Check of Bending Stress

（Reference）

Calculation of neutral axis

Find x from:

$$x^3 - 3 \cdot \left(\frac{h}{2} - e\right) \cdot x^2 - \frac{6n}{b} \left\{As \cdot \left(\frac{h}{2} - d - e\right)\right\} \cdot x$$

$$+ \frac{6n}{b} \left\{As \cdot d \cdot \left(\frac{h}{2} - d - e\right)\right\} = 0.0$$

Stress Calculation

$$\sigma_c = \frac{N}{\frac{b \cdot x}{2} - n \cdot As \cdot \frac{d - x}{x}}$$

$$\sigma_s = n \cdot \sigma_c \cdot \frac{d - x}{x}$$

Where,

x : Distance from concrete compression edge to the neutral axis (mm)
h : Member cross-section height (mm), h ＝ 500.000
b : Member cross-section width (mm), b ＝ 1000.000

d : Effective height of the member (mm)
As : Total cross-sectional area of the tension side rebar (mm^2)
n : Rebar and concrete's coefficient of Young's modulus, n = 15.00
e : Distance from the sectional member's centroid to the axial force (mm)
σc : Concrete bending compressive stress (N/mm^2)
σs : Rebar tensile stress (N/mm^2)
N : Axial Force (N)

Load Condition (Water Level)	M (kN.m)	N (kN)	x (cm)	Comp. Stress (N/mm^2)		Tensile Stress (N/mm^2)		Judgement
				Calculated	Allowable	Calculated	Allowable	
Ordinary	-1.332	21.328	59.944	0.072 ≦	8.000	-0.956 ≦	180.000	○
Earthquake	1.813	17.981	44.671	0.080 ≦	12.000	-0.032 ≦	300.000	○
Collision	25.361	17.602	8.433	1.668 ≦	12.000	104.041 ≦	300.000	○

(5) Check of Shear Stress

$$\tau_m = \frac{S_h}{b \cdot d} \leqq \tau_{a1}$$

Where,
τ_m : Concrete average shear stress (N/mm^2)
S_h : Shear force (N)
d : Sectional member's effective height (mm)
b : Sectional member's effective width (mm)
τ_{a1} : Allowable shear stress (N/mm^2)

$$\tau_{a1} = Ce \cdot Cpt \cdot CN \cdot \tau_{a1}'$$

$$CN = 1 + \frac{M_o}{M} \quad (1 \leqq CN \leqq 2)$$

Where,
τ_{a1}' : Allowable shear stress when shear force acts in concrete (N/mm^2)
Ce : Correction factor for the sectional member's effective height
Cpt : Correction factor for the tensile main reinforcement ratio, Pt
CN : Correction factor from axial compression force
M_o : Bending moment at which the concrete stress is zero at the tension edge due to the axial compressive force (kN.m)

Loading Condition (Water Level)	Shear Force S_h(kN)	Effective Height d (cm)	Shear Stress (N/mm^2)			Correction Factor			Judge ment
			Calc. τ	Allow. τ_{a1}	Allow. τ_{a2}	Ce	Cpt	CN	
Ordinary	4.629	6.500	0.071 ≦	0.881	1.700	1.40	1.37	2.00	○
Earthquake	5.095	43.500	0.012 ≦	0.620	2.550	1.32	0.73	1.83	○
Collision	19.159	43.500	0.044 ≦	0.354	2.550	1.32	0.73	1.06	○

ROAD DESIGN

MECHANICALLY-STABILIZED EARTH WALL AND RETAINING WALL

(PACKAGE 3)

Contents

1. DESIGN CONDITION

1.1 Detailed Design of Retaining Wall

1.1.1 Changes from Basic Design

(1) Resetting Soil Modulus

Table 3.5.5.19 shows a list of soil modulus reflecting additional boring and indoor test results in non-target area.

Table 1.1.1 List of Soil Modulus

Layer	N Average [*1]	Unit Weight “γ” (kN/m3)	Cohesion “c” (kN/m2)	Friction Angle “φ” [*5] (°)	Modulus of Deformation “E” (kN/m2)
FILLED SOIL	4	18 [*3]	24 [*4]	0	1300 [*6]
CLAY-I	4	18 [*2]	24 [*1]	0	1300 [*6]
SILTY SAND-I	10	18 [*2]	0 [*4]	32	5000 [*8]
SANDY SILT	8	17 [*3]	48 [*4]	0	5600 [*7]
SILTY SAND-II	22	19 [*3]	0 [*4]	33	15400 [*7]
CLAY-II	21	18 [*3]	126 [*4]	0	14700 [*7]
CLAYEY SAND-I	35	19 [*3]	0 [*4]	33	24500 [*7]
CLAY-III	35	18 [*3]	210 [*4]	0	24500 [*7]
CLAYEY SAND-II	50	19 [*3]	0 [*4]	37	35000 [*7]
CLAY-IV	50	18 [*3]	300 [*4]	0	35000 [*7]

Source: JICA Study Team

*1 Maximum N value is 50
*2 Average values obtained by each tests
*3 Referenced by Japanese Standard (NEXCO)
*4 Calculated by C=6N (referenced by Japanese Standard (NEXCO)). The value of sandy soil is 0.
*5 Calculated with N value using effective overburden pressure
*6 Test value obtained by unconfined compression test
*7 E=700N according to the worth value obtained by borehole lateral load test
*8 E=500N according to the worth value obtained by borehole lateral load test

Range of Ground Improvement

In the section of reinforced earth wall, the ground improvement will be provided by deep-layer mixing method to the lower end of SANDY SILT layer. In the section of L-shaped retaining wall, it will be provided by middle layer mixing method to the lower end of CLAY-I layer.

1.1.2 Design conditions

(1) Application criteria

1) Japan Road Association (2012). *Road Earthwork: Retaining Wall Guidelines 2012.*

2) Japan Road Association (2012). *Specifications for Highway Bridges, the Commentary.*

3) Civil Engineering Research Center (2014). *Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed.*

4) American Association of State Highway and Transportation Officials (2012), *AASHTO LRFD Bridge Design Specifications, 6th Ed (US)*

(2) Soil Condition

1) Embankment Material γ =19 kN/m^3,C=0 kN/m^2

2) Foundation Ground

Reinforced earth wall γ =19.0 kN/m^3, ϕ =33.0,C=0.0 kN/m^2 (SILTY SAND-Ⅱ)

L-shaped retaining wall γ =18.0 kN/m^3, ϕ =32.0,C=0.0 kN/m^2 (SILTY SAND-Ⅰ)

(3) Materials

1) Concrete (Protective fence foundation, L type retaining wall) σ ck=24 N/mm^2

2) Leveling Concrete σ ck=18 N/mm^2

3) Rebar SD345

4) Reinforced earth wall Bolt and Strip Dimensions

Bolt Nominal diameter M12(mm)

Strip PL-SS400(Width b=80.0mm × Thickness t=4.0mm × Corrosion Allowance Cm=1.0mm)

(5) Load condition

1) Dead Loads

Plain concrete	23.0 kN/m^3
Reinforced concrete	24.5 kN/m^3
Backfilling material	18.0 kN/m^3
Water	10.0 kN/m^3

2) Working Load

Roadway 11.6 kN/m^2 (AASHTO LRFD 2012 Bridge)

3) Collision Load

Rigid protection fence (SC Type) P=43 kN/m

(7) Allowable Stress

1) Concrete Allowable Stress

The allowable compressive stress level and allowable shear strength of concrete are in principle based on the values in the table below.

Table 1.2 Allowable compressive stress level of concrete and allowable shear strength (N/mm^2)

Types of stress intensity	Concrete design standard strength (σ_{ck})	21	24
Compressive stress	Bending compressive stress	7.0	8.0
	Axial compressive stress	5.5	6.5
Shear stress	In case of bearing shear force only with concrete (τa_1)	0.22	0.23
	In case of bearing shear force with concrete and diagonal tension bar (τ_{a2})	1.6	1.7
	Punching shear stress intensity (τ_{a3})	0.85	0.90

The table below shows the allowable unit bond stress of concrete for rebar with a diameter of 5 mm or less.

Table 1.3 Allowable Unit Bond Stress of Concrete(N/mm^2)

Concrete design standard strength (σ_{ck})	21	24
Bond Stress (deformed rebars)	1.4	1.6

2) Allowable Stress Level of Rebar

The table below shows the allowable stress level of rebar for rebar with a diameter of 51 mm or less.

Table1.4 Allowable Stress Level of Rebar (N/mm^2)

Types of Rebars Stress Level, Types of Materials			SD345
Tensile stress	Load combination includes neither impact load nor influence of earthquake.	1) General Materials	180
		2)Materials in water or under groundwater level	160
	3) Basic value of allowable stress. Load combination includes collision load or influence of earthquake.		200
	4) Calculating lap joint length or fixing length of rebar		200
5) Compressive stress			200

Table1.5 Allowable Stress and Design Safety Factor

TYPE OF ALLOWABLE STRESS AND DESIGN SAFETY FACTOR	ORDINARY	EARTHQUAKE
STRIP TENSILE STRESS （N/mm^2）	σa=140.0	σaＥ=210.0
BOLT SHEAR STRESS （N/mm^2）	τa=200.00	τaＥ=300.00
STRIP PULL OUT	Ｆs≧2.00	ＦsＥ≧1.20
SLIDING	Ｆs≧1.50	ＦsＥ≧1.20
OVER-TURNING	ｅ ≦Ｌ／6	ｅ ≦Ｌ／3
BEARING CAPACITY	Ｆs≧3.00	ＦsＥ≧2.00
CIRCULAR SLIP	Ｆs≧1.20	ＦsＥ≧1.00

Table1.6 Material

(1) SKIN : CONCRETE SKIN

(2) BOLT AND STRIP DIMENSIONS

ITEM	ITEM	CODE	UNIT	GROUND (UNDER WATER)
BOLT	NOMINAL DIAMETER	d	mm	M12
	BOLT THREAD STRESS AREA	Ae	mm^2	84.3
NUMBER OF BOLT PER CONNECTION		n	PIECE	1
NUMBER OF BOLT ACROSS THE WIDTH		n'	PIECE	1
NUMBER OF SHEAR		j	POINT	2
STRIP	STRIP WIDTH	b	mm	80.0
	STRIP THICKNESS	t	mm	4.0
	CORROSION ALLOWANCE	Cm	mm	1.0 (1.5)

(9) Horizontal Seismic Coefficient

Horizontal seismic coefficient is as follows.

1) Safety of Elements (Internal Stability) Note: Same for L type retaining wall.

$$k_{h1} = C_z \cdot k_{h0} = 0.18$$

k_{h1} : Horizontal seismic coefficient allowing for safety of elements

2) Safety of Terre Armee（External stability）

$$k_{h2} = C_z \cdot k_{h0} \cdot \upsilon = 0.13$$

k_{h2} : Horizontal seismic coefficient allowing for safety of Terre Armee

υ : Correction factor = 0.70

3) Total Stability including Terre Armee（Stability for circular slip）

$$k_{h3} = C_z \cdot k_{h0} = 0.18$$

k_{h3} : Design horizontal seismic coefficient

k_{h0} : Standard value of design horizontal seismic coefficient = 0.18

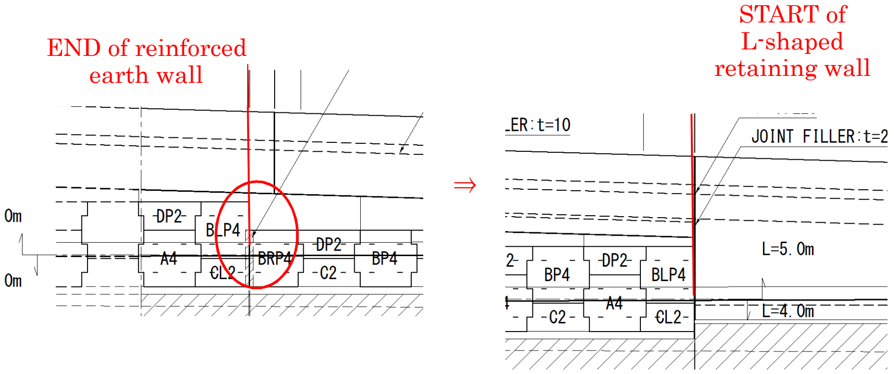
[Ground type：Level 1, Earthquake ground motion: TypeⅢ]

Source: *Road Earthwork: Retaining Wall Guidelines 2012.*

2. Retaining Wall Structure Plan

(1) Range of Reinforced Earth Wall

Panels of Reinforced earth wall needs to be installed horizontally, so they will be adjusted to the vertical profile of the road with coping concrete. In this design, L-shaped retaining wall will be installed from the changing point to 1 or less wall panel, balancing with the base of ground improvement.



Follow range setting flow

Place panels horizontally
↓
Adjust with coping concrete
↓
Come to changing point to 1 panel
↓
END of reinforced earth wall
↓
START of L-shaped retaining wall

Fig. 1.1 Evidence Chart of Reinforced Earth Wall Range

(2) Foundation Embedment

For the reinforced earth wall, the depth of foundation embedment is 0.5 m or more from the upper surface of the foundation concrete. For the L-shaped retaining wall, it is 0.5 m or more from the bottom of the bottom plate. Foundation concrete of reinforced earth wall, unlike the bottom plate, has a role as an adjustment to construct wall panels horizontally. There is little fear of scouring for the street in front of the L-shaped retaining wall that will be embedded from the bottom of the bottom plate.

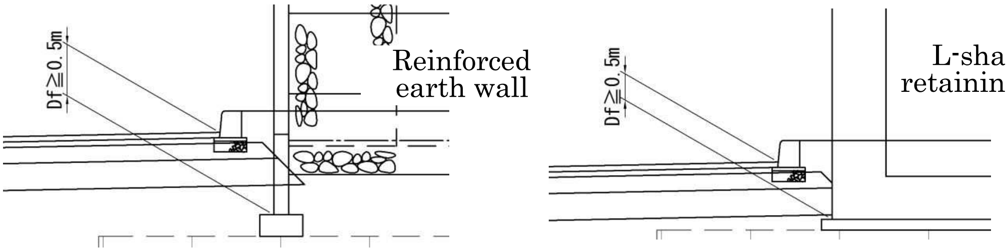


Fig. 1.2 (2)   Depth of Foundation Embedment

(3) Adoption of L-shaped Retaining Wall

The depth of the drainage facility in the approach section is 650 mm: 50 mm (necessary thickness of pavement) + 560 mm (waterway shape) + 40 mm (thick bed mortar). The necessary height of the retaining wall is 900 mm: 650m + 250mm (height of curb). Generally gravity type retaining walls are used for those less than 3 m in height. But we adopt an L type retaining wall because 1/3 or more than 1/2 of the cross section is cut for installing drainage facility. It is superior over the gravity type stabilized by its own weight. Joint, integrated with handrails, is basically 10m that is the same as the expansion joint of handrails.

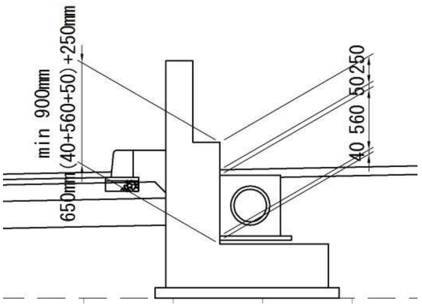


図 1.3 Waterway Shape and Retaining Wall

(4) Curbs and Handrails

Curbs and handrails are continuous from the approach to the bridge section. The conditions such as impact loads are the same in both sections. Taking into account the landscape, the design of them will be the same in both sections.

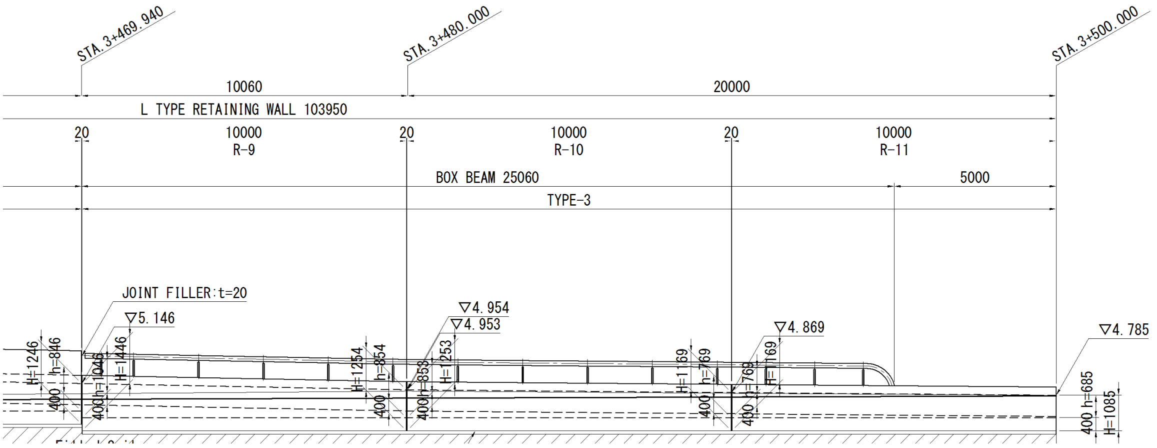


Fig. 1.4 Box Beam Elevation

3. Calculation Result

(1) Reinforced earth wall

Maximum Wall High

1) Strip Length and Safety Factor of Pullout

STAGE (i)	DEPTH z i(m)	VERTICAL ΔH(m)	HORIZONTAL ΔB(m)	DESIGN LENGTH L(m)	CALCULATION LENGTH		SAFETY FACTOR PULL OUT		JUDGE
					ORDINARY Lr(m)	EARTH QUAKE LrE(m)	ORDINARY Fs	EARTH QUAKE FsE	
1	2.185	0.750	0.750	6.500	5.961	6.297	2.280	1.268	OK
2	2.935	0.750	0.750	6.000	5.476	5.544	2.312	1.395	OK
3	3.685	0.750	0.750	6.000	5.114	5.026	2.574	1.688	OK
4	4.435	0.750	0.750	5.000	4.667	4.440	2.215	1.481	OK
5	5.185	0.750	0.750	5.000	4.325	4.000	2.422	1.731	OK
6	5.935	0.750	0.750	5.000	4.040	4.000	2.570	1.925	OK
7	6.685	0.750	0.750	4.000	4.000	4.000	2.250	1.758	OK

2) Degree of Material Stress

STAGE (i)	ORDINARY			EARTHQUAKE			JUDGE
	σti (N/mm²)	σoti (N/mm²)	τoi (N/mm²)	σtEi (N/mm²)	σotEi (N/mm²)	τoEi (N/mm²)	
1	82.9	76.5	88.5	99.6	91.9	106.3	OK
2	84.6	78.0	90.3	97.0	89.5	103.5	OK
3	86.1	79.5	91.9	95.4	88.1	101.8	OK
4	91.7	84.6	97.9	103.9	95.9	111.0	OK
5	97.7	90.1	104.3	111.6	103.0	119.2	OK
6	102.5	94.6	109.4	118.3	109.2	126.3	OK
7	112.6	103.9	120.2	129.7	119.7	138.5	OK

σti , σtEi : TENSILE STRESS OF GENERAL SECTION OF STRIP（N/mm²）

σoti, σotEi : TENSILE STRESS OF STRIP IN SECTION JOINED TO SKIN ELEMENT（N/mm²）

τoi , τoEi : BOLT shear STRESS IN SECTION JOINED TO SKIN ELEMENT（N/mm²）

4) Examination of Stability of R/E Itself (Examination of External Stability)

( ) Allowable Value

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR F s	7.642 (1.500)	OK	2.339 (1.200)	OK
OVERTUENING	ECCENTLICT Y e (m)	-1.024 (0.655)	OK	-0.310 (1.310)	OK
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION q s (kN/m^2)	223.045 (223.770)	OK	201.455 (335.650)	OK
BEARING CAPACITY STABILITY OF REPLACED FOUNDATION UNDER R/E BACK FILL	VERTICAL REACTION q os (kN/m^2)	354.527 (1012.063)	OK	340.247 (1296.009)	OK
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION q w (kN/m^2)	208.838 (223.770)	OK	186.596 (335.650)	OK

5) Overall Stability Examination Including R/E (Examination of Stability Concerning Circuler Slip)

CLASSIFICATION WITHIN AND OUTSIDE REINFORCED AREA	CASE	COORDINATE OF CIRCULAR CENTER		RADIUS R(m)	F smin	F s	JUDGE
		X(m)	Y(m)				
WITHIN REINFORCED AREA	ORDINARY-3	-12.500	18.000	21.303	1.414	1.200	OK
	EARTHQUAKE-3	-5.000	19.000	18.923	1.345	1.000	OK
OUTSIDE THE REINFORCING AREA	ORDINARY-12	-6.500	5.500	20.524	3.112	1.200	OK
	EARTHQUAKE-12	-7.500	21.500	35.288	1.795	1.000	OK

(2) L-shaped retaining wall

Maximum Wall High

Examination of Stability of Wall Itself (Examination of External Stability)

1) OVERTUENING

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Here

d : Distance from toe to the point of force (m)

ΣMr : Moment of resistance around the toes (kN.m)

ΣMt : Overturning moment around the toes (kN.m)

ΣV : Base design surface in all vertical load (kN)

$$e = \frac{B}{2} - d$$

Here

e : From the bottom version middle of force eccentricity (m)

B : Width bottom Edition (m), B = 2.500

$e_a = B/n$

Here

e_a : Permissible eccentricity (m)

n : Factor of safety

CASE	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	JUDGE
ORDINARY (Top Loading)	246.537	56.371	208.023	0.914	0.336	≦	0.417	OK
ORDINARY (Back)	211.737	56.371	184.823	0.841	0.409	≦	0.417	OK
EARTHQUAKE	261.083	108.858	204.587	0.744	0.506	≦	0.833	OK
COLLISION	211.737	92.502	184.823	0.645	0.605	≦	0.833	OK
WIND	209.637	41.476	184.823	0.910	0.340	≦	0.833	OK

2) SLIDING

$$F_s = \frac{\Sigma V \cdot \mu + C_B \cdot B'}{\Sigma H}$$

Here

ΣV : Base design surface in full load (kN)

ΣH : All horizontal loading in the bottom design (kN)

μ : Coefficient of friction between the floor and the bearing, μ=0.600

C_B : Bond strength between the floor and the bearing (kN/m^2), C_B = 0.000

B' : Loading loading width (m), B' = B −2e

B : Width bottom Edition (m), B = 2.500

e : Eccentricity (m)

CASE	ECCENTRICITY e(m)	LOADING LOADING WIDTH B'(m)
ORDINARY (Top Loading)	0.336	1.828
ORDINARY (Back)	0.409	1.682
EARTHQUAKE	0.506	1.488
COLLISION	0.605	1.290
WIND	0.340	1.820

CASE	VERTICAL LOAD ΣV(kN)	HRAIZONTA LLOAD ΣH(kN)	SAFETY FACTOR F_s		REQUIRED SAFETY FACTOR F_{sa}	JUDGE
ORDINARY (Top Loading)	208.023	49.754	2.509	≧	1.500	OK
ORDINARY (Back)	184.823	49.754	2.229	≧	1.500	OK
EARTHQUAKE	204.587	77.166	1.591	≧	1.200	OK
COLLISION	184.823	48.074	2.307	≧	1.200	OK
WIND	184.823	36.607	3.029	≧	1.200	OK

3) BEARING CAPACITY

A. In the bottom version width 1/3 bottom Edition Central (middle third) force action

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

B. In the bottom version Central bottom Edition width 2/3 force action

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

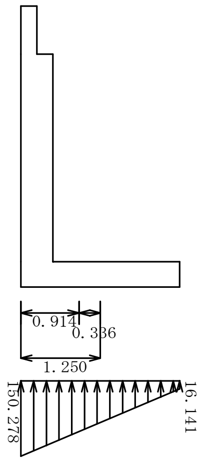
Here

ΣV : Load all lead pressure on bottom design (kN)

B : Width bottom Edition (m), B = 2.500

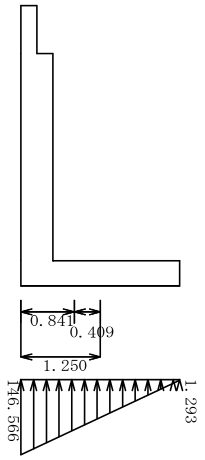
e : Eccentricity (m)

[1] ORDINARY (Top Loading)



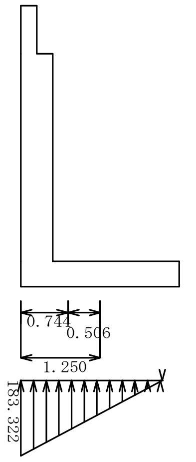
Width of subgrade reaction between (m)	Shape of subgrade reaction	VERTICAL REACTION (kN/m^2)	
		qmin	qmax
2.500	Trapezoid	16.141	150.278 ≦150.700

[2] ORDINARY (Back)



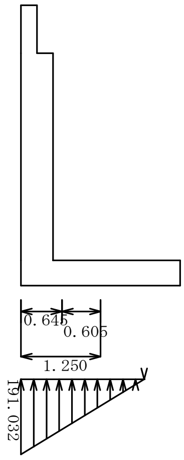
Width of subgrade reaction between (m)	Shape of subgrade reaction	VERTICAL REACTION (kN/m^2)	
		qmin	qmax
2.500	Trapezoid	1.293	146.566 ≦ 150.700

[3] EARTHQUAKE



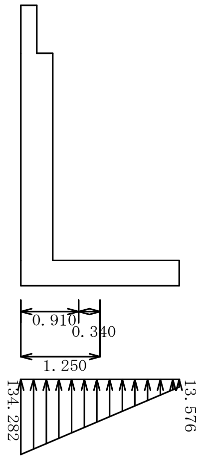
Width of subgrade reaction between (m)	Shape of subgrade reaction	VERTICAL REACTION (kN/m^2)	
		qmin	qmax
2.232	Triangle	0.000	183.322 ≦ 226.050

[4] COLLISION



Width of subgrade reaction between (m)	Shape of subgrade reaction	VERTICAL REACTION (kN/m^2)	
		qmin	qmax
1.935	Triangle	0.000	191.032 ≦ 226.050

[5]WIND



Width of subgrade reaction between (m)	Shape of subgrade reaction	VERTICAL REACTION (kN/m^2)	
		qmin	qmax
2.500	Trapezoid	13.576	134.282 ≦ 226.050

2) IMPROVEMENTS UNDER THE BEARING CAPACITY

( ) Allowable Value

ITEM	VERTICAL REACTION (kN)	
	RESULT	JUDGE
ORDINARY (Top Loading)	532.023 (1705.782)	OK
ORDINARY (Back)	508.823 (1687.532)	OK
EARTHQUAKE	528.587 (2212.718)	OK
COLLISION	508.823 (2552.467)	OK
WIND	508.823 (2702.338)	OK

ROAD DESIGN

SOFT SOIL TREATMENT
(PACKAGE 1 & 2)

Design statement
Settlement analysis

Consolidation Settlement Statement

Table of contents

1. Calculation condition

1.1 Condition of modeling

Input data used for calculation is shown as follows, taking the cross section of STA.0+240 as an example.

1.1.1 Node data

Enter the node data of the section to be used for analysis.

Node No	X coordinate value (m)	Y coordinate value (m)
2	-50.00	-2.14
7	50.00	-2.14
10	-50.00	-8.11
15	50.00	-8.11
18	-50.00	-14.20
23	50.00	-14.20
25	-50.00	-18.21
30	50.00	-18.21
32	-50.00	-27.16
37	50.00	-27.16

Node No	X coordinate value (m)	Y coordinate value (m)
39	-50.00	0.15
40	-30.00	0.15
41	-28.37	0.13
42	-21.17	-0.51
43	-19.95	-1.38
44	-3.34	-1.16
45	-1.96	-0.63
46	0.00	-0.03
47	4.40	-0.47
48	10.19	-0.79

Node No	X coordinate value (m)	Y coordinate value (m)
49	23.01	0.00
50	24.69	0.10
51	30.00	1.00
52	50.00	1.00
53	-14.06	0.00
54	-10.91	2.09
55	10.91	2.09
57	14.07	0.00
58	-26.90	0.00

1.1.2 Shape data

From the node data, create cross section data to be used for analysis and input the formation type.

Shape No	Configuration node										Formation type	Floor digging／backfilling
1	53	57	55	54							Embankment	Not
2	52	51	50	49	48	47	46	45	44	43	Clay layer	Not
	42	58	41	40	39	2	7					
3	2	10	15	7							Clay layer	Not
4	10	18	23	15							Clay layer	Not
5	18	25	30	23							Clay layer	Not
6	25	32	37	30							Clay layer	Not
7	58	42	43	44	45	46	47	48	49	57	Sand Layer	Not
	53											

1.1.3 Embankment condition

The data such as the unit weight of embankment to be constructed and the number of days of embankment construction are input.

Construction phase	Shape No	Embankment name	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Construction period		Removal date	Re-construction period		Re-removal date
					Start date	End date		Restart date	Re-end date	
1	1	Embankment	20.0	19.0	0	43	---	---	---	---

1.1.4 Soil condition

Enter the unit weight of each soil layer and soil, calculation method, and other constants.

Soil (Shape) No	Soil name	Method of calculation	Saturated weight (kN/m³)	Wet weight (kN/m³)	Type of soil layer
2	Filled Soil	Δe method	18.0	18.0	Clay layer
3	CLAY-Ⅰ	Δe method	17.5	17.5	Clay layer
4	SandyCLAY-Ⅰ	Δe method	17.5	17.5	Clay layer
5	SiltySAND-Ⅰ	B.K.Hough	17.5	16.5	Sand layer
6	CLAY-AⅡ	Non-consolidated layer	17.5	17.5	Clay layer
7	Filling	B.K.Hough	20.0	19.0	Sand layer

Soil (Shape) No	e-logP Curve	logMv -logP Curve	logCv -logP Curve	Water Content Ratio (%)	N-value	Conpression index C_c		Compaction yield stress P_c (kN/m²)	Increase amount of preceding consolidation q_0(kN/m²)	Coefficient at t=□Tv □(d)
						Normal compaction	Overconsolidation			
2	6	0	1	0	0	0.00	0.00	0.00	0.00	0.00
3	6	0	1	0	0	0.00	0.00	0.00	0.00	0.00
4	7	0	2	0	0	0.00	0.00	0.00	0.00	0.00
5	3	0	0	0	0	0.00	0.00	0.00	0.00	0.00
6	0	0	0	0	0	0.00	0.00	0.00	0.00	0.00
7	3	0	1	0	0	0.00	0.00	0.00	0.00	0.00

1.1.5 Arbitrary load condition

Enter load conditions related to settlement. Here, traffic load is input as band load.

(1)Band load

Load No	Left end			Right end			Construction period		Removal date
	X coordinate value (m)	Y coordinate value (m)	Load value (kN/m²)	X coordinate value (m)	Y coordinate value (m)	Load value (kN/m²)	Start date	End date	
1	-10.91	2.09	20.0	10.91	2.09	20.0	960	960	---

1.1.6 Water level condition

Enter groundwater level.

Unit volume weight of water :10.0 kN/m³

Buoyancy correction : Not

Node No	X coordinate value (m)	Y coordinate value (m)
1	-50.00	-0.75
2	50.00	-0.75

1.2 Common items on calculation

1.2.1 Calculation condition (Common items)

Set the Boussinesq method to the calculation method.

Method of calculating underground stress : Boussinesq method

Soil No	Settling reduction factor β
2	1.00
3	1.00
4	1.00
5	1.00
6	1.00
7	1.00

1.2.2 Condition of residual settlement

Enter the number of days to obtain the residual settlement amount.

Calculation of settlement amount Post-embankment period 480 days

1.3 Conditions for calculation time calculation

1.3.1 Calculation condition (Settling time)

Enter calculation period settlement calculation and output date of settlement amount.

History condition ：Number of days elapsed

Calculation period：2000 days

Output condition of transverse shape / relationship disgram

Output date： 480 days

How to create a time settlement curve ：Translation method

1.3.2 Compaction condition

Set drainage conditions and calculation method for each layer.

Method of calculating the consolidation coefficient of multiple soil layers

Layer thickness conversion method ： Average consolidation method

Drainage condition

Soil (Shape) No	Drainage condition
2	One side(Top drainage)
3	Both sides
4	One side(Bottom drainage)
5	Sand layer
6	Non-consolidation layer
7	Sand layer

1.4　Registration curve

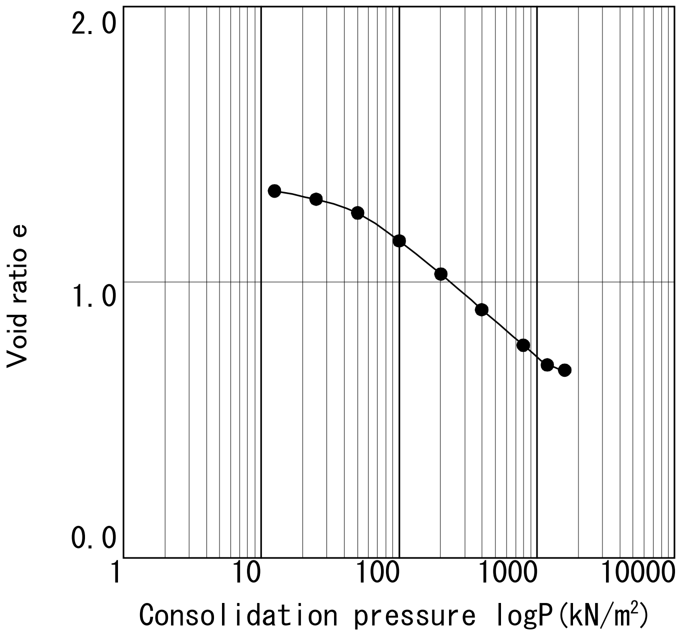
1.4.1　e-logP curve

Enter constant used for calculation of clay layer.

curve 6　　　　　：CLAY-Ⅰ

Method of interpolation between data：Curve interpolation

Used formation No　:　　2　3



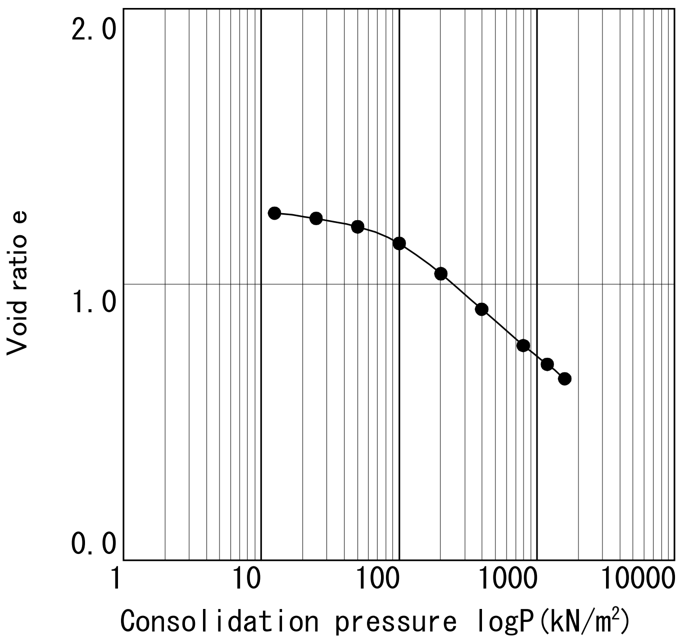
No.	1	2	3	4	5	6	7	8
Consolidation Pressure (kN/m²)	12.46	25.02	50.03	100.06	200.03	400.15	800.30	1200.55
Void ratio e	1.330	1.300	1.250	1.150	1.030	0.900	0.770	0.700

No.	9
Consolidation Pressure (kN/m²)	1600.50
Void ratio e	0.680

Curve 7 : Sandy CLAY

Method of interpolation between data：Curve interpolation

Used formation No : 4



No.	1	2	3	4	5	6	7	8
Consolidation Pressure (kN/m^2)	12.46	25.02	50.03	100.06	200.03	400.15	800.30	1200.55
Void ratio e	1.260	1.240	1.210	1.150	1.040	0.910	0.780	0.710

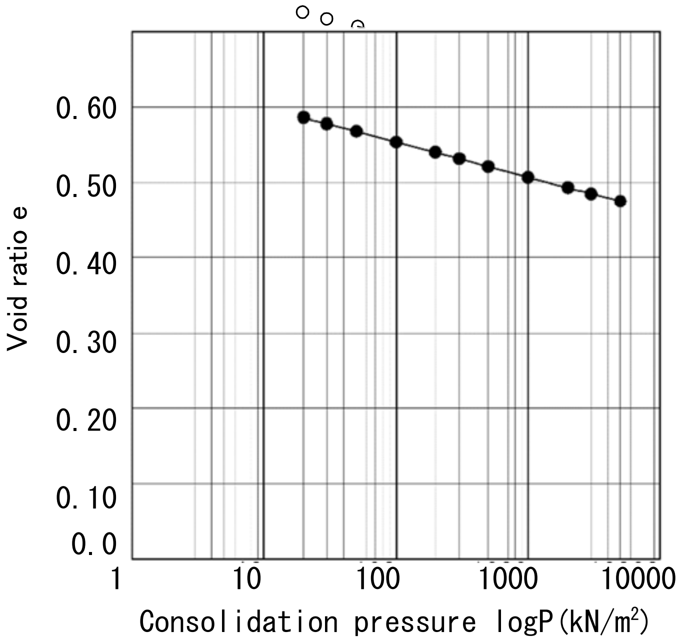
No.	9
Consolidation Pressure (kN/m^2)	1600.50
Void ratio e	0.660

Enter constant used for calculation of sand layer.

Curve 3 : Sand tightened in the middle

Method of interpolation between data : Curve interpolation

Used formation No : 5 7



No.	1	2	3	4	5	6	7	8
Consolidation Pressure (kN/m^2)	20.00	30.00	50.00	100.00	200.00	300.00	500.00	1000.00
Void ratio e	0.586	0.578	0.568	0.554	0.540	0.532	0.521	0.507

No.	9	10	11
Consolidation Pressure (kN/m2)	2000.00	3000.00	5000.00
Void ratio e	0.493	0.485	0.475

1.4.2 logCv-logP Curve

Enter constant used for calculation of clay layer.

Curve 1 : CLAY-Ⅰ

Method of interpolation between data：Curve interpolation

Calculation method of consolidation pressure：Arithmetic average

Used layer No : 2 3

No.	1	2	3	4	5	6	7	8
Consolidation Pressure (kN/m^2)	6.28	17.66	35.41	70.73	141.46	282.92	565.94	980.22
Consolidation coefficient Cv (cm2/day)	507.17	447.00	414.50	361.60	270.17	229.93	341.00	202.63

No.	9
Consolidation Pressure (kN/m^2)	1386.15
Consolidation coefficient Cv (cm2/day)	201.43

Curve 2 : Sandy-CLAY-Ⅰ

Method of interpolation between data：Curve interpolation

Calculation method of consolidation pressure：Arithmetic average

Used layer No : 4

No.	1	2	3	4	5	6	7	8
Consolidation Pressure (kN/m^2)	6.28	17.66	35.41	70.73	141.46	282.92	565.94	980.22
Consolidation Coefficient Cv (cm2/day)	965.50	1400.00	1133.50	965.50	782.50	694.50	583.00	522.00

No.	9
Consolidation Pressure (kN/m^2)	1386.15
Consolidation Coefficient Cv (cm2/day)	161.50

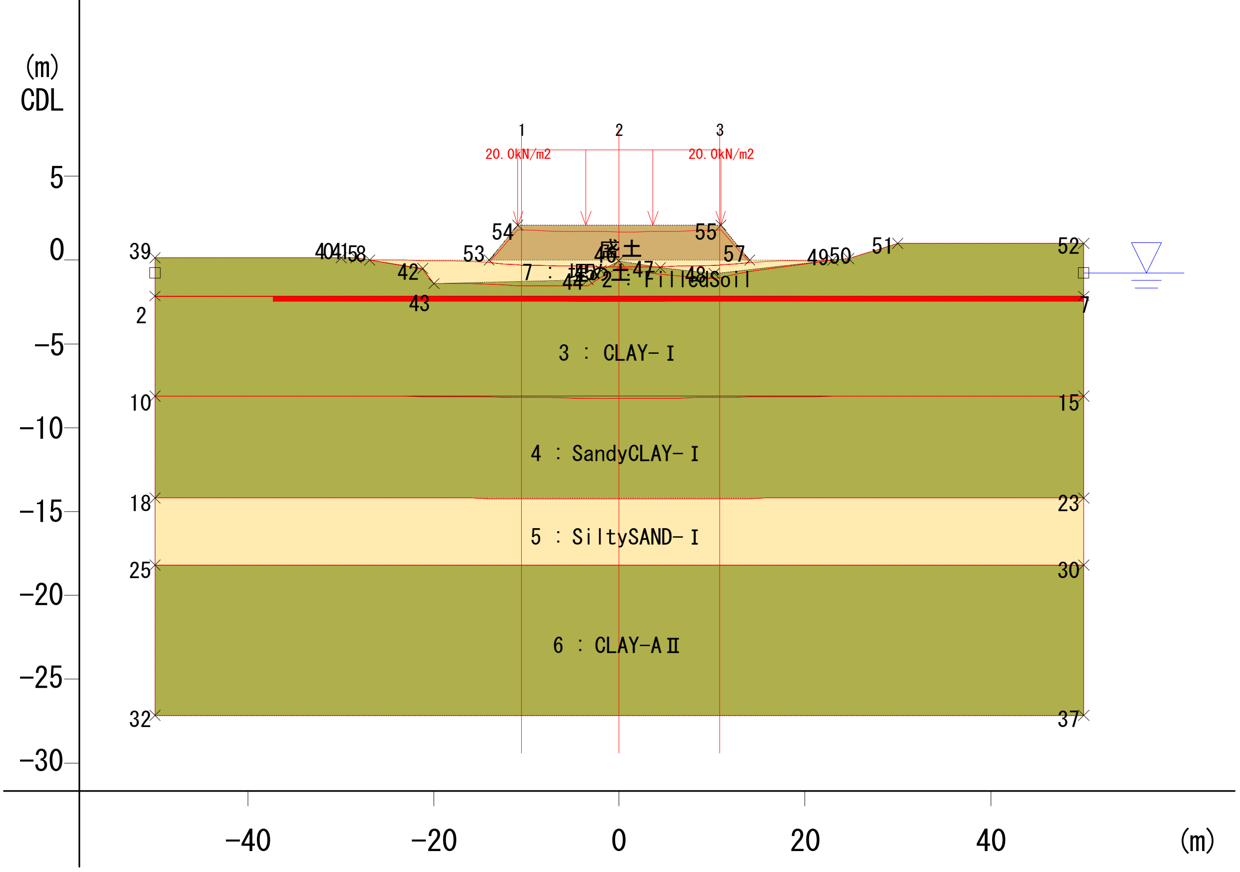
2. Calculation result

Cross-section view of calculation result.

STA. 0+240

Estimated embankment section (480days)

Item	1	2	3
Surface height	CDL 1.79	CDL 1.68	CDL 1.79
Settlement amount	0.30m	0.41m	0.30m



2.1 Result by calculation point

The total settlement amount at each calculation point and the settlement amount for each layer are output.

Calculation point 1 : 1

Calculation position : -10.500 m

Groundwater level : -0.750 m

Layer No	Soil layer name	Method of calculation	Layer thickness (m)	Effective overburden pressure p_0(kN/m^2)	Increasing stress ⊿p(kN/m^2)	Increase amount of preceding consolidation q_0(kN/m^2)	Settlement amount (cm)
7	Filling	B.K.Hough	1.255	11.92	50.73	0.00	2.570
2	Filled Soil	Δe method	0.885	22.84	46.89	0.00	3.739
3	CLAY-Ⅰ	Δe method	5.970	48.77	38.89	0.00	21.488
4	Sandy CLAY-Ⅰ	Δe method	6.090	93.99	33.17	0.00	11.534
5	Silty SAND-Ⅰ	B.K.Hough	4.010	131.87	30.01	0.00	1.075
6	CLAY-AⅡ	Non-consolidation layer	8.950	---	---	---	---

Subtotal(cm) : 40.405

Layer No	Initial void ratio e_0	Void ratio e_1	Coefficient of volume compressibility Mv (m^2/kN)	Conpression index C_{c1}	Expansion index C_{c2}	Geometric average √p_0(p_0+⊿p) (kN/m^2)	Arithmetic average p_0+⊿p/2 (kN/m^2)	Consolidation coefficientCv (cm^2/day)
7	0.5962	0.5635	---	---	---	---	--	0.000
2	1.3044	1.2071	---	---	---	---	46.28	398.431
3	1.2527	1.1716	---	---	---	---	68.21	365.434
4	1.1575	1.1167	---	---	---	---	110.58	835.929
5	0.5483	0.5442	---	---	---	---	--	---
6	---	--	---	---	---	---	--	---

[Transverse shape/relationship chart]　　[Residual settlement amount]

Output date： 480 days　　post-embankment period： 480 days

Layer No	Output consolidation degree (%)	Settlement amount (cm)	Settlement consolidation degree of post-embankment period (%)	Converted layer consolidation degree of post-embankment period (%)	Settlement amount of post-embankment period (cm)
7	87.15	2.240	87.157	---	2.240
2	74.77	2.796	74.777	---	2.796
3	74.40	15.989	74.409	---	15.989
4	66.81	7.706	66.815	---	7.706
5	72.48	0.779	72.488	---	0.779
6	---	---	---	---	---

Subtotal(settlement amount)： 29.510(cm)

Subtotal(Settlement amount of post-embankment period)： 29.510(cm)

Residual settlement amount： 10.895(cm)

※When the immediate settlement amount is included in the final settlement amount, the output consolidation degree × final settlement amount＝settlement amount does not become.

Converted layer thickness

Layer No.	Formation Type	Drainage surface	Conversion layer	Converted layer thickness	Representative Cv
7	sand layer	sand layer	--	1.255	0.000
2	clay layer	one side (top drainage)	--	0.885	398.431
3	clay layer	both sides	--	5.970	365.434
4	clay layer	one side(bottom drainage)	--	6.090	835.929
5	sand layer	sand layer	--	4.010	0.000
6	clay layer	non-consolidated layer	--	8.950	0.000

Calculation point 2 : 2

Calculation position : 0.000 m

Groundwater level : -0.750 m

Layer No	Soil layer name	Method of calculation	Layer thickness (m)	Effective overburden pressure p_0(kN/m^2)	Increasing stress ⊿p(kN/m^2)	Increase amount of preceding consolidation q_0(kN/m^2)	settlement amount (cm)
7	Filling	B.K.Hough	0.030	0.28	59.65	0.00	0.137
2	FilledSoil	Δe method	2.110	16.21	59.51	0.00	11.343
3	CLAY-Ⅰ	Δe method	5.970	47.04	57.14	0.00	29.928
4	SandyCLAY-Ⅰ	Δe method	6.090	92.26	48.79	0.00	16.719
5	SiltySAND-Ⅰ	B.K.Hough	4.010	130.14	41.21	0.00	1.439
6	CLAY-AⅡ	non-consolidated layer	8.950	---	---	---	---

Subtotal (cm) : 59.566

Layer No	Initial void ratio e_0	Void ratio e_1	Coefficient of volume compressibility Mv(m^2/kN)	Compression index C_{c1}	Expansion index C_{c2}	Geometric average $\sqrt{p_0(p_0+⊿p)}$ (kN/m^2)	Arithmetic average $p_0+⊿p/2$ (kN/m^2)	Consolidation coefficient Cv (cm^2/day)
7	0.6395	0.5644	---	---	---	---	---	0.000
2	1.3194	1.1947	---	---	---	---	45.96	398.903
3	1.2564	1.1433	---	---	---	---	75.61	354.196
4	1.1597	1.1004	---	---	---	---	116.66	822.180
5	0.5486	0.5431	---	---	---	---	---	---
6	---	--	---	---	---	---	---	---

[Transverse shape/relationship chart]　　[Residual settlement amount]

Output date： 480 days　　　　　　　post-embankment period： 480days

Layer No	Output consolidation degree (%)	Settlement amount (cm)	Settlement consolidation degree of post-embankment period (%)	Converted layer consolidation degree of post-embankment period (%)	Settlement amount of post-embankment period (cm)
7	89.3	0.123	89.359	---	0.123
2	66.0	7.487	66.009	---	7.487
3	71.3	21.340	71.305	---	21.340
4	65.8	11.006	65.830	---	11.006
5	72.8	1.047	72.803	---	1.047
6	---	---	---	---	---

Subtotal(settlement amount)：	41.004(cm)
Subtotal(Settlement amount of post-embankment period)：	41.004(cm)
Residual settlement amount：	18.562(cm)

※When the immediate settlement amount is included in the final settlement amount, the output consolidation degree × final settlement amount＝settlement amount does not become.

Converted layer thickness

Layer No	Formation Type	Drainage surface	Conversion layer	Converted layer thickness	Representative Cv
7	sand layer	sand layer	--	0.030	0.000
2	clay layer	one side(top drainage)	--	2.110	398.903
3	clay layer	both sides	--	5.970	354.196
4	clay layer	one side(bottom drainage)	--	6.090	822.180
5	sand layer	sand layer	--	4.010	0.000
6	clay layer	non-consolidated layer	--	8.950	0.000

Calculation point 3 :3

Calculation position : 10.750 m

Groundwater level : -0.750 m

Layer No	Soil layer name	Method of calculation	Layer thickness (m)	Effective overburden pressure p_0(kN/m^2)	Increasing Stress ⊿p(kN/m^2)	Increase amount of preceding consolidation q_0(kN/m^2)	Settlement amount (cm)
7	Filling	B.K.Hough	0.755	7.18	49.77	0.00	1.903
2	FilledSoil	Δ e method	1.385	19.84	45.74	0.00	5.686
3	CLAY-Ⅰ	Δ e method	5.970	47.77	37.53	0.00	20.875
4	SandyCLAY-Ⅰ	Δ e method	6.090	92.99	32.45	0.00	11.292
5	SiltySAND-Ⅰ	B.K.Hough	4.010	130.87	29.54	0.00	1.067
6	CLAY-AⅡ	non-consolidated layer	8.950	---	---	---	---

Subtotal (cm) :40.822

Layer No	initial void ratio e_0	Void Ratio e_1	Coefficient of volume compressibility Mv (m^2/kN)	Compression index C_{c1}	Expansion index C_{c2}	Geometric average $\sqrt{p_0(p_0+⊿p)}$ (kN/m^2)	Arithmetic average $p_0+⊿p/2$ (kN/m^2)	Consolidation coefficient Cv (cm^2/day)
7	0.6059	0.5654	---	---	---	---	---	0.000
2	1.3108	1.2159	---	---	---	---	42.71	403.694
3	1.2549	1.1760	---	---	---	---	66.53	367.989
4	1.1587	1.1187	---	---	---	---	109.22	839.239
5	0.5485	0.5444	---	---	---	---	---	---
6	---	---	---	---	---	---	---	---

[Transverse shape/relationship chart]　　[Residual settlement amount]

Output date：　480日　　　　　　　　　post-embankment period：480日

Layer No	Output consolidation degree (%)	Settlement amount (cm)	Settlement consolidation degree of post-embankment period (%)	Converted layer consolidation degree of post-embankment period (%)	Settlement amount of post-embankment period (cm)
7	89.6	1.706	89.678	---	1.706
2	75.6	4.299	75.610	---	4.299
3	74.3	15.521	74.353	---	15.521
4	66.8	7.546	66.823	---	7.546
5	72.4	0.774	72.477	---	0.774
6	---	---	---	---	---

Subtotal(settlement amount)：　29.846(cm)

Subtotal(Settlement amount of post-embankment period)：　29.846(cm)

Residual settlement amount：　10.977(cm)

※When the immediate settlement amount is included in the final settlement amount, the output consolidation degree × final settlement amount＝settlement amount does not become.

Converted layer thickness

Layer No	Formation Type	Drainage surface	Conversion layer	Converted layer thickness	Representative Cv
7	sand layer	sand layer	--	0.755	0.000
2	clay layer	one side(top drainage)	--	1.385	403.694
3	clay layer	both sides	--	5.970	367.989
4	clay layer	one side(bottom drainage)	--	6.090	839.239
5	sand layer	sand layer	--	4.010	0.000
6	clay layer	non-consolidated layer	--	8.950	0.000

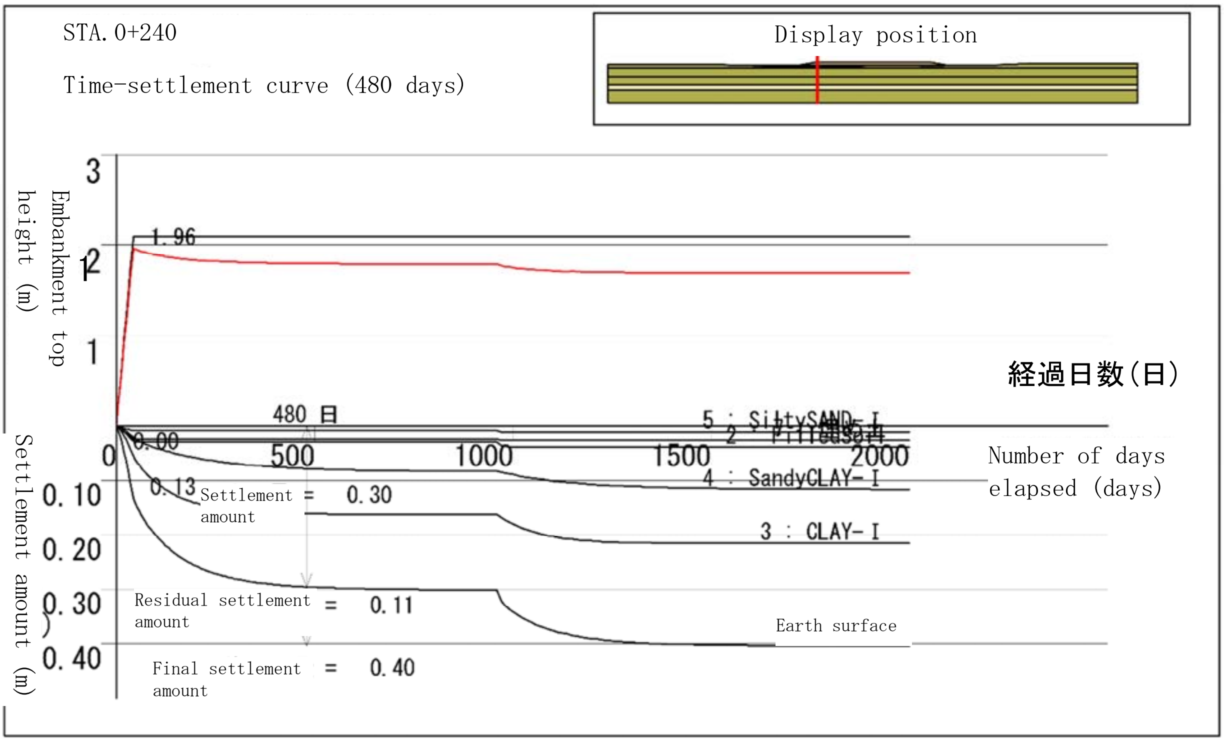
2.2 Consolidation degree — Number of days elapsed

Output of figure showing relationship between time, settlement amount and consolidation degree each point.

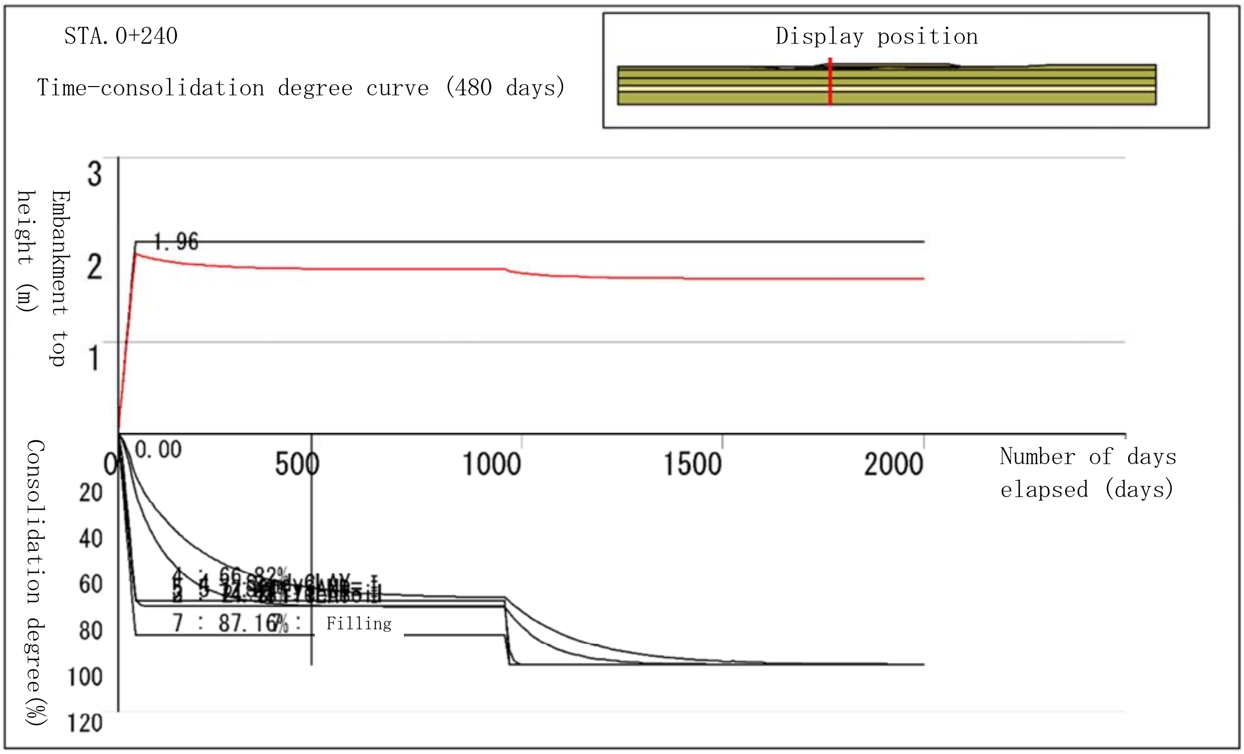
Calculation point 1 : 1

Calculation position : -10.500 m

Time-settlement curve



Time-consolidation degree curve



Consolidation degree of all layer

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	18	30	41	63	102	166	320	973	1056	---
Settlement amount	4.041	8.081	12.122	16.162	20.203	24.243	28.284	32.324	36.365	40.405

No. 7 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name: Filling

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	5	10	15	20	25	30	35	40	963	---
Settlement amount	0.257	0.514	0.771	1.028	1.285	1.542	1.799	2.056	2.313	2.570

No. 2 Layer Soil layer name : Filled Soil

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	11	17	22	27	33	38	43	964	969	---
Settlement amount	0.374	0.748	1.122	1.495	1.869	2.243	2.617	2.991	3.365	3.739

No. 3 Layer Soil layer name: CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	24	37	52	76	108	158	265	983	1051	---
Settlement amount	2.149	4.298	6.446	8.595	10.744	12.893	15.042	17.190	19.339	21.488

No. 4 Layer Soil layer name : Sandy CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	30	49	83	131	200	313	715	1028	1153	---
Settlement amount	1.153	2.307	3.460	4.613	5.767	6.920	8.074	9.227	10.380	11.534

No. 5 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name : Silty SAND-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	6	12	18	24	30	36	42	964	968	---
Settlement amount	0.107	0.215	0.322	0.430	0.537	0.645	0.752	0.860	0.967	1.075

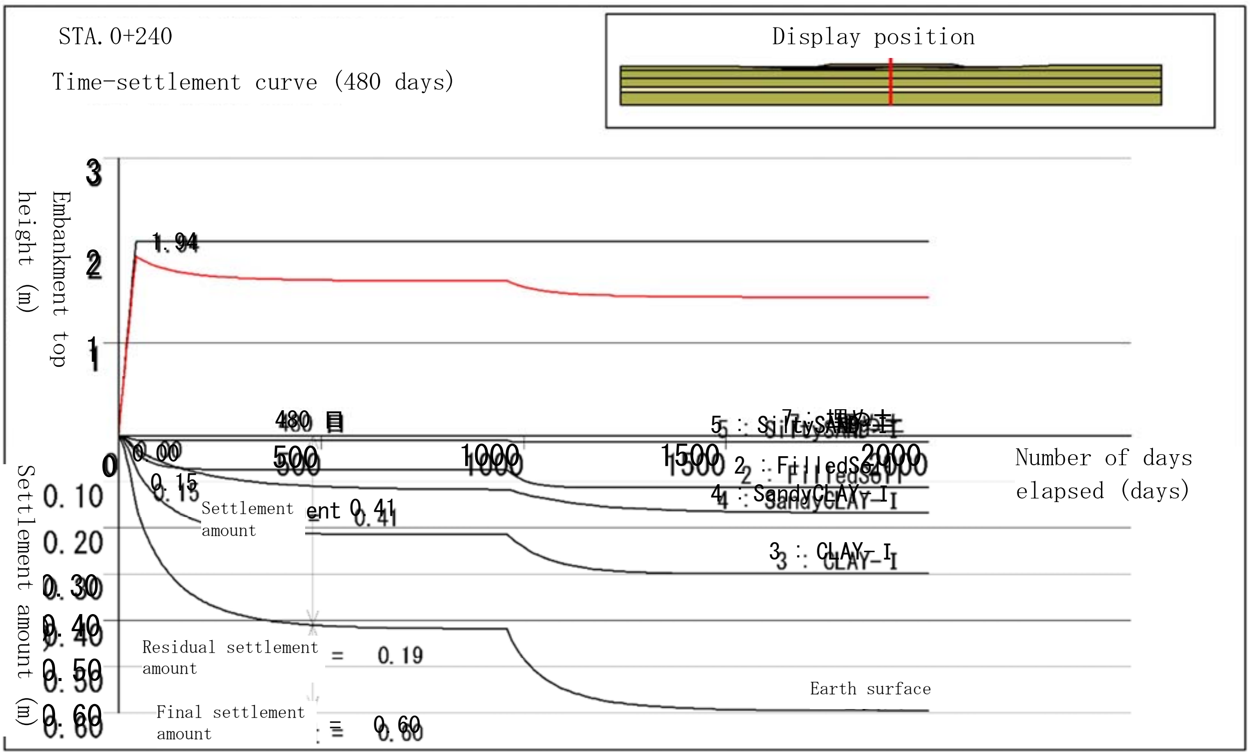
No. 6 Layer Soil layer name : CLAY-AⅡ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	---	---	---	---	---	---	---	---	---	---
Settlement amount	---	---	---	---	---	---	---	---	---	---

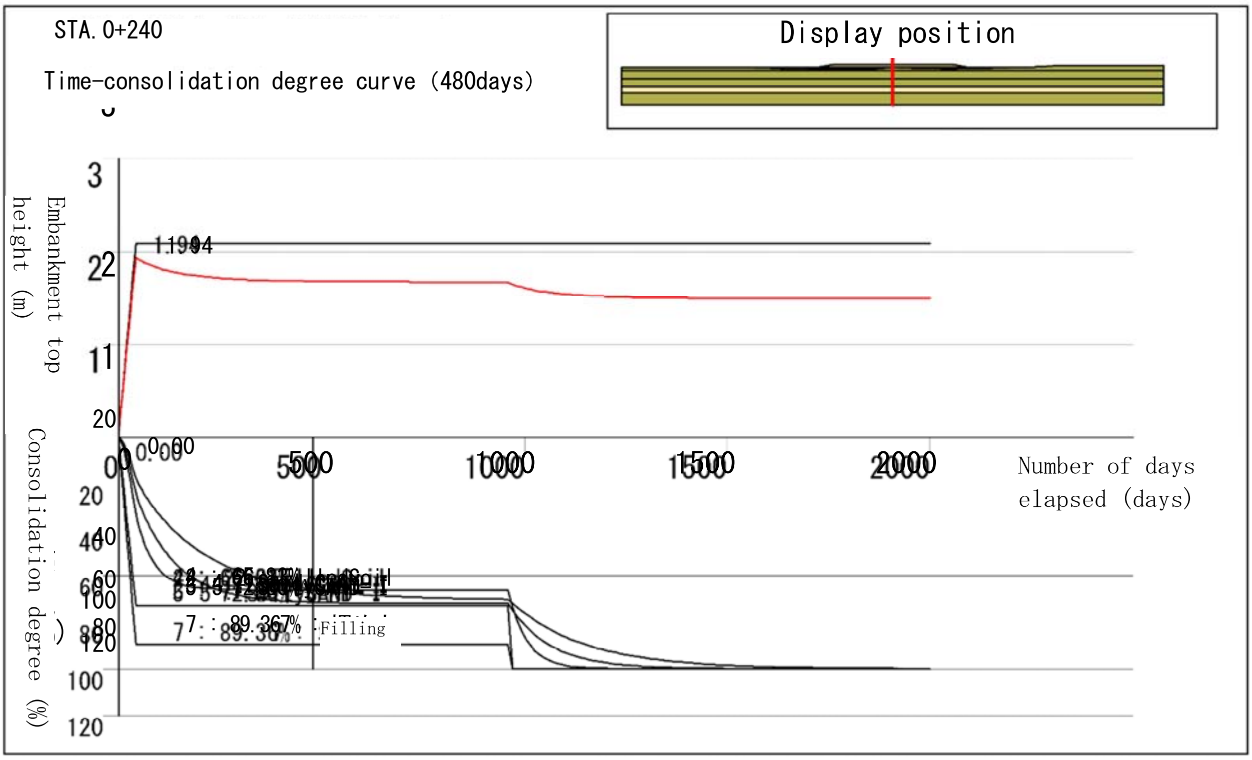
Calculation point 2 : 2

Calculation position : 0.000 m

Time-settlement curve



Time-consolidation degree curve



Consolidation degree of all layer

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	23	37	53	81	124	206	721	996	1071	---
settlement amount	5.957	11.913	17.870	23.826	29.783	35.739	41.696	47.653	53.609	59.566

No. 7 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name： Filling

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	5	10	15	20	25	29	34	39	961	---
settlement amount	0.014	0.027	0.041	0.055	0.069	0.082	0.096	0.110	0.124	0.137

No. 2 Layer Soil layer name : Filled Soil

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	20	31	41	53	75	117	966	985	1016	---
settlement amount	1.134	2.269	3.403	4.537	5.671	6.806	7.940	9.074	10.209	11.343

No. 3 Layer Soil layer name： CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	24	38	55	81	118	178	360	995	1066	---
settlement amount	2.993	5.986	8.978	11.971	14.964	17.957	20.949	23.942	26.935	29.928

No. 4 Layer Soil layer name : Sandy CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	30	50	85	135	207	329	961	1035	1162	---
settlement amount	1.672	3.344	5.016	6.688	8.360	10.031	11.703	13.375	15.047	16.719

No. 5 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name : Silty SAND-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	6	12	18	24	30	36	42	963	967	---
settlement amount	0.144	0.288	0.432	0.575	0.719	0.863	1.007	1.151	1.295	1.439

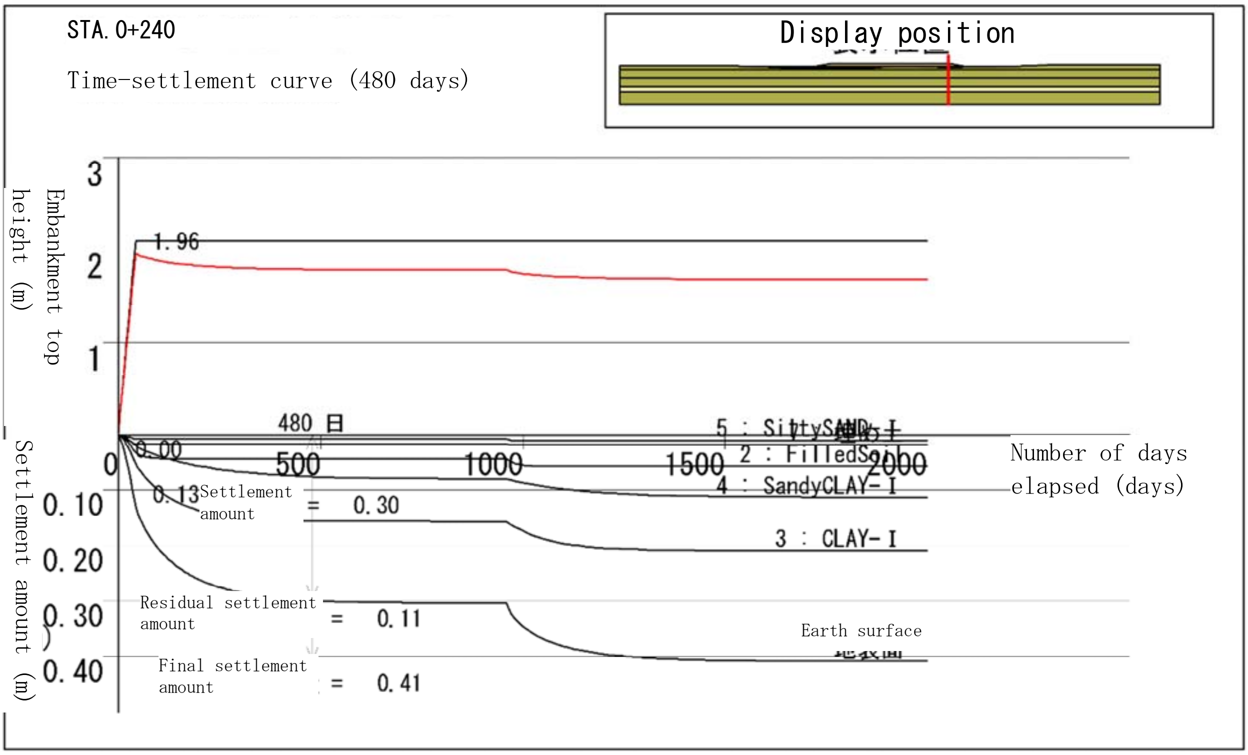
No. 6 Layer Soil layer name : CLAY-AⅡ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	---	---	---	---	---	---	---	---	---	---
settlement amount	---	---	---	---	---	---	---	---	---	---

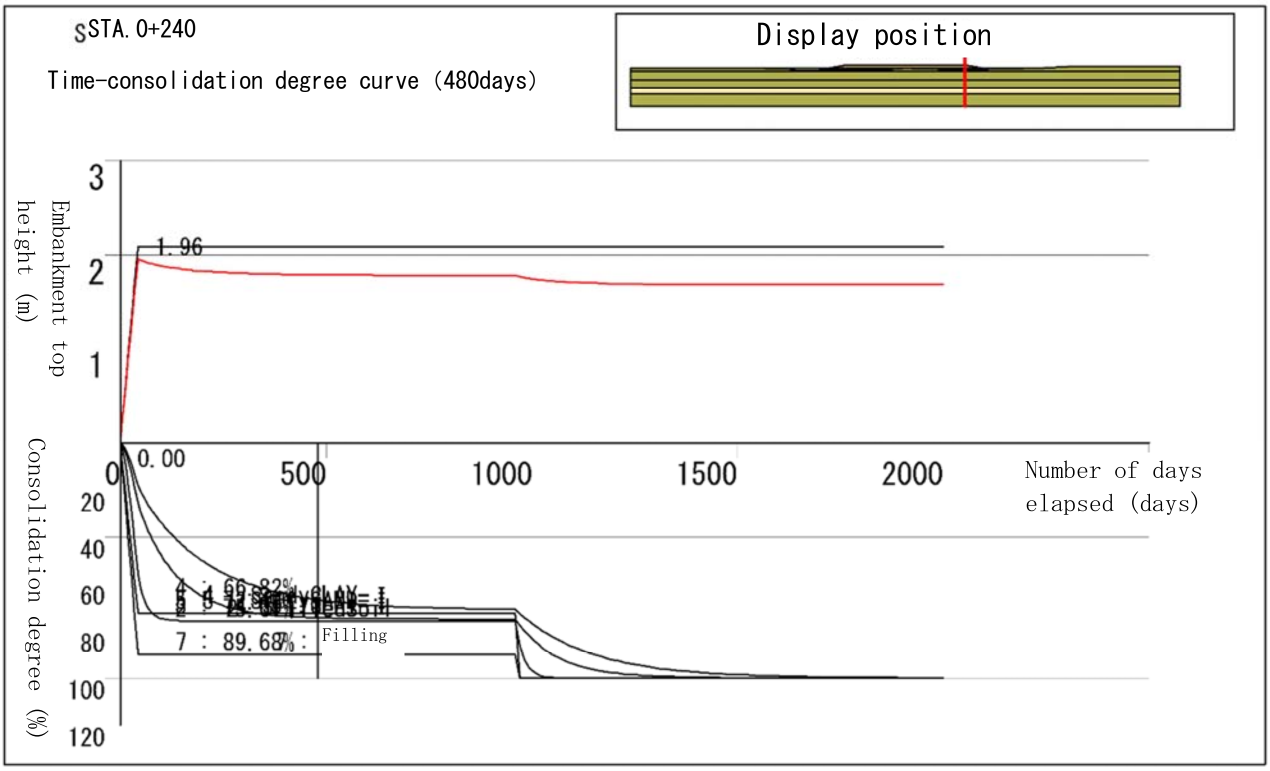
Calculation point 3 : 3

Calculation position : 10.750 m

Time-settlement curve



Time-consolidation degree curve



Consolidation degree of all layer

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	19	31	42	62	98	161	313	976	1051	---
settlement amount	4.082	8.164	12.247	16.329	20.411	24.493	28.576	32.658	36.740	40.822

No. 7 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name: Filling

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	5	10	15	20	24	29	34	39	961	---
settlement amount	0.190	0.381	0.571	0.761	0.951	1.142	1.332	1.522	1.713	1.903

No. 2 Layer Soil layer name：Filled Soil

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	14	22	28	34	40	48	67	965	978	---
settlement amount	0.569	1.137	1.706	2.274	2.843	3.411	3.980	4.549	5.117	5.686

No. 3 Layer Soil layer name：CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	24	37	52	75	108	157	265	983	1051	---
settlement amount	2.087	4.175	6.262	8.350	10.437	12.525	14.612	16.700	18.787	20.875

No. 4 Layer Soil layer name：Sandy CLAY-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	30	49	83	131	199	313	718	1028	1152	---
settlement amount	1.129	2.258	3.388	4.517	5.646	6.775	7.904	9.033	10.163	11.292

No. 5 Layer Calculation of the immediate settlement amount (B.K.Hough or DeBeer)
Soil layer name：Silty SAND-Ⅰ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	6	12	18	24	30	36	42	964	968	---
settlement amount	0.107	0.213	0.320	0.427	0.534	0.640	0.747	0.854	0.961	1.067

No. 6 Layer Soil layer name：CLAY-AⅡ

Consolidation degree	10	20	30	40	50	60	70	80	90	100
Number of days elapsed	---	---	---	---	---	---	---	---	---	---
settlement amount	---	---	---	---	---	---	---	---	---	---

2.3 Calculation result for each load step

When embankment and traffic load are applied, the output of each subsidence amount.

480 days later

Calculation point : 1

Calculation position : -10.500 m

When Transverse shape/relationship chart is "Output date" by superposition method andcalculation period setting, consolidation degree U, time settlement amount St, total settlement amount Si+St are output.

Phase 1 Embankment name: embankment

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient Cv cm^2/day)	Instantaneous settlemtnt Si (cm)	Consolidation settlement amount Sc (cm)	Consolidation amount U (%)	Time settlement amount St (cm)	total settlement amount Si＋St (cm)
7	38.85	0.0	0.000	2.240	-	-	-
2	35.55	406.8	0.000	2.796	-	-	-
3	28.31	373.5	0.000	16.103	-	-	-
4	23.46	848.2	0.000	8.207	-	-	-
5	21.09	0.0	0.000	0.779	-	-	-
6	-	-	-	-	-	-	-

Phase 2 Band load 1

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient Cv (cm^2/day)	Instantaneous settlemtnt Si (cm)	Consolidation settlement amount Sc (cm)	Consolidation amount U (%)	Time settlement amount St (cm)	total settlement amount Si＋St (cm)
7	11.88	0.0	0.000	0.330	-	-	-
2	11.35	398.4	0.000	0.943	-	-	-
3	10.58	365.4	0.000	5.385	-	-	-
4	9.72	835.9	0.000	3.327	-	-	-
5	8.93	0.0	0.000	0.296	-	-	-
6	-	-	-	-	-	-	-

Calculation point : 2

Calculation position : 0.000 m

Phase 1 Embankment name: embankment

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient Cv (cm^2/day)	Instantaneous settlemtnt Si (cm)	Consolidation settlement amount Sc (cm)	Consolidation amount U (%)	Time settlement amount St (cm)	total settlement amount Si＋St (cm)
7	39.71	0.0	0.000	0.123	-	-	-
2	39.70	413.5	0.000	7.488	-	-	-
3	38.72	368.2	0.000	21.502	-	-	-
4	33.77	839.4	0.000	11.742	-	-	-
5	28.76	0.0	0.000	1.047	-	-	-
6	-	-	-	-	-	-	-

Phase 2 band load 1

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient C v (cm^2/day)	Instantaneous settlemtnt S i (cm)	Consolidation settlement amount S c (cm)	Consolidation amount U (%)	Time settlement amount S t (cm)	total settlement amount S i＋S t (cm)
7	19.94	0.0	0.000	0.015	-	-	-
2	19.81	398.9	0.000	3.855	-	-	-
3	18.42	354.2	0.000	8.425	-	-	-
4	15.02	822.2	0.000	4.977	-	-	-
5	12.45	0.0	0.000	0.391	-	-	-
6	-	-	-	-	-	-	-

Calculation point : 3

Calculation position : 10.750 m

Phase 1 Embankment name: embankment

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient C v (cm^2/day)	Instantaneous settlemtnt S i (cm)	Consolidation settlement amount S c (cm)	Consolidation amount U (%)	Time settlement amount S t (cm)	total settlement amount S i＋S t (cm)
7	38.95	0.0	0.000	1.706	-	-	-
2	35.18	411.5	0.000	4.299	-	-	-
3	27.38	375.7	0.000	15.629	-	-	-
4	22.96	851.5	0.000	8.030	-	-	-
5	20.76	0.0	0.000	0.774	-	-	-
6	-	-	-	-	-	-	-

Phase 2 band load 1

Layer No	Increasing stress ⊿P (kN/m^2)	Consolidation coefficient C v (cm^2/day)	Instantaneous settlemtnt S i (cm)	Consolidation settlement amount S c (cm)	Consolidation amount U (%)	Time settlement amount S t (cm)	total settlement amount S i＋S t (cm)
7	10.82	0.0	0.000	0.196	-	-	-
2	10.56	403.7	0.000	1.387	-	-	-
3	10.14	368.0	0.000	5.246	-	-	-
4	9.49	839.2	0.000	3.261	-	-	-
5	8.78	0.0	0.000	0.294	-	-	-
6	-	-	-	-	-	-	-

Settlement analysis output data list

Area	Section	Embankment height (m)	Calculation position		Total settlement amount (cm)	Residual settlement amount (cm)	post-embankment period (days)
Thanlyin side	STA.0+240	2.09	Left		40.405	10.895	480
			Center		59.566	18.562	
			Right		40.822	10.977	
	STA.0+320	3.51	Left		44.242	7.202	480
			Center		73.543	10.786	
			Right		58.510	7.250	
	STA.0+340	3.97	Left		53.317	7.176	480
			Center		75.908	9.752	
			Right		68.588	7.396	
Thaketa side	STA.2+400	4.57	Left		45.679	16.498	390
			Center		69.851	27.632	
			Right		61.438	22.601	
	STA.2+620	4.81	Left		19.206	1.916	390
			Center		34.530	2.915	
			Right		27.653	1.789	
	STA.2+680	2.85	Left embankment	Left	13.984	2.566	390
				Center	21.671	3.465	
				Right	19.157	2.377	
			Right embankment	Left	19.335	2.339	
				Center	22.677	3.436	
				Right	19.102	2.383	
On-ramp	STA.0+360	2.57	Left		40.621	6.433	480
			Center		47.125	8.146	
			Right		38.627	7.086	
	STA.0+400	4.55	Left		43.376	5.281	480
			Center		54.301	5.620	
			Right		42.123	5.169	

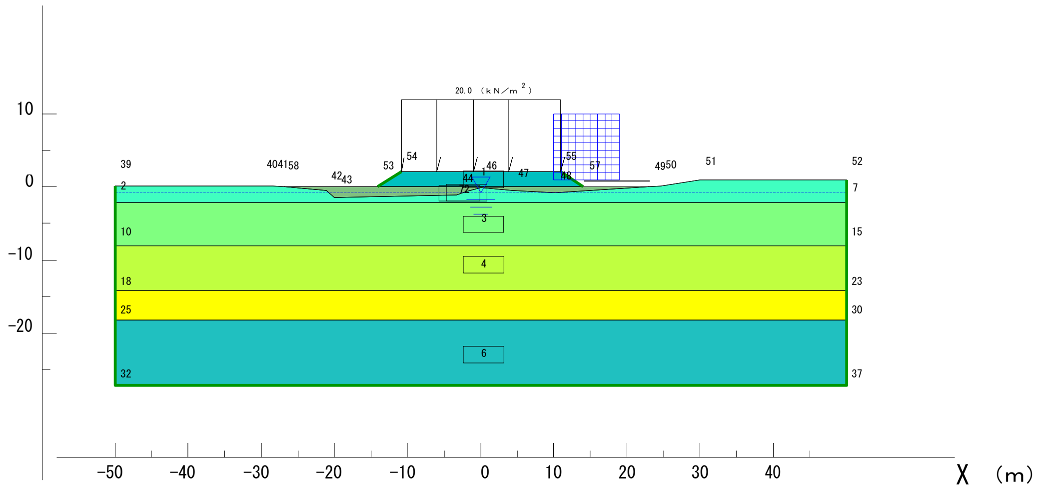
Design statement
Stability analysis

Arc slip calculation

1. Input data list

In the following, the conditions necessary for analysis are input.

// Design section //



(DATA 01.)　// Analysis type //

It shows the analysis method to be used and the condition of its calculation formula.

Title　　　　..... STA.0+240

Analysis type ..... Minimum safety factor calculation

Method of calculation ..... Guideline for Road Earth Work, Measure of Soft Soil Ground, issued by Japan Road Association (1986) (Effective stress method)

Formula

$$F_s = \frac{R\Sigma\,(CL + W'\cos\alpha\tan\phi)}{R\Sigma W\cdot\sin\alpha}$$

Here,

Fs　: Safety factor

R　: Radius of sliding arc (m)

C　: Cohesion (kN/m^2)

L　: Length of sliced bottom surface (m)

W　: Total weight of slice (kN/m)

α　: Angle formed by the slicing bottom face with the horizontal plane (°)

W'　: Weight of slice

(Water level line under water unit volume weight) (kN/m)

ϕ　: Internal friction angle (°)

(DATA 02.)　// Control //

Set data output condition.

Number of secondary pursuits ..... 0 (=0 Not performed)
(>0 To do)

Instructuon of secondary pursuit ..... 0 (=0 Always)
(=1 Earthquake)

Execution ..... 0 (=0 Perform input data check and stable calculation)
(=1 Perform input data check only)

Standard value of minimum safety factor 0.000

(DATA 06.)　// Node //

Enter embankment and node information of the formation.

Node No.	Coordinate value of node		Y coordinate value before settlement(m)
	X (m)	Y (m)	
2	-50.00	-2.14	0.00
7	50.00	-2.14	0.00
10	-50.00	-8.11	0.00
15	50.00	-8.11	0.00
18	-50.00	-14.20	0.00
23	50.00	-14.20	0.00
25	-50.00	-18.21	0.00
30	50.00	-18.21	0.00
32	-50.00	-27.16	0.00
37	50.00	-27.16	0.00
39	-50.00	0.15	0.00
40	-30.00	0.15	0.00
41	-28.37	0.13	0.00
42	-21.17	-0.51	0.00
43	-19.95	-1.38	0.00
44	-3.34	-1.16	0.00
45	-1.96	-0.63	0.00
46	0.00	-0.03	0.00
47	4.40	-0.47	0.00
48	10.19	-0.79	0.00
49	23.01	0.00	0.00
50	24.69	0.10	0.00
51	30.00	1.00	0.00
52	50.00	1.00	0.00
53	-14.06	0.00	0.00
54	-10.91	2.09	0.00
55	10.91	2.09	0.00
57	14.07	0.00	0.00
58	-26.90	0.00	0.00

(DATA 07.) // Water level line //

Enter groundwater level information.
Unit weight of water shall be 10 uniformly.

Unit weight of water (kN/m^3) = 10.00

Node No.	Node coordinates of water level line	
	X (m)	Y (m)
1	-50.00	-0.75
2	50.00	-0.75

(DATA 08.) // Layer shape //

Based on the node information set in DATA 6, select embankment and create embankment and geological form.

Soil No.	Characteristic value number of the soil	Number of constituent nodes	Node number making up the formation										
1	1	4	53	57	55	54							
2	2	17	52	51	50	49	48	47	46	45	44	43	42
			58	41	40	39	2	7					
3	3	4	2	10	15	7							
4	4	4	10	18	23	15							
5	5	4	18	25	30	23							
6	6	4	25	32	37	30							
7	7	11	58	42	43	44	45	46	47	48	49	57	53

(DATA 09.1) // Characteristics of soil //

For the embankment and geological formations set with DATA 8, enter the appropriate constants respectively.

Characteristic value No	Saturated unit volume weight (kN/m^3)	Wet unit volume weught (kN/m^3)	Pore water pressure calculation unit volume weight (kN/m^3)	Coefficient α	Cohesion C (kN/m2)	First order coefficient of adhesive force	Internal friction angle ϕ (°)	Circumferential frictional resistance τ (kN/m^2)
1	20.00	19.00	0.00	0.000	0.00	0.00	30.00	0.0
2	18.00	18.00	0.00	0.000	15.00	0.00	0.00	0.0
3	17.50	17.50	0.00	0.000	21.82	0.00	0.00	0.0
4	17.50	17.50	0.00	0.000	24.46	0.00	0.00	0.0
5	17.50	16.50	0.00	0.000	0.00	0.00	33.00	0.0
6	17.50	17.50	0.00	0.000	30.00	0.00	0.00	0.0
7	20.00	19.00	0.00	0.000	30.00	0.00	25.00	0.0

(DATA 09.3)　// Setting of strength increase by sand compaction pile consolidation //

Enter the condition constant of clay to consolidate by load.

Soil No.	Replacement rate by sand piles	Saturated unit volume weight of sand pile (kN/m^3)	Wet unit weight of sand pile (kN/m^3)	Internal friction angle of sand pile (°)	Stress sharing ratio	Strength increase rate of raw ground cohesive soil	Consolidation degree of cohesive soil part	Difference between compaction yield stress and clay covering pressure (kN/m^2)
3	0.000	0.00	0.00	0.00	0	0.300	0.90	55.90
4	0.000	0.00	0.00	0.00	0	0.300	0.90	44.40

(DATA 09.4)　// Embankment //

Set the stratum to be calculated as embankment.

Embankment formation No.	Stratum number to be calculated as embankment
1	1

Embankment load action position .....　1(=0 Reference Y cordinate)
(=1 Bottom surface of embankment)

(DATA 11.)　// Grid //

Enter grid point information of the arc to be calculated, split pitch, etc.

Grid No.	Sliding direction	Double cut of the ground	The upper left coordinate value of the		Number of grid points		Split pitch		Slope of grid
			X (m)	Y (m)	X direction	Y direction	X direction (m)	Y direction (m)	
1	Right slide	Left sediment	10.00	10.00	10	10	1.00	1.00	0.000

(DATA 12.)　// Radius// (Normal line method)

Enter the radius and position of the node of the arc to be calculated.

Radius ..... Search by ⊿R pitch from maximum radius
Pitch of radius ΔR (m) 1.00
Y coordinate value giving minimum radius (m) 50.00
Y coordinate value giving the maximum radius (m) ..... -27.16

(DATA 15.1) // Distributed load //(Virtical)

Enter load area and load amount of traffic load acting on embankment.

Load No.	Left end				Right end				Coefficient of resistance	Earthquake inertial force consideration	
	X (m)	Y (m)	load(kN)		X (m)	Y (m)	Load(kN)			Horizontal seismic intensity	Vertical seismic intensity
			Normal	Earthquake			Normal	Earthquake			
1	-10.91	2.0	20	0.0	10.91	2.09	20.0	0.0	0.000	0.000	0.000

(DATA 16.1) // NEVER line // (N = 3)

Set a line on which the arc does not pass in calculating.
It is necessary to set it so that arcs which are impossible to reality are outputted by calculation.

Number of constituent nodes	X (m)	Y (m)
2	14.07	0.00
	10.91	2.09
4	-50.00	0.15
	-50.00	-27.16
	50.00	-27.16
	50.00	1.00
2	-10.91	2.09
	-14.06	0.00

2. Minimum safety factor list

Minimum safety factor (Group number of the lattice = 1)

It is the calculation result.

It is a list of nodal conditions of arcs with minimum safety factor shown.

Normal (Number of secondary pursuits) = 0)

Minimum safety factor = 1.450

Center of circle X coordinate value(m) = 12.00

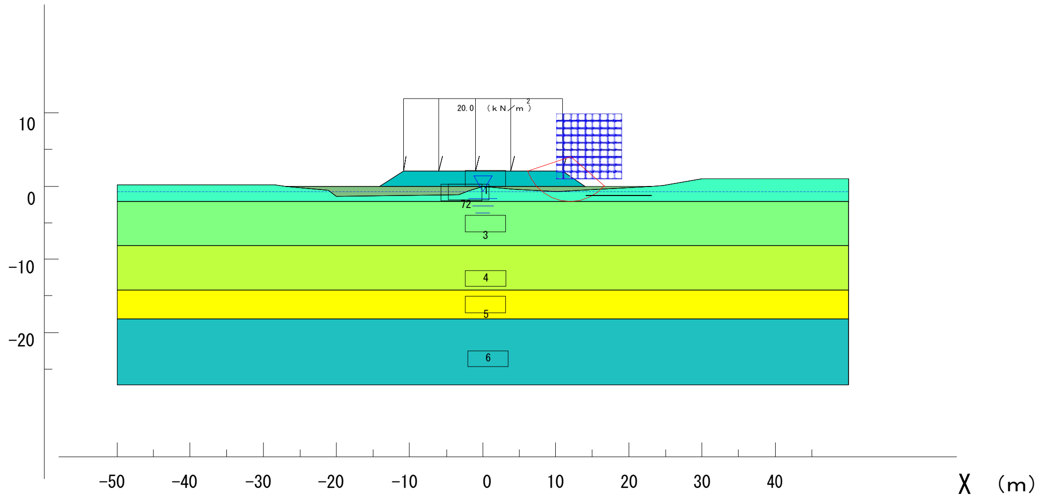
Y coordinate value(m) = 4.00

Radius of circle R(m) = 6.16

Resistance moment(kNm) = 1282.1

Starting moment(kNm) = 884.2

// Safety factor diagram //



STA. 0+240

This is a list of input values and output results of arc slip calculation.

Scale ; 1/ 792

Minimum safety factor F $_{S\,MIN}$ = 1.450

Centerof arc X = 12.00 （m）

Y = 4.00 （m）

Radius R = 6.16 （m）

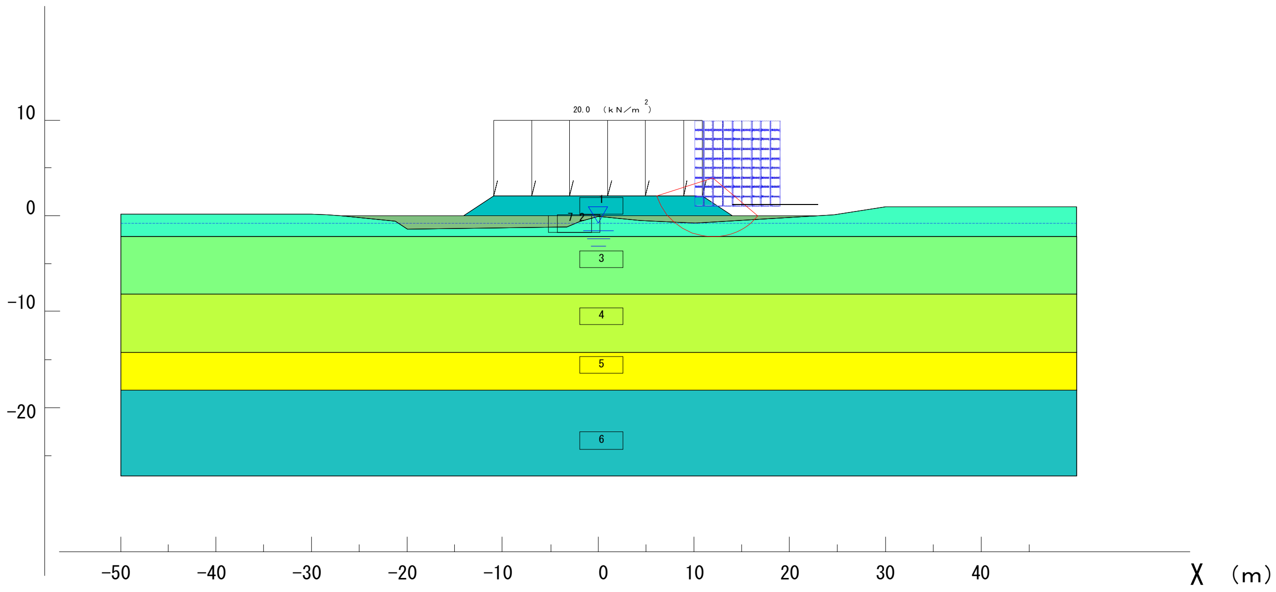
Resistance moment M $_R$ = 1282.1 （kN・m）

Starting moment M $_D$ = 884.2 （kN・m）

Layer No.	Saturated weight (kN/m^3)	Wet weight (kN/m^3)	Internal friction angle(°)	Cohesion (kN/m^2)	First order coefficient of cohesion	Horizontal seismic intensity	Vertical seismic intensity
1	20.00	19.00	30.00	0.00	0.00	0.000	0.000
2	18.00	18.00	0.00	15.00	0.00	0.000	0.000
3	17.50	17.50	0.00	21.82	0.00	0.000	0.000
4	17.50	17.50	0.00	24.46	0.00	0.000	0.000
5	17.50	16.50	33.00	0.00	0.00	0.000	0.000
6	17.50	17.50	0.00	30.00	0.00	0.000	0.000
7	20.00	19.00	25.00	30.00	0.00	0.000	0.000

Layer No.	Strength increase rate of raw ground cohesive soil	Consolidation degree of raw ground	P$_c$-P$_0$ of raw ground (kN/m^2)
3	0.300	0.90	55.90
4	0.300	0.90	44.40

Unit volume weight of water = 10.00 (kN/m^3)



Safety factor diagram (Normal)

Stability analysis output data list

Area	Section	Embankment height (m)	Minimum safety factor Fsmin						
			No measures				After measures		
			Normal		Time of earthquake	Time of liquefaction	Normal	Time of earthquake	Time of liquefaction
			Time of construction	Time of service			Time of service		
Thanlyin side	STA.0+240	2.09	1.450	1.450	1.226	1.564	–	–	–
	STA.0+320	3.51	1.434	1.434	0.876	1.199	2.433	1.288	1.848
	STA.0+340	3.97	1.202	1.202	0.840	1.047	1.798	1.260	1.816
Thaketa side	STA.2+400	4.57	1.226	1.226	0.895	1.379	1.825	1.217	1.432
	STA.2+620	4.81	1.282	1.282	1.027	1.347	2.329	1.394	1.461
	STA.2+680	2.85	1.880	1.880	1.466	1.772	–	–	–
On-ramp	STA.0+360	2.57	1.635	1.635	1.207	2.413	–	–	–
	STA.0+400	4.55	1.161	1.161	1.072	0.819	1.312	1.314	1.447

ROAD DESIGN

SOFT SOIL TREATMENT
(PACKAGE 3)

Contents

１. Introduction

Design Requirements for Countermeasures Bago Bridge Approach Retaining Wall Beneath, and the Results Recorded.

２. Design condition

2-1 Design Standards

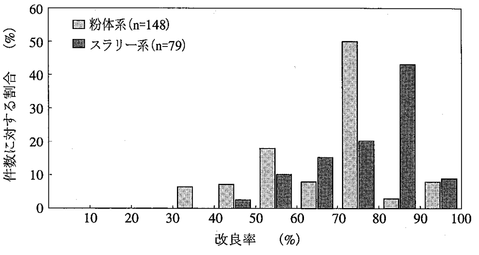
- *Japan Road Association (2012). Specifications for Highway Bridges, the Commentary.*
- *Japan Road Association (2012). Road Earthwork: Soft Ground Measures Guidelines 2012.*
- *Japan Road Association (2009). Road Earthwork Guidelines 2009.*
- *Japan Road Association (2010). Road Earthwork: Banking Construction Method Guidelines 2010.*
- *Japan Road Association (2012). Road Earthwork: Retaining Wall Guidelines 2012.*
- *Civil Engineering Research Center (2004). Deep Mixing Method in Land-based Construction Design and Construction Manual, 2th Rev Ed.*
- *Seismic technology research center,Faculty of Civil Engineering,Construction Ministry (1999). Cooperative Research Report No.186 Liquefaction Process Design and Construction Manual 1999.*
- *Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed.*

2-2 Basic Concept for Determination of Improvement Rate

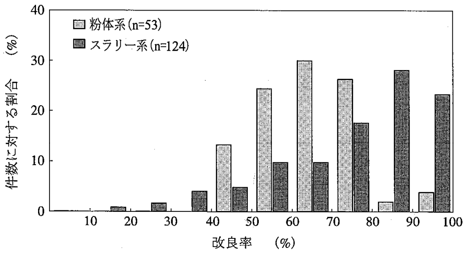
As shown in "*Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed*", the improvement ratio of 50% to 80% is practically adopted to the deep mixing method for the improvement of bearing capacity. On the other hand, for the countermeasure to the liquefaction, the rate is practically the range of 50% to 100%.

Considering the above practical values, the improvement ratio shall be set to be more than 50%.

(Bearing capacity reinforcement of retaining wall) (Liquefaction)



資図ー2.8 擁壁の支持力増強を主目的とした施工実績(改良率)



資図ー2.16 液状化防止を主目的とした施工実績(改良率)

Source: Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed

３． Setting of design conditions

3-1 Embankment material

The]design parameter for embankment material is shown below.

- ANGLE OF SHEARING RESISTANCE : $\varphi = 30°$
- UNIT WEIGHT OF SOIL : $\gamma = 19\text{kN/m}^3$

Talble4-6 Unit weight of soil (kN/m2)

Ground	Soils and foundations	One with a loose	A dense
Natural Ground	Sand and gravel	18	20
	Sandy soils	17	19
	Cohesive soils	14	18
And the Backfill Soil embankment	Sand and gravel	20	
	Sandy soils	19	
	Cohesive soils (However w_L＜50%)	18	

※ Groundwater less than weight per unit volume of soil is good as 9(kN/m3) minus the value in the table.

Talble4-5 And the backfill soil strength parameters of embankment

And the Backfill Soil embankment	Shear Resistance Angle (φ)	Adhesive Force (C)※2
Gravelly soils	35°	—
Sandy soils※1	30°	—
Cohesive soils (However w_L＜50%)	25°	—

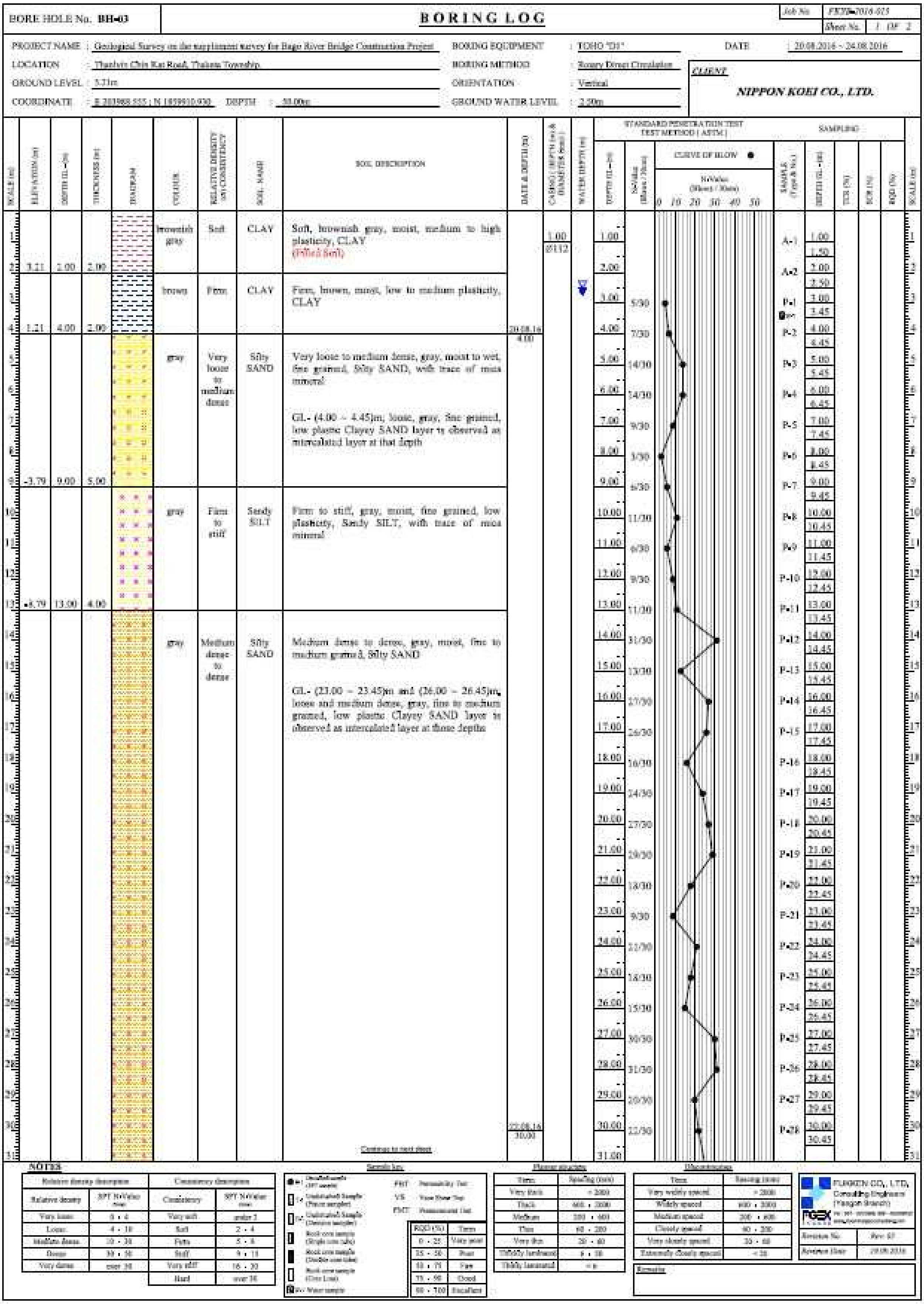
※1 Little fines sand, gravelly soils of good use.

※2 To estimate the soil parameters from the table above to ignore sticky C.

Source : Road Earthwork: Retaining Wall Guidelines 2012, Japan Road Association

3-2 Classification of strata of clay Foundation

The clay strata was identified by referring the following geological profile on “BH-03” conducted in the Supplemental F/S.



BORE HOLE No. BH-03

BORING LOG

Job No. FKYB-2016-015
Sheet No. 1 OF 2

PROJECT NAME : Geological Survey on the supplement survey for Bago River Bridge Construction Project
LOCATION : Thanlyin Chin Kat Road, Thaketa Township
GROUND LEVEL : 5.21m
COORDINATE : E 203988.555 ; N 1859910.930 DEPTH : 50.00m
BORING EQUIPMENT : TOHO "D1"
BORING METHOD : Rotary Direct Circulation
ORIENTATION : Vertical
GROUND WATER LEVEL : 2.50m
DATE : 20.08.2016 ~ 24.08.2016
CLIENT
NIPPON KOEI CO., LTD.

ELEVATION (m)	DEPTH GL-(m)	THICKNESS (m)	COLOUR	RELATIVE DENSITY OR CONSISTENCY	SOIL NAME	SOIL DESCRIPTION
3.21	2.00	2.00	brownish gray	Soft	CLAY	Soft, brownish gray, moist, medium to high plasticity, CLAY (Filled Soil)
1.21	4.00	2.00	brown	Firm	CLAY	Firm, brown, moist, low to medium plasticity, CLAY
-3.79	9.00	5.00	gray	Very loose to medium dense	Silty SAND	Very loose to medium dense, gray, moist to wet, fine grained, Silty SAND, with trace of mica mineral GL- (4.00 ~ 4.45)m, loose, gray, fine grained, low plastic Clayey SAND layer is observed as intercalated layer at that depth
-8.79	13.00	4.00	gray	Firm to stiff	Sandy SILT	Firm to stiff, gray, moist, fine grained, low plasticity, Sandy SILT, with trace of mica mineral
			gray	Medium dense to dense	Silty SAND	Medium dense to dense, gray, moist, fine to medium grained, Silty SAND GL- (23.00 ~ 23.45)m and (26.00 ~ 26.45)m, loose and medium dense, gray, fine to medium grained, low plastic Clayey SAND layer is observed as intercalated layer at those depths

DATE & DEPTH (m): 20.08.16 4.00; 22.08.16 30.00
CASING (DEPTH (m) & DIAMETER (mm)): 1.00 Ø112

DEPTH GL-(m)	N-Value (Blows / 30cm)	SAMPLE (Type & No.)	DEPTH GL-(m)
1.00		A-1	1.00 1.50
2.00		A-2	2.00 2.50
3.00	5/30	P-1	3.00 3.45
4.00	7/30	P-2	4.00 4.45
5.00	14/30	P-3	5.00 5.45
6.00	14/30	P-4	6.00 6.45
7.00	9/30	P-5	7.00 7.45
8.00	3/30	P-6	8.00 8.45
9.00	6/30	P-7	9.00 9.45
10.00	11/30	P-8	10.00 10.45
11.00	6/30	P-9	11.00 11.45
12.00	9/30	P-10	12.00 12.45
13.00	11/30	P-11	13.00 13.45
14.00	31/30	P-12	14.00 14.45
15.00	13/30	P-13	15.00 15.45
16.00	27/30	P-14	16.00 16.45
17.00	26/30	P-15	17.00 17.45
18.00	16/30	P-16	18.00 18.45
19.00	24/30	P-17	19.00 19.45
20.00	27/30	P-18	20.00 20.45
21.00	29/30	P-19	21.00 21.45
22.00	18/30	P-20	22.00 22.45
23.00	9/30	P-21	23.00 23.45
24.00	21/30	P-22	24.00 24.45
25.00	18/30	P-23	25.00 25.45
26.00	15/30	P-24	26.00 26.45
27.00	30/30	P-25	27.00 27.45
28.00	31/30	P-26	28.00 28.45
29.00	20/30	P-27	29.00 29.45
30.00	22/30	P-28	30.00 30.45
31.00			

Continues to next sheet

NOTES

Relative density	SPT N-Value (blow)	Consistency	SPT N-Value (blow)
Very loose	0 - 4	Very soft	under 2
Loose	4 - 10	Soft	2 - 4
Medium dense	10 - 30	Firm	5 - 8
Dense	30 - 50	Stiff	9 - 15
Very dense	over 50	Very stiff	16 - 30
		Hard	over 30

Sample key
- Disturbed sample (SPT sample)
- Undisturbed Sample (Piston sampler)
- Undisturbed Sample (Denison sampler)
- Rock core sample (Single core tube)
- Rock core sample (Double core tube)
- Rock core sample (Core Loss)
- Water sample

PMT Permeability Test
VS Vane Shear Test
PMT Pressuremeter Test

RQD (%)	Term
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Planar structure

Term	Spacing (mm)
Very thick	> 2000
Thick	600 - 2000
Medium	200 - 600
Thin	60 - 200
Very thin	20 - 60
Thickly laminated	6 - 20
Thinly laminated	< 6

Discontinuities

Term	Spacing (mm)
Very widely spaced	> 2000
Widely spaced	600 - 2000
Medium spaced	200 - 600
Closely spaced	60 - 200
Very closely spaced	20 - 60
Extremely closely spaced	< 20

Remarks

FUGEN CO., LTD.
Consulting Engineers
(Yangon Branch)

Revision No. Rev: 01
Revision Date 20.09.2016

BORE HOLE No. **BH-03**

BORING LOG

Job No. FKYB-2016-013
Sheet No. 2 OF 2

PROJECT NAME : Geological Survey on the supplement survey for Bago River Bridge Construction Project
LOCATION : Thanlyin Chin Kai Road, Thaketa Township
GROUND LEVEL : 5.21m
COORDINATE : E 203988.555 ; N 1859910.930 DEPTH : 50.00m
BORING EQUIPMENT : TOHO "D1"
BORING METHOD : Rotary Direct Circulation
ORIENTATION : Vertical
GROUND WATER LEVEL : 2.50m
DATE : 20.08.2016 ~ 24.08.2016

CLIENT

NIPPON KOEI CO., LTD.

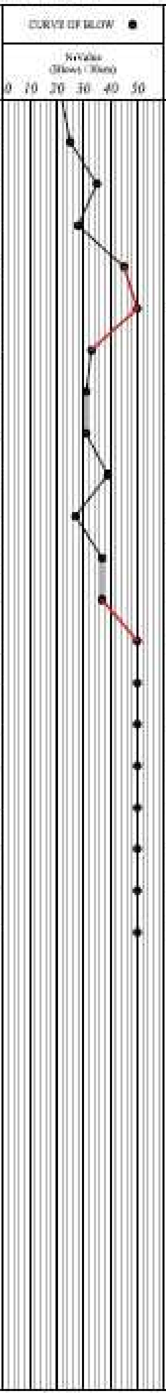
SCALE (m)	ELEVATION (m)	DEPTH (GL -m)	THICKNESS (m)	DIAGRAM	COLOUR	RELATIVE DENSITY OR CONSISTENCY	SOIL NAME	SOIL DESCRIPTION	DATE & DEPTH (m)	CASING (DEPTH (m) & DIAMETER mm)	WATER DEPTH (m)
31	-25.79	31.00	18.00		gray	Medium dense to dense	Silty SAND	Medium dense to dense, gray, moist, fine to medium grained, Silty SAND			
32	-26.79	32.00	1.00		gray	Very stiff	CLAY	Very stiff, gray, moist, low to medium plasticity, CLAY with fine grained sand			
36	-30.79	36.00	4.00		gray	Medium dense to very dense	Clayey SAND	Medium dense to dense, gray, moist, fine to medium grained, low plastic Clayey SAND			
44	-38.79	44.00	8.00		gray	Very stiff to hard	CLAY	Very stiff to hard, gray, moist, low to medium plasticity, CLAY with fine grained sand			
50	-45.16	50.37	6.37		gray to brownish gray	Very dense	Clayey SAND	Very dense, gray to brownish gray, moist, fine to medium grained, low plastic Clayey SAND	23.08.16 45.00 24.08.16 50.00		
51–61								This borehole is terminated at 50.00m, according to the termination criteria.			

STANDARD PENETRATION TEST
TEST METHOD (ASTM)

SAMPLING

DEPTH (GL -m)	N-Value (Blows / 30cm)	SAMPLE (Type & No.)	DEPTH (GL -m)	TCR (%)	SCR (%)	RQD (%)
31.00	25/30	P-29	31.00 31.45			
32.00	35/30	P-30	32.00 32.45			
33.00	28/30	P-31	33.00 33.45			
34.00	45/30	P-32	34.00 34.45			
35.00	50/27	P-33	35.00 35.42			
36.00	33/30	P-34	36.00 36.45			
37.00	31/30	P-35	37.00 37.45			
38.00	31/30	P-36	38.00 38.45			
39.00	39/30	P-37	39.00 39.45			
40.00	27/30	P-38	40.00 40.45			
41.00	37/30	P-39	41.00 41.45			
42.00	37/30	P-40	42.00 42.45			
43.00	50/30	P-41	43.00 43.45			
44.00	50/25	P-42	44.00 44.40			
45.00	50/25	P-43	45.00 45.40			
46.00	50/18	P-44	46.00 46.33			
47.00	50/18	P-45	47.00 47.33			
48.00	50/17	P-46	48.00 48.32			
49.00	50/17	P-47	49.00 49.32			
50.00	50/22	P-48	50.00 50.37			
51.00						
52.00						
53.00						
54.00						
55.00						
56.00						
57.00						
58.00						
59.00						
60.00						
61.00						

CURVE OF BLOW ●



NOTES

Relative density description		Consistency description	
Relative density	SPT N-Value	Consistency	SPT N-Value
Very loose	0 - 4	Very soft	under 2
Loose	4 - 10	Soft	2 - 4
Medium dense	10 - 30	Firm	5 - 8
Dense	30 - 50	Stiff	9 - 15
Very dense	over 50	Very stiff	16 - 30
		Hard	over 30

Sample key

●- 1	Disturbed sample (SPT sample)
▯ 1-	Undisturbed Sample (Piston sampler)
▯ 1-	Undisturbed Sample (Denison sampler)
▯	Rock core sample (Single core tube)
■	Rock core sample (Double core tube)
□	Rock core sample (Core Loss)
W-	Water sample
PRT	Permeability Test
VS	Vane Shear Test
PMT	Pressuremeter Test

RQD (%)	Term
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Planar structure

Term	Spacing (mm)
Very thick	> 2000
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Very thin	20 - 60
Thickly laminated	6 - 20
Thinly laminated	< 6

Discontinuities

Term	Spacing (mm)
Very widely spaced	> 2000
Widely spaced	600 - 2000
Medium spaced	200 - 600
Closely spaced	60 - 200
Very closely spaced	20 - 60
Extremely closely spaced	< 20

Remarks

FUKKEN CO., LTD.
Consulting Engineers
(Yangon Branch)

Revision No.	Rev. 02
Revision Date	19.09.2016

3-3 Design geotechnical parameters

The design geotechnical parameter is shown in below table.

Layer	N Average *1	Unit Weight “γ” (kN/m3)	Cohesion “c” (kN/m2)	Friction Angle “φ” *5 (°)	Modulus of Deformation “E” (kN/m2)
FILLED SOIL	4	18 *3	24 *4	0	1300 *6
CLAY-I	4	18 *2	24 *1	0	1300 *6
SILTY SAND-I	10	18 *2	0 *4	32	5000 *8
SANDY SILT	8	17 *3	48 *4	0	5600 *7
SILTY SAND-II	22	19 *3	0 *4	33	15400 *7
CLAY-II	21	18 *3	126 *4	0	14700 *7
CLAYEY SAND-I	35	19 *3	0 *4	33	24500 *7
CLAY-III	35	18 *3	210 *4	0	24500 *7
CLAYEY SAND-II	50	19 *3	0 *4	37	35000 *7
CLAY-IV	50	18 *3	300 *4	0	35000 *7

Source: JICA Study Team

*1 Maximum N value is 50
*2 Average values obtained by each test
*3 Referenced by Japanese Standard (NEXCO)
*4 Calculated by C=6N (referenced by Japanese Standard (NEXCO)). The value of sandy soil is 0.
*5 Calculated with N value using effective overburden pressure
*6 Test value obtained by unconfined compression test
*7 E=700N according to the worth value obtained by borehole lateral load test
*8 E=500N according to the worth value obtained by borehole lateral load test

3-4 Liquefaction evaluation of foundations

Possibility of liquefaction in the project area has been examined based on the results of soil investigation survey in the Supplemental F/S, and the reduction coefficient of geotechnical parameter was established as below table.

(a) 0≦x≦10

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01			5.922	1.263	1.093	0.269				
BH-02	3.393	0.617	1.827	0.407	1.078	0.293				
BH-03			3.953	0.780	1.044	0.247	0.910	0.231		
BH-04	2.111	0.395	2.517	0.548	1.432	0.365				
BH-05			0.942	0.186	1.396	0.324	0.912	0.232		
BH-06					1.103	0.267				
BH-07	1.109	0.197	0.968	0.186	0.953	0.242				
BH-08					1.425	0.315				
BH-09			1.433	0.269	1.207	0.264				
BH-10					1.155	0.248				
BH-11					1.130	0.257				
BH-12					3.210	0.859				
BH-13			10.138	2.207	6.886	1.920				
BH-14			1.832	0.407	1.400	0.366				
BH-5(13)					0.991	0.225				
Average	2.204	0.403	3.281	0.695	1.700	0.431	0.911	0.232		
DE	1		1		1		2/3		–	

(b) 10<x≦20

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01							0.975	0.250	1.163	0.286
BH-02							2.588	0.717	1.434	0.374
BH-03							1.034	0.266	1.147	0.283
BH-04									1.076	0.272
BH-05							1.409	0.362	1.071	0.263
BH-06							1.089	0.285	1.021	0.255
BH-07					0.970	0.261	1.301	0.348	0.974	0.247
BH-08					1.128	0.276	1.221	0.301	1.075	0.256
BH-09					1.089	0.256	1.263	0.301	1.224	0.283
BH-10							1.888	0.447	1.228	0.285
BH-11							1.214	0.287	1.200	0.277
BH-12					1.010	0.278	0.995	0.268	0.854	0.218
BH-13					1.031	0.286	1.007	0.272	1.182	0.302
BH-14					1.168	0.310	13.839	3.613	1.319	0.330
BH-5(13)					0.851	0.201			1.386	0.320
Average					1.035	0.267	2.294	0.594	1.157	0.283
DE	–		–		1		1		1	

Source: JICA Study Team

3-5 Horizontal Seismic Coefficient

Horizontal seismic coefficient is as follows.

1) Safety of Elements (Internal Stability) Note: Same for L type retaining wall.

$$k_{h1} = C_z \cdot k_{h0} = 0.18$$

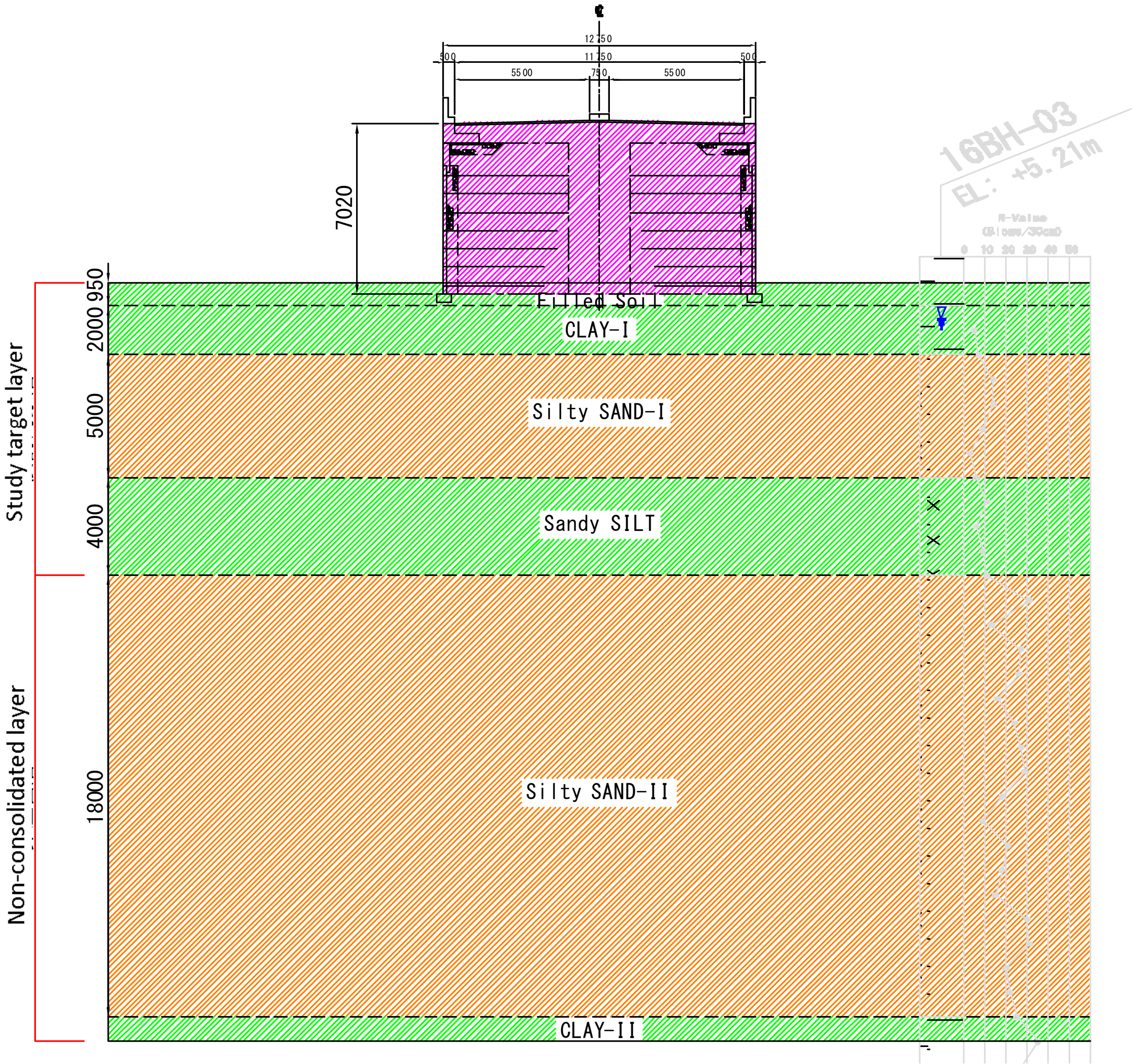
[Ground type：Level 1, Earthquake ground motion: TypeⅢ]

3-6 Live load on the wall

Unit live load of ql=11.6kN/m2 is subjected on the top of wall by following the AASHTO LRFD Bridge Design Specification.

3-7 Design calculation model

The design calculation was conducted at the section where the wall height is the maximum (7.020m). "Silty SAND-II." is considered as the expected bearing layer for the mechanically-stabilised earth wall because N-value is continuously more than 20.



4. Considerations for Consolidation Settlement

Because of consolidation test has not been pressure consolidation settlement study do in the following ways.

(1) Cohesive soil

The natural moisture of the soil test results based on road earthworks-consider using the oedometer test results for General viscous soil layer on soft ground countermeasure construction guidelines P.119, moisture (Wn) each.

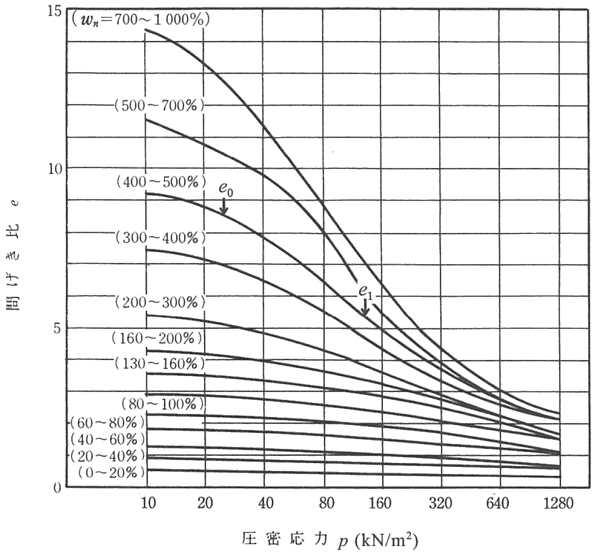
(2) Sand layer

About sand N values of each layer based on road earthworks-on soft ground countermeasure construction guidelines P.125

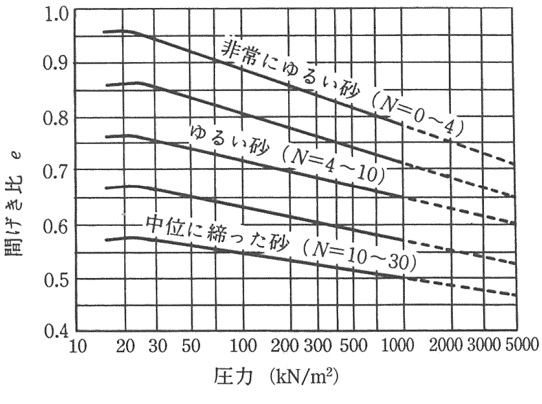
Sand pressure-consider using a void ratio curve (B.K.Hough).

Soil parameters	N values How to calculate	UNIT WEIGHT OF SOIL (kN/m^3)	Evaluation Division
FILLED SOIL	Δe method	18.0	(W$_n$=20～40%)
CLAY- Ⅰ	Δe method	18.0	(W$_n$=20～40%)
SILTY SAND- Ⅰ	B.K.Hough	18.0	Shut down to medium sand (N=10～30)
SANDY SILT	Δe method	17.0	(W$_n$=20～40%)
SILTY SAND- Ⅱ	Non-compacted layers※1	19.0	―
CLAY- Ⅱ	Non-compacted layers※1	18.0	―

※1. Road earth retaining wall guideline than a sandy soil N ≥ 20, clay N ≥ and non-compacted layers to 15, which can be considered the support layer.



解図 3－7 自然含水比 w_n と e-log$p$ 曲線の関係（高速道路，一般国道）



参図 5－1 砂の圧力－間隙比曲線（B. K. Hough）[4)]

―*Soft Ground Measures Guidelines* P.50― ―*Soft Ground Measures Guidelines* P.125―

Soft ground countermeasure construction Guide and (Terre Armee)-reinforced soil wall for the acceptable settlement in the consolidation,

Construction design and construction documentation, listed below.

Considering the safety side, this time in the retaining wall of the bridge connecting from the settlement allowance shall be 10 cm.

	Allowable Settlement	
Road Earthwork: Soft Ground Measures Guidelines 2012.	10〜30cm (Commencement of operations in 3 year's)	
Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed.	10〜20cm (Connection of the bridges and viaduct)	15〜30cm (Other structures)

On permanent settlement of the verification does not exceed the settlement amount goals in designing earthworks structures on soft ground construction and operations at the expected settlement amount.

—*Road Earthwork: Soft Ground Measures Guidelines 2012.* P.119—

To evaluate residual settlement in the design goals and the tolerance the corresponding effect of settlement on on earthwork structures feature, based on Office Edition structure mounting structure and road facilities and maintenance management. As the tolerance value of residual settlement in the design goals and the mounting structure in Central fill paving complete or acceptable after 3 years in 10 cm-many cases came as 30 cm.

—*Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed.* P.118—

5. Calculation Results

5-1 Necessity of countermeasures and depth of improvement

The necessity of countermeasures is as follows from (1) consolidation settlement analysis and (2) external stability study of reinforced soil wall.

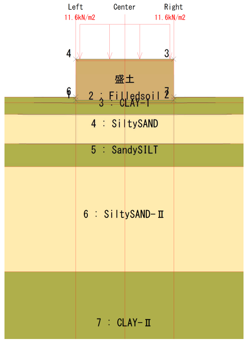
(1) Consolidation settlement analysis

From the consolidation analysis result of the largest settlement at Center part, the total settlement is 31 cm. And it cannot satisfy the allowable value of 10 cm without treatment. Therefore, the ground improvement is necessary as a countermeasure.

As a countermeasure, if the ground improvement is made up to the silty sand layers from the 2nd to the 4th layers, and then the remaining settlement of the silty sand layer becomes 7.345cm which satisfy the allowable value of 10cm.

From the above, the required improvement depth in consolidation settlement analysis is up to the silty sand layer (depth: 7.150 m).

項目	Left	Center	Right
地表高	CDL 10.49	CDL 10.36	CDL 10.45
沈下量	0.18m	0.31m	0.22m



◆ Center In the pneumatic dense study results

All layer consolidation

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	5	8	11	13	16	22	31	45	77	---
Settlement	3.116	6.232	9.348	12.464	15.580	18.696	21.812	24.928	28.045	31.161

The second layer:Filledsoil

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	6	9	12	15	18	23	29	38	55	---
Settlement	0.386	0.772	1.158	1.543	1.929	2.315	2.701	3.087	3.473	3.859

The third layer:CLAY-1

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	6	10	12	15	18	23	29	38	56	---
Settlement	1.407	2.813	4.220	5.626	7.033	8.439	9.846	11.253	12.659	14.066

The fourth layer:SiltySAND

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	2	4	5	7	8	10	11	13	14	---
Settlement	0.576	1.152	1.728	2.303	2.879	3.455	4.031	4.607	5.183	5.759

The fifth layer:SandySILT

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	9	13	19	27	38	52	70	97	149	---
Settlement	0.748	1.496	2.243	2.991	3.739	4.487	5.234	5.982	6.730	7.478

The sixth layer:SiltySAND-Ⅱ

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	---	---	---	---	---	---	---	---	---	---
Settlement	---	---	---	---	---	---	---	---	---	---

The seventh layer:CLAY-Ⅱ

Consolidation ratio	10	20	30	40	50	60	70	80	90	100
The progress days	---	---	---	---	---	---	---	---	---	---
Settlement	---	---	---	---	---	---	---	---	---	---

(2) External stability study of reinforced soil wall

① Analysis results: Without improvement

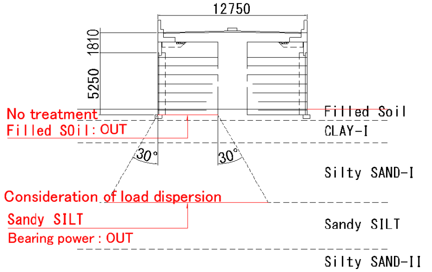
From the analysis results of external stability study of reinforced soil wall, it is impossible to satisfy the allowable value / necessary safety factor for each analysis item except for overturning analysis without improvement. Therefore, the ground improvement is necessary as a countermeasure.

In addition, as an analysis on the depth of improvement, when the vertical reaction force degree of the reinforced soil wall was dispersed to the lower side of the silty sand layer which was determined by consolidation settlement analysis and it cannot satisfy the allowable value of bearing force at the sandy silt layer.

EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	1.454 (1.500)	×	0.482 (1.200)	×
OVERTUENING	ECCENTLICTY e (m)	-1.011 (0.655)	○	-0.310 (1.310)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m²)	220.046 (31.177)	×	201.455 (36.353)	×
BEARING CAPACITY STABILITY OF REPLACED FOUNDATION UNDER R/E BACK FILL	VERTICAL REACTION qos (kN/m²)	203.115 (68.389)	×	197.284 (98.104)	×
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m²)	141.354 (45.933)	×	128.231 (68.899)	×



←No processing of Filled Soil bearing capacity study results (below the embankment of the reinforced soil wall)

←Bearing capacity results in the top surface SandySILT

←No processing of Filled Soil bearing capacity results (Foundation beneath the wall of reinforced soil wall)

② Analysis results: With improvement

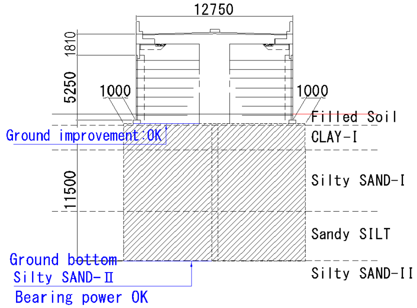
Based on results of analysis without improvement, we decided the silty sand - II layer for the supporting layer by making the sandy silt layer with improvement, and it can satisfy the allowable value of bearing force.

From the above, the required improvement depth in external stability study of reinforced soil wall analysis is up to the sandy silt layer (depth: 11.500 m).

EXAMINATION OF STABILITY OF R/E ITSELF (EXAMINATION OF EXTERNAL STABILITY)

( ) ALLOWABLE VALUE

ITEM		ORDINARY		EARTHQUAKE	
		RESULT	JUDGE	RESULT	JUDGE
SLIDING	SLIDING SAFETY FACTOR Fs	7.642 (1.500)	○	2.339 (1.200)	○
OVERTUENING	ECCENTLICTY e (m)	-1.024 (0.655)	○	-0.310 (1.310)	○
BEARING CAPACITY UNDER R/E BACL FILL	VERTICAL REACTION qs (kN/m²)	223.045 (223.770)	○	201.455 (335.650)	○
BEARING CAPACITY STABILITY OF REPLACED FOUNDATION UNDER R/E BACK FILL	VERTICAL REACTION qos (kN/m²)	354.527 (1012.063)	○	340.247 (1296.009)	○
BEARING CAPACITY UNDER CONCRETE FOUNDATION	VERTICAL REACTION qw (kN/m²)	208.838 (223.770)	○	186.596 (335.650)	○



←No processing of Filled Soil bearing capacity study results (below the embankment of the reinforced soil wall)

←Bearing capacity results in the top surface SandySILT

←No processing of Filled Soil bearing capacity results (Foundation beneath the wall of reinforced soil wall)

(3) Summary

In this place, it is difficult to build a retaining wall without any treatment, so ground improvement is necessary.

About the depth of improvement, 11.5m shall be used since the required depth (11.500 m) determined by the bearing force of the reinforced soil wall is deeper than the depth (7.500 m) determined by consolidation settlement,

5-2 Study on soil improvement

Here the soil improved format, and stability calculation.

(1) Liquefaction evaluation

SandySILT formation in the article any where near liquefaction potential of the target layer of soil liquefaction test Has become.
Format from this soil liquefaction for can and should be.

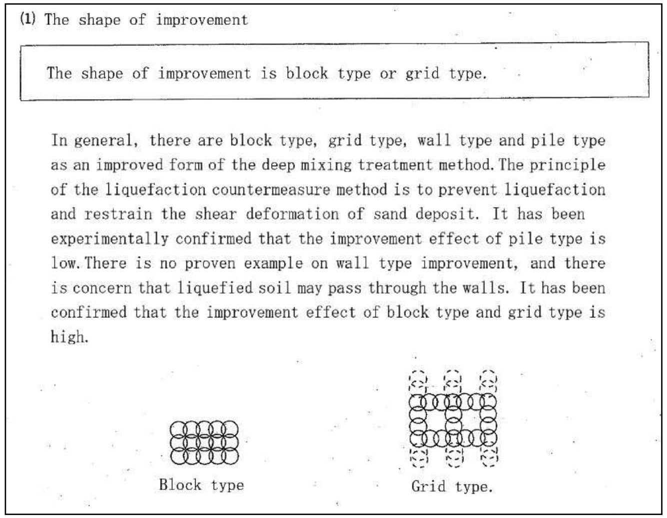
(a) 0≦x≦10

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01			5.922	1.263	1.093	0.269				
BH-02	3.393	0.617	1.827	0.407	1.078	0.293				
BH-03			3.953	0.780	1.044	0.247	0.910	0.231		
BH-04	2.111	0.395	2.517	0.548	1.432	0.365				
BH-05			0.942	0.186	1.396	0.324	0.912	0.232		
BH-06					1.103	0.267				
BH-07	1.109	0.197	0.968	0.186	0.953	0.242				
BH-08					1.425	0.315				
BH-09			1.433	0.269	1.207	0.264				
BH-10					1.155	0.248				
BH-11					1.130	0.257				
BH-12					3.210	0.859				
BH-13			10.138	2.207	6.886	1.920				
BH-14			1.832	0.407	1.400	0.366				
BH-5(13)					0.991	0.225				
ave	**2.204**	**0.403**	**3.281**	**0.695**	**1.700**	**0.431**	**0.911**	**0.232**		
DE	**1**		**1**		**1**		**2/3**		-	

Range of FL	Depth−x	Dynamic Shear Strength Ratio−R	
		R≦0.3	0.3<R
FL≦1/3	0≦x≦10	0	1/6
	10<x≦20	1/3	1/3
1/3<FL≦2/3	0≦x≦10	1/3	2/3
	10<x≦20	2/3	2/3
2/3<FL≦1	0≦x≦10	2/3	1
	10<x≦20	1	1

(2) Type of soil

As considering liquefaction and liquefaction soil improvement format protection design and construction manual (draft) than block-shaped , underlying lattice. More blocky, grid is a modified pile sporadically and is advantageous in economic efficiency by segment , adopts the format of a lattice, we consider.



(1) The shape of improvement

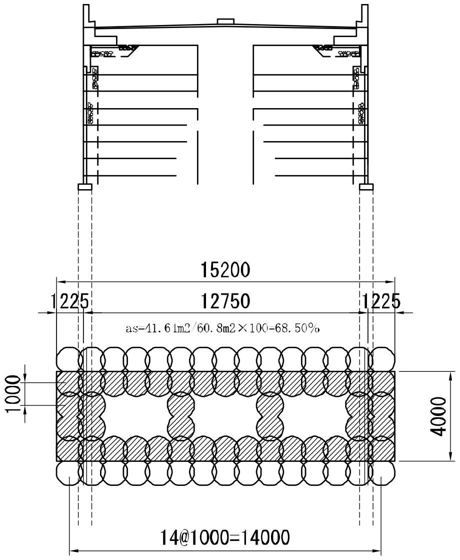
The shape of improvement is block type or grid type.

In general, there are block type, grid type, wall type and pile type as an improved form of the deep mixing treatment method. The principle of the liquefaction countermeasure method is to prevent liquefaction and restrain the shear deformation of sand deposit. It has been experimentally confirmed that the improvement effect of pile type is low. There is no proven example on wall type improvement, and there is concern that liquefied soil may pass through the walls. It has been confirmed that the improvement effect of block type and grid type is high.

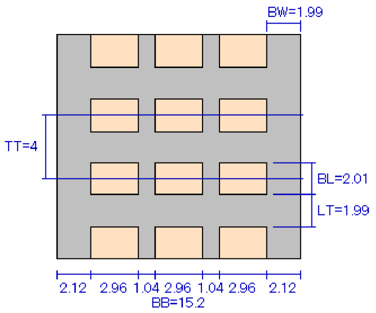
—*Soft Ground Measures Guidelines* P.114—

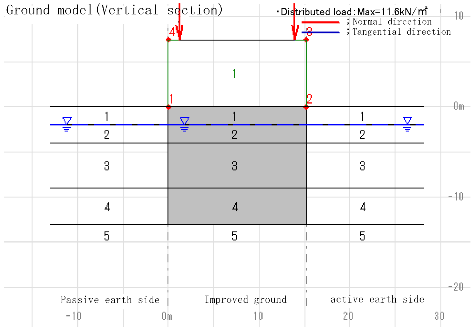
(3) Stability study of soil strength

Indicates required improved depth of reinforcement to the aforementioned mud and improved width and improved form based on study results.



Horizontal cross section of improved body (m)





```
1. Design condition

1.1. Standard safety factor · allowable unit stress (Earthquake)
 ─────────────────────────────────────────────────
     ・Slide safety factor             Fs  : 1.000
     ・Fall safety factor              Fs  : 1.100
 ─────────────────────────────────────────────────

1.2. Design horizontal seismic intensity
 ─────────────────────────────────────────────────
     ・Design seismic intensity for earth
      pressure calculation                kh1:        0.180
     ・Design seismic intensity for
      hydrodynamic pressure calculation   kh2:        0.180
     ・Design seismic intensity for improved
      ground inertial force calculation   kh3:        0.145
     ・Design seismic intensity for embankment
      inertial force calculation          kh4:        0.180
 ─────────────────────────────────────────────────

1.3. Improvement method
 ─────────────────────────────────────────────────
     ・Improvement method              :  Improvement underneath

 ─────────────────────────────────────────────────

1.4. Shape of the improved ground
 ─────────────────────────────────────────────────
     ・Improvement rate (%)                         ap  :          68.500
     ・Improvement width (m)                         B  :          15.200
     ・Improved depth (m)                            D  :          11.300
     ・Extension direction 1 unit length(m)         TT:             4.000
     ・Improvement length at the
      examination point of vertical shear (m)      LT:             1.990
     ・Extension length of extraction
      grid lattice                      (m)        BL:             2.010
     ・Thickness of the lattice improvement wall (m) BW:            1.990
     ・Dimensions of the sectional direction (m) [From the left] :
          2.12, 2.96, 1.04, 2.96, 1.04, 2.96, 2.12
 ─────────────────────────────────────────────────


4.5. Stability study result list
 ─────────────────────────────────────────────────────────
            Study item                        result Allowable value Judgment
 ─────────────────────────────────────────────────────────
 External :  Sliding        Fs                  2.186      1.000      OK
 stability ──────────────────────────────────────────────
             Overturning    Fs                  3.370      1.100      OK
           ──────────────────────────────────────────────
             Vertical       q   (KN/m2)       326.960      ---       ---
             reaction       qa  (KN/m2)                  1012.630    OK
 ─────────────────────────────────────────────────────────
```