5.2 Comparison of Two Cases of one inlet pipe and two inlet pipes

5.2.1 Case Settings

In order to justify the trunk sewer plan in which the trunk sewer of 2,200mm in diameter reaches to the WWTP at the invert level of about 28.175m below the ground surface, the following study was conducted at the early stage in the JICA Supplementary Study.

In addition to the above plan and design as of Case 1: the original case in which the inlet pipe of 2,200mm in diameter to the WWTP is installed at about 28.175m below the ground surface (invert level).

the alternative Case 2 is set as follows: the alternative case in which two inlet pipes come to the WWTP, one inlet pipe of 1,800mm is the main trunk sewer covering major sewerage service area and another inlet pipe of 1,350mm is a sub trunk sewer covering the areas near the WWTP.

Figure 5.2.1 shows the major service areas covered by the main trunk sewer (1) and the service area near the WWTP covered by the trunk sewer (2). The service area near the WWTP and the trunk sewers (2) were selected among several options considering the road conditions.

Figure 5.2.1 Major service areas (green color) covered by the main trunk sewer (1) and the service area near the WWTP (blue color) covered by the trunk sewer (2)

Major sewerage facilities needed are summarized in the table below.

Table 5.2.1 Major features of the alternatives of trunk sewer planning

Item	Case 1	Case 2	Remarks
1. Design Flows			
1.1 Ave. Daily Flow	197,900 m^3/d =200,000 m^3/d	1) Main Area: 141,700 m^3/d 2) Area near WWTP: 56,200m^3/d	
1.2 Max. Hourly Flow	382,800 m^3/d	1) Main Area: 266,400 m^3/d 2) Area near WWTP: 112,400m^3/d	
2.1 Trunk Sewers (1)	a) 2,000mm, b) 1,396 m, followed by a) 2,200mm, b) 955m, c) -28.175m	a) 1,800mm, b) 2,351m c) -29.970m	a) diameter, b) length, c) invert level
2.2 Trunk Sewer (2)		a) 1,350 mm, b) 955m c) -15.910m	
3. Pumping Facilities			
3.1 Type	Vertical shaft Volute type mixed flow pump	same as Case 1	
3.2 Diameter	700 mm	600 mm	
3.3 Capacity	67 m^3/min	46 m^3/min and 39 m^3/min	
3.4 Pump Head	34.90 m	35.0m and 23.0m	
3.5 Motor Output	560 kW	400 kW and 250 kW	
3.6 Numbers	5 nos. include one stand-by	5 nos. include one stand-by and 3 nos. include one stand-by	

The sewer capacity calculation for Case 2 is referred to Table 5.2.2. The profiles of sewers needed are referred to the drawings attached finally for you reference.

The pumping facilities design for both cases are referred to Table 5.2.3.

5.2.2 Cost Comparisons

The cost required for both cases are estimated and compared.

First, the difference of construction cost of two cases are estimated as shown in Table 5.2.4.
The construction cost of Case 2 is higher as 128 million Japanese yen than that of Case 1.

Second, the power cost as of major operation costs for two cases are estimated and compared as shown in Table 5.2.5. The annual power cost of Case 2 is lower as 9.3 million Japanese yen than that of Case 1.

Therefore, the construction cost difference in two cases of 128 million Japanese yen equivalents to the about 14 year of power costs needed.

5.2.3 Selection of the appropriate trunk sewer plan

In addition to the cost comparison, a construction work for the structure of pumping station at the WWTP is studied.

In the Case 2, since two trunk sewers having different invert levels come into the WWTP, then the receiving pumping well structures are more complicated to construct at a limited land space at the Pluit site. The construction of complicated civil structures needs higher cost.

The required pumps number are larger than that of Case 1 as shown in the previous section. More O&M of pumping facilities are required for the Case 2 due to the increased number of pumping equipment.

These comparison results suggest that the trunk sewer plan in Case 1 would be appropriate.

Table 5.2.2 Trunk Sewer Capacity Calculation for Case 2

Trunk Sewers West, covering the area near WWTP **200** lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (o/oo)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end
ST-73	MT-32	**756**	**756**	**55.0**	**55.0**	**40,142**	**40,142**	**8,028**	**0**	**8,028**	**2.917**	**0.272**	**0**	**0.272**	**700**	**2.2**	**1.129**	**0.434**		
ST-74	MT-32	615	615	10.7	10.7	4,498	4,498	900	0	900	4.086	0.043	0	0.043	350	3.5	0.897	0.086		
MT-32	MT-33	**704**	**2,075**	**30.3**	**96.0**	**12,746**	**57,386**	**11,477**	**0**	**11,477**	**2.761**	**0.367**	**0**	**0.367**	**800**	**2.0**	**1.176**	**0.591**		
ST-75	MT-33	1,147		85.0	85.0	43,702	43,702	8,740	0	8,740	2.879	0.292	0	0.292	700	2.4	1.179	0.454		
MT-33	MT-35	**629**		**81.1**	**262.1**	**18,470**	**119,558**	**23,912**	**0**	**23,912**	**2.466**	**0.683**	**0**	**0.683**	**1,000**	**1.8**	**1.295**	**1.017**		
MT-17	MT-18	1,524		96.7	96.7	43,135	43,135	8,627	0	8,627	2.885	0.289	0	0.289	700	2.4	1.179	0.454		
ST-72	MT-18	482		18.7	18.7	4,160	4,160	832	0	832	4.135	0.040	0	0.040	350	3.5	0.897	0.086		
MT-18	MT-35	34		0.0	115.4	0	47,295	9,459	0	9,459	2.844	0.312	0	0.312	800	2.0	1.176	0.591		
MT-35	MT-34	**545**		**72.2**	**449.7**	**16,441**	**183,294**	**36,659**	**0**	**36,659**	**2.309**	**0.980**	**0**	**0.980**	**1,100**	**1.6**	**1.301**	**1.237**		
MT-34	**ST-88**	**671**		**40.0**	**489.7**	**8,897**	**192,191**	**38,438**	**0**	**38,438**	**2.292**	**1.020**	**0**	**1.020**	**1,200**	**1.6**	**1.379**	**1.559**		
ST-80	ST-77	**787**		**64.4**	**64.4**	**6,357**	**6,357**	**1,271**	**0**	**1,271**	**3.874**	**0.057**	**0**	**0.057**	**350**	**3.5**	**0.897**	**0.086**		
ST-76	ST-77	567		110.1	110.1	10,866	10,866	2,173	0	2,173	3.567	0.090	0	0.090	450	2.8	0.949	0.151		
ST-77	ST-78	**2,558**		**116.9**	**291.4**	**11,534**	**28,757**	**5,751**	**0**	**5,751**	**3.071**	**0.205**	**0**	**0.205**	**600**	**2.6**	**1.107**	**0.313**		
ST-81	ST-78	468		40.1	40.1	8,921	8,921	1,784	0	1,784	3.677	0.076	0	0.076	400	3.0	0.908	0.114		
ST-78	ST-79	**128**		**0.0**	**331.5**	**0**	**37,678**	**7,536**	**0**	**7,536**	**2.945**	**0.257**	**0**	**0.257**	**700**	**2.2**	**1.129**	**0.434**		
ST-82	ST-79	790		29.4	29.4	2,899	2,899	580	0	580	4.372	0.030	0	0.030	300	2.8	0.941	0.067		
ST-83	ST-79	475		43.4	43.4	9,654	9,654	1,931	0	1,931	3.633	0.082	0	0.082	400	3.5	0.980	0.123		
ST-79	**ST-88**	**836**		**0.0**	**404.3**	**0**	**50,231**	**10,046**	**0**	**10,046**	**2.818**	**0.328**	**0**	**0.328**	**800**	**2.2**	**1.234**	**0.620**		

Table 5.2.2 Trunk Sewer Capacity Calculation for Case 2

Trunk Sewers West, covering the area near WWTP **200** lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (o/oo)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end
ST-88	ST-89	**323**		**0.0**	**894.0**	**0**	**242,422**	**48,484**	**0**	**48,484**	**2.211**	**1.241**	**0**	**1.241**	**1,350**	**1.5**	**1.444**	**2.067**	**-13.925**	**-14.410**
ST-89	**ST-90**	**640**		**144.0**	**1,038.0**	**14,204**	**256,626**	**51,325**	**0**	**51,325**	**2.192**	**1.303**	**0**	**1.303**	**1,350**	**1.5**	**1.444**	**2.067**	**-14.460**	**-15.360**
This trunk sewers are installed parallel along the trunk sewer east																				
ST-84	ST-87	**2,411**		**190.2**	**190.2**	**18,759**	**18,759**	**3,752**	**0**	**3,752**	**3.279**	**0.143**	**0**	**0.143**	**600**	**2.4**	**1.064**	**0.301**	**-2.000**	**-7.343**
ST-86	ST-87	1,677		55.9	55.9	5,518	5,518	1,104	0	1,104	3.959	0.051	0	0.051	350	3.5	0.897	0.086	-2.320	-8.939
ST-87	**ST-90**	**100**		**0.0**	**246.1**	**0**	**24,277**	**4,855**	**0**	**4,855**	**3.152**	**0.178**	**0**	**0.178**	**600**	**2.4**	**1.064**	**0.301**	**-9.388**	**-9.678**
ST-90	**WWTP**	**315**		**0.0**	**1,284.1**	**0**	**280,903**	**56,181**	**0**	**56,181**	**2.162**	**1.406**	**0**	**1.406**	**1,350**	**1.5**	**1.444**	**2.067**	**-15.510**	**-15.910**
This trunk sewers are installed parallel along the trunk sewer east																				
										0.2837										

Trunk Sewer Central, covering major sewerage area **200** lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (o/oo)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end
ST-6	ST-7	1,543	1,543	57.7	57.7	2,358	2,358	472	0	472	4.513	0.025	0	0.025						
ST-8	ST-7	168	168	2.0	2.0	81	81	16	0	16	7.600	0.002	0	0.002						
ST-7	ST-2	27	1,738	0.0	59.7	0	2,439	488	0	488	4.490	0.026	0	0.026						
ST-1	ST-2	2,434	2,434	125.5	125.5	12,614	12,614	2,523	0	2,523	3.486	0.102	0	0.102						
ST-2	ST-3	389	2,823	13.9	199.1	1,399	16,452	3,290	0	6,301	3.028	0.116	0	0.116						
ST-9	ST-3	752	752	74.3	74.3	7,471	7,471	1,494	0	1,494	3.779	0.066	0	0.066						
ST-3	ST-4	40		0.0	273.4	0	23,923	4,785	0	4,785	3.159	0.175	0	0.175						
ST-10	ST-4	637	637	16.3	16.3	1,635	1,635	327	0	327	4.775	0.019	0	0.019						
ST-4	ST-5	520		14.0	303.7	1,406	26,964	5,393	0	5,393	3.101	0.194	0	0.194						
ST-11	ST-5	602	602	31.7	31.7	9,566	9,566	1,913	0	1,913	3.638	0.081	0	0.081						
ST-5	MT-1	278	880	0.0	335.4	0	36,530	7,306	0	7,306	2.960	0.251	0	0.251						
ST-12	MT-1	653	653	90.1	90.1	19,640	19,640	3,928	0	3,928	3.256	0.149	0	0.149						
MT-1	MT-2	939	1,592	0.0	425.5	0	56,170	11,234	0	11,234	2.770	0.361	0	0.361						
ST-13	MT-2	964	964	54.5	54.5	13,430	13,430	2,686	0	2,686	3.453	0.108	0	0.108						
MT-2	MT-3	108		0.0	480.0	0	69,600	13,920	0	13,920	2.680	0.432	0	0.432						
ST-14	MT-3	814	814	107.2	107.2	9,830	9,830	1,966	0	1,966	3.623	0.083	0	0.083						
MT-3	MT-4	621		0.0	587.2	0	79,430	15,886	0	15,886	2.626	0.483	0	0.483						
ST-15	MT-4	588	588	34.8	34.8	5,806	5,806	1,161	0	1,161	3.929	0.053	0	0.053						
MT-4	MT-5	38		0.0	622.0	0	85,236	17,047	0	17,047	2.597	0.513	0	0.513						
ST-16	MT-5	634	634	65.5	65.5	6,703	6,703	1,341	0	1,341	3.842	0.060	0	0.060						
ST-17	MT-5	1,001	1,001	38.2	38.2	6,382	6,382	1,276	0	1,276	3.872	0.058	0	0.058						
MT-5	MT-6	803		6.1	731.8	2,631	100,952	20,190	0	20,190	2.531	0.592	0	0.592						
ST-18	MT-6	579	579	96.8	96.8	1,028	1,028	206	0	206	5.127	0.013	0	0.013						
MT-6	MT-7	710		7.2	835.8	3,114	105,094	21,019	0	21,019	2.515	0.612	0	0.612						

Trunk Sewer Central, covering major sewerage area **200** lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (‰)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end
ST-19	MT-7	263	263	42.4	42.4	450	450	90	0	90	5.825	0.007	0	0.007						
MT-7	MT-8	1,588		14.1	892.3	6,072	111,616	22,323	0	22,323	2.492	0.644	0	0.644						
ST-20	MT-8	734	734	33.6	33.6	6,280	6,280	1,256	0	1,256	3.881	0.057	0	0.057						
MT-8	MT-9	113		0.0	925.9	0	117,896	23,579	0	23,579	2.471	0.675	0	0.675						
ST-21	MT-9	105	105	33.4	33.4	6,232	6,232	1,246	0	1,246	3.886	0.057	0	0.057						
MT-9	MT-10	118		35.2	994.5	6,574	130,702	26,140	0	26,140	2.432	0.736	0	0.736						
ST-22	MT-10	2,248	2,248	25.4	25.4	8,562	8,562	1,712	0	1,712	3.701	0.074	0	0.074						
MT-10	MT-11	37		0.0	1,019.9	0	139,264	27,853	0	27,853	2.408	0.777	0	0.777						
ST-23	ST-24	1,124	1,124	28.4	28.4	4,020	4,020	804	0	804	4.157	0.039	0	0.039						
ST-25	ST-24	542	542	28.4	28.4	4,020	4,020	804	0	804	4.157	0.039	0	0.039						
ST-24	MT-11	1,222		34.4	91.2	7,582	15,622	3,124	0	3,124	3.373	0.122	0	0.122						
MT-11	MT-12	293		0.0	1,111.1	0	154,886	30,977	0	30,977	2.369	0.850	0	0.850						
ST-26	ST-27	1,048	1,048	49.6	49.6	7,022	7,022	1,404	0	1,404	3.815	0.062	0	0.062						
ST-28	ST-27	53	53	21.1	21.1	6,830	6,830	1,366	0	1,366	3.831	0.061	0	0.061						
ST-27	MT-12	1,222	2,323	35.1	105.8	11,390	25,242	5,048	0	5,048	3.133	0.184	0	0.184						
MT-12	MT-13	1,479		0.0	1,216.9	0	180,128	36,026	0	36,026	2.315	0.966	0	0.966						
ST-29	MT-20	663	663	81.1	81.1	41,245	41,245	8,249	0	8,249	2.905	0.278	0	0.278						
MT-20	MT-13	1,285	1,948	67.9	149.0	61,102	102,347	20,469	0	20,469	2.525	0.599	0	0.599						
MT-13	MT-14	33		0.0	1,365.9	0	282,475	56,495	0	56,495	2.160	1.413	0	1.413						
MT-21	MT-14	1,647	1,647	139.0	139.0	100,270	100,270	20,054	0	20,054	2.533	0.588	0	0.588						
MT-14	MT-15	642		0.0	1,504.9	0	382,745	76,549	0	76,549	2.061	1.827	0	1.827						
ST-30	MT-15	961	961	20.4	20.4	11,126	11,126	2,225	0	2,225	3.554	0.092	0	0.092						
MT-15	**AT-1**	329		0.0	1,525.3	0	393,871	78,774	0	78,774	2.052	1.871	0	1.871						

Trunk Sewer East, covering major sewerage area 200 lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)		Ground Elevation (m)		Earth Covering (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (o/oo)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end	Upper end	Lower end	Upper end	Lower end
ST-31	ST-32	563	563	9.0	9.0	1,999	1,999	400	0	400	4.629	0.022	0	0.022	300	2.8	0.941	0.067	6.869	6.118	8.69	10.09	1.39	3.54
ST-42	ST-32	217		27.0	27.0	6,013	6,013	1,203	0	1,203	3.907	0.055	0	0.055	400	3.0	0.908	0.114	6.068	3.432	10.09	7.53	3.59	3.66
ST-32	ST-33	696		0.0	36.0	0	8,012	1,602	0	1,602	3.739	0.070	0	0.070	400	3.0	0.908	0.114						
ST-43	ST-33	246	246	40.8	40.8	9,098	9,098	1,820	0	1,820	3.666	0.078	0	0.078	450	3.0	0.982	0.156						
ST-33	ST-34	540	540	0.0	76.8	0	17,110	3,422	0	3,422	3.326	0.132	0	0.132	600	2.6	1.107	0.313	3.232	1.478	7.53	8.87	3.65	6.74
ST-34	ST-35	60		0.0	76.8	0	17,110	3,422	0	3,422	3.326	0.132	0	0.132	600	2.6	1.107	0.313	1.428	1.272	8.87	8.54	6.79	6.62
ST-44	ST-35	364	364	22.8	22.8	4,555	4,555	911	0	911	4.078	0.043	0	0.043	350	3.5	0.897	0.086						
ST-35	ST-36	351		0.0	99.6	0	21,665	4,333	0	4,333	3.207	0.161	0	0.161	600	2.6	1.107	0.313	1.222	0.160	8.54	6.95	6.67	6.14
ST-45	ST-36	512	512	32.6	32.6	3,554	3,554	711	0	711	4.237	0.035	0	0.035	350	3.5	0.897	0.086						
ST-36	ST-37	803		0.0	132.2	0	25,219	5,044	0	5,044	3.133	0.183	0	0.183	600	2.4	1.064	0.301	0.110	−2.267	6.95	6.51	6.19	8.13
ST-46	ST-37	561	561	45.8	45.8	5,002	5,002	1,000	0	1,000	4.020	0.047	0	0.047	350	3.5	0.897	0.086						
ST-37	ST-38	459		0.0	178.0	0	30,221	6,044	0	6,044	3.047	0.214	0	0.214	700	2.4	1.179	0.454	−2.367	−3.718	6.51	5.01	8.12	7.97
ST-47	ST-38	296	296	31.9	31.9	4,086	4,086	817	0	817	4.147	0.040	0	0.040	350	3.5	0.897	0.086						
ST-38	ST-39	905		0.0	209.9	0	34,307	6,861	0	6,861	2.988	0.238	0	0.238	700	2.4	1.179	0.454	−3.768	−6.439	5.01	3.98	8.02	9.66
ST-48	ST-39	180	180	39.6	39.6	421	421	84	0	84	5.887	0.006	0	0.006	200	3.0	0.743	0.023						
ST-39	ST-40	941		0.0	249.5	0	34,728	6,946	0	6,946	2.983	0.240	0	0.240	700	2.4	1.179	0.454	−6.489	−9.300	3.98	4.45	9.71	12.99
ST-49	ST-40	954	954	64.7	64.7	687	687	137	0	137	5.460	0.009	0	0.009	200	3.0	0.743	0.023						
ST-40	ST-41	471		0.0	314.2	0	35,415	7,083	0	7,083	2.974	0.244	0	0.244	700	2.4	1.179	0.454	−9.350	−10.781	4.45	3.39	13.04	13.41
ST-50	ST-41	390	390	12.6	12.6	1,434	1,434	287	0	287	4.872	0.017	0	0.017	250	2.8	0.833	0.041						
ST-41	MT-22	1,084		72.7	399.5	8,634	45,483	9,097	0	9,097	2.861	0.302	0	0.302	800	2.2	1.234	0.620	−10.881	−13.466	3.39	1.86	13.41	14.46
MT-22	MT-23'	982		82.1	481.6	41,549	87,032	17,406	0	17,406	2.589	0.522	0	0.522	900	1.8	1.207	0.768	−13.566	−15.383	1.86	1.71	14.45	16.12
MT-23'	MT-24'	690		0.0	481.6	0	87,032	17,406	0	17,406	2.589	0.522	0	0.522	900	1.8	1.207	0.768	−15.433	−16.825	1.71	1.38	16.17	17.23
MT-23	MT-24	968		28.4	28.4	9,840	9,840	1,968	0	1,968	3.622	0.083	0	0.083	400	3.5	0.980	0.123						
ST-51	ST-52	1,542	1,542	61.5	61.5	2,590	2,590	518	0	518	4.449	0.027	0	0.027	300	2.8	0.941	0.067						
ST-52	ST-53	1,315	2,857	60.6	122.1	21,212	23,802	4,760	0	4,760	3.161	0.175	0	0.175	600	2.4	1.064	0.301						
ST-53	MT-24	1,368		41.8	163.9	11,657	35,459	7,092	0	7,092	2.973	0.245	0	0.245	700	2.2	1.129	0.434						
MT-24	MT-25	290		0.0	192.3	0	45,299	9,060	0	9,060	2.863	0.301	0	0.301	800	2.2	1.234	0.620						
MT-30	MT-25	1,116		45.6	45.6	14,198	14,198	2,840	0	2,840	3.423	0.113	0	0.113	500	2.6	0.981	0.193						
MT-25	MT-26	462		0.0	237.9	0	59,497	11,899	0	11,899	2.745	0.379	0	0.379	800	2.2	1.234	0.620						
ST-55	ST-56	65	65	125.3	125.3	23,042	23,042	4,608	0	4,608	3.177	0.170	0	0.170	700	2.4	1.179	0.454						
ST-56	MT-26	1,616	1,681	20.9	146.2	2,242	25,284	5,057	0	5,057	3.132	0.184	0	0.184	700	2.4	1.179	0.454						

Trunk Sewer East, covering major sewerage area 200 lpcd

Line No. of Upper Sewer	Line No. of Lower Sewer	Sewer Length (m)		Sewerage Area (ha)		Population		Average Flow (m3/d)			Peak Factor	Max. Flow (m^3/s)			Sewer Line				Sewer Invert Elevation (m)		Ground Elevation (m)		Earth Covering (m)	
		Increment	Total	Increment	Total	Increment	Total	Sewage	Inlet	Total		Sewage	Infilt.	Total	Dia. (mm)	Slope (o/oo)	V (ms/)	Cap. (m^3/s)	Upper end	Lower end	Upper end	Lower end	Upper end	Lower end
MT-26	MT-27	97		0.0	384.1	0	84,781	16,956	0	16,956	2.600	0.511	0	0.511	900	2.0	1.273	0.810						
ST-54	MT-24'	583		130.1	130.1	44,960	44,960	8,992	0	8,992	2.866	0.299	0	0.299	800	2.2	1.234	0.620						
MT-24'	MT-25'	190		0.0	611.7	0	131,992	26,398	0	26,398	2.428	0.742	0	0.742	1,000	1.8	1.295	1.017	−16.925	−17.267	1.38	1.48	17.22	17.67
MT-25'	MT-27	1,330		19.1	630.8	3,156	135,148	27,030	0	27,030	2.419	0.757	0	0.757	1,000	1.8	1.295	1.017	−17.317	−20.011	1.48	0.87	17.72	19.80
(旧ST-57)																								
MT-27	MT-28	152		0.0	1,014.9	0	219,929	43,986	0	43,986	2.245	1.143	0	1.143	1,200	1.6	1.379	1.559						
ST-58	MT-28	931		33.5	33.5	4,951	4,951	990	0	990	4.026	0.047	0	0.047	350	3.5	0.897	0.086						
MT-28	MT-29	420		0.0	1,048.4	0	224,880	44,976	0	44,976	2.237	1.165	0	1.165	1,200	1.6	1.379	1.559						
ST-64	ST-65	365		28.0	28.0	11,184	11,184	2,237	0	2,237	3.551	0.092	0	0.092	450	2.8	0.949	0.151						
ST-66	ST-65	171		7.6	7.6	4,182	4,182	836	0	836	4.132	0.040	0	0.040	350	3.5	0.897	0.086						
ST-65	MT-29	248		9.2	44.8	1,210	16,576	3,315	0	3,315	3.343	0.129	0	0.129	600	2.6	1.107	0.313						
ST-59	ST-60	508		31.9	31.9	1,262	1,262	252	0	252	4.971	0.015	0	0.015	250	2.8	0.833	0.041						
ST-62	ST-60	291		14.5	14.5	1,554	1,554	311	0	311	4.812	0.018	0	0.018	300	2.8	0.941	0.067						
ST-60	ST-61	378		31.9	78.3	3,420	6,236	1,247	0	1,247	3.886	0.057	0	0.057	400	3.0	0.908	0.114						
ST-63	ST-61	625		34.6	34.6	4,556	4,556	911	0	911	4.078	0.043	0	0.043	350	3.5	0.897	0.086						
ST-61	MT-29	256		9.2	122.1	1,210	12,002	2,400	0	2,400	3.513	0.098	0	0.098	500	2.6	0.981	0.193						
MT-29	AT-1	432		0.0	1,215.3	0	253,458	50,692	0	50,692	2.196	1.289	0	1.289	1,350	1.5	1.444	2.067						
AT-1	AT-2	420		0.0	2,740.6	0	647,329	129,466	0	129,466	1.901	2.849	0	2.849	1,800	1.2	1.565	3.982						
AT-2	AT-3	890		0.0	2,740.6	0	647,329	129,466	0	129,466	1.901	2.849	0	2.849	1,800	1.2	1.565	3.982						
ST-69	ST-70	3,335		193.5	193.5	7,658	7,658	1,532	0	1,532	3.764	0.067	0	0.067	400	3.0	0.908	0.114						
ST-71	ST-70	1,368		39.2	39.2	1,551	1,551	310	0	310	4.815	0.018	0	0.018	250	2.8	0.833	0.041						
ST-70	AT-3	742		10.6	243.3	2,371	11,580	2,316	0	2,316	3.532	0.095	0	0.095	450	2.8	0.949	0.151						
ST-67	ST-68	645		92.8	92.8	20,674	20,674	4,135	0	4,135	3.231	0.155	0	0.155	600	2.6	1.107	0.313						
ST-68	AT-3	1,227		74.4	167.2	16,566	37,240	7,448	0	7,448	2.951	0.255	0	0.255	700	2.2	1.129	0.434						
AT-3	AT-4	492		55.4	3,206.5	12,334	708,483	141,697	0	141,697	1.875	3.076	0	3.076	1,800	1.2	1.565	3.982						
AT-4	AT-5	904		0.0	3,206.5	0	708,483	141,697	0	141,697	1.875	3.076	0	3.076	1,800	1.2	1.565	3.982						
AT-5	AT-6	640		0.0	3,206.5	0	708,483	141,697	0	141,697	1.875	3.076	0	3.076	1,800	1.2	1.565	3.982						
AT-6	WWTP	315		0.0	3,206.5	0	708,483	141,697	0	141,697	1.875	3.076	0	3.076	1,800	1.2	1.565	3.982						
							In the area near WWTP			56,181														
							In Total Area			197,878														

Table 5.2.3 for Case 1: One Trunk Sewer (Dia. 2200mm)

Item	Final Phase Calculation
1. Inlet Pipe	
1.1 Pipe Condition	
Design Flow rate	
Avegrage Daily Flow rate	= 200,000 m^3/d = 2.315 m^3/s
Maximum Daily Flow rate	=
Maximum hourly Flow rate	= 382,800 m^3/d = 4.431 m^3/s
Pipe Diameter	= 2,200 mm
Pipe Gradient	= 1.1 permillage
Invert Level	= -28.175 M
Manning's "n" value	= 0.013
Full Flow rate	= 6.51 m^3/s
Full Flow Velocity	= 1.713 m/s
Water Depth	
Avegrage Daily Flow rate	= 0.837 m
Maximum Daily Flow rate	= m
Maximum hourly Flow rate	= 1.331 m
Water Level (above sea level)	
Avegrage Daily Flow rate	= -28.175 + 0.837 = -27.338 M
Maximum Daily Flow rate	=
Maximum hourly Flow rate	= -28.175 + 1.331 = -26.844 M
1.2 Inlet Chamber	
Invert Elevation	= -28.500 M (above sea level)
Water depth at upstream of inlet Gate	
Avegrage Daily Flow rate	= 1.162 m
Maximum hourly Flow rate	= 1.656 m

Item	Final Phase Calculation
2. Inlet Gate	
Invert Elevation	= -28.500 M (above sea level)
Gate Width	= 0.800 m
Gate Height	= 1.800 m
Number of Gate	= 4 Nos.
Passing Velocity	
Avegrage Daily Flow rate	= 2.315 /(0.80 x 1.162 x 4) = 0.623 m/s
Maximum hourly Flow rate	= 4.431 /(0.80 x 1.656 x 4) = 0.836 m/s
Headloss at Inlet Gate	
Avegrage Daily Flow rate	= 1.5 x 0.623 ^2 / 19.6 = 0.03 m
Maximum hourly Flow rate	= 1.5 x 0.836 ^2 / 19.6 = 0.053 m
Water Level at downstream of inlet Gate	
Avegrage Daily Flow rate	= -27.338 - 0.03 = -27.368 M
Maximum hourly Flow rate	= -26.844 - 0.053 = -26.897 M
3. Screen	
3.1 Coarse Screen	
Openning	= 100 mm
Type	= Manual Rake
Bottom Elevation	= -28.550 M
Channel width	= 1.400 m
Number of Screen	= 4 Nos.
Approch water depth	
Avegrage Daily Flow rate	= -27.368 - -28.550 = 1.182 m
Maximum hourly Flow rate	= -26.897 - -28.550 = 1.653 m
Approch velocity	
Avegrage Daily Flow rate	= 2.315 / (1.40 x 1.182 x 4) = 0.350 m/s
Maximum hourly Flow rate	= 4.431 / (1.40 x 1.653 x 4) = 0.479 m/s
Headloss at coarse screen	= neglect

Item	Final Phase Calculation
3.2 Fine Screen	
Openning	= 15 mm
Type	= Mechanical Rake
Bottom Elevation	= -28.550 M
Channel width	= 1.400 m
Number of Screen	= 4 Nos.
Approch water depth	
Avegrage Daily Flow rate	= -27.368 - -28.550 = 1.182 m
Maximum hourly Flow rate	= -26.897 - -28.550 = 1.653 m
Approch velocity	
Avegrage Daily Flow rate	= 2.315 / (1.40 x 1.182 x 4) = 0.350 m/s
Maximum hourly Flow rate	= 4.431 / (1.40 x 1.653 x 4) = 0.479 m/s
Headloss at fine screen	
Avegrage Daily Flow rate	= 2.34 x sin70 x (9/15)^(4/3) x(0.350 x 2)^2 / 19.6 = 0.023
Maximum hourly Flow rate	= 2.34 x sin70 x (9/15)^(4/3) x(0.479 x 2)^2 / 19.6 = 0.043
Water Level at downstream of Fine screen	
Avegrage Daily Flow rate	= -27.368 - 0.023 = -27.391 M
Maximum hourly Flow rate	= -26.897 - 0.043 = -26.940 M
4. Lift Pump	
Type	Vertical shaft Volute type mixed flow pump
Design Flow rate	
Average Daily Flow rate	= 200,000 m^3/d = 139 m^3/min
Maximum Daily Flow rate	= 0 m^3/d = 0 m^3/min
Maximum hourly Flow rate	= 382,800 m^3/d = 266 m^3/min
Pump Capacity	
Number of Pump	= 4 nos
Pump Capacity	= 266.0 / 4 = 67 m^3/min
Pump Diameter	= 146 × (67.0 / 3)^0.5 = 690 = 700 mm

Item	Final Phase Calculation
Pump Head	
Discharge water level	= 5.500 M
Suction water level	= -27.400 M
Actual Pump Head	= 5.500 − -27.400 = 32.900 m
Others head loss	= 2.000 m (assumed)
Total Pump Head	= 32.900 + 2.000 = 34.900 = 35.000 m
Motor output	$P=\frac{\rho \times g \times Q \times H}{60 \times 1000 \times \eta} \times (1 + \alpha)$ ρ : Water Density 1,000 kg/m3 g : Acceleration of Gravity 9.8 m/sec2 Q : Flow rate 67 m3/min H : Pump head 35.00 m η : Pump Efficiency 0.8 α : Surplus ratio 0.15 $= \frac{1,000 \times 9.8 \times 67.0 \times 35.0}{60 \times 1,000 \times 0.8} \times (1 + 0.15)$ = 550.6 → 560 kW
Lift Pimp Specification	Type Vertical shaft Volute type mixed flow pump Diamater 700 mm Capacity 67 m3/min Pump Head 34.90 m Motor output 560 kW Numbers 5 nos (Include 1 stand-by)

Table 5.2.4 for Case 2: Two (2) Trunk Sewers (Dia. 1800mm & 1350mm)

Item	Trunk Sewer Main Covering Areas (低段施設)	Trunk Sewer Covering Area near WWTP (高段施設)
1. Inlet Pipe		
1.1 Pipe Condition		
Design Flow rate		
Avegrage Daily Flow rate	= 141,700 m^3/d = 1.64 m^3/s	= 56,200 m^3/d = 0.65 m^3/s
Maximum Daily Flow rate	= m^3/d = 0 m^3/s	= m^3/d = 0 m^3/s
Maximum hourly Flow rate	= 266,400 m^3/d = 3.083 m^3/s	= 112,400 m^3/d = 1.301 m^3/s
Pipe Diameter	= 1,800 mm	= 1,350 mm
Pipe Gradient	= 1.2 permillage	= 1.5 permillage
Invert Level	= -27.970 M	= -15.910 M
Manning's "n" value	= 0.013	= 0.013
Full Flow rate	= 3.982 m^3/s	= 2.067 m^3/s
Full Flow Velocity	= 1.565 m/s	= 1.444 m/s
Water Depth		
Avegrage Daily Flow rate	= 0.805 m	= 0.520 m
Maximum Daily Flow rate	= m	= m
Maximum hourly Flow rate	= 1.189 m	= 0.777 m
Water Level (above sea level)		
Avegrage Daily Flow rate	= -27.970 + 0.805 = -27.165 M	= -15.910 + 0.520 = -15.390 M
Maximum Daily Flow rate	=	=
Maximum hourly Flow rate	= -27.970 + 1.189 = -26.781 M	= -15.910 + 0.777 = -15.133 M
1.2 Inlet Chamber		
Invert Elevation	= -28.300 M (above sea level)	= -16.300 M (above sea level)
Water depth at upstream of inlet Gate		
Avegrage Daily Flow rate	= 1.135 m	= 0.910 m
Maximum hourly Flow rate	= 1.519 m	= 1.167 m
2. Inlet Gate		
Invert Elevation	= -28.300 M (above sea level)	= -16.300 M (above sea level)
Gate Width	= 0.600 m	= 0.600 m
Gate Height	= 1.700 m	= 1.200 m
Number of Gate	= 4 Nos.	= 2 Nos.
Passing Velocity		
Avegrage Daily Flow rate	= 1.640 /(0.60 x 1.135 x 4) = 0.602 m/s	= 0.650 /(0.60 x 0.910 x 2) = 0.595 m/s
Maximum hourly Flow rate	= 3.083 /(0.60 x 1.519 x 4) = 0.846 m/s	= 1.301 /(0.60 x 1.167 x 2) = 0.929 m/s
Headloss at Inlet Gate		
Avegrage Daily Flow rate	= 1.5 x 0.602 ^2 / 19.6 = 0.028 m	= 1.5 x 0.595 ^2 / 19.6 = 0.027 m
Maximum hourly Flow rate	= 1.5 x 0.846 ^2 / 19.6 = 0.055 m	= 1.5 x 0.929 ^2 / 19.6 = 0.066 m

Item	Trunk Sewer Main Covering Areas (低段施設)	Trunk Sewer Covering Area near WWTP (高段施設)
Water Level at downstream of inlet Gate		
Avegrage Daily Flow rate	= -27.165 - 0.028 = -27.193 M	= -15.390 - 0.027 = -15.417 M
Maximum hourly Flow rate	= -26.781 - 0.055 = -26.836 M	= -15.133 - 0.066 = -15.199 M
3. Screen		
3.1 Coarse Screen		
Openning	= 100 mm	= 100 mm
Type	= Manual Rake	= Manual Rake
Bottom Elevation	= -28.350 M	= -16.350 M
Channel width	= 1.800 m	= 1.000 m
Number of Screen	= 2 Nos.	= 2 Nos.
Approch water depth		
Avegrage Daily Flow rate	= -27.193 - -28.350 = 1.157 m	= -15.417 - -16.350 = 0.933 m
Maximum hourly Flow rate	= -26.836 - -28.350 = 1.514 m	= -15.199 - -16.350 = 1.151 m
Approch velocity		
Avegrage Daily Flow rate	= 1.640 / (1.80 x 1.157 x 2) = 0.394 m/s	= 0.650 / (1.00 x 0.933 x 2)= 0.348 m/s
Maximum hourly Flow rate	= 3.083 / (1.80 x 1.514 x 2) = 0.566 m/s	= 1.301 / (1.00 x 1.151 x 2)= 0.565 m/s
Headloss at coarse screen	= neglect	= neglect
3.2 Fine Screen		
Openning	= 15 mm	= 15 mm
Type	= Mechanical Rake	= Mechanical Rake
Bottom Elevation	= -28.350 M	= -16.350 M
Channel width	= 1.800 m	= 1.000 m
Number of Screen	= 2 Nos.	= 2 Nos.
Approch water depth		
Avegrage Daily Flow rate	= -27.193 - -28.350 = 1.157 m	= -15.417 - -16.350 = 0.933 m
Maximum hourly Flow rate	= -26.836 - -28.350 = 1.514 m	= -15.199 - -16.350 = 1.151 m
Approch velocity		
Avegrage Daily Flow rate	= 1.640 / (1.80 x 1.157 x 2) = 0.394 m/s	= 0.650 / (1.00 x 0.933 x 2)= 0.348 m/s
Maximum hourly Flow rate	= 3.083 / (1.80 x 1.514 x 2) = 0.566 m/s	= 1.301 / (1.00 x 1.151 x 2)= 0.565 m/s
Headloss at fine screen		
Avegrage Daily Flow rate	= 2.34 x sin70 x (9/15)^(4/3) x(0.394 x 2)^2 / 19.6 = 0.029	= 2.34 x sin70 x (9/15)^(4/3) x(0.348 x 2)^2 / 19.6 = 0.023
Maximum hourly Flow rate	= 2.34 x sin70 x (9/15)^(4/3) x(0.566 x 2)^2 / 19.6 = 0.060	= 2.34 x sin70 x (9/15)^(4/3) x(0.565 x 2)^2 / 19.6 = 0.060
Water Level at downstream of Fine screen		
Avegrage Daily Flow rate	= -27.193 - 0.029 = -27.222 M	= -15.417 - 0.023 = -15.440 M
Maximum hourly Flow rate	= -26.836 - 0.060 = -26.896 M	= -15.199 - 0.060 = -15.259 M

Item	Trunk Sewer Main Covering Areas (低段施設)	Trunk Sewer Covering Area near WWTP (高段施設)
4. Lift Pump		
Type	Vertical shaft Volute type mixed flow pump	
Design Flow rate		
Average Daily Flow rate	= 141,700 m^3/d = 98 m^3/min	= 56,200 m^3/d = 39 m^3/min
Maximum Daily Flow rate	= 0 m^3/d = 0 m^3/min	= 0 m^3/d = 0 m^3/min
Maximum hourly Flow rate	= 266,400 m^3/d = 185 m^3/min	= 112,400 m^3/d = 78 m^3/min
Pump Capacity		
Number of Pump	= 4 nos	= 2 nos
Pump Capacity	= 185.0 / 4 = 46 m^3/min	= 78.0 / 2 = 39 m^3/min
Pump Diameter	= 146 × (46.0 / 3)^0.5 = 572 = 600 mm	= 146 × (39.0 / 3)^0.5 = 526 = 600 mm
Pump Head		
Discharge water level	= 5.500 M	= 5.500 M
Suction water level	= -27.300 M	= -15.500 M
Actual Pump Head	= 5.500 − -27.300 = 32.800 m	= 5.500 − -15.500 = 21.000 m
Others head loss	= 2.000 m (assumed)	= 2.000 m (assumed)
Total Pump Head	= 32.800 + 2.000 = 34.800 = 35.000 m	= 21.000 + 2.000 = 23.000 m
Motor output	$P=\dfrac{\rho \times g \times Q \times H}{60 \times 1000 \times \eta} \times (1 + \alpha)$	
	ρ : Water Density 1,000 kg/m3	ρ : Water Density 1,000 kg/m3
	g : Acceleration of Gravity 9.8 m/sec2	g : Acceleration of Gravity 9.8 m/sec2
	Q : Flow rate 46 m3/min	Q : Flow rate 39 m3/min
	H : Pump head 35.00 m	H : Pump head 23.00 m
	η : Pump Efficiency 0.8	η : Pump Efficiency 0.8
	α : Surplus ratio 0.15	α : Surplus ratio 0.15

Item	Trunk Sewer Main Covering Areas (低段施設)	Trunk Sewer Covering Area near WWTP (高段施設)
Lift Pimp Specification	$= \dfrac{1,000 \times 9.8 \times 46.0 \times 35.0}{60 \times 1,000 \times 0.8} \times (1 + 0.15)$ = 378 → 400 kW Type Vertical shaft Volute type mixed flow pump Diamater 600 mm Capacity 46 m3/min Pump Head 35.00 m Motor output 400 kW Numbers 5 nos (Include 1 stand-by)	$= \dfrac{1,000 \times 9.8 \times 39.0 \times 23.0}{60 \times 1,000 \times 0.8} \times (1 + 0.15)$ = 210.6 → 250 kW Type Vertical shaft Volute type mixed flow pump Diamater 600 mm Capacity 39 m3/min Pump Head 23.00 m Motor output 250 kW Numbers 3 nos (Include 1 stand-by)

Table 5.2.5 Comparison of Construction Cost of Sewers for Case 1 and Case 2

(1) Construction Cost of Case 1 (Sewer Dia. Φ2,200mm, Excavation 31.5m)

Unit : Thousand Yen

Item	Quantity	unit	Unit Price	Cost
Departure Shaft	2	locations	280,000	560,000
Arriving Shaft	1	location	135,000	135,000
Pipe Jacking Work	960	m	700	672,000
Total				**1,367,000**

(2-1) Constrution Cost of Case 2 (Sewer Dia. Φ1,350mm, Excavation Depth 18.0m)

Unit : Thousand Yen

Item	Quantity	unit	Unit Price	Cost
Departure Shaft	3	locations	58,000	174,000
Arriving Shaft	5	location	3,800	19,000
Pipe Jacking Work	960	m	250	240,000
Total				433,000

(2-2) Construction Cost of Case 2 (Sewer Dia. Φ1,800mm, Excavation Depth 31.5m)

Unit : Thousand Yen

Item	Quantity	unit	Unit Price	Cost
Departure Shaft	2	locations	260,000	520,000
Arriving Shaft	1	location	110,000	110,000
Pipe Jacking Work	960	m	450	432,000
Total				1,062,000

(2-3) Total Construction Cost of Case 2 **1,495,000**

(3) Differnce

The construction cost of Case 2 is higher than that of Case 1.

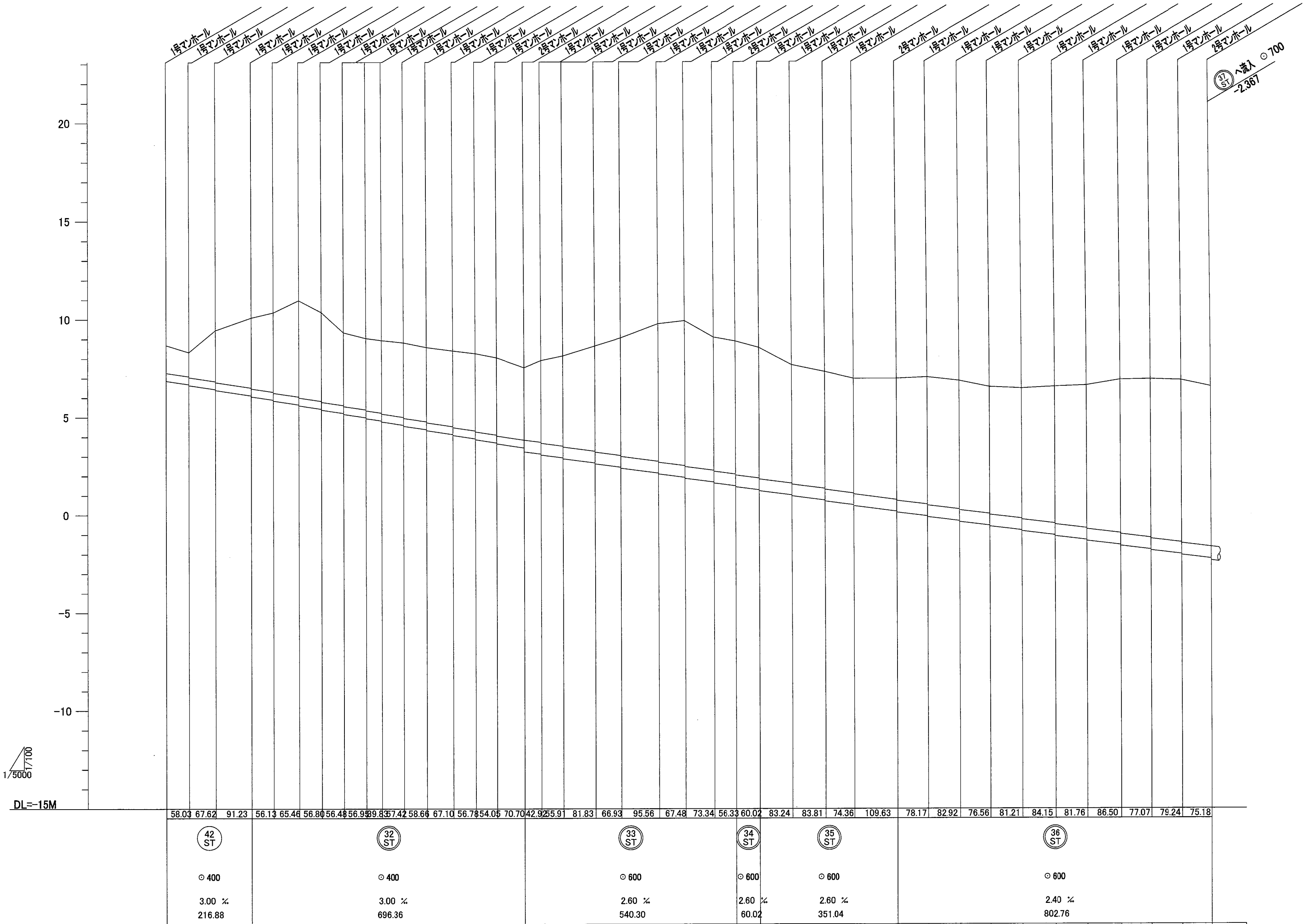
The difernece is : **128,000** Thousand Yen

Table 5.2.6 Comparison of Power Cost for Two Cases

Item	Case 1	Case 2
Average Flow Rate	= 200,000 m^3/day = 138.9 m^3/min	Lower = 141,700 m^3/day = 98.4 m^3/min Higher = 56,200 m^3/day = 39 m^3/min
Pump Head	= 35.0 m	Lower = 35.0 m Higher = 23.0 m
Required Power	$= \frac{\rho \times g \times Q \times H}{60 \times 1000 \times \eta}$	Lower Higher
	ρ : Water Density 1,000 kg/m3 g : Acceleration of Gravity 9.8 m/sec2 Q : Flow rate 138.9 m3/min H : Pump head 35.00 m η : Pump Efficiency 0.8	ρ : 1,000 kg/m3 1,000 kg/m3 g : 9.8 m/sec2 9.8 m/sec2 Q : 98.4 m3/min 39 m3/min H : 35.00 m 23.00 m η : 0.8 0.8
	$= \frac{1{,}000 \times 9.8 \times 138.9 \times 35.0}{60 \times 1{,}000 \times 0.8} =$ 992.6 kW	Lower $= \frac{1{,}000 \times 9.8 \times 98.4 \times 35.0}{60 \times 1{,}000 \times 0.8} =$ 703.2 kW Higher $= \frac{1{,}000 \times 9.8 \times 39.0 \times 23.0}{60 \times 1{,}000 \times 0.8} =$ 183.1 kW
Operation Time	= 24 hr	= 24 hr
Power Consumption	= 992.6 × 24 = 23,822.4 kWh/day	Lower = 703.2 × 24 = 16,876.8 kWh/day Higher = 183.1 × 24 = 4,394.4 kWh/day Total = 21,271.2 kWh/day
Unit rate of power	= 10 yen/kWh	= 10 yen/kWh
Cost of power per day	= 23,822.4 × 10 = 238,224 yen	= 21,271.2 × 10 = 212,712 yen
Cost of power per year	= 23,822.4 × 10 × 365.0 = 86,951,760 yen	= 21,271.2 × 10 × 365.0 = 77,639,880 yen

処理区	ジャカルタ
分 区	東幹線(第1B案)

管 番 号 表					
42 ST	32 ST	33 ST	34 ST	35 ST	36 ST



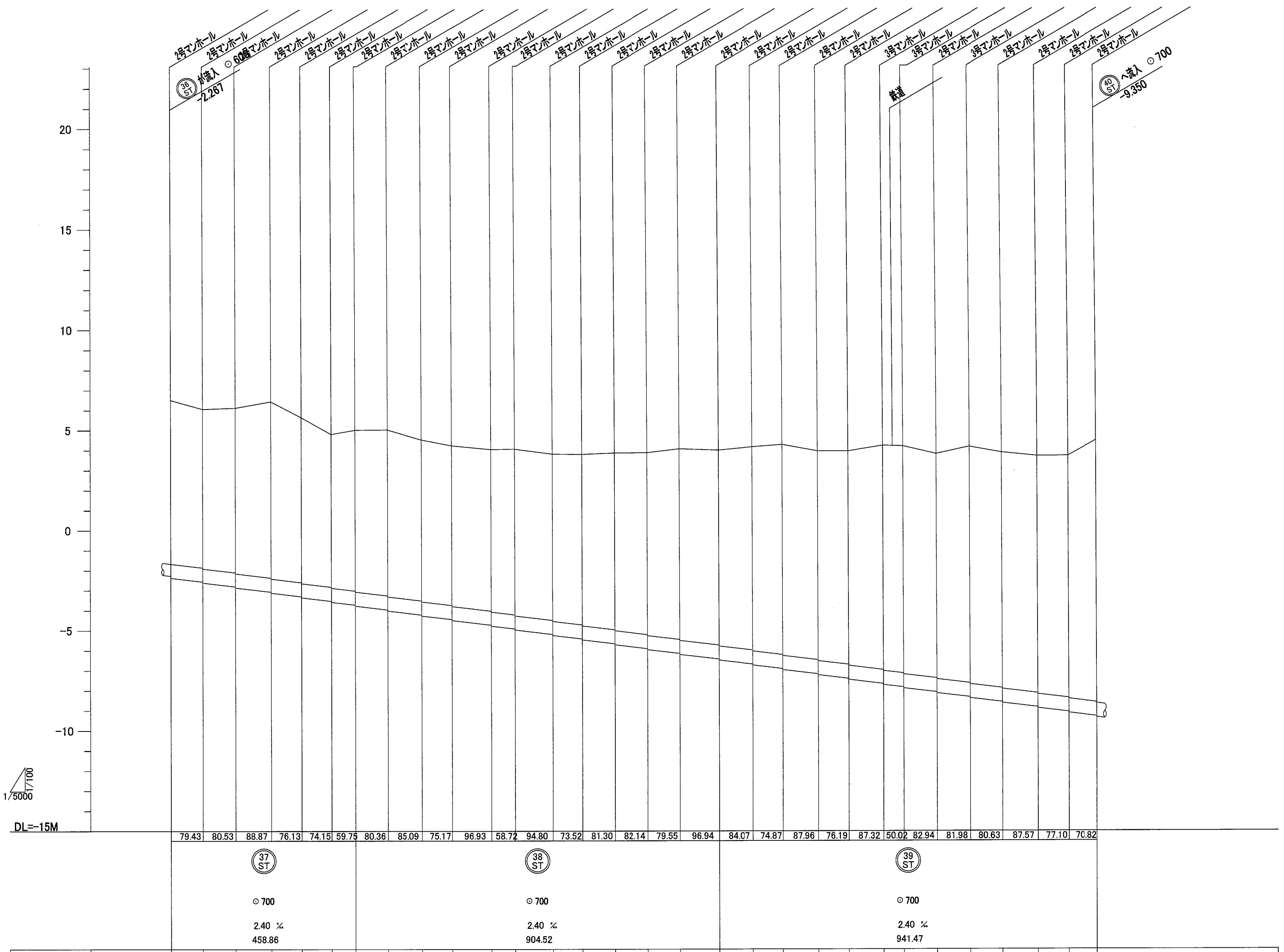
	42 ST	32 ST	33 ST	34 ST	35 ST	36 ST
	⊙ 400	⊙ 400	⊙ 600	⊙ 600	⊙ 600	⊙ 600
	3.00 ‰	3.00 ‰	2.60 ‰	2.60 ‰	2.60 ‰	2.40 ‰
	216.88	696.36	540.30	60.02	351.04	802.76

地盤高	8.69	8.33	9.45	10.09	10.35	10.97	10.37	9.33	9.04	8.93	8.81	8.57	8.39
土被り	1.39	1.20 1.25	2.57 2.62	3.54 3.59	4.02 4.07	4.88 4.93	4.50 4.55	3.68 3.73	3.61 3.66	3.67 3.72	3.77 3.82	3.76 3.81	3.83 3.88
管底高	6.869	6.695 6.645	6.442 6.392	6.118 6.068	5.900 5.850	5.654 5.604	5.434 5.384	5.215 5.165	4.994 4.944	4.825 4.775	4.603 4.553	4.377 4.327	4.126 4.076
逓加距離	0.00	58.03	125.65	216.88	273.01	338.47	395.27	451.75	508.70	548.53	605.95	664.61	731.71

地盤高	8.25	8.04	7.53	7.89	8.13	8.63	9.04	9.76	9.91	9.07	8.87	8.54	7.65
土被り	3.91 3.96	3.91 3.96	3.66 3.65	4.12 4.17	4.56 4.61	5.32 5.37	5.95 6.00	6.97 7.02	7.35 7.40	6.75 6.80	6.74 6.79	6.62 6.67	5.99 6.04
管底高	3.906 3.856	3.694 3.644	3.432 3.232	3.120 3.070	2.925 2.875	2.662 2.612	2.438 2.388	2.140 2.090	1.915 1.865	1.674 1.624	1.478 1.428	1.272 1.222	1.006 0.956
逓加距離	788.49	842.54	913.24	956.16	1012.07	1093.90	1160.83	1256.39	1323.87	1397.21	1453.54	1513.56	1596.80

地盤高	7.30	6.95	6.95	7.01	6.83	6.51	6.43	6.52	6.58	6.86	6.89	6.83	6.51
土被り	5.91 5.96	5.81 5.86	6.14 6.19	6.44 6.49	6.51 6.56	6.42 6.47	6.59 6.64	6.93 6.98	7.23 7.28	7.77 7.82	8.04 8.09	8.22 8.27	8.13
管底高	0.738 0.688	0.495 0.445	0.160 0.110	-0.078 -0.128	-0.327 -0.377	-0.561 -0.611	-0.806 -0.856	-1.058 -1.108	-1.304 -1.354	-1.562 -1.612	-1.797 -1.847	-2.037 -2.087	-2.267
逓加距離	1680.61	1754.97	1864.60	1942.77	2025.69	2102.25	2183.46	2267.61	2349.37	2435.87	2512.94	2592.18	2667.36

ジャカルタ東幹線			
図 面 名	高段流入縮小ー第1B案		汚水
縮 尺	縦 1:100 横 1:5000	図面番号	1 7
作製年月	平成26年 5月		
部長	課長	係長	係員



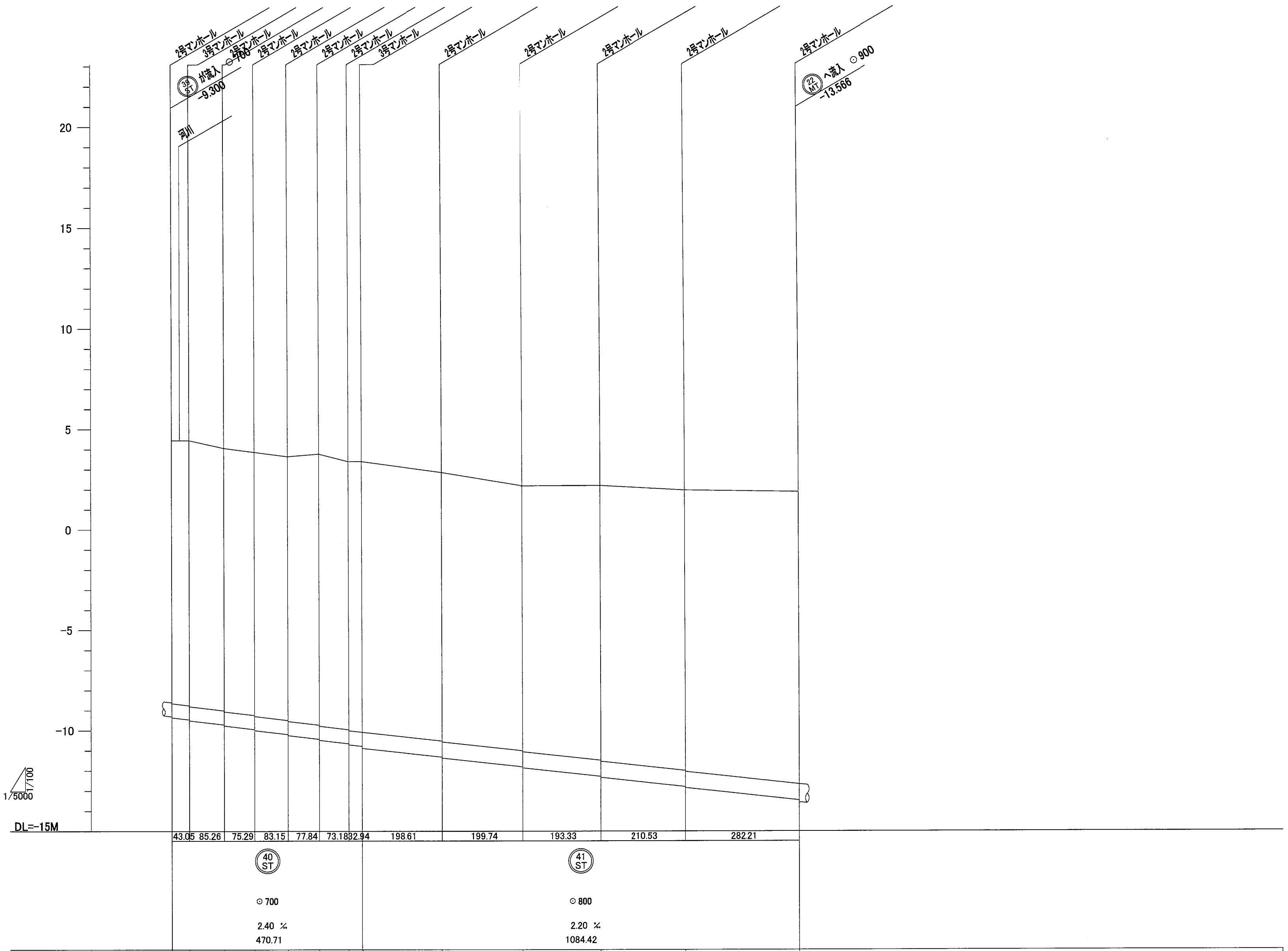
地盤高	6.51	6.06	6.12	6.43	5.64	4.80	5.01	5.02	4.52	4.22	4.03	4.05	3.80	3.78	3.85
土被り	8.12	7.86 7.91	8.16 8.21	8.74 8.79	8.18 8.23	7.57 7.62	7.97 8.02	8.22 8.27	7.98 8.03	7.91 7.96	8.00 8.05	8.21 8.26	8.24 8.29	8.45 8.50	8.76 8.81
管底高	-2.367	-2.558 -2.608	-2.801 -2.851	-3.064 -3.114	-3.297 -3.347	-3.525 -3.575	-3.718 -3.768	-3.961 -4.011	-4.215 -4.265	-4.445 -4.495	-4.728 -4.778	-4.919 -4.969	-5.197 -5.247	-5.423 -5.473	-5.668 -5.718
逓加距離	2667.36	2746.79	2827.32	2916.19	2992.32	3066.47	3126.22	3206.58	3291.67	3366.84	3463.77	3522.49	3617.29	3690.81	3772.11

地盤高	3.86	4.05	3.98	4.14	4.25	3.93	3.92	4.20	4.17	3.78	4.14	3.85	3.67	3.68	4.45
土被り	9.02 9.07	9.45 9.50	9.66 9.71	10.07 10.12	10.41 10.46	10.35 10.40	10.58 10.63	11.12 11.17	11.26 11.31	11.12 11.17	11.72 11.77	11.68 11.73	11.76 11.81	12.00 12.05	12.99
管底高	-5.915 -5.965	-6.156 -6.206	-6.439 -6.489	-6.691 -6.741	-6.921 -6.971	-7.182 -7.232	-7.415 -7.465	-7.675 -7.725	-7.845 -7.895	-8.094 -8.144	-8.341 -8.391	-8.585 -8.635	-8.845 -8.895	-9.080 -9.130	-9.300
逓加距離	3854.25	3933.80	4030.74	4114.81	4189.68	4277.64	4353.83	4441.15	4491.17	4574.11	4656.09	4736.72	4824.29	4901.39	4972.21

処理区	ジャカルタ
分　区	東幹線(第1B案)

管　番　号　表						
37 ST	38 ST	39 ST				

ジャカルタ東幹線							
図面名	高段流入縮小ー第1B案　汚水						
縮　尺	縦 1: 100 横 1:5000			図面番号	2 7		
作製年月	平成26年 5月						
部長		課長		係長		係員	



地盤高	4.45	4.45	4.06	3.86	3.64	3.77	3.39	3.39	2.84	2.17	2.18	1.95	1.86
土被り	13.04	13.15 13.20	13.01 13.06	13.04 13.09	13.07 13.12	13.44 13.49	13.28 13.33	13.41	13.29 13.34	13.11 13.16	13.60 13.65	13.88 13.93	14.46
管底高	-9.350	-9.453 -9.503	-9.708 -9.758	-9.939 -9.989	-10.189 -10.239	-10.426 -10.476	-10.652 -10.702	-10.781 -10.881	-11.318 -11.368	-11.807 -11.857	-12.282 -12.332	-12.795 -12.845	-13.466
逓加距離	4972.21	5015.26	5100.52	5175.81	5258.96	5336.80	5409.98	5442.92	5641.53	5841.27	6034.60	6245.13	6527.34

処理区	ジャカルタ
分　区	東幹線(第1B案)

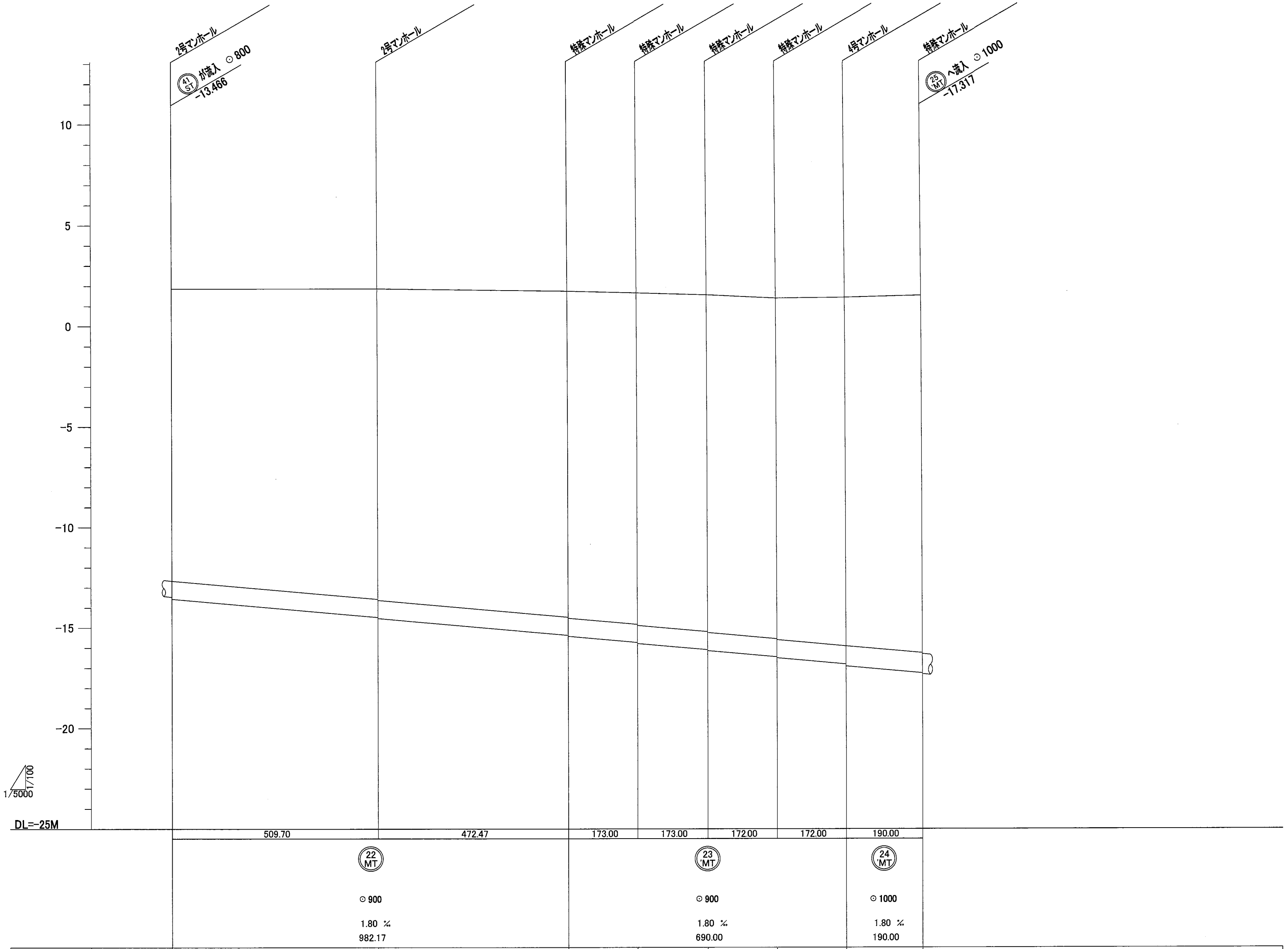
管　番　号　表						
40 ST	41 ST					

ジャカルタ東幹線

図面名	高段流入縮小一第1B案　汚水						
縮　尺	縦 1:100 横 1:5000			図面番号	3/7		
作製年月	平成26年 5月						
部長		課長		係長		係員	

処理区	ジャカルタ
分 区	東幹線(第1B案)

管 番 号 表						
㉒MT	㉓MT	㉔MT				

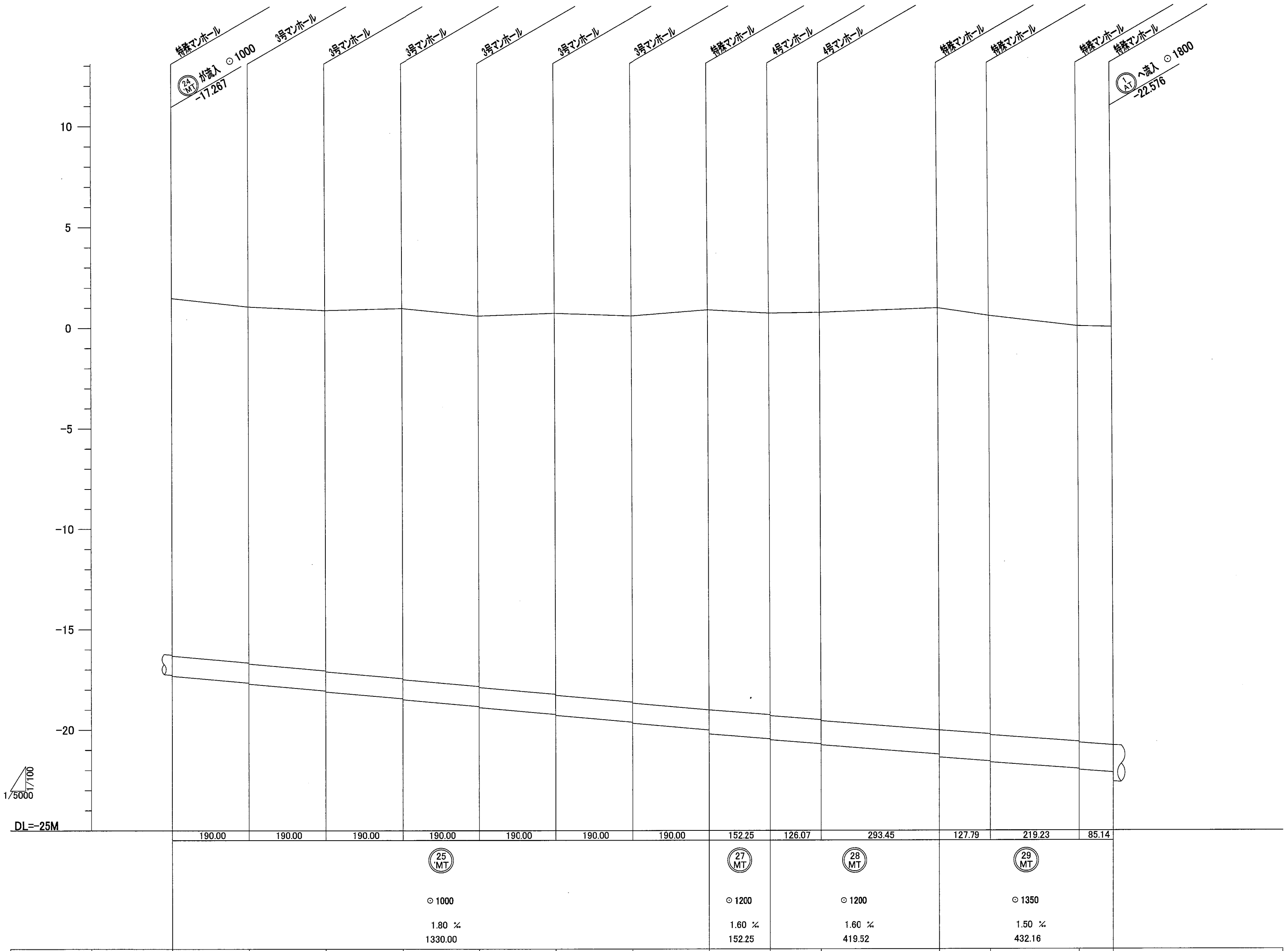


地盤高	1.86	1.85	1.71	1.62	1.52	1.35	1.38	1.48
土被り	14.45	15.36 15.41	16.12 16.17	16.39 16.44	16.65 16.70	16.84 16.89	17.23 17.22	17.67
管底高	-13.566	-14.483 -14.533	-15.383 -15.433	-15.744 -15.794	-16.105 -16.155	-16.465 -16.515	-16.825 -16.925	-17.267
逓加距離	6527.34	7037.04	7509.51	7682.51	7855.51	8027.51	8199.51	8389.51

ジャカルタ東幹線			
図 面 名	高段流入縮小ー第1B案　汚水		
縮　尺	縦 1:100 横 1:5000	図面番号	4/7
作製年月	平成26年 5月		
部長		課長	
係長		係員	

処理区	ジャカルタ
分 区	東幹線(第1B案)

管 番 号 表						
㉕ 'MT	㉗ MT	㉘ MT	㉙ MT			

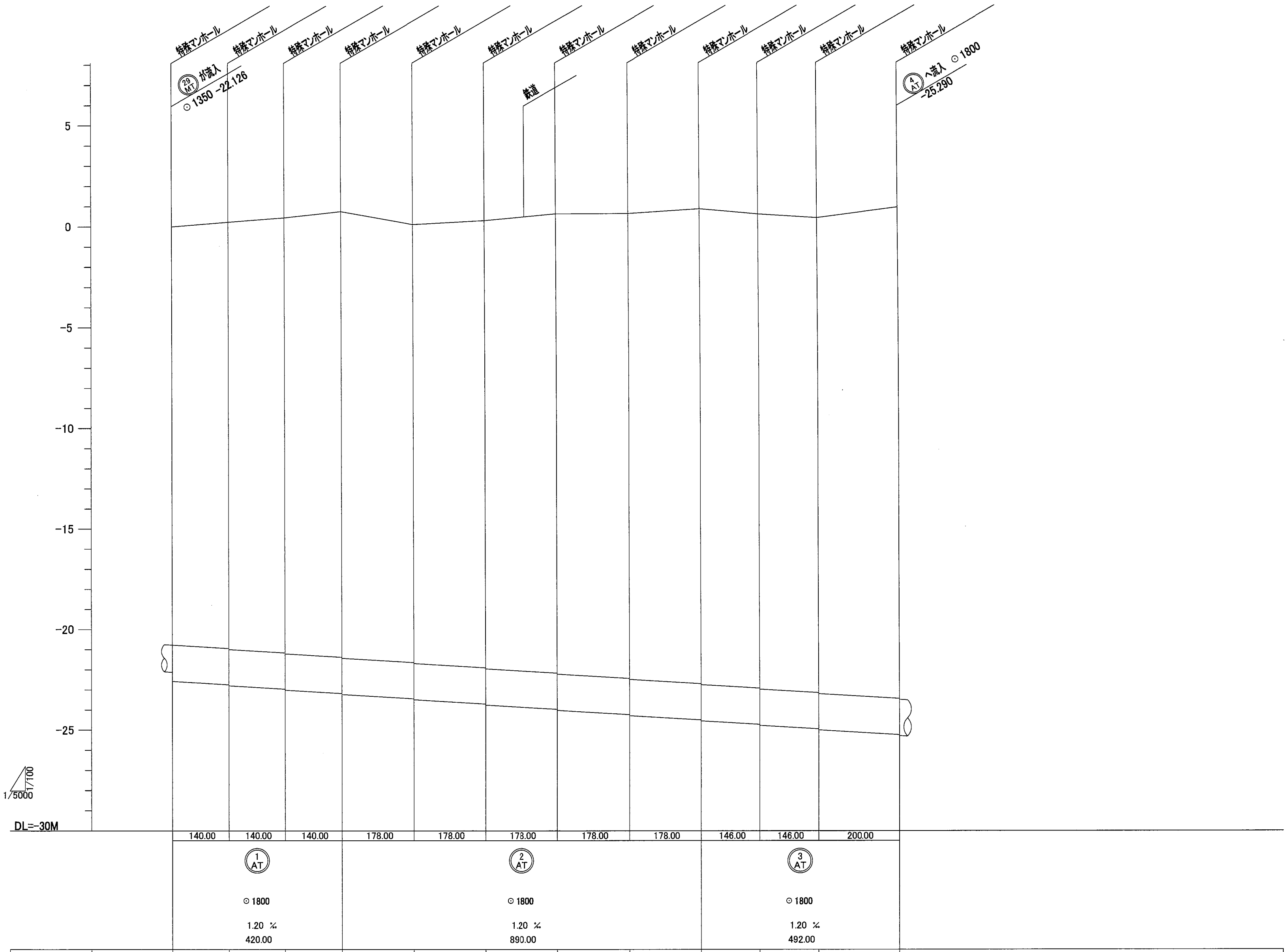


地盤高	1.48	1.05	0.87	0.96	0.58	0.71	0.57	0.87	0.70	0.73	0.94	0.56	0.04	0.00
土被り	17.72	17.63 17.68	17.84 17.89	18.32 18.37	18.33 18.38	18.86 18.91	19.11 19.16	19.80 19.79	19.86 19.91	20.14 20.19	20.87 20.86	20.68 20.73	20.54 20.59	20.67
管底高	-17.317	-17.659 -17.709	-18.051 -18.101	-18.443 -18.493	-18.835 -18.885	-19.227 -19.277	-19.619 -19.669	-20.011 -20.211	-20.455 -20.505	-20.707 -20.757	-21.227 -21.377	-21.569 -21.619	-21.948 -21.998	-22.126
逓加距離	8389.51	8579.51	8769.51	8959.51	9149.51	9339.51	9529.51	9719.51	9871.76	9997.83	10291.28	10419.07	10638.30	10723.44

ジャカルタ東幹線							
図面名	高段流入縮小一第1B案 汚水						
縮 尺	縦 1: 100 横 1:5000			図面番号	5 7		
作製年月	平成26年 5月						
部長		課長		係長		係員	

処理区	ジャカルタ
分　区	東幹線(第1B案)

管　番　号　表						
1 AT	2 AT	3 AT				

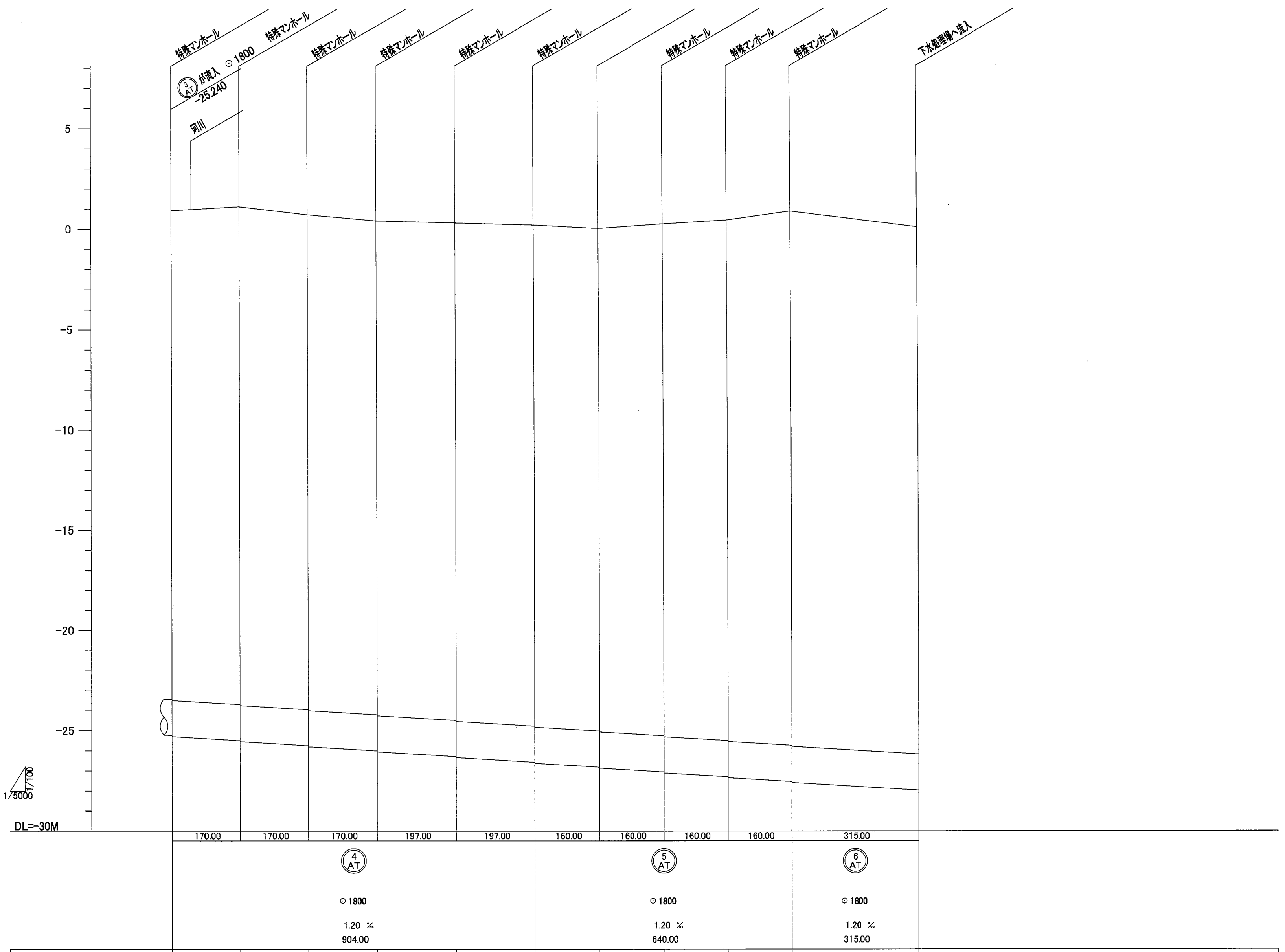


地盤高	0.00	0.23	0.44	0.75	0.10	0.29	0.62	0.63	0.86	0.60	0.41	0.93
土被り	20.62	21.01 21.06	21.44 21.49	21.97 22.02	21.58 21.63	22.04 22.09	22.63 22.68	22.91 22.96	23.40 23.45	23.37 23.42	23.40 23.45	24.21
管底高	-22.576	-22.744 -22.794	-22.962 -23.012	-23.180 -23.230	-23.444 -23.494	-23.708 -23.758	-23.972 -24.022	-24.236 -24.286	-24.500 -24.550	-24.725 -24.775	-24.950 -25.000	-25.240
逓加距離	12723.44	12863.44	13003.44	13143.44	13321.44	13499.44	13677.44	13855.44	14033.44	14179.44	14325.44	14525.44

ジャカルタ東幹線							
図面名	高段流入縮小ー第1B案　汚水						
縮　尺	縦 1:100 横 1:5000			図面番号	6/7		
作製年月	平成26年 5月						
部長		課長		係長		係員	

処理区	ジャカルタ
分　区	東幹線(第1B案)

管　番　号　表						
(4 AT)	(5 AT)	(6 AT)				



地盤高	0.93	1.12	0.72	0.41	0.30	0.19	0.02	0.24	0.43	0.87	0.08
土被り	24.26	24.65 24.70	24.51 24.56	24.45 24.50	24.63 24.68	24.80 24.85	24.88 24.93	25.34 25.39	25.77 25.82	26.45 26.50	26.09
管底高	-25.290	-25.494 -25.544	-25.748 -25.798	-26.002 -26.052	-26.288 -26.338	-26.574 -26.624	-26.816 -26.866	-27.058 -27.108	-27.300 -27.350	-27.542 -27.592	-27.970
逓加距離	14525.44	14695.44	14865.44	15035.44	15232.44	15429.44	15589.44	15749.44	15909.44	16069.44	16384.44

ジャカルタ東幹線			
図面名	高段流入縮小ー第1B案　汚水		
縮　尺	縦 1: 100 横 1:5000	図面番号	7 7
作製年月	平成26年 5月		

部長		課長		係長		係員	

Appendix-6

DETAILED DESIGN OF THE SHAFT A, B AND C AND PIPE JACKING

Table of Contents

1. Structural Calculation Sheet for No. A and No. C Shaft

Contents

1. Design condition

The calculation of shaft A is adopted as a representative of the calculation of shaft C because both forms are almost same and the depth of A is deeper than shaft C.

1-1. Structural type

Structural type : Strucuture of reinforced concrete
Foundation type : Open Caisson foundation

1-2. Load

1) Deal load

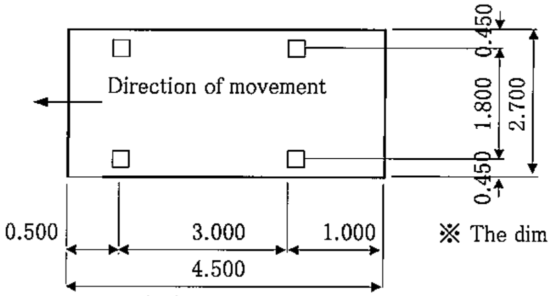
Material	Unit Weight	Notes	
	kN/m^3		
Reinforced concrete	24.5		
Plain concrete	23.0		
Backfill soil (wet weight)	19.0	Internal frictional angle	ϕ = 30.0 °
Backfill soil (submerged weight)	10.0		
Unit weight of water	10.0		

2) Vehicle load

If vehicle load is loaded, "load T-4" is considered.

The standard is shown in the following figure.

Gross weight W = 40.0 kN



※ The dimension of 6t vehicle is referred.

$$\text{Rear wheel}: P_{l1} = \frac{2\times\text{Load of rear wheel(kN)}}{\text{Occupied width of a set of T load(m}} \times (1+\text{ impact factor})$$

$$= \frac{2 \times 15.0}{2.700} \times (1 + i) \quad \text{kN/m}$$

$$\text{Front Wheel}: P_{l2} = \frac{2\times\text{Load of front wheel(kN)}}{\text{Occupied width of a set of T load(m}} \times (1+\text{ impact factor})$$

$$= \frac{2 \times 5.0}{2.700} \times (1 + i) \quad \text{kN/m}$$

To the above formula

i : Coefficient of impact

Type of culvert	Earth covering(h)	Coefficient of impact
・Box culvert	h<4m	0.3
・Arch culvert		
・Portal culvert	4m≦h	0
・Corrugated metal culvert		
・Concrete pipe culvert	h<1.5m	0.5
・Ceramic pipe culvert	1.5m≦h<6.5m	0.65-0.1h
・Rigid polyvinyl chloride pipe culvert		
・Reinforced plastic composite pipe culvert	6.5m≦h	0

a) Vertical load by live load which applies to top slab

i) Case of earth covering under 4 m

$$\text{Rear wheel}: p_{l1} = \frac{P_{l1} \cdot \beta}{W_1} = \frac{P_{l1}}{2 \cdot h + 0.2} \ \text{kN/m}^2$$

$$\text{Front Wheel}: p_{l2} = \frac{P_{l2}}{W_1} = \frac{P_{l2}}{2 \cdot h + 0.2} \ \text{kN/m}^2$$

To the above formula

P_{l1} : Load of rear wheel per unit longitudianal length of calvert (kN/m)

P_{l2} : Load of front wheel per unit longitudianal length of calvert (kN/m)

W_1 : Distribution width of wheel load(m)

ii). Case of earth covering of 4 m and over

In case of earth covering of 4m and over, load, 10kN/m2 equally to top side of top slab as vertical live load is considered.

b). Horizontal load by live load which applies to the side of a manhole

Load, 10 KN/m2 equally as live load of ground surface without considering impact is considered.

c). Sidewalk live load which applies to middle slab of a manhole

Sidewalk live load, 5.0 kN/m2 as live load loading to middle slab is considered.

3) Earth pressure

a) At Ordinary condition

Horizontal earth pressure in an optional depth is considered to be earth pressure at rest.

$$pa = k_0 \cdot \gamma \cdot h$$

To the above formula

pa : Earth pressure at rest(kN/m^2)

k_0 : Coefficient of earth pressure at rest (k_0= 0.5)

γ : Unit weight of soil (kN/m3)

h : Optional depth (m)

※When considering unit volume weight of soil in the calculation of earth pressure, earth pressure is generally seperated from water pressure.

b) At Earthquake condition

Earth pressure in earthquake is considered to be affected by load from earth pressure at rest at ordinary conditi including earth pressure calculated from response displacement method.

1-3. Soil condition

■ Location	Bor.No.A
■ Height of ground	E.L.+ 0.180 m
■ Groundwater level	E.L.+ -2.250 m
■ Basement level	E.L.+ -24.820 m

Elevation	Layer thickness	sign	N value	γ	γ'	c	ϕ	E_0	α	$\alpha \cdot E_0$
m	m			kN/m^3	kN/m^3	kN/m^2	Degree	kN/m^2		kN/m^2
-14.820	15.000	Ac1	1.0	16.0	7.0	21.0	0.0	2,800	1	2,800
-24.820	10.000	Ac2	60.0	16.0	7.0	8.0	0.0	168,000	1	168,000
-41.320	16.500	Ac3	18.0	16.0	7.0	144.0	0.0	50,400	1	50,400
-44.320	3.000	Ac4	50.0	16.0	7.0	-	0.0	140,000	1	140,000
-54.320	10.000	Ac5	25.0	16.0	7.0	-	0.0	70,000	1	70,000
-59.820	5.500	Ac6	40.0	16.0	7.0	-	0.0	112,000	1	112,000
-63.820	4.000	Ac7	60.0	16.0	7.0	-	0.0	168,000	1	168,000
-70.500	6.680	Ac8	18.0	16.0	7.0	-	0.0	50,400	1	50,400

No	Modulus of deformation in each following testing methodology $E_0(kN/m^2)$	α	
		Regular time	Earthquake
①	A half of modulus of deformation calculated from endurance cu of plate loading test by rigid disk of diameter with 0.3m.	1	1
②	Modulus of deformation measured inside borehole.	4	4
③	Modulus of deformation calculated from unconsolidated compression test and triaxial compression test of specimen.	4	4
④	Modulus of deformation estimated with E0=2800N by N value from standard penetration test.	1	1

1-4. Use material and allowable stress

1) Reinforced concerete

Unit:N/mm^2

Design strength		24.0
Compressive stress	Compressive stress due to bending	8.0
	Axial compressive stress	6.5
Shearing stress	In case of shearing stress burdened by only concrete (τ_{a1})	0.23
	In case of being burdened cooperated with diagonal tension bar (τ_{a2})	1.7
	Punching shear unit stress (τ_{a3})	0.90
Bonding sress	To deformed reinforce bars	1.6
Bearing stress		7.2

Note1. Punching shear unit stress does not consider extra according to combination of load.

Note2. If there is no haunch, allowable compressive stress due to bending of conrner is decreased to "3/4".

Elastic modulus	E =	2.5 × 10^7	kN/m^2
Linear expansion coefficient	T =	1.0 × 10^{-5}	$℃^{-1}$

If shear force is caused only by concerete, allowable shearing stress intensity τ al is corrected considering following influence.

① Influence of effective depth, d of member section

Correction coefficient, Ce related to effective depth, d of member section.

Effective depth, d (mm)	300 or lower	1,000	3,000	5,000	10,000and over
C_e	1.4	1.0	0.7	0.6	0.5

② Influence of ration of axial stretched reinforcing bar, p_t

Correction coefficient, Cpt related to ration of axial stretched reinforcing bar, pt

Ration of axial stretched reinforcing bar, p_t (%)	0.1	0.2	0.3	0.5	1.0 and over
C_{pt}	0.7	0.9	1.0	1.2	1.5

③ If axial compressive force of member is large, correction coefficient, CN by axial compressive force calculated from the following formula is multiplied by τ al.

$$C_N = 1 + \frac{M_0}{M}$$

To the above formula

C_N : Correction coefficient by axial compressive force

M_0 : Bending moment N•mm with stress intensity of concrete with zero in the edge of tension member due to axial compressive force

$$= \frac{N}{Ac} \cdot \frac{Ic}{y}$$

M	:	Bending moment applying member section	N•mm
N	:	Axial stress in compression applying member section	N
Ic	:	Inertia moment related to centroid axis of member section	mm^4
Ac	:	Sectional area of member	mm^2
y	:	Distance to the edge of tension member from centroid of sectional area of member	mm

2) Plain concrete

Unit: N/mm^2

Type of stress intensity	Allowable stress intensity	Design strength
		24.0
Compressive stress intensity	$\frac{\sigma ck}{4} \leqq 5.5$	5.5
Tensile stress intensity due to bending	$\frac{\sigma ck}{7} \leqq 0.3$	0.3
Bearing stress intensity	0.3 σck ≦ 6.0	6.0
Shearing unit stress	$\frac{\sigma ck}{100} + 0.15$ Notes 1)	0.39

Notes 1. Extra increase is not added according to combination of load.

3) Reinforcing bar

Unit:N/mm^2

<table>
<tr><th colspan="3">Variety of reinforcing bar
Variety of stress intensity and member</th><th>SD 345</th></tr>
<tr><td rowspan="6">Tensile stress intensity</td><td colspan="2">1) In case that main load without live load and impact is applied (like beam member</td><td>100</td></tr>
<tr><td rowspan="2">Basic value in case that influence of collision load and earthquake is not included in the combination of load</td><td>2) General member</td><td>180</td></tr>
<tr><td>3) Member installed in water level or under groundwater level</td><td>160</td></tr>
<tr><td rowspan="2">Basic value in case that influence of collision load and earthquake is included in the combination of load</td><td>4) Axial reinforcing bar</td><td>200</td></tr>
<tr><td>5) Other than that above</td><td>200</td></tr>
<tr><td colspan="2">6) Basic value in case of calculating the length of lap joint of reinforcing bar or fixing length</td><td>200</td></tr>
<tr><td colspan="3">7) Compressive stress intensity</td><td>200</td></tr>
</table>

3) As for extra increase for allowable stress intensity

Extra increase of allowable tensile stress intensity is the following according to the combination of load.

Combinations of loads	Overdesign factor	Notes
Regular time	1.0	
Construction time	1.5	
Earthquake time(L1)	1.5	

1-5. Application specification and references

*1 Earthworks of road-guideline of culvert work	Corporate juridical person Japan Road Association
*2 Specification of highway bridge and the manual, I common version	Corporate juridical person Japan Road Association
*3 Specification of highway bridge and the manual, Ⅲconcrete bridge version	Corporate juridical person Japan Road Association
*4 Specification of highway bridge and the manual, ⅣSubstructure version	Corporate juridical person Japan Road Association
*5 Design manual of civil engineering(draft)-Civil engineering structure・Bridge version-	Ministry of Land, Transport and Tourism
*6 Guideline and the manual for earthquake countermeasure of sewage facility	Corporate juridical person Japan Sewage Works Association
*7 Calculation examples for earthquake resistance of sewage facility	Corporate juridical person Japan Sewage Works Association
*8 Standard specification for tunnel [Open cut method version]・the manual	Japan Society of Civil Engineering
*9 Structural calculation criterion of reinforced concerete・the manual	Architectural Institute of Japan

1-6. The others

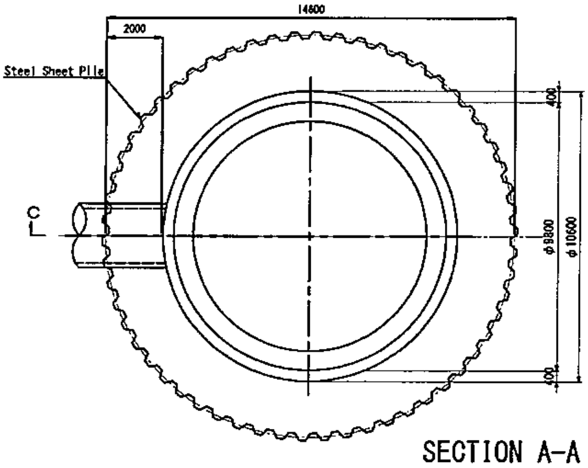
- As for minimum reinforcement content

Minimum reinforcement content is 0.2 and over of effective sectional area of member.

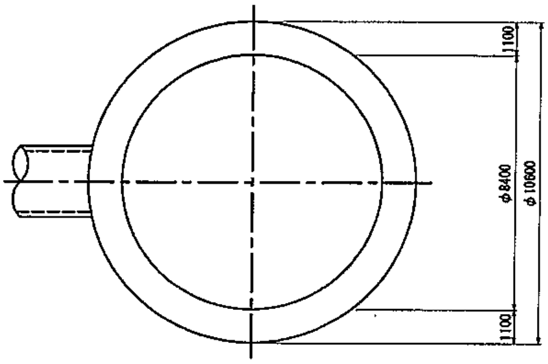
2. Structural drawing

Section A-A

Section B-B

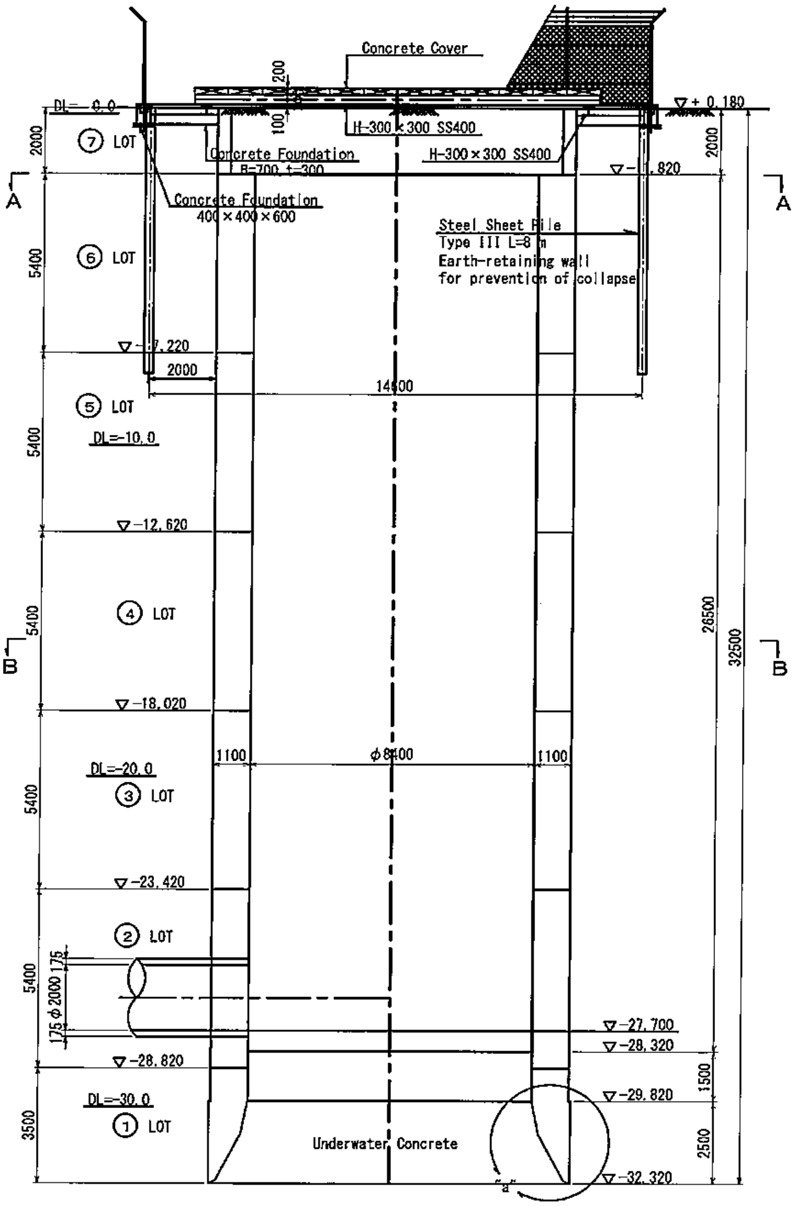


SECTION A-A



SECTION B-B

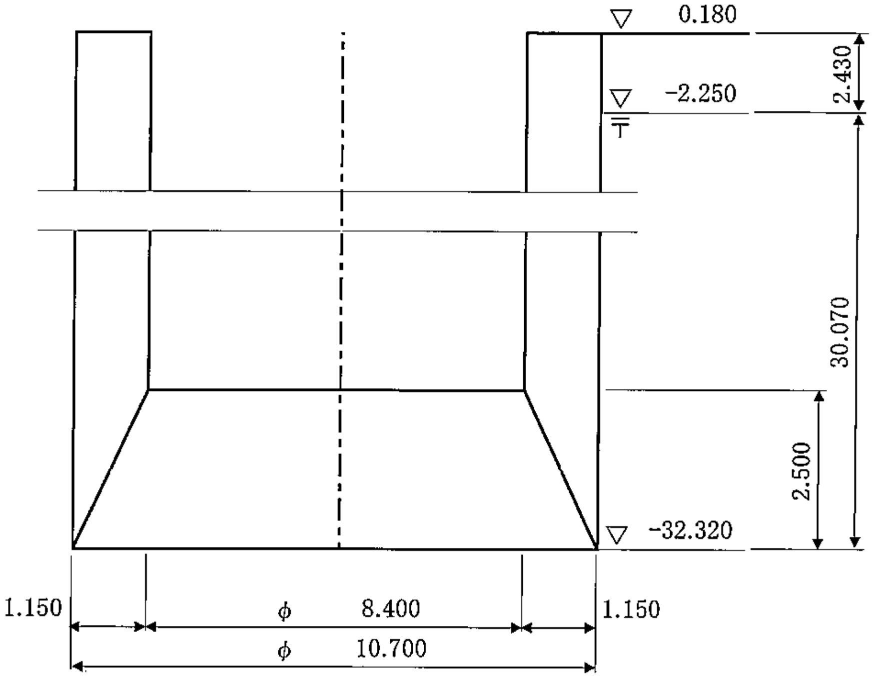
Section C-C



3. Stability computation

3-1. Design of bottom slab (underwater concrete)

Bottom slab (underwater concerete) is treated as plain concrete constructed in water.



1) Load calculation

As for design load, uplift pressure and self weight of bottom slab are considered.

Uplift pressure

w_u = 10.0 × 30.070 = 300.70 kN/m^2

Self weight of bottom slab of concrete

w_c = 23.0 × 2.500 = 57.50 kN/m^2

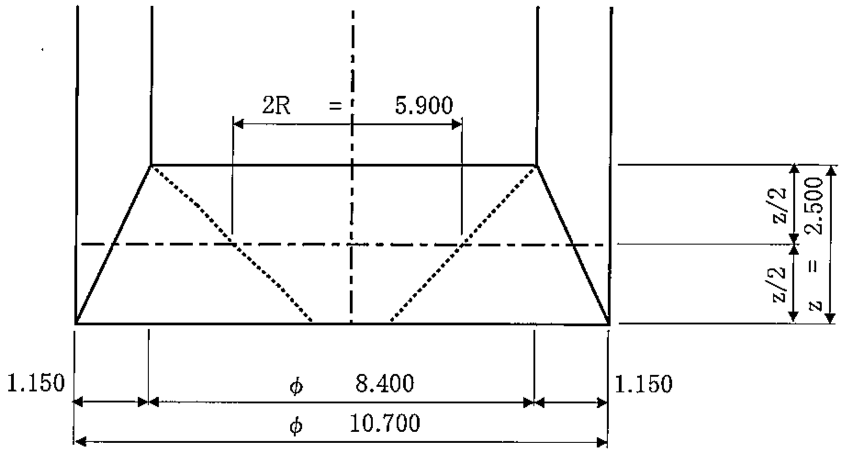
Design load

w = w_u - w_c

= 300.70 - 57.50 = 243.20 kN/m^2

2) Calculation of section force

As for cross sectional area, bottom slab is considered to be slab that the surrounding is simply supported.



Bending moement

$$\text{Mmax} = (3 + \nu)\cdot\frac{w \cdot R^2}{16}$$

$$= (3 + \frac{1}{6})\times\frac{243.20 \times 2.950^2}{16} = 418.88 \text{ kN}\cdot\text{m/m}$$

Shearing strength

$$S = \frac{w \cdot R}{2}$$

$$= \frac{243.20 \times 2.950}{2} = 358.72 \text{ kN/m}$$

3) Checking sectional area

	Bottom slab			
M kN·m	418.88			
N kN	0.00			
S kN	358.72			
b mm	1,000			
h mm	2,500			
Z mm^3	1,041,666,667			
A mm^2	2,500,000			
σc N/mm^2	0.4 < 8.25			
σt N/mm^2	0.4 < 0.45			
τ N/mm^2	0.14 < 0.39			
Judgement	OK			

Section modulus $Z = \frac{1}{6} b\cdot h^2$

Sectional area $A = b\cdot h$

3-2. Consideration for lift

1) Construction time

a) Load calculation

- Body part

Elevation	Height	External diameter	Internal diameter	Cross sectional area	Average cross section area	Volume
m	m	m	m	m^2	m^2	m^3
0.180	2.000	10.600	9.800	12.818	12.818	25.635
-1.820		10.600	9.800	12.818		
	28.000	10.600	8.400	32.830	32.830	919.230
-29.820		10.600	8.400	32.830		
	1.000	10.700	8.400	34.503	31.801	31.801
-30.820		10.700	8.800	29.099		
	1.400	10.700	8.800	29.099	17.848	24.987
-32.220		10.700	10.300	6.597		
	0.100	10.700	10.300	6.597	3.299	0.330
-32.320		10.700	10.700	0.000		
Sum	32.500	-	-	-	-	1,001.983

- Bottom slab part (underwater concrete)

Elevation	Height	External diameter	Internal diameter	Cross sectional area	Average cross section area	Volume
m	m	m	m	m^2	m^2	m^3
-29.820	1.000	8.400	0.000	55.418	58.119	58.119
-30.820		8.800	0.000	60.821		
	1.400	8.800	0.000	60.821	72.072	100.901
-32.220		10.300	0.000	83.323		
	0.100	10.300	0.000	83.323	86.622	8.662
-32.320		10.700	0.000	89.920		
Sum	2.500	-	-	-	-	167.683

- Total weight

$$\Sigma W = 24.5 \times 1{,}001.983 + 23.0 \times 167.683 = 28{,}405.29 \text{ kN}$$

- Buoyancy

$$W_u = 10.0 \times 30.070 \times \frac{\pi}{4} \times 10.700^2 = 27{,}039.01 \text{ kN}$$

b) Checking buoyancy

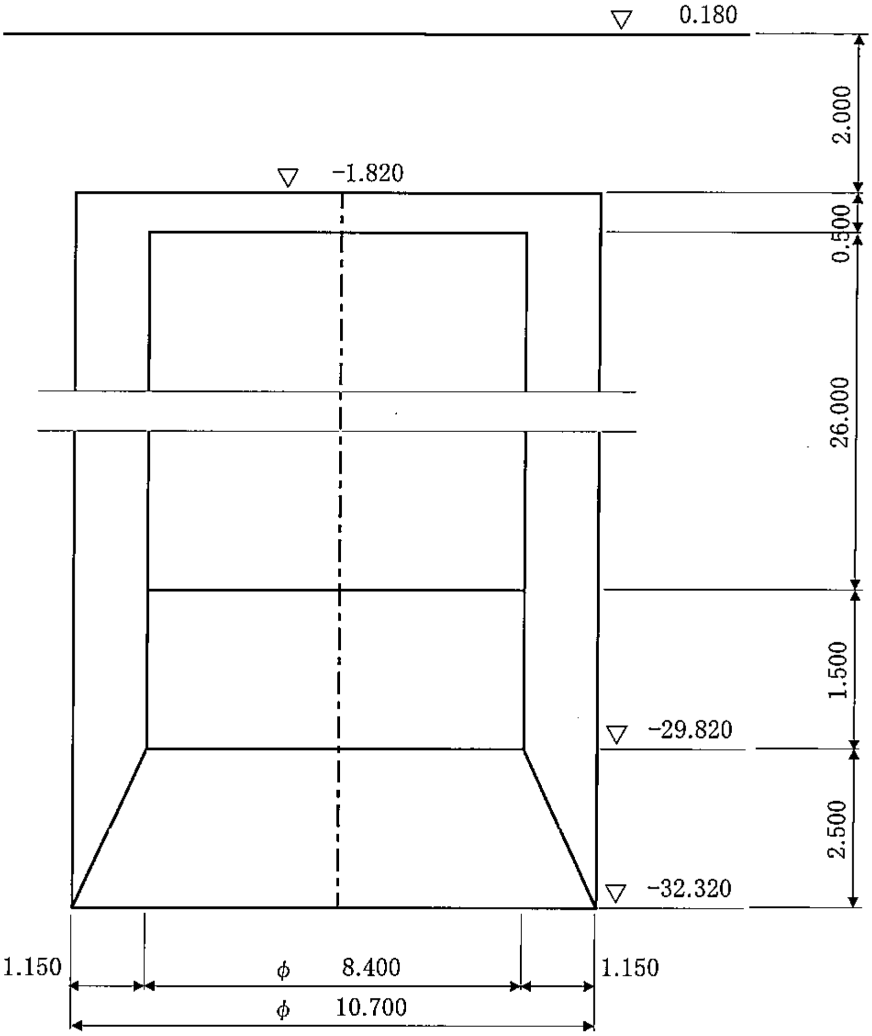
$$F = \frac{\Sigma W}{W_u}$$

$$= \frac{28{,}405.29}{27{,}039.01}$$

$$= 1.05 > Fs = 1.0$$ -- OK

2) Completion time

a) Load calculation



Overburden load

$$W_s = 19.0 \times 2.000 \times \frac{\pi}{4} \times 10.600^2 = 3{,}353.40 \text{ kN}$$

Self weight of top slab

$$W_t = 24.5 \times 0.500 \times \frac{\pi}{4} \times 10.600^2 = 1{,}081.03 \text{ kN}$$

Self weight of lateral wall

$$W_w = 24.5 \times \frac{\pi}{4} \times (10.600^2 - 8.400^2) \times 26.000 = 20{,}912.48 \text{ kN}$$

Self weight of bottom slab

$$W_f = 24.5 \times 1.500 \times \frac{\pi}{4} \times 10.600^2 = 3{,}243.09 \text{ kN}$$

Cutting edge part (lower part than ∇−29.530m)

$$W_n = 24.5 \times (31.801 + 24.987 + 0.330) = 1{,}399.39 \text{ kN}$$

Self weight of middle slab

$$W_m = 6 \times 24.5 \times 0.400 \times \frac{\pi}{4} \times 8.400^2 = 3{,}258.56 \text{ kN}$$

Bottom slab (underwater concrete)

$$W = 23.0 \times 167.683 = 3{,}856.70 \text{ kN}$$

$$\Sigma W = 37{,}104.65 \text{ kN}$$

- Buoyancy

$$W_u = 27{,}039.01 \text{ kN}$$

b) Checking to buoyancy

$$F = \frac{\Sigma W}{W_u}$$

$$= \frac{37{,}104.65}{27{,}039.01}$$

$$= 1.37 > F_s = 1.2$$ --- OK

3-3. consideration for bearing capacity

1) Calculation for ultimate bearing capacity

$q_d \ = \ \alpha \cdot c \cdot Nc + 1/2 \cdot \beta \cdot \gamma_1 \cdot B \cdot N_\gamma + \gamma_2 \cdot Df \cdot Nq$

To the above formula

q_d : Ultimate bearing capacity(kN/m^2)

c : Adhesive force intensity of ground under faoudation base(kN/m^2)

γ_1 : Unit volume weight of ground under foundation base(kN/m3)

γ_2 : Weight per unit volume of ground over foundation base(kN/m3)

α、β : Form coefficient indicated in a table

Form coefficient

Shape for load side of base	Shape like belt	Squuare, circle	Rectangle, oval
α	1.0	1.3	1 + 0.3 ・B/D
β	1.0	0.6	1 − 0.4 ・B/D

B : Base width(m)

Df : Effective depth of foundation(m)

Nc,Nr,Nq : Coefficient of bearing capacity shown in graph

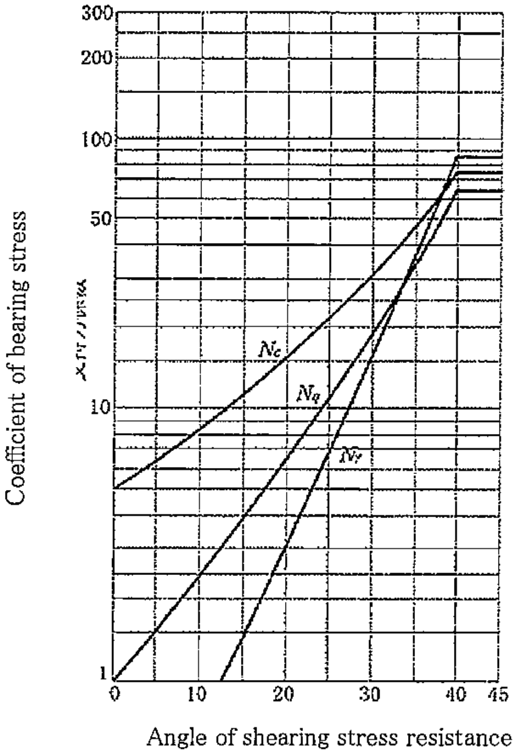


Figure 11.4.2 Figure for coefficient of bearing stress

$$q_d = 1.30 \times 144.0 \times 7.0 + \frac{1}{2} \times 0.60 \times 7.0 \times 10.700 \times 0.0 + 249.37 \times 1.0 = 1{,}559.77\ \mathrm{kN/m^2}$$

$c = 144.0\ \mathrm{kN/m^2}$

$\gamma_1 = 7.0\ \mathrm{kN/m^3}$

$\alpha = 1.30$

$\beta = 0.60$

$B = 10.700\ \mathrm{m}$

$\gamma_2 \cdot Df = 249.37\ \mathrm{kN/m^2}$

Calculation for $\gamma_2 \cdot Df$

Soil	Elevation	Depth	Thickness of layer	γ	γ'	Vertical load	Notes
	m	m	m	$\mathrm{kN/m^3}$	$\mathrm{kN/m^3}$	$\mathrm{kN/m^2}$	
Ac1	0.180	0.000	0.000	16.0	7.0	0.00	Ground level
	-2.250	2.430	2.430	16.0	7.0	38.88	Groundwater level
	-14.820	15.000	12.570	16.0	7.0	126.87	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	126.87	-
	-24.820	25.000	10.000	16.0	7.0	196.87	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	196.87	-
	-32.320	32.500	7.500	16.0	7.0	249.37	Cutting edge

$Nc = 7.0 \qquad Nq = 1.0 \qquad Nr = 0.0$

2) Checking bearing strength

By consideration for uplift in completion time

$\Sigma W = 37{,}104.65\ \mathrm{kN}$

$$q = \frac{37{,}104.65}{\pi / 4 \times 10.700^2} + \overset{\text{Live load}}{10.00}$$

$$= 422.6\ \mathrm{kN/m^2} < q_a = \frac{1}{3} \times 1{,}559.77 = 519.9\ \mathrm{kN/m^2} \cdots\cdots \text{OK}$$

4. Checking member in construction

4-1. Calculation of sidawall

As for checking lateral wall in construction, consideration for the case of occurrence of difference of head of water in working state of sinking and after work of sinking

- As for working state of sinking

① Active earth pressure adding hydrostatic pressure is acted into 4 directions. The acting directions are orthogonal direction towards lateral wall.

② A half of active earth pressure is acted into one direction as unbalanced load at the same time with ①. The acting direction is the direction with its decenterizing.

Active earth pressure is evaluated by formula of Coulomb's earth pressure. However, if coefficient of active earth pressure is under 0.5, the coefficient is set with 0.5.

Moreover, decrease of earth pressure by adhesion is not considered.

③ In case of open caisson, external pressure is not different as the case of pneumatic caisson. However, internal pressure considers hydrastatic stress having the difference between external hydrastatic stress and internal pressure with 3.0 m.

- In case of occurrence of difference of head of water after sinking work

Stratified pressure including hydraostatic stress are acted into 4 directions in the situation of occurrence of difference of head of water between internal and external Caisson due to pump up after sinking. The acting directions are orthogonal direction towards lateral walls.

1) consideration in sinking working state

a) Load calculation

Calculation for coeffieicnet of active earth pressure

Coefficient of active earth pressure is calculated by the following formula. If the coefficient is under 0.5, the value is set with 0.5.

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

To the above formula

K_A : Coefficient of active earth pressure by Coulomb's earth pressure

ϕ : Angle of internal friction of soil (°)

α : Angle between ground surface and horizontal surface (°)

θ : Angle between rear side of wall and vertical plane (°)

δ : Wall friction angle between rear side of wall and ground (°) = 1/3 ϕ

Soil	ϕ	α	θ	δ	K_A		
	(°)	(°)	(°)	(°)	Calculated value	Minimum value	Adopted value
Ac1	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac2	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac3	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac4	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac5	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac6	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac7	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac8	0.0	0.0	0.0	0.0	1.000	0.5	1.000

Calculation for earth pressure intensity

Active earth pressure

$p_a = K_A \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

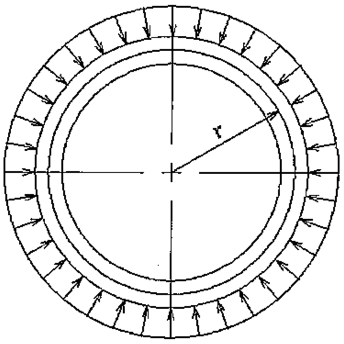
$p_w = \gamma_w \cdot \Sigma h_n$

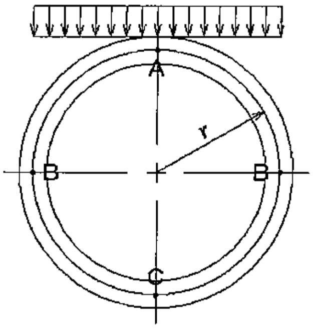
To the above formula

p_a : Active earth pressure
p_w : Hydrostatic stress (kN/m^2)
K_A : Coefficient of active earth pressure by Coulomb's earth pressure
q_0 : Vertical load (kN/m^2)
= 10.0 kN/m^2
γ_n : Unit volume weight of soil in each strat· (kN/$m^{3)}$
(in case of under groundwater level, submerged weight)
γ_w : Weight per unit volume of groundwater (kN/$m^{3)}$
h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.180	0.000	0.000	16.0	7.0	10.0	10.00	1.000	10.00	0.00	Ground level
	-1.820	2.000	2.000	16.0	7.0	10.0	42.00	1.000	42.00	0.00	7R soffit
	-2.250	2.430	0.430	16.0	7.0	10.0	48.88	1.000	48.88	0.00	Groundwater level
	-7.220	7.400	4.970	16.0	7.0	10.0	83.67	1.000	83.67	49.70	6R soffit
	-12.620	12.800	5.400	16.0	7.0	10.0	121.47	1.000	121.47	103.70	5R soffit
	-14.820	15.000	2.200	16.0	7.0	10.0	136.87	1.000	136.87	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	1.000	136.87	125.70	-
	-18.020	18.200	3.200	16.0	7.0	10.0	159.27	1.000	159.27	157.70	4R soffit
	-23.420	23.600	5.400	16.0	7.0	10.0	197.07	1.000	197.07	211.70	3R soffit
	-24.820	25.000	1.400	16.0	7.0	10.0	206.87	1.000	206.87	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	1.000	206.87	225.70	-
	-28.820	29.000	4.000	16.0	7.0	10.0	234.87	1.000	234.87	265.70	2R soffit
	-29.820	30.000	1.000	16.0	7.0	10.0	241.87	1.000	241.87	275.70	Undersurface of bottom slab

b) Calculation of sectional force





- In case of bearing even load from 4 directions (in case this, there is no bending moment)

Axial force

$N = 1.000 \cdot p \cdot r$

- In case of bearing unbalanced load from 1 direction

Bending moment

$M_A = 0.163 \cdot p' \cdot r^2$

$M_B = -0.125 \cdot p' \cdot r^2$

$M_C = 0.087 \cdot p' \cdot r^2$

Axial force

$N_A = 0.212 \cdot p' \cdot r$

$N_B = 1.000 \cdot p' \cdot r$

$N_C = -0.212 \cdot p' \cdot r$

Form and working load

Checking location		Internal diameter	Thickness of member	Shaft diameter of member	Radius of axis of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m		kN/m^2	kN/m^2	kN/m^2
1	6R soffit	8.400	1.100	9.500	4.750	83.67	30.00	41.84
2	5R soffit	8.400	1.100	9.500	4.750	121.47	30.00	60.74
3	4R soffit	8.400	1.100	9.500	4.750	159.27	30.00	79.64
4	3R soffit	8.400	1.100	9.500	4.750	197.07	30.00	98.54
5	2R soffit	8.400	1.100	9.500	4.750	234.87	30.00	117.44
6	Soffit of bottom slab	8.400	1.100	9.500	4.750	241.87	30.00	120.94

Calculation of sectional force

6R soffit		Coefficient	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN･m/m	kN/m
Bending moment	M_A	0.163	-	41.84	4.750	153.86	-
	M_B	-0.125	-	41.84	4.750	-117.99	-
	M_C	0.087	-	41.84	4.750	82.12	-
Axial force	N_A	0.212	113.67	41.84	4.750	-	582.06
	N_B	1.000	113.67	41.84	4.750	-	738.65
	N_C	-0.212	113.67	41.84	4.750	-	497.80

5R soffit		Coeffiicient	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN･m/m	kN/m
Bending moment	M_A	0.163	-	60.74	4.750	223.36	-
	M_B	-0.125	-	60.74	4.750	-171.29	-
	M_C	0.087	-	60.74	4.750	119.22	-
Axial force	N_A	0.212	151.47	60.74	4.750	-	780.64
	N_B	1.000	151.47	60.74	4.750	-	1,007.97
	N_C	-0.212	151.47	60.74	4.750	-	658.32

4R soffit		Coeffieicnet	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN･m/m	kN/m
Bending moment	M_A	0.163	-	79.64	4.750	292.87	-
	M_B	-0.125	-	79.64	4.750	-224.60	-
	M_C	0.087	-	79.64	4.750	156.32	-
Axial moment	N_A	0.212	189.27	79.64	4.750	-	979.22
	N_B	1.000	189.27	79.64	4.750	-	1,277.30
	N_C	-0.212	189.27	79.64	4.750	-	818.84

3R soffit		Coeffieicnet	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN･m/m	kN/m
Bending moment	M_A	0.163	-	98.54	4.750	362.38	-
	M_B	-0.125	-	98.54	4.750	-277.90	-
	M_C	0.087	-	98.54	4.750	193.42	-
Axial force	N_A	0.212	227.07	98.54	4.750	-	1,177.81
	N_B	1.000	227.07	98.54	4.750	-	1,546.62
	N_C	-0.212	227.07	98.54	4.750	-	979.36

2R soffit		Coeffieicnet	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN•m/m	kN/m
Bending moment	M_A	0.163	-	117.44	4.750	431.89	-
	M_B	-0.125	-	117.44	4.750	-331.20	-
	M_C	0.087	-	117.44	4.750	230.52	-
Axial force	N_A	0.212	264.87	117.44	4.750	-	1,376.39
	N_B	1.000	264.87	117.44	4.750	-	1,815.95
	N_C	-0.212	264.87	117.44	4.750	-	1,139.88

Soffit of bottom slab		Coeffieicnet	Uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN•m/m	kN/m
Bending moment	M_A	0.163	-	120.94	4.750	444.76	-
	M_B	-0.125	-	120.94	4.750	-341.07	-
	M_C	0.087	-	120.94	4.750	237.39	-
Axial force	N_A	0.212	271.87	120.94	4.750	-	1,413.16
	N_B	1.000	271.87	120.94	4.750	-	1,865.82
	N_C	-0.212	271.87	120.94	4.750	-	1,169.60

c) Checking section

- As for minimum amount of reinforcing bar

Minimum amount of reinforcing bar is 0.2 % and over of effective sectional area of member.

Member	b	h	d'	Formula							Arrangement of minimum reinforcing bar				
	mm	mm	mm	mm^2							mm^2				
Lateral wall	1000.0	1100.0	100.0	1000.0	×	1000.0	×	0.002	=	2,000.0	D	25	@	250	2,026.8

6R soffit		A point				B point				C point			
		Inner surface				Exterior surface				Internal surface			
M	kN・m	153.86				117.99				82.12			
N	kN	582.06				738.65				497.80			
b	mm	1000				1000				1000			
h	mm	1100				1100				1100			
d	mm	1000				1000				1000			
d'	mm	100				100				100			
As	cm^2	D	25	@	250	D	25	@	250	D	25	@	250
		20.268				20.268				20.268			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00203				0.00203				0.00203			
k		0.218				0.218				0.218			
j		0.927				0.927				0.927			
σ c	N/mm^2	1.3		<	12.0	1.2		<	12.0	0.8		<	12.0
σ s	N/mm^2	2.8		<	240	-16.6		<	240	-11.3		<	240
n		15				15				15			

5RSoffit		A point				B point				Cpoint			
		Inner surface				Exterior surface				Inner surface			
M	kN・m	223.36				171.29				119.22			
N	kN	780.64				1,007.97				668.32			
b	mm	1000				1000				1000			
h	mm	1100				1100				1100			
d	mm	1000				1000				1000			
d'	mm	100				100				100			
As	cm^2	D	25	@	250	D	25	@	250	D	25	@	250
		20.268				20.268				20.268			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00203				0.00203				0.00203			
k		0.218				0.218				0.218			
j		0.927				0.927				0.927			
σ c	N/mm^2	1.9		<	12.0	1.7		<	12.0	1.1		<	12.0
σ s	N/mm^2	6.1		<	240	-23.3		<	240	-15.6		<	240
n		15				15				15			

4R soffit		A point	B point	Cpoint
		Inner surface	Exterior surface	Inner surface
M	kN･m	292.87	224.60	156.32
N	kN	979.22	1,277.30	818.84
b	mm	1000	1000	1000
h	mm	1100	1100	1100
d	mm	1000	1000	1000
d'	mm	100	100	100
As	cm^2	D 25 @ 250	D 25 @ 250	D 25 @ 250
		20.268	20.268	20.268
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00203	0.00203	0.00203
k		0.218	0.218	0.218
j		0.927	0.927	0.927
σc	N/mm^2	2.5 < 12.0	2.2 < 12.0	1.5 < 12.0
σs	N/mm^2	9.6 < 240	-29.9 < 240	-1.5 < 240
n		15	15	15

3R soffit		A point	B point	Cpoint
		Inner surface	Exterior surface	Inner surface
M	kN･m	362.38	277.90	193.42
N	kN	1,177.81	1,546.62	979.36
b	mm	1000	1000	1000
h	mm	1100	1100	1100
d	mm	1000	1000	1000
d'	mm	100	100	100
As	cm^2	D 25 @ 250	D 25 @ 250	D 25 @ 250
		20.268	20.268	20.268
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00203	0.00203	0.00203
k		0.218	0.218	0.218
j		0.927	0.927	0.927
σc	N/mm^2	3.1 < 12.0	2.7 < 12.0	1.9 < 12.0
σs	N/mm^2	13.3 < 240	-36.5 < 240	-1.4 < 240
n		15	15	15

2R soffit		A point				B point				Cpoint			
		Inner surface				Exterior surface				Inner surface			
M	kN·m	431.89				331.20				230.52			
N	kN	1,376.39				1,815.95				1,139.88			
b	mm	1000				1000				1000			
h	mm	1100				1100				1100			
d	mm	1000				1000				1000			
d'	mm	100				100				100			
As	cm^2	D	25	@	250	D	25	@	250	D	25	@	250
		20.268				20.268				20.268			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00203				0.00203				0.00203			
k		0.218				0.218				0.218			
j		0.927				0.927				0.927			
σc	N/mm^2	3.7	<	12.0		3.2	<	12.0		2.2	<	12.0	
σs	N/mm^2	17.0	<	240		-43.2	<	240		-1.2	<	240	
n		15				15				15			

Undersurface of base plate		A point				B point				Cpoint			
		Inner surface				Exterior surface				Inner surface			
M	kN·m	444.76				341.07				237.39			
N	kN	1,413.16				1,865.82				1,169.60			
b	mm	1000				1000				1000			
h	mm	1100				1100				1100			
d	mm	1000				1000				1000			
d'	mm	100				100				100			
As	cm^2	D	25	@	250	D	25	@	250	D	25	@	250
		20.268				20.268				20.268			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00203				0.00203				0.00203			
k		0.218				0.218				0.218			
j		0.927				0.927				0.927			
σc	N/mm^2	3.8	<	12.0		3.3	<	12.0		2.3	<	12.0	
σs	N/mm^2	17.7	<	240		-44.4	<	240		-1.2	<	240	
n		15				15				15			

1) Consideration in case of occurence of difference of head of water after sinking

a) Load calculation

Calculation of earth pressure intensity

Earth pressure at rest

$p_a \;=\; K_0\cdot[q_0+\Sigma(\gamma_n\cdot h_n)]$

Hydrostatic pressure

$p_w \;=\; \gamma_w\cdot \Sigma h_n$

To the above formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Vertical load (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil of each strat (kN/m$^{3)}$

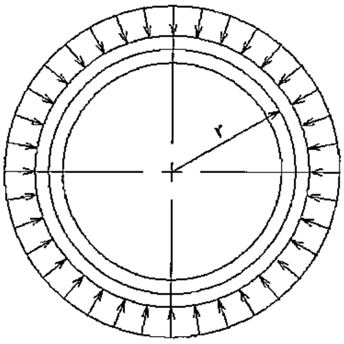
(in case under groundwater level, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Thickness of each stratum (m)

Soil	Elevation	Depth	Thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.820	2.000	2.000	16.0	7.0	10.0	42.00	0.500	21.00	0.00	7R soffit
	-2.250	2.430	0.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Groundwater level
	-7.220	7.400	4.970	16.0	7.0	10.0	83.67	0.500	41.84	49.70	6R soffit
	-12.620	12.800	5.400	16.0	7.0	10.0	121.47	0.500	60.74	103.70	5R soffit
	-14.820	15.000	2.200	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-18.020	18.200	3.200	16.0	7.0	10.0	159.27	0.500	79.64	157.70	4R soffit
	-23.420	23.600	5.400	16.0	7.0	10.0	197.07	0.500	98.54	211.70	3R soffit
	-24.820	25.000	1.400	16.0	7.0	10.0	206.87	0.500	103.44	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	103.44	225.70	-
	-28.820	29.000	4.000	16.0	7.0	10.0	234.87	0.500	117.44	265.70	2R soffit
	-29.820	30.000	1.000	16.0	7.0	10.0	241.87	0.500	120.94	275.70	Undersurface of bottom slab

b) Calculation of section force



- In case of bearing equal load from 4 directions
  (In this case, bending moment does not occure)
  Axial force
  N = 1.000 ・ p ・ r

Form and working load

Checking locatior		Internal diameter	Member thickness	Diamter of center line of member	Radius of center line of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m	m	kN/m^2	kN/m^2	kN/m^2
1	6R soffit	8.400	1.100	9.500	4.750	41.84	49.70	0.00
2	5R soffit	8.400	1.100	9.500	4.750	60.74	103.70	0.00
3	4R soffit	8.400	1.100	9.500	4.750	79.64	157.70	0.00
4	3R soffit	8.400	1.100	9.500	4.750	98.54	211.70	0.00
5	2R soffit	8.400	1.100	9.500	4.750	117.44	265.70	0.00
6	Undersurface of bottom slab	8.400	1.100	9.500	4.750	120.94	275.70	0.00

Calculation for section force

Axial force	Uniform load	Radius	N
	kN/m^2	m	kN/m
6R soffit	91.54	4.750	434.79
5R soffit	164.44	4.750	781.07
4R soffit	237.34	4.750	1,127.34
3R soffit	310.24	4.750	1,473.62
2R soffit	383.14	4.750	1,819.89
Undersurface of bottom slab	396.64	4.750	1,884.02

c) Reviewing section

$$\sigma_c = \frac{N}{A}$$

To the above forumla

σ_c : Compressive stress (N/mm^2)

A : Sectional area of member mm^2

Checking location		N	b	h	A	σ_c	σ_{ca}	Judgement
		kN/m	mm	mm	mm^2	N/mm^2	N/mm^2	
1	6R soffit	434.79	1,000	1,100	1,100,000	0.40	12.00	○
2	5R soffit	781.07	1,000	1,100	1,100,000	0.71	12.00	○
3	4R soffit	1,127.34	1,000	1,100	1,100,000	1.02	12.00	○
4	3R soffit	1,473.62	1,000	1,100	1,100,000	1.34	12.00	○
5	2R soffit	1,819.89	1,000	1,100	1,100,000	1.65	12.00	○
6	Undersurface of bottom slab	1,884.02	1,000	1,100	1,100,000	1.71	12.00	○

4-2. Calculation of cutting edge

1) Consideration of vertical direction

Design for cutting edge is for just before final settlement of Caisson. In the design, design load from outside considers earth pressure at rest plus hydrostatic pressure, while design load from inside considers hydrostatic pressure having the difference of head of water with 3.0 m to outside hydrostatic pressure. In analytical model, span from cutting edge to bottom slab is regarded as cantilever. However, if there is no bottom slab, the span is set with 1.5 m.

a) Load calculation

Calculation of earth pressure intensity

Earth pressure at rest

$p_a \quad = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \quad = \gamma_w \cdot \Sigma h_n$

To the above formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stra (kN/m$^{3)}$

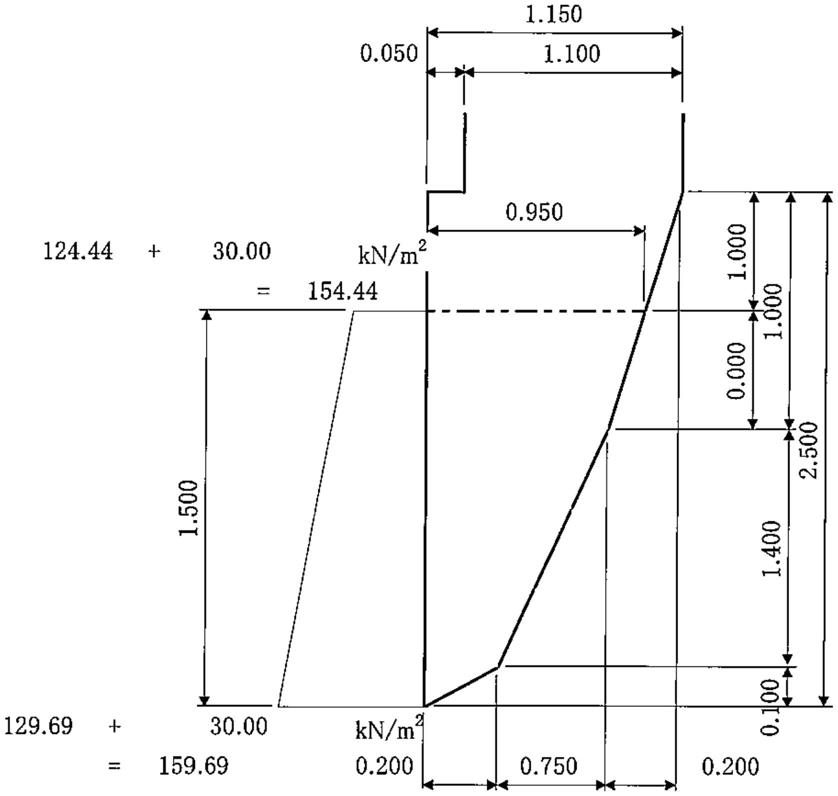
(submerged weight in case under groundwater level)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-2.250	2.430	2.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Groundwater level
	-14.820	15.000	12.570	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-24.820	25.000	10.000	16.0	7.0	10.0	206.87	0.500	103.44	225.70	Change point of
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	103.44	225.70	-
	-29.820	30.000	5.000	16.0	7.0	10.0	241.87	0.500	120.94	275.70	Undersurface of bottom slab
	-30.820	31.000	1.000	16.0	7.0	10.0	248.87	0.500	124.44	285.70	Supporting point of cutting edge
	-32.320	32.500	1.500	16.0	7.0	10.0	259.37	0.500	129.69	300.70	Cutting edge

b) Calculation of sectional force



Bending moment

$$M = \left(\frac{1}{6} \times 154.44 + \frac{1}{3} \times 159.69\right) \times 1.500^2 = 177.68 \text{ kN·m/m}$$

Shear force

$$S = \frac{1}{2} \times (154.44 + 159.69) \times 1.500 = 235.59 \text{ kN/m}$$

c) Reviewing section

- As for minimum volume of reinforcing bar

Minimum volume of reinforcing bar is 0.2% and over of effective sectional area of member.

Member	b	h	d'	Formula							Arrangement of minimum reinforcing bar				
	mm	mm	mm	mm^2							mm^2				
Lateral wall	1000.0	950.0	100.0	1000.0	×	850.0	×	0.002	=	1,700.0	D	22	@	200	1,935.5

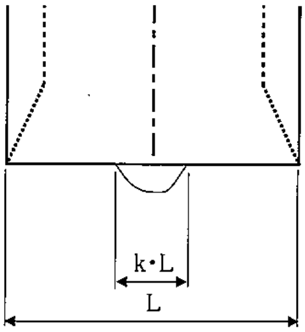
		Cutting edge			
		Exterior surface			
M	kN·m	177.68			
N	kN	0.00			
S	kN	235.59			
b	mm	1000			
h	mm	950			
d'	mm	100			
d	mm	850			
As	cm^2	D 22 @ 250			
		15.484			
p		0.00182			
k		0.208			
j		0.931			
σc	N/mm^2	2.5 < 12.0			
σs	N/mm^2	145.1 < 240			
τ	N/mm^2	0.28 < 0.32			
τ_{al}	N/mm^2	0.35			
C_e		1.086			
C_{pt}		0.864			
C_N		1.000			
n		15			

2) Consideration just after immersion of first lot

After assumption of condition of simple support partially without ground reaction just after sinking work of Caissor and condition of supporting by cantilever of partial bottom slab, considereation is carried out.

Case-1

Simple support



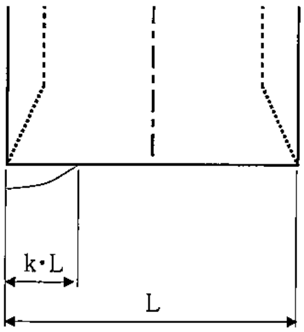
L = 10.700 m

k = 1 / 4

k•L = 2.675 m

Case-2

Cantilever



L = 10.700 m

k = 1 / 5

k•L = 2.140 m

a) Load calculation

Self weight of first lot

Elevation	Height	External diameter	Internal diameter	Sectional area	Average sectional area	Volume
m	m	m	m	m^2	m^2	m^3
-28.820	1.000	10.600	8.400	32.830	32.830	32.830
-29.820		10.600	8.400	32.830		
	1.000	10.700	8.400	34.503	31.801	31.801
-30.820		10.700	8.800	29.099		
	1.400	10.700	8.800	29.099	17.848	24.987
-32.220		10.700	10.300	6.597		
	0.100	10.700	10.300	6.597	3.299	0.330
-32.320		10.700	10.700	0.000		
Total	3.500	-	-	-	-	89.948

$W = 24.5 \times 89.948 = 2{,}203.72$ kN

Perimeter of first lot

$U = \pi \times 10.700 = 33.615$ m

Design load

$$q = \frac{W}{U}$$

$$= \frac{2{,}203.72}{33.615} = 65.56 \text{ kN/m}$$

b) Calculation of section force

- Case−1 : Condition of simple supproting

  Bending moment (tension of underside)

  $$M = \frac{1}{8} \times 65.56 \times 2.675^{2} = 58.64 \quad \text{kN·m}$$

  Shear force

  $$S = \frac{1}{2} \times 65.56 \times 2.675 = 87.68 \quad \text{kN/m}$$

- Case−2 : Condition of canitilever supporting

  Bending moment (Upper side of tension)

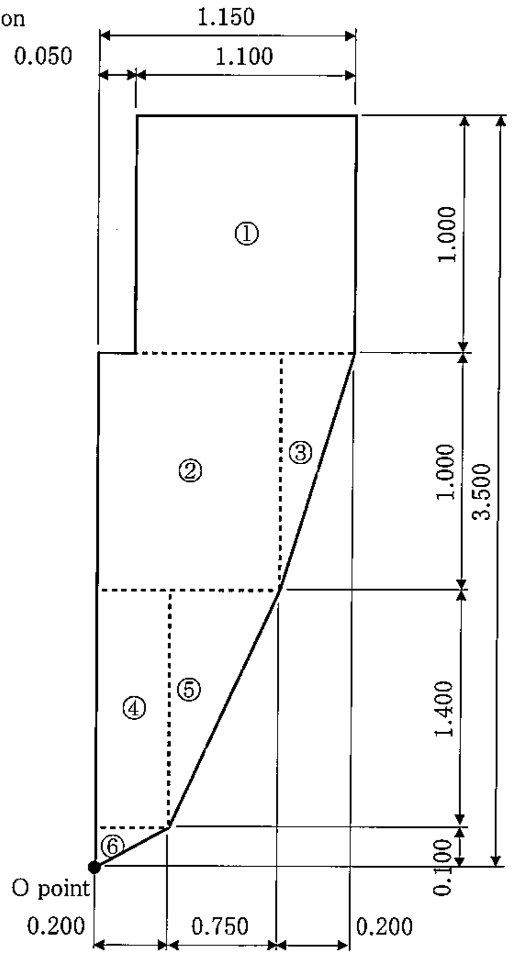
  $$M = \frac{1}{2} \times 65.56 \times 2.140^{2} = 150.11 \quad \text{kN·m}$$

  Shear force

  $$S = 65.56 \times 2.140 = 140.29 \quad \text{kN/m}$$

c) Checking section

Various constant of section



	Formula				A	y	Ay	Ay^2	I
					m^2	m	m^3	m^4	m^4
1		1.100	×	1.000	1.100	3.000	3.300	9.900	0.092
2		0.950	×	1.000	0.950	2.000	1.900	3.800	0.079
3	1/2×	0.200	×	1.000	0.100	2.167	0.217	0.469	0.006
4		0.200	×	1.400	0.280	0.800	0.224	0.179	0.046
5	1/2×	0.750	×	1.400	0.525	1.033	0.543	0.561	0.057
6	1/2×	0.200	×	0.100	0.010	0.067	0.001	0.000	0.000
Total					2.965	2.086	6.184	14.909	0.279

Various constants of section in centroid axis

Geometrical accuracy moment of inertia

$$I = 14.909 + 0.279 - 2.965 \times 2.086^2 = 2.292 \ m^4$$

Modulus of section

$$Z_U = \frac{2.292}{3.500 - 2.086} = 1.620 \ m^3$$

$$Z_L = \frac{2.292}{2.086} = 1.099 \ m^3$$

Checking section

		Case-1			Case-2								
		Under side tension			Upper side tension								
M	kN•m	58.64			150.11								
N	kN	0.00			0.00								
S	kN	87.68			140.29								
Z_U	m^3	1.620			1.620								
Z_L	m^3	1.099			1.099								
A	m^2	2.965			2.965								
σc	N/mm^2	0.04	<	8.25	0.14	<	8.25						
σt	N/mm^2	0.05	<	0.45	0.09	<	0.45						
τ	N/mm^2	0.03	<	0.39	0.05	<	0.39						
Judgement		OK			OK								

4-3. Calculation for earth retaining wall

Design of earth retaining wall for temporaray work is carried out.
Load in that situation is considered to be active earth pressure plus hydrostatic pressure plus uneven earth pressure.

1) Calculation for load

Calculation for earth pressure intensity

Active earth pressure

$p_a \;=\; K_A \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \;=\; \gamma_w \cdot \Sigma h_n$

To the above formula

p_a : Active earth pressure

p_w : Hydrostatic pressure (kN/m^2)

K_A : Coefficient of active earth pressure by Coulomb's earth pressure

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

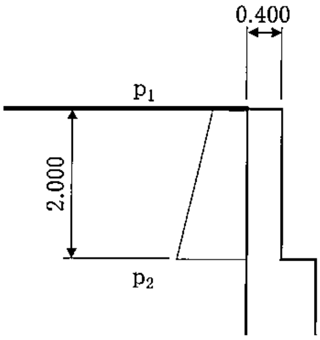
(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vercial load	Earth pressure coefficient	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.180	0.000	0.000	16.0	7.0	10.0	10.00	1.000	10.00	0.00	Ground level
	-1.820	2.000	2.000	16.0	7.0	10.0	42.00	1.000	42.00	0.00	7R soffit
	-2.250	2.430	0.430	16.0	7.0	10.0	48.88	1.000	48.88	0.00	Groundwater level

2) Calculation for sectional force



p_1 = 10.00 + 1/2 × 10.00 = 15.00 kN/m^2

p_1 = 42.00 + 1/2 × 42.00 = 63.00 kN/m^2

Bending moement

$$M = \left(\frac{1}{3} \times 15.00 + \frac{1}{6} \times 63.00\right) \times 2.000^{2} = 62.00 \text{ kN·m/m}$$

Shear force

$$S = \frac{1}{2} \times (15.00 + 63.00) \times 2.000 = 78.00 \text{ kN/m}$$

3) Checking section

- As for minimu volueme of reinforcing bar

Minimum volume of reinforcing bar is set with 0.2 and over of effective sectional area of member.

Member	b	h	d’	Formula							Arrangement of minimum reinforcing bar				
	mm	mm	mm	mm^2							mm^2				
Lateral wall	1000.0	400.0	100.0	1000.0	×	300.0	×	0.002	=	600.0	D	16	@	250	794.4

		Earth retaining wall			
		Exterior surface			
M	kN･m	62.00			
N	kN	0.00			
S	kN	78.00			
b	mm	1000			
h	mm	400			
d’	mm	100			
d	mm	300			
As	cm^2	D 19 @ 250			
		11.460			
p		0.00382			
k		0.286			
j		0.905			
σc	N/mm^2	5.3 < 12.0			
σs	N/mm^2	199.3 < 240			
τ	N/mm^2	0.26 < 0.52			
τ_{al}	N/mm^2	0.35			
C_e		1.400			
C_{pt}		1.082			
C_N		1.000			
n		15			

5. Checking member in regular time

5-1. Calculation for lateral wall

In regular time, only earth pressure at rest plus hydrostatic pressure is set as targes. The pressures are acted towards lateral wall with right angle from 4 directions.

Coefficient of earth pressure at rest adopts 0.5 without difference of sandy soil and cohesive soil. As for distribution of intensity of earth pressure at rest, if the depth is within 15m, the distribution is set as triangular distribution, while if the depth is over 15m, the distribution is considered to be same as intensity of earth pressure at rest.

1) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$$p_a \;=\; K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$$

Hydrostatic pressure

$$p_w \;=\; \gamma_w \cdot \Sigma h_n$$

To the above formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

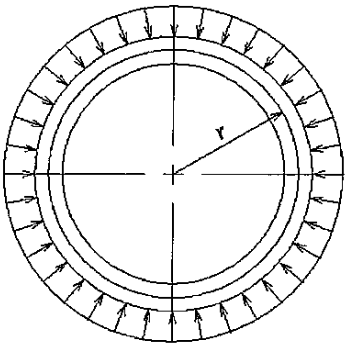
(In case under groundwater level, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.820	2.000	2.000	16.0	7.0	10.0	42.00	0.500	21.00	0.00	7R soffit
	-2.250	2.430	0.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Groundwater level
	-7.220	7.400	4.970	16.0	7.0	10.0	83.67	0.500	41.84	49.70	6R soffit
	-12.620	12.800	5.400	16.0	7.0	10.0	121.47	0.500	60.74	103.70	5R soffit
	-14.820	15.000	2.200	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	15m
	-18.020	18.200	3.200	16.0	7.0	10.0	159.27	0.500	68.44	157.70	4R soffit
	-23.420	23.600	5.400	16.0	7.0	10.0	197.07	0.500	68.44	211.70	3R soffit
	-24.820	25.000	1.400	16.0	7.0	10.0	206.87	0.500	68.44	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	-
	-28.820	29.000	4.000	16.0	7.0	10.0	234.87	0.500	68.44	265.70	2R soffit
	-29.820	30.000	1.000	16.0	7.0	10.0	241.87	0.500	68.44	275.70	Undersurface of bottom slab

2) Calculation for sectional force



- In case of receiving equal loads from 4 directions

  (In this case, there is no occurrence of bending momenet.)

  Axial force

  N = 1.000 · p · r

Form and working load

Checking location		Interior diameter	Thickness of member	Diamter of axis of member	Radius of axis of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m	m	kN/m^2	kN/m^2	kN/m^2
1	6R soffit	8.400	1.100	9.500	4.750	41.84	49.70	0.00
2	5R soffit	8.400	1.100	9.500	4.750	60.74	103.70	0.00
3	4R soffit	8.400	1.100	9.500	4.750	68.44	157.70	0.00
4	3R soffit	8.400	1.100	9.500	4.750	68.44	211.70	0.00
5	2R soffit	8.400	1.100	9.500	4.750	68.44	265.70	0.00
6	Undersurface of bottom slab	8.400	1.100	9.500	4.750	68.44	275.70	0.00

Calculation for sectional force

Axial force	Uniform load	Radius	N
	kN/m^2	m	kN/m
6R soffit	91.54	4.750	434.79
5R soffit	164.44	4.750	781.07
4R soffit	226.14	4.750	1,074.14
3R soffit	280.14	4.750	1,330.64
2R soffit	334.14	4.750	1,587.14
Undersurface of bottom slab	344.14	4.750	1,634.64

c) Checking section

$$\sigma_c = \frac{N}{A}$$

To the above formula

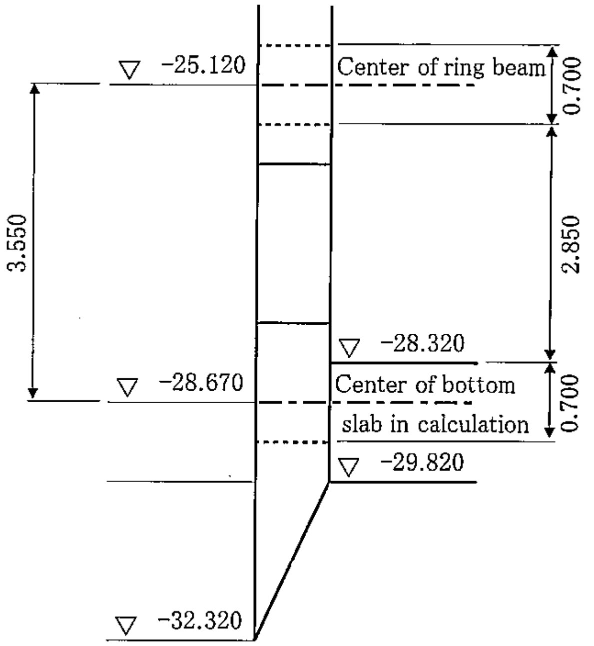
σ_c : Compressive stress (N/mm^2)

A : Sectional area of member mm^2

Checking location		N	b	h	A	σ_c	σ_{ca}	Judgement
		kN/m	mm	mm	mm^2	N/mm^2	N/mm^2	
1	6R soffit	434.79	1,000	1,100	1,100,000	0.40	8.00	○
2	5R soffit	781.07	1,000	1,100	1,100,000	0.71	8.00	○
3	4R soffit	1,074.14	1,000	1,100	1,100,000	0.98	8.00	○
4	3R soffit	1,330.64	1,000	1,100	1,100,000	1.21	8.00	○
5	2R soffit	1,587.14	1,000	1,100	1,100,000	1.44	8.00	○
6	Undersurface of bottom slab	1,634.64	1,000	1,100	1,100,000	1.49	8.00	○

5-2. Calculation for opening of lateral wall

1) Calculation for peripheral of opening of lateral wall (part of both ends fixed beam)



a) Calculation for load

Calclation for earth pressure intensity

Earth pressure at rest

$p_a = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w = \gamma_w \cdot \Sigma h_n$

To this formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

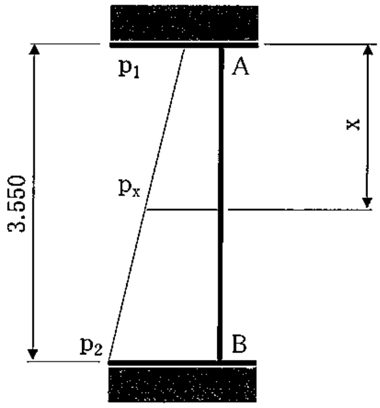
h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficint of earth pressure	Horizaontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-2.250	2.430	2.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Ground water
	-14.820	15.000	12.570	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	15m
	-24.820	25.000	10.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	-
	-25.120	25.300	0.300	16.0	7.0	10.0	208.97	0.500	68.44	228.70	Center of ring beam
	-28.670	28.850	3.550	16.0	7.0	10.0	233.82	0.500	68.44	264.20	Center of bottom slab
	-29.820	30.000	1.150	16.0	7.0	10.0	241.87	0.500	68.44	275.70	Undersurface of bottom slab

p_1 = 68.44 + 228.70 = 297.14 kN/m^2

p_2 = 68.44 + 264.20 = 332.64 kN/m^2

b) Calculation for sectional force



Bending moment of supporting point

$$M_A = (\frac{1}{20} \times 297.14 + \frac{1}{30} \times 332.64) \times 3.550^2 = 326.97 \text{ kN·m/m}$$

$$M_B = (\frac{1}{20} \times 332.64 + \frac{1}{30} \times 297.14) \times 3.550^2 = 334.42 \text{ kN·m/m}$$

Bending moment of span

$$p_x = p_1 + \frac{p_2 - p_1}{L} x$$

$$S_A = \frac{1}{2} \times (p_1 + p_x) \times x$$

From these

$$\frac{p_2 - p_1}{2 \cdot L} x^2 + p_1 x - S_A = 0$$

Therefore

$x = 1.785$ m

Load intensity

$$p_x = 297.14 + \frac{332.64 - 297.14}{3.550} \times 1.785 = 314.99 \text{ kN/m}^2$$

$$M_{max} = - 326.97 - 546.32 \times 1.785$$

$$+ (\frac{1}{3} \times 297.14 + \frac{1}{6} \times 314.99) \times 1.785^2 = 165.36 \text{ kN·m/m}$$

Shear force

$$S_A = (\frac{7}{20} \times 297.14 + \frac{3}{20} \times 332.64) \times 3.550 = 546.32 \text{ kN/m}$$

$$S_B = (\frac{7}{20} \times 332.64 + \frac{3}{20} \times 297.14) \times 3.550 = 571.52 \text{ kN/m}$$

c) Checking section

	On supporting point	Span	Under supporting point	
	Exterior surface	Inner surface	Exterior surface	
M kN·m	326.97	165.36	334.42	
N kN	0.00	0.00	0.00	
S kN	546.32	0.00	571.52	
b mm	1000	1000	1000	
h mm	1100	1100	1100	
d' mm	127	127	127	
d mm	973	973	973	
As cm^2	D 29 @ 250	D 25 @ 250	D 29 @ 250	
	25.696	20.268	25.696	
p	0.00264	0.00208	0.00264	
k	0.245	0.221	0.245	
j	0.918	0.926	0.918	
σc N/mm^2	3.1 $<$ 8.0	1.7 $<$ 8.0	3.1 $<$ 8.0	
σs N/mm^2	142.4 $<$ 160	90.5 $<$ 160	145.6 $<$ 160	
τ N/mm^2	0.56 $>$ 0.23	0.00 $<$ 0.21	0.59 $>$ 0.23	
τ_{a1} N/mm^2	0.23	0.23	0.23	
C_e	1.015	1.015	1.015	
C_{pt}	0.964	0.908	0.964	
C_N	1.000	1.000	1.000	
n	15	15	15	

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 352.44 \times 10^3 \times 250}{160 \times 973} \times 10^{-2}$$

$= 6.51$ cm^2/m $<$ 4 Number D 16 (= 7.944 cm^2) are arranged.

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.23 \times 1000 \times 973 \times 10^{-3}$

$= 219.08$ kN

$\tau a = 0.23$ N/mm^2

$b = 1000$ mm

$d = 973$ mm

Shear force received by diagonal tension bar

$Sh = S - Sc$

$= 571.52 - 219.08$

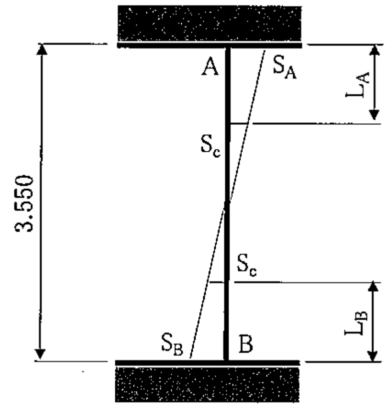
$= 352.44$ kN

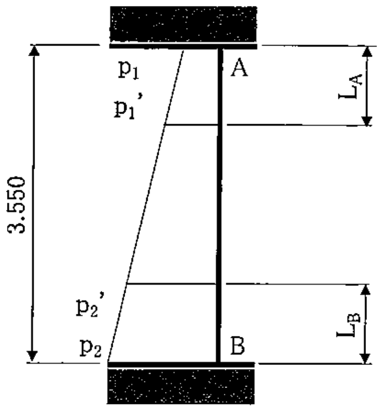
$S = 571.52$ kN

$a = 250$ mm

$\sigma sa = 160$ N/mm^2

Arrangement of sphere of diagonal tension bar





p_1 = 297.14 kN/m^2
p_2 = 332.64 kN/m^2

- Calculation for L_A

$$p_1' = p_1 + \frac{p_2 - p_1}{L} L_A$$

$$S_A = \frac{1}{2} \times (p_1 + p_1') \times L_A + S_c$$

From these

$$\frac{p_2 - p_1}{2 \cdot L} L_A^2 + p_1 L_A + S_c - S_A = 0$$

Therefore

L_A = 1.082 m

- Calculation for L_B

$$p_2' = p_2 - \frac{p_2 - p_1}{L} L_B$$

$$S_B = \frac{1}{2} \times (p_2' + p_2) \times L_B + S_c$$

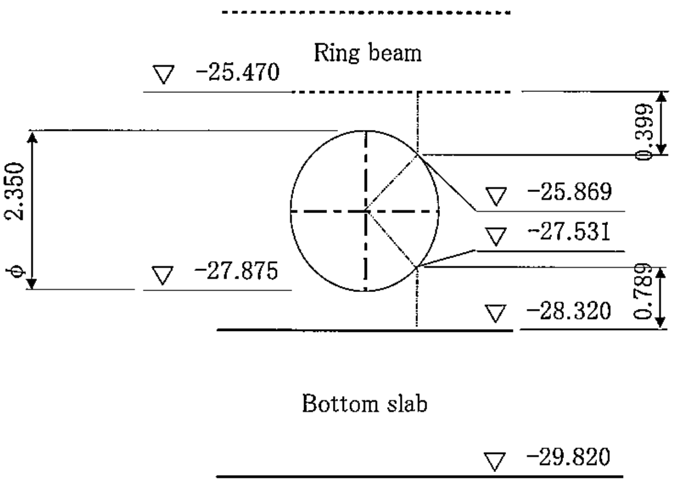
From these

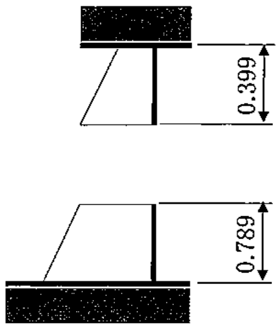
$$-\frac{p_2 - p_1}{2 \cdot L} L_B^2 + p_2 L_B + S_c - S_B = 0$$

Therefore

L_A = 1.077 m

2) Calculatation for peripheral of opening of lateral wall(part of cantilever)





a) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$p_a \ = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \ = \gamma_w \cdot \Sigma h_n$

To this

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil of each stratum (kN/m$^{3)}$

(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coeffieicne of earth	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-2.250	2.430	2.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Groundwater level
	-14.820	15.000	12.570	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	15m
	-24.820	25.000	10.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	-
	-25.470	25.650	0.650	16.0	7.0	10.0	211.42	0.500	68.44	232.20	Soffit of ring beam
	-25.869	26.049	0.399	16.0	7.0	10.0	214.21	0.500	68.44	236.19	Soffit of upper side of opening
	-27.531	27.711	1.662	16.0	7.0	10.0	225.85	0.500	68.44	252.81	Soffit of lower side of opening
	-28.320	28.500	0.789	16.0	7.0	10.0	231.37	0.500	68.44	260.70	Upper side of bottom slab

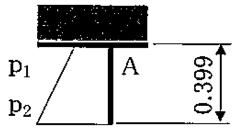
$p_1 = 68.44 + 232.20 = 300.64$ kN/m^2

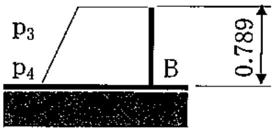
$p_2 = 68.44 + 236.19 = 304.63$ kN/m^2

$p_3 = 68.44 + 252.81 = 321.24$ kN/m^2

$p_4 = 68.44 + 260.70 = 329.14$ kN/m^2

b) Calculation for sectional force





Bending moement of supporting point

$$M_A = \left(\frac{1}{6} \times 300.64 + \frac{1}{3} \times 304.63\right) \times 0.399^2 = 24.16 \ \mathrm{kN{\cdot}m/m}$$

$$M_B = \left(\frac{1}{3} \times 321.24 + \frac{1}{6} \times 329.14\right) \times 0.789^2 = 100.85 \ \mathrm{kN{\cdot}m/m}$$

Shear force

$$S_A = \frac{1}{2} \times (300.64 + 304.63) \times 0.399 = 120.79 \ \mathrm{kN/m}$$

$$S_B = \frac{1}{2} \times (321.24 + 329.14) \times 0.789 = 256.62 \ \mathrm{kN/m}$$

c) Checking section

	On supporting point	Under supporting point		
	Exterior surface	Exterior surface		
M kN·m	24.16	100.85		
N kN	0.00	0.00		
S kN	120.79	256.62		
b mm	1000	1000		
h mm	1100	1100		
d' mm	125	125		
d mm	975	975		
As cm^2	D 25 @ 250	D 25 @ 250		
	20.268	20.268		
p	0.00208	0.00208		
k	0.220	0.220		
j	0.927	0.927		
σc N/mm^2	0.2 < 8.0	1.0 < 8.0		
σs N/mm^2	13.2 < 160	55.1 < 160		
τ N/mm^2	0.12 < 0.21	0.26 > 0.21		
τ_{a1} N/mm^2	0.23	0.23		
C_e	1.014	1.014		
C_{pt}	0.908	0.908		
C_N	1.000	1.000		
n	15	15		

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 50.12 \times 10^3 \times 250}{160 \times 975} \times 10^{-2}$$

$= 0.92\ cm^2/m < 4$ 本 D 13 (= 5.068 cm^2) is arranged/

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.21 \times 1000 \times 975 \times 10^{-3}$

$= 206.50$ kN

$\tau a = 0.21\ N/mm^2$

b = 1000 mm

d = 975 mm

Shear force received by diagonal tension bar

Sh = S − Sc

= 256.62 − 206.50

= 50.12 kN

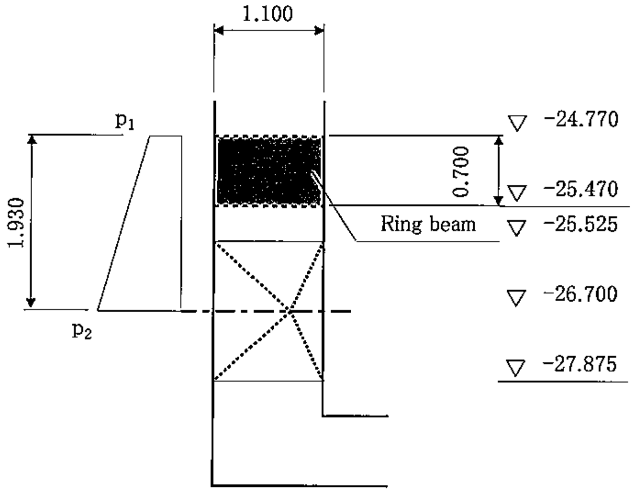
S = 256.62 kN

a = 250 mm

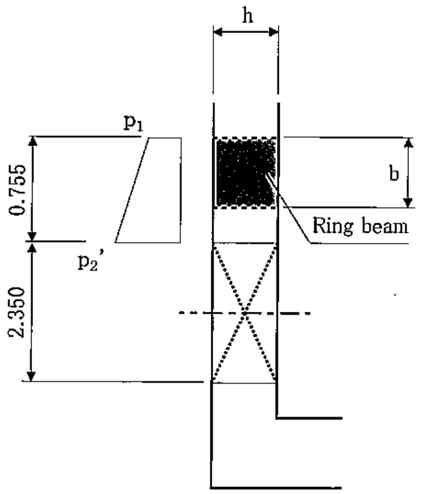
$\sigma sa = 160\ N/mm^2$

3) Calculation for ring beam

Load applied into ring beam



Location of edge of opening part



Location of center of opening part

a) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$p_a = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w = \gamma_w \cdot \Sigma h_n$

To this formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coeffieicnet of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soild of each stratum (kN/m$^{3)}$

(In case under groundwater level, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.180	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-2.250	2.430	2.430	16.0	7.0	10.0	48.88	0.500	24.44	0.00	Ground water level
	-14.820	15.000	12.570	16.0	7.0	10.0	136.87	0.500	68.44	125.70	Change point of stratum
Ac2	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	-
	-14.820	15.000	0.000	16.0	7.0	10.0	136.87	0.500	68.44	125.70	15m
	-24.820	25.000	10.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	Change point of stratum
Ac3	-24.820	25.000	0.000	16.0	7.0	10.0	206.87	0.500	68.44	225.70	-
	-24.770	24.950	-0.050	16.0	7.0	10.0	206.52	0.500	68.44	225.20	Upper bed of ring
	-25.470	25.650	0.700	16.0	7.0	10.0	211.42	0.500	68.44	232.20	Soffit of ring
	-25.525	25.705	0.055	16.0	7.0	10.0	211.81	0.500	68.44	232.75	Upper bed of opening
	-26.700	26.880	1.175	16.0	7.0	10.0	220.03	0.500	68.44	244.50	Center of opening

$p_1 = 68.44 + 225.20 = 293.64$ kN/m^2

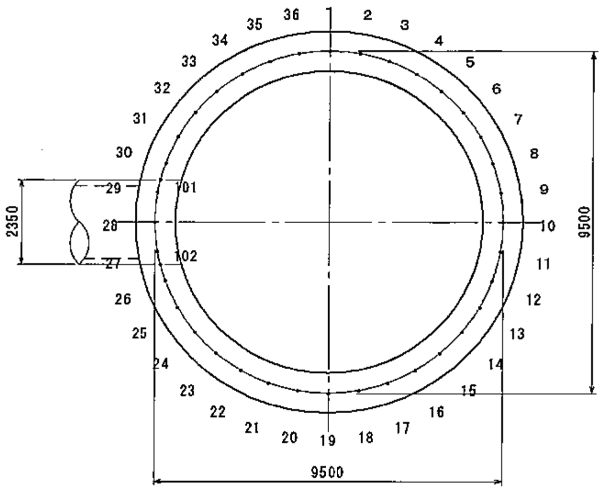
$p_2 = 68.44 + 244.50 = 312.94$ kN/m^2

$p_2' = 68.44 + 232.75 = 301.19$ kN/m^2

$$p_{w1} = \frac{1}{2} \times (\ 293.64 + 312.94\ ) \times 1.930 = 585.34 \text{ kN/m}$$

$$p_{w2} = \frac{1}{2} \times (\ 293.64 + 301.19\ ) \times 0.755 = 224.54 \text{ kN/m}$$

b) Skeleton diagram



Coordinates of panel point

Panel point Number	Coordinates		Panel point Number	Coordinates		Panel point Number	Coordinates	
	x (m)	y (m)		x (m)	y (m)		x (m)	y (m)
1	0.000	4.750	19	0.000	-4.750	101	-4.602	1.175
2	0.825	4.678	20	-0.825	-4.678	102	-4.602	-1.175
3	1.625	4.464	21	-1.625	-4.464	-	-	-
4	2.375	4.114	22	-2.375	-4.114	-	-	-
5	3.053	3.639	23	-3.053	-3.639	-	-	-
6	3.639	3.053	24	-3.639	-3.053	-	-	-
7	4.114	2.375	25	-4.114	-2.375	-	-	-
8	4.464	1.625	26	-4.464	-1.625	-	-	-
9	4.678	0.825	27	-4.678	-0.825	-	-	-
10	4.750	0.000	28	-4.750	0.000	-	-	-
11	4.678	-0.825	29	-4.678	0.825	-	-	-
12	4.464	-1.625	30	-4.464	1.625	-	-	-
13	4.114	-2.375	31	-4.114	2.375	-	-	-
14	3.639	-3.053	32	-3.639	3.053	-	-	-
15	3.053	-3.639	33	-3.053	3.639	-	-	-
16	2.375	-4.114	34	-2.375	4.114	-	-	-
17	1.625	-4.464	35	-1.625	4.464	-	-	-
18	0.825	-4.678	36	-0.825	4.678	-	-	-

Sectional area and moment of second order

	Width	Height	Sectional area	Moment of second order
	b	h	A	I
	m	m	m^2	m^4
Ring beam	0.700	1.100	0.770	0.077642

Sectional area $A = b \cdot h$

Second moment of area $I = \frac{1}{12} b \cdot h^3$

Calculation for coefficient of ground reaction

Coefficient of horizontal ground reaction

$$k_H = k_{H0} \cdot \left(\frac{B_H}{0.3}\right)^{-3/4}$$

$$k_{H0} = \frac{1}{0.3} \cdot \alpha \cdot E_0$$

$$\alpha = 1$$

$$E_0 = 50{,}400 \ \mathrm{kN/m^2}$$

$$B_H = \sqrt{A_H}$$

$$= \sqrt{0.8 \times 10.600 \times 28.000}$$

$$= 15.409 \ \mathrm{m} \geqq 10.0 \ \mathrm{m}$$

$$k_H = 168{,}000 \times \left(\frac{10.000}{0.3}\right)^{-3/4}$$

$$= 12{,}110 \ \mathrm{kN/m^3}$$

$$k_{H0} = \frac{1}{0.3} \times 1 \times 50{,}400$$

$$= 168{,}000 \ \mathrm{kN/m^3}$$

$$K_H = 12{,}110 \times 0.700$$

$$= 8{,}477 \ \mathrm{kN/m^2}$$

※ Only compression spring is valid.

c) Calculation for sectional force

Diagram of load

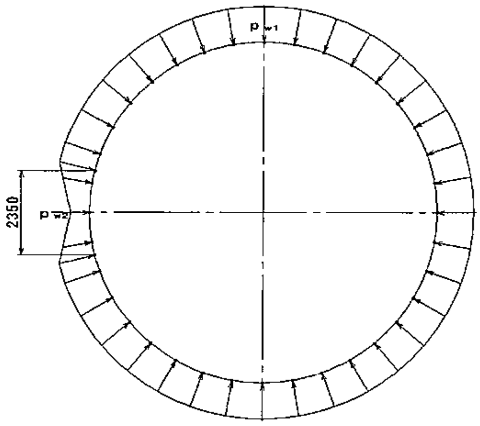
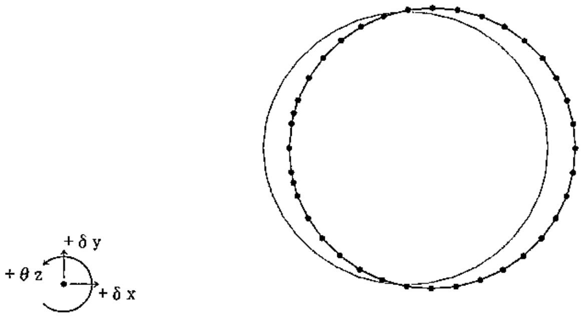
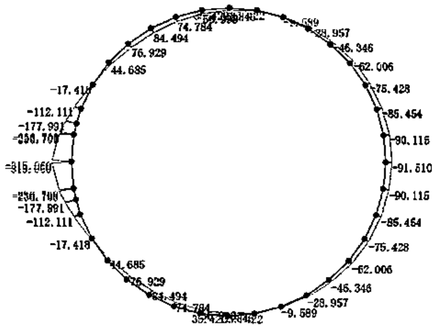


Diagram of transposition



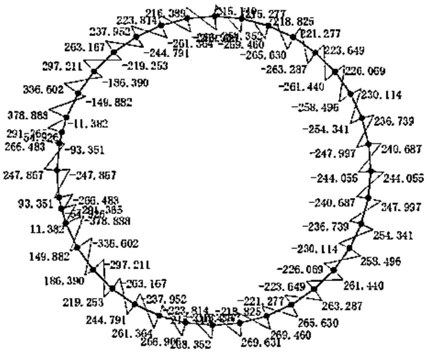
Stress diagram

Diagram Mz of sectional force



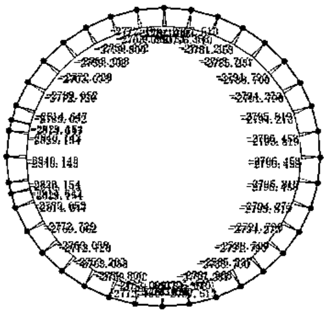
+M (i——j) +M

Diagram Sy of sectional force



+S ↑ i——j ↓ +S

Diagram Nx of sectional force



+N ← i——j → +N

c) Checking section

- Checking to bending

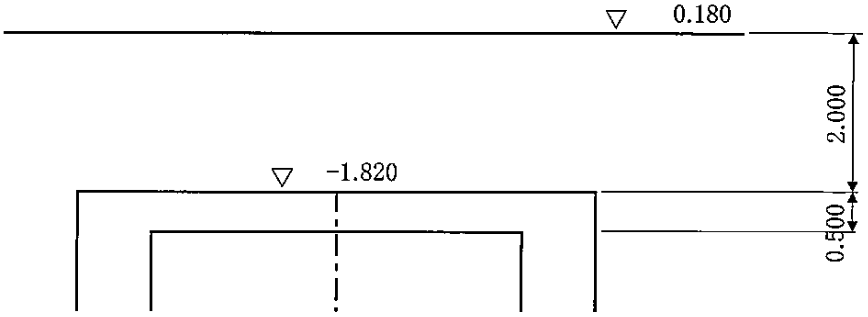
		Panel point 10	Panel point 22	Panel point 28	
		Exterior surface	Inner surface	Exterior surface	
M	kN･m	91.51	84.49	315.06	
N	kN	2,796.46	2,759.50	2,840.14	
S	kN	0.00	0.00	0.00	
b	mm	700	700	700	
h	mm	1100	1100	1100	
d'	mm	100	100	100	
d	mm	1000	1000	1000	
As	cm^2	3 Number D 25	3 Number D 25	3 Number D 25	
		15.201	15.201	15.201	
p		0.00217	0.00217	0.00217	
k		0.225	0.225	0.225	
j		0.925	0.925	0.925	
σc	N/mm^2	4.1 < 8.0	4.0 < 8.0	5.7 < 8.0	
σs	N/mm^2	-62.1 < 160	-60.7 < 160	-85.3 < 160	
τ	N/mm^2	0.00 < 0.42	0.00 < 0.42	0.00 < 0.42	
τ_{a1}	N/mm^2	0.23	0.23	0.23	
C_e		1.000	1.000	1.000	
C_{pt}		0.917	0.917	0.917	
C_N		2.000	2.000	2.000	
n		15	15	15	

- Checking to shear

		Panel point26						
		Exterior surface						
M	kN·m	112.11						
N	kN	2,801.85						
S	kN	183.75						
b	mm	700						
h	mm	1100						
d'	mm	100						
d	mm	1000						
As	cm^2	3	本	D	25			
		15.201						
p		0.00217						
k		0.225						
j		0.925						
σc	N/mm^2	4.3	<	8.0				
σs	N/mm^2	-64.3	<	160				
τ	N/mm^2	0.26	<	0.42				
τ_{al}	N/mm^2	0.23						
C_e		1.000						
C_{pt}		0.917						
C_N		2.000						
n		15						

5-3. Design of top slab

1) Calculation for load



Load of earth covering

$w_s = 19.0 \times 2.000 = 38.00$ kN/m^2

Empty load of top slab

$w_t = 24.5 \times 0.500 = 12.25$ kN/m^2

Live load

$q = 10.00$ kN/m^2

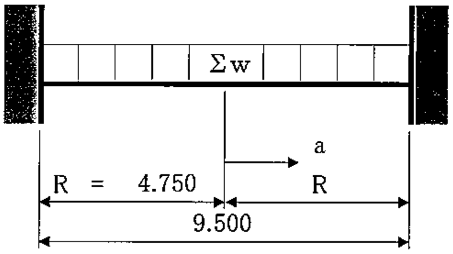
$\Sigma w = 60.25$ kN/m^2

(Reference)

$$w_l = \frac{2 \times 15 \times (1 + 0.3) \times 1.0}{2.700 \times (2 \times 2.000 + 0.200)} = 3.44 \text{ kN/m}^2$$

2) Calculation for sectional force

Peripheral of circular plate is supposed to be fixed on lateral wall and sectional force is calculated.



Bending moement

$$Mr = \frac{1}{16} \cdot \Sigma w \cdot R^{2} \cdot [(1 + \nu) - (3 + \nu) \cdot (\frac{a}{R})^{2}]$$

Shear force

$$Qr = \frac{1}{2} \cdot \Sigma w \cdot a$$

To this

Mr : Bending moment applied to top slab(kN·m/m)
Qr : Shear force applied to top slab (kN/m)
Σw : Applied load (kN/m^2)
R : Radius of circular plate (m)
ν : Poisson's ration = 1 / 6
a : Distance from center of cirbular plate(m)

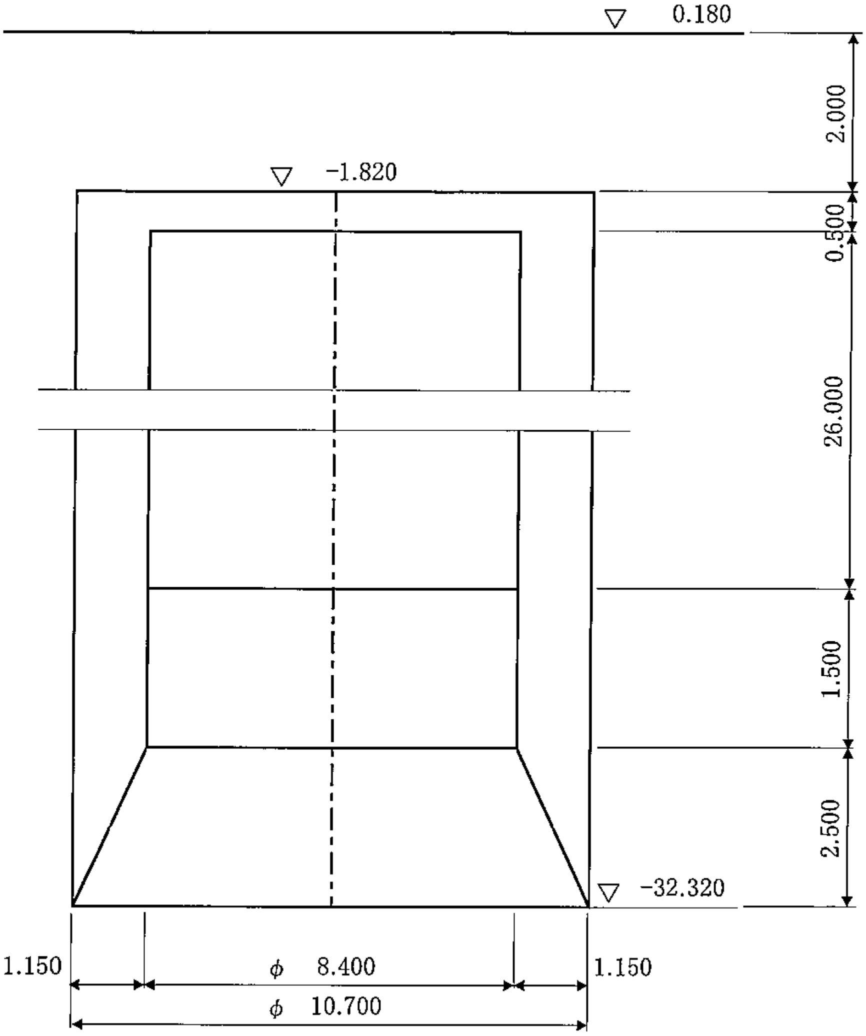
	R	a	Σw	Mr	Qr	Notes
	m	m	kN/m^2	kN·m/m	kN/m	
0	4.750	0.000	60.25	99.12	0.00	Center part
1	4.750	1.000	60.25	87.20	30.13	
2	4.750	2.000	60.25	51.42	60.25	
3	4.750	2.883	60.25	0.01	86.85	Inflection point
4	4.750	3.000	60.25	-8.20	90.38	
5	4.750	3.950	60.25	-86.93	118.99	Checking shear(H/2)
6	4.750	4.000	60.25	-91.67	120.50	
7	4.750	4.750	60.25	-169.92	143.09	Edge

3) Checking section

		Top slab												
		Edge				Center				Checking shear				
M	kN·m	169.92				99.12				86.93				
N	kN	0.00				0.00				0.00				
S	kN	0.00				0.00				118.99				
b	mm	1000				1000				1000				
h	mm	500				500				500				
d’	mm	100				100				100				
d	mm	400				400				400				
As	cm^2	D	22	@	125	D	25	@	250	D	22	@	125	
		30.968				20.268				30.968				
p		0.00774				0.00507				0.00774				
k		0.380				0.321				0.380				
j		0.873				0.893				0.873				
σ c	N/mm^2	6.4		<	8.0	4.3		<	8.0	3.3		<	8.0	
σ s	N/mm^2	157.0		<	160	136.9		<	160	80.3		<	160	
τ	N/mm^2	0.00		<	0.42	0.00		<	0.37	0.30		<	0.42	
τ_{al}	N/mm^2	0.23				0.23				0.23				
C_e		1.343				1.343				1.343				
C_{pt}		1.365				1.204				1.365				
C_N		1.000				1.000				1.000				
n		15				15				15				

5-4. Design of bottom slab

1) Calculation for load



Load of earth covering

$$w_s = 19.0 \times 2.000 = 38.00 \text{ kN/m}^2$$

Empty weight of top slab

$$w_t = 24.5 \times 0.500 = 12.25 \text{ kN/m}^2$$

Live load

$$q = 10.00 \text{ kN/m}^2$$

Empty weight of lateral wall

$$w_w = \frac{24.5 \times (10.600^2 - 8.400^2)}{10.700^2} \times 26.000 = 567.29 \text{ kN/m}^2$$

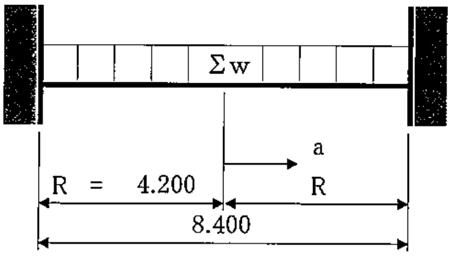
Empty weight of medium slab

$$w_m = 5 \times 24.5 \times 0.400 = 49.00 \text{ kN/m}^2$$

$$\Sigma w = 676.54 \text{ kN/m}^2$$

2) Calculation for sectional force

Circular plate, with the peripheral fixed on lateral wall is suuposed and sectional force is calculated.



Bending moment

$$Mr = \frac{1}{16} \cdot \Sigma w \cdot R^{2} \cdot [(1 + \nu) - (3 + \nu) \cdot (\frac{a}{R})^{2}]$$

Shear force

$$Qr = \frac{1}{2} \cdot \Sigma w \cdot a$$

To this

Mr : Bending moment applied to bottom slab (kN·m/m)
Qr : Shear force applied to bottom slab(kN/m)
Σw : Applied load(kN/m^2)
R : Radius of circular plate (m)
ν : Poisson's ration = 1 / 6
a : Distance from the center of circular plates(m)

	R	a	Σw	Mr	Qr	Notes
	m	m	kN/m^2	kN·m/m	kN/m	
0	4.200	0.000	676.54	870.19	0.00	Center
1	4.200	1.000	676.54	736.30	338.27	
2	4.200	2.000	676.54	334.60	676.54	
3	4.200	2.549	676.54	0.21	862.24	Inflection point
4	4.200	3.000	676.54	-334.89	1014.80	
5	4.200	3.450	676.54	-723.52	1167.02	Checking shear (H/2)
6	4.200	3.581	676.54	-846.85	1211.34	1/2As
7	4.200	4.000	676.54	-1272.17	1353.07	
8	4.200	4.200	676.54	-1491.76	1420.72	Edge

3) Checking section

		Bottom slab			1/2As
		Edge	Center	Checking shear	Edge
M	kN･m	1491.76	870.19	723.52	846.85
N	kN	0.00	0.00	0.00	0.00
S	kN	0.00	0.00	1167.02	0.00
b	mm	1000	1000	1000	1000
h	mm	1500	1500	1500	1500
d’	mm	110	110	110	110
d_1	mm	1390	1390	1390	1390
d_2	mm	1290	–	1290	1290
A_{s1}	cm^2	D 38 @ 250	D 29 @ 125	D 38 @ 250	D 38 @ 500
		45.600	51.392	45.600	22.800
A_{s2}	cm^2	D 38 @ 250		D 38 @ 250	D 38 @ 500
		45.600		45.600	22.800
p		0.00656	0.00370	0.00656	0.00328
k		0.356	0.282	0.356	0.268
j		0.881	0.906	0.881	0.911
σc	N/mm^2	5.1 < 8.0	3.5 < 8.0	2.5 < 8.0	3.8 < 8.0
σs	N/mm^2	143.8 < 160	134.5 < 160	69.8 < 160	159.9 < 160
τ	N/mm^2	0.00 < 0.28	0.00 < 0.23	0.85 > 0.29	0.00 < 0.22
τ_{a1}	N/mm^2	0.23	0.23	0.23	0.23
C_e		0.942	0.942	0.949	0.942
C_{pt}		1.294	1.070	1.307	1.028
C_N		1.000	1.000	1.000	1.000
n		15	15	15	15

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 770.49 \times 10^{3} \times 250}{160 \times 1390} \times 10^{-2}$$

$= 9.96\ cm^2/m$ < 4 Number D 19 (= 11.460 cm^2) are arranged.

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.29 \times 1000 \times 1390 \times 10^{-3}$

$= 396.54$ kN

$\tau a = 0.29\ N/mm^2$

$b = 1000$ mm

$d = 1390$ mm

Shear force received by diagonal tension bar

$Sh = S - Sc$

$= 1167.02 - 396.54$

$= 770.49$ kN

$S = 1167.02$ kN

$a = 250$ mm

$\sigma sa = 160\ N/mm^2$

Arranging distribution of diagonal tension bar

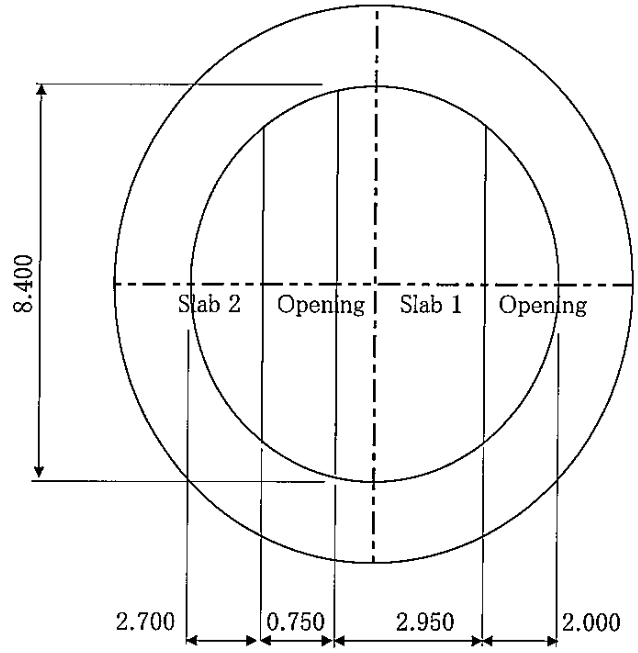
$$L = r - \frac{a}{S} \times S_c$$

$$= 4.200 - \frac{3.450}{1167.02} \times 396.54$$

$= 3.028$ m

5-5. Design of medium slab

1) Calculation of from B1F to B4F



a). Calculation for load

Self weight of medium slab

w_s = 24.5 × 0.400 = 9.80 kN/m^2

Sidewalk live load

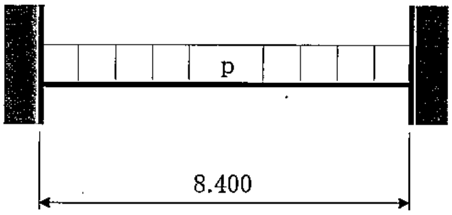
q = 5.00 kN/m^2

p = 14.80 kN/m^2

b). Calculation for stress

- Slab 1

Slab 1 is recognised as beam with both ends built-in and sectional force is evaluated.



Bending moment of supporting point

$$M = \frac{1}{12} \times 14.80 \times 8.400^{2} = 87.02 \text{ kN·m/m}$$

Bending moment of span

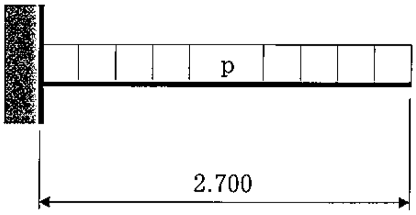
$$M_B = \frac{1}{24} \times 14.80 \times 8.400^{2} = 43.51 \text{ kN·m/m}$$

Shear force

$$S = \frac{1}{2} \times 14.80 \times 8.400 = 62.16 \text{ kN/m}$$

- Slab 2

Slab 2 is recognized as cantilever and sectional force is estimated.



Bending moment of supporting point

$$M = \frac{1}{2} \times 14.80 \times 2.700^{2} = 53.95 \text{ kN·m/m}$$

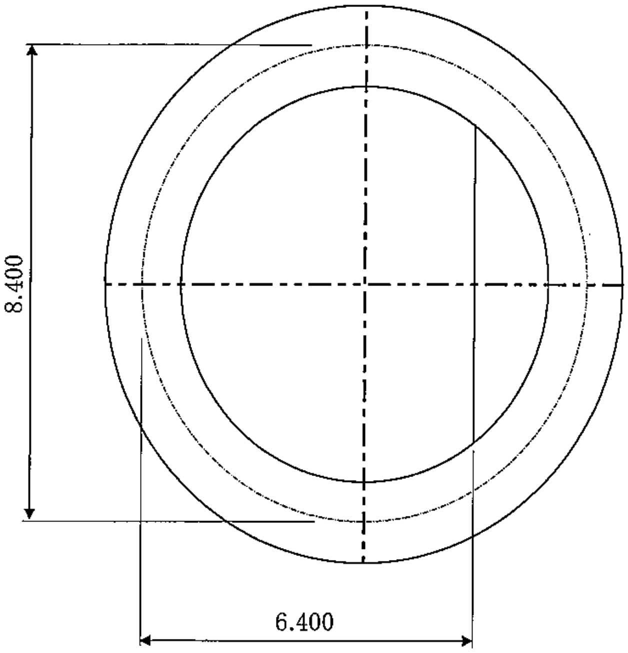
Shear force

$$S = 14.80 \times 2.700 = 39.96 \text{ kN/m}$$

c). Checking section

		Slab 1		Slab 2	
		Supporting point	Span	Supporting point	
M	kN·m	87.02	43.51	53.95	
N	kN	0.00	0.00	0.00	
S	kN	62.16	0.00	39.96	
b	mm	1000	1000	1000	
h	mm	400	400	400	
d’	mm	100	100	100	
d	mm	300	300	300	
As	cm^2	D 25 @ 250	D 19 @ 250	D 19 @ 250	
		20.268	11.460	11.460	
p		0.00676	0.00382	0.00382	
k		0.360	0.286	0.286	
j		0.880	0.905	0.905	
σc	N/mm^2	6.1 < 8.0	3.7 < 8.0	4.6 < 8.0	
σs	N/mm^2	162.6 < 180	139.9 < 180	173.4 < 180	
τ	N/mm^2	0.21 < 0.42	0.00 < 0.35	0.13 < 0.35	
τ_{a1}	N/mm^2	0.23	0.23	0.23	
C_e		1.400	1.400	1.400	
C_{pt}		1.305	1.082	1.082	
C_N		1.000	1.000	1.000	
n		15	15	15	

3) Calculation for B5F



a). Calculation for load

Self weight of medium slab

w_s = 24.5 × 0.400 = 9.80 kN/m^2

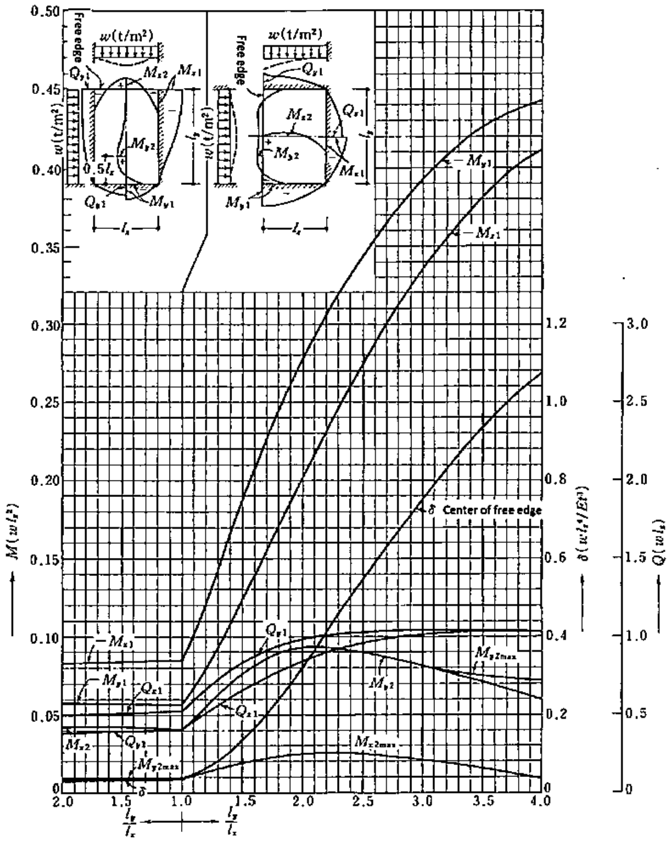
Sidewalk live load

q = 5.00 kN/m^2

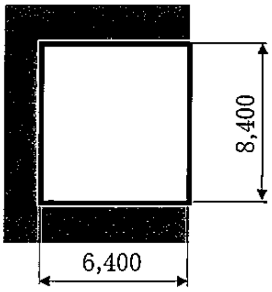
p = 14.80 kN/m^2

b). Calculation for stress

It is recognised as trilateral fixed slab by making axis of member side length.



Attached diagram 10.3 stress diagram of slab with three clamped edges and one free edge and deformation σ11(v=0) of center of free edge.



$$\frac{L_y}{L_x} = \frac{8.400}{6.400} = 1.31$$

From left diagram

$-M_{x1} = 0.095$

$M_{x2} = 0.015$

$Q_{x1} = 0.23$

$-M_{y1} = 0.145$

$M_{y2max} = 0.064$

$Q_{y1} = 0.30$

X direction (Direction for short span)

Bending moment of supporting point

$M_{\text{supporting}} = 0.095 \times 14.80 \times 6.400^{2} = 57.59$ kN·m/m

Bending moment of span

$M_{\text{diameter}} = 0.015 \times 14.80 \times 6.400^{2} = 9.09$ kN·m/m

Shear force

- $x = 0$

$S = 0.23 \times 14.80 \times 6.400 = 21.79$ kN/m

- $x = 0.200$ m (located h/2 away from inside of lateral wall)

$S = \frac{1 \times 21.79}{6.400} \times (\frac{6.400}{1} - 0.200) = 21.10$ kN/m

Y direction (direction for long span)

Bending moment of supporting point

$M_{\text{supporting point}} = 0.145 \times 14.80 \times 6.400^{2} = 87.90$ kN·m/m

Bending moement of span

$M_{\text{diamete}} = 0.064 \times 14.80 \times 6.400^{2} = 38.80$ kN·m/m

Shear force

- $x = 0$

$S = 0.30 \times 14.80 \times 6.400 = 28.42$ kN/m

- $x = 0.200$ m (Loacated h/2 away from inside of lateral wall)

$S = \frac{2 \times 28.42}{8.400} \times (\frac{8.400}{2} - 0.200) = 27.06$ kN/m

c). Checking section

		X direction (direction for short span)		Y direction (direction for long span)	
		Supporting point	Span	Supporting point	Span
M	kN·m	57.59	9.09	87.90	38.80
N	kN	0.00	0.00	0.00	0.00
S	kN	21.10	0.00	27.06	0.00
b	mm	1000	1000	1000	1000
h	mm	400	400	400	400
d’	mm	100	100	100	100
d	mm	300	300	300	300
As	cm^2	D 22 @ 250	D 16 @ 250	D 25 @ 250	D 16 @ 250
		15.484	7.944	20.268	7.944
p		0.00516	0.00265	0.00676	0.00265
k		0.324	0.245	0.360	0.245
j		0.892	0.918	0.880	0.918
σc	N/mm^2	4.4 < 8.0	0.9 < 8.0	6.2 < 8.0	3.8 < 8.0
σs	N/mm^2	139.0 < 180	41.5 < 180	164.3 < 180	177.3 < 180
τ	N/mm^2	0.07 < 0.39	0.00 < 0.31	0.09 < 0.42	0.00 < 0.31
τ_{al}	N/mm^2	0.23	0.23	0.23	0.23
C_e		1.400	1.400	1.400	1.400
C_{pt}		1.210	0.965	1.305	0.965
C_N		1.000	1.000	1.000	1.000
n		15	15	15	15

5-6. Design of stairs

a). Calculation for load

Self weight of slab

$w_s = 24.5 \times 0.400 \qquad = \quad 9.80$ kN/m^2

Load on part of step

$w_s = 24.5 \times \frac{1}{2} \times 0.200 \qquad = \quad 2.45$ kN/m^2

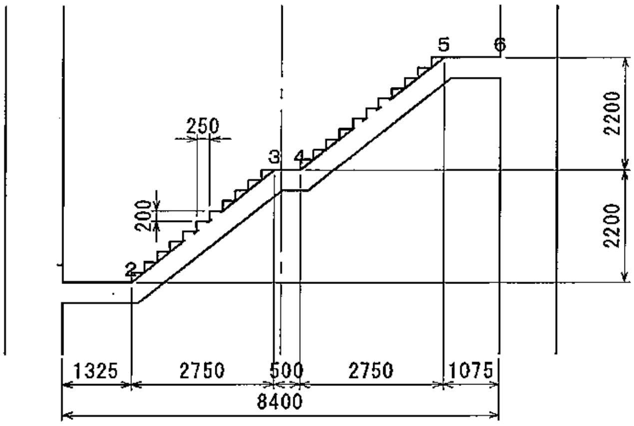
Sidewalk live load

$q \qquad = \quad 5.00$ kN/m^2

$p = 17.25$ kN/m^2

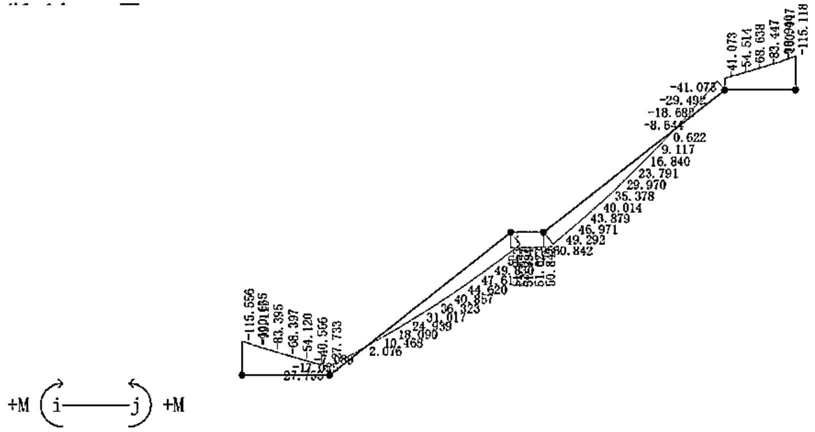
b). Calculation for stress

The model is set as beam with both ends built-in as shown in below diagram.

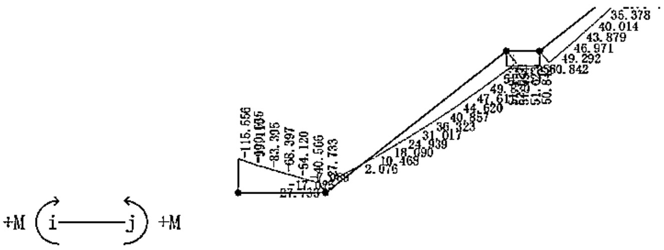


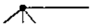
Stress diagram

Mz diagram for sectional force

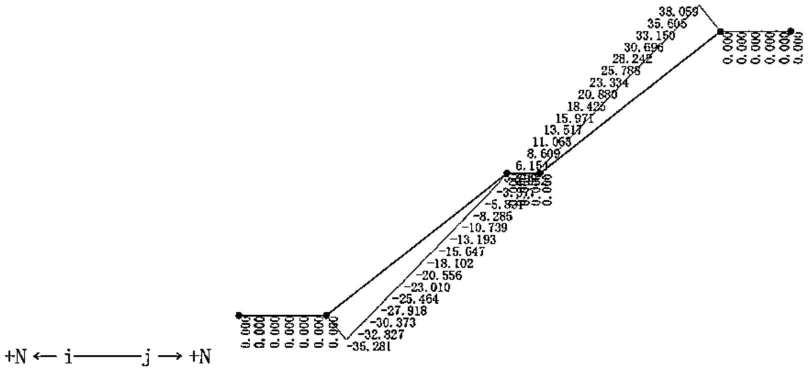


Sy diagram for sectional force





Nx diagram for sectional force



c). Checking section

- Checking to bending

		Stairs			
		Supporting point 1	Span	Supporting point 6	
M	kN·m	115.56	52.03	115.12	
N	kN	0.00	0.00	0.00	
S	kN	0.00	0.00	0.00	
b	mm	1000	1000	1000	
h	mm	400	400	400	
d’	mm	100	100	100	
d	mm	300	300	300	
As	cm^2	D 29 @ 250	D 19 @ 250	D 29 @ 250	
		25.696	11.460	25.696	
p		0.00857	0.00382	0.00857	
k		0.394	0.286	0.394	
j		0.869	0.905	0.869	
σc	N/mm^2	7.5 < 8.0	4.5 < 8.0	7.5 < 8.0	
σs	N/mm^2	172.6 < 180	167.3 < 180	171.9 < 180	
τ	N/mm^2	0.00 < 0.46	0.00 < 0.35	0.00 < 0.46	
τ_{al}	N/mm^2	0.23	0.23	0.23	
C_e		1.400	1.400	1.400	
C_{pt}		1.414	1.082	1.414	
C_N		1.000	1.000	1.000	
n		15	15	15	

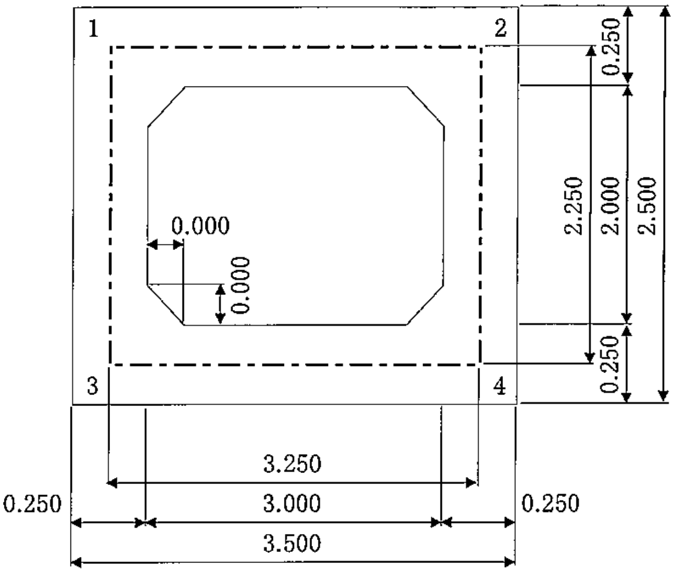
- Checking to shear

		Stairs									
		Supporting point 1				Supporting point 6					
M	kN・m	100.64				100.05					
N	kN	0.00				0.00					
S	kN	73.13				73.87					
b	mm	1000				1000					
h	mm	400				400					
d’	mm	100				100					
d	mm	300				300					
As	cm^2	D	29	@	250	D	29	@	250		
		25.696				25.696					
p		0.00857				0.00857					
k		0.394				0.394					
j		0.869				0.869					
σc	N/mm^2	6.5		<	8.0	6.5		<	8.0		
σs	N/mm^2	150.3		<	180	149.4		<	180		
τ	N/mm^2	0.24		<	0.46	0.25		<	0.46		
τ_{al}	N/mm^2	0.23				0.23					
C_e		1.400				1.400					
C_{pt}		1.414				1.414					
C_N		1.000				1.000					
n		15				15					

5-7. Calculation for cleaning hole

1). Skelton diagram

a). Skelton diagram



b). Sectional area and second moment of area

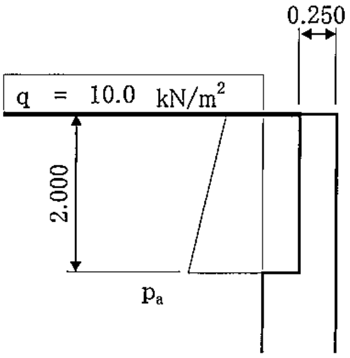
Sectional area A = 0.250 m^2/m

Second moment of area $I = \frac{1}{12} \times 0.250^3 = 0.001302$ m^4/m

c). Point specified for calculation

Member	Point specified for calculation (m)							
	Front face of bearing	(Haunch)	Haunch	h/2	h/2	Haunch	(Haunch)	Front face of bearing
1-2	0.125	0.000	0.125	0.250	3.000	3.125	3.250	3.125
3-4	0.125	0.000	0.125	0.250	3.000	3.125	3.250	3.125
1-3	0.125	0.000	0.125	0.250	2.000	2.125	2.250	2.125
2-4	0.125	0.000	0.125	0.250	2.000	2.125	2.250	2.125

2). Calculation for load



Eartht pressure

p_{a1} = 0.5 × 19.0 × 2.000 = 19.00 kN/m^2

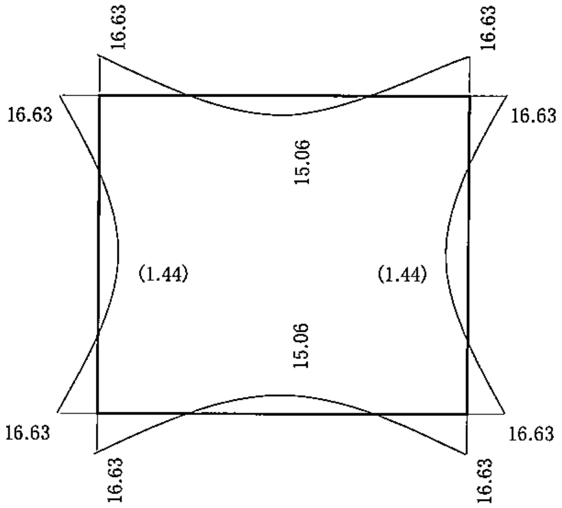
Earth pressure of live load

p_q = 0.5 × 10.0 = 5.00 kN/m^2

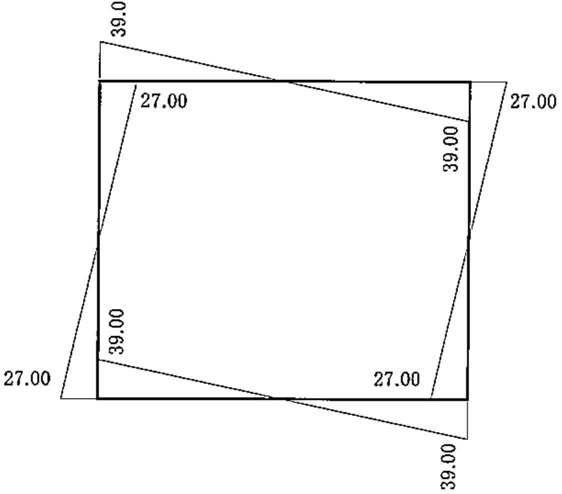
p_a = 24.00 kN/m^2

3). Calculation for stress

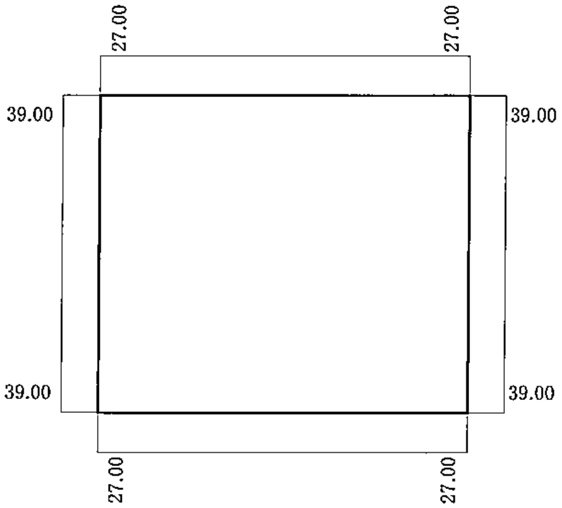
- Diagram for bending moment



- Diagram for shear force



- Diagram for axial force



4). Checking section

- Checking to bending

	1-2,3-4		1-3,2-4	
	Supporting point (Exterior surface)	Span (inner surface)	Supporting point (Exterior surface)	Span (inner surface)
M kN·m	16.63	15.06	16.63	-1.44
N kN	27.00	27.00	39.00	39.00
b mm	1000	1000	1000	1000
h mm	250	250	250	250
d' mm	100	150	100	150
d mm	150	100	150	100
As cm^2	D 19 @ 250	D 19 @ 250	D 19 @ 250	D 19 @ 250
	11.460	11.460	11.460	11.460
p	0.00764	0.01146	0.00764	0.01146
k	0.378	0.439	0.378	0.439
j	0.874	0.854	0.874	0.854
σ c N/mm^2	4.3 < 8.0	7.3 < 8.0	4.3 < 8.0	0.3 < 8.0
σ s N/mm^2	93.1 < 160	125.1 < 160	85.4 < 160	-1.8 < 160
n	15	15	15	15

- Checking to shear force

	Long side		Shohrt side	
	Supporting point		Supporting point	
M kN·m	7.63		10.63	
N kN	27.00		39.00	
S kN	33.00		21.00	
b mm	1000		1000	
h mm	250		250	
d' mm	100		100	
d mm	150		150	
As cm^2	D 19 @ 250		D 19 @ 250	
	11.460		11.460	
p	0.00764		0.00764	
τ N/mm^2	0.22 < 0.50		0.14 < 0.50	
τ_{al} N/mm^2	0.23		0.23	
C_e	1.400		1.400	
C_{pt}	1.358		1.358	
C_N	1.148		1.153	

6. Checking section in earthquake (level 1)

Thickness of member is determined in analysis for buoyancy in construction. The amount of reinforcing bar is determined by the thickness of member.

Moreover, in calculation of shaft B, there is ample proof stress in checking section in earthquake. Therefore, it is assumed that pit A also have simlar trend.

Therefore, chekcing section in earthquake is omitted.

2. Study on Press-in Force for No. A and No. C Shaft

Contents

1. Plan for theory of settlement

In order for Caisson to reach a fixed depth due to gravitational act, the following formula shall be satisfied.

Insertion pressure ＋ Self weight ≧ Buoyancy ＋ Resistance force of skin friction ＋ Resistance force on cutting edge

(Condition)
The calculation of shaft A is adopted as a representative of the calculation of shaft C because both forms are almost same and the depth of A is deeper than shaft C.

(1) Self weight (Wc)

Figure section 1

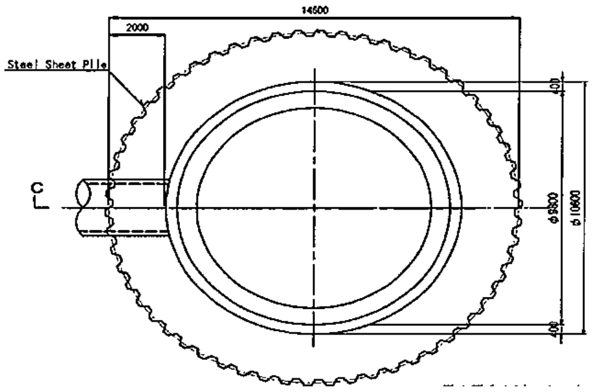


Figure section 2

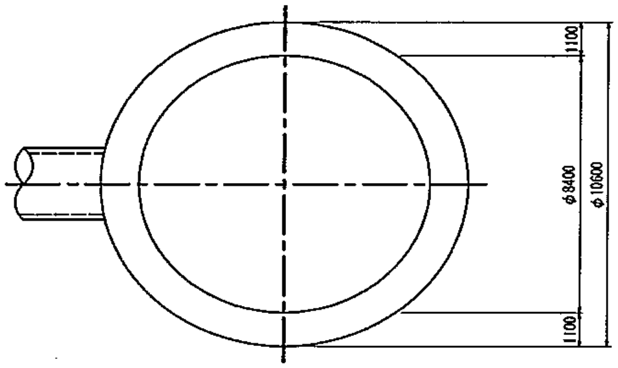
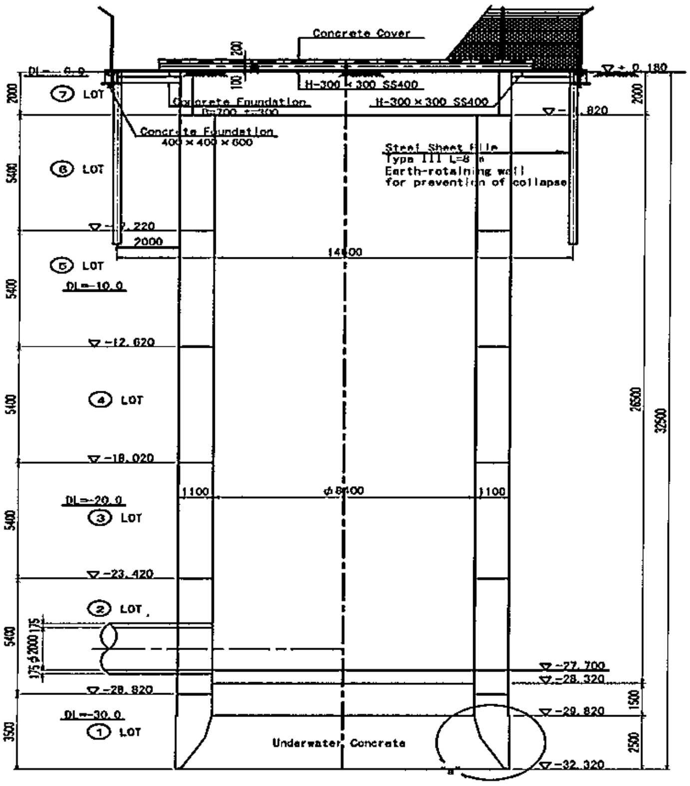


Figure section 3



As setting unit volume weight of reinforcing concrete at 25.0 (KN/m^3),

Lot	Volume of concrete (m³)	Self weight(kN)	
		Weight within interval	Self weight
①	90.4	2260.0	2260.0
②	177.3	4432.5	6692.5
③	177.3	4432.5	11125.0
④	177.3	4432.5	15557.5
⑤	177.3	4432.5	19990.0
⑥	177.3	4432.5	24422.5
⑦	25.6	640.0	25062.5

(2)Buoyancy (U)

As for setting groundwater level at −2.83m

Lot	Depth(m)	Buoyancy (kN)
①	3.000	8.4
②	8.400	1583.4
③	13.800	3356.2
④	19.200	5129.0
⑤	24.600	6901.8
⑥	30.000	8674.6
⑦	32.500	9495.3

1

(3) Resistance force of skin friction (F)

$$F = L \cdot Ha \cdot Fa$$

To this, F : Resistance force of skin friction (kN)
L : Perimeter of Caisson (m)
Ha : Ground contact height of perimeter of Caisson (m)
fa : Skin friction (kN/m²)

The value of skin friction adopts recommended value from "Design guideline of press-in open Caisson" of the Hanshin Expressway Public Corporation as shown in below table.
However, in order to put NF sheet to the spot of friction cut, the value without combined use of promotion of settlement process in the interval from cutting edge to friction cut is calculated,
while the value with combined use of promotion of settlement in the interval of NF sheet is calculated.

Illustration by table-3.2(1) Table of skin friction (kN/m2) (In case without combined use of promotion of settlement process)

Soil	Resources			Actual measurements		Recommended value
	Design and construction of intrusion method (Ohmsha)	Civil engineering handbook	Specification of highway bridge	Section of Uozakihama	Section of Sukematsu	
Clay	———	50.0～200.0	5.0～10.0	32.0	19.0～25.0	30.0
Silt	2.0～ 7.0	———		———	———	
Well tight silt	5.0～ 10.0	———		———	———	
Well tight sand	12.0～22.0	35.0～ 70.0	14.0～24.0	15.0～ 25.0	20.0～36.0	30.0
Sand mixed with gravel	14.0～24.0	———		———	24.0～30.0	
Gravel mixed with sand	17.0～26.0	———	22.0～31.0	———	24.0～44.0	100.0
Well tight gravel	22.0～31.0	50.0～100.0		80.0～130.0	———	
Notes			Actual values in good condtion without resistance of cutting edge attached with friction cut	Interval without NF sheet	Interval without NF sheet	

Illustration by table-3.2(2) Table of skin friction (kN/m2) (In case with combined use of promotion of settlement process)

Depth (m)	Literature *1	Actual measurements by other organization			Actual measurements		Recommended value
		Bypass of Hamadera	Shinfujigawa river	Kishuoohashi bridge	Section of Uozakihama	Section of Sukematsu	
0～ 5	2.0	25.0～39.0 (Average value)	15.0 (Average value)	17.0 (Average value)	6.0	7.0	5.0
5～10	6.0				6.0	8.0	10.0
10～15	10.0				10.0～20.0	14.0	15.0
15～	12.0				10.0～20.0	14.0～17.0	20.0
Notes		NF sheet jetting	NF sheet jetting	NF sheet jetting	NF sheet jetting	NF sheet jetting	

Literature*1 : Recommended value of catalog of NF construction method e

Resistance force of skin friction

Lot	Depth of cutting edge (m)	Resistance force of skin friction (kN)		
		Interval from cutting edge to friction cut	Interval of NF sheet	Total
①	3.000	33.62 × (2.330 × 30.0 + 0.170 × 30.0) = 2521.5	33.30 × (0.500 × 5.0) = 83.3	2604.8
②	8.400	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 0.900 × 10.0) = 1132.2	3653.7
③	13.800	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 5.000 × 10.0 + 1.300 × 15.0) = 3146.9	5668.4
④	19.200	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 5.000 × 10.0 + 5.000 × 15.0 + 1.700 × 20.0) = 6127.2	8648.7
⑤	24.600	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 5.000 × 10.0 + 5.000 × 15.0 + 7.100 × 20.0) = 9723.6	12245.1
⑥	30.000	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 5.000 × 10.0 + 5.000 × 15.0 + 12.500 × 20.0) = 13320.0	15841.5
⑦	32.500	33.62 × (2.500 × 30.0) = 2521.5	33.30 × (5.000 × 5.0 + 5.000 × 10.0 + 5.000 × 15.0 + 15.000 × 20.0) = 14985.0	17506.5

Perimeter of Caisson

Interval between cutting edge and friction cut $\pi \times 10.700 = 33.62$ (m)

Interval of NF sheet $\pi \times 10.600 = 33.30$ (m)

3

(4) Resistance of cutting edge (Q) (from design and construction of intrusion method published by Ohmsha)

As for the case of press–in Caisson, cutting edge is generally embedded in ground. In this situation, it cann be supposed that resistance force of cutting edge is bearring capacity in shallow ground.

Therefore, resistance on cutting edge is calculated from the formula which is generally used in construction of press–in Caisson.

$$Q=A\cdot qd$$

To this Q : Resistance force on cutting edge (kN)
A : Ground contact area of cutting edge (㎡)
qd : Ultimate bearing capacity of ground contacted with cutting edge (kN／㎡)

General formula $qd=C\cdot Nc'+\gamma_1\cdot B'\cdot (Nr'/2)+\gamma_2\cdot Df'\cdot Nq'$

To this C : Cohesion of soil (kN／㎡)
$\gamma 1, \gamma 2$: Unit volume weight of soil above and below cutting edge (kN／㎥)
B/ : Ground contact width of cutting edge (m)
Df/ : Ground contact height of cutting edge (m)
Nc/,Nr/,Nq/ : Coefficient of bearing capacity

Coefficients of bearing capacity, Nc.Nr, and Nq decrease due to excavation condition in Caisson.

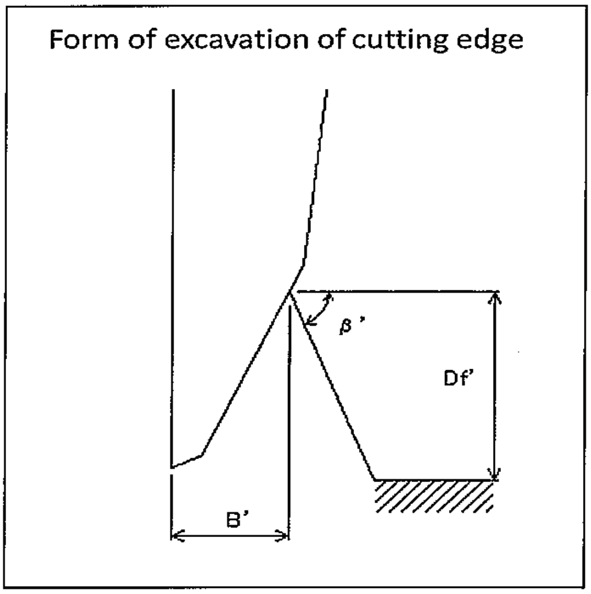
This relatshionship is shown below formula with approximate reduction coefficient, kc, kr related to β, ϕ.

Reduction formula $qd=kc\cdot C\cdot Nc/+kr\cdot \gamma 1\cdot B/\cdot (Nr//2)+\gamma 2\cdot Df/\cdot Nq/$

To this, kc, kr : Reduction coefficient of bearing capacity

As for calculation for resistance on cutting edge, the values of various factors of soil, (C, ϕ), and embedment depth of cutting edge shall be noted because they influence resistance greatly.

Therefore, embedment depth, (Df') and width of resistance of cutting edge, B' shall be determined based on workability, and assuming the condition of engulfment of earth and sand around cutting edge, and the condition of the tightness of earth and sand by press–in.
Accuracy of calculation of resistance of cutting edge is influenced by whether the above assumption is good or bad, which affects economy of construction.
Therefore, deliberate consideration shall be necessary.



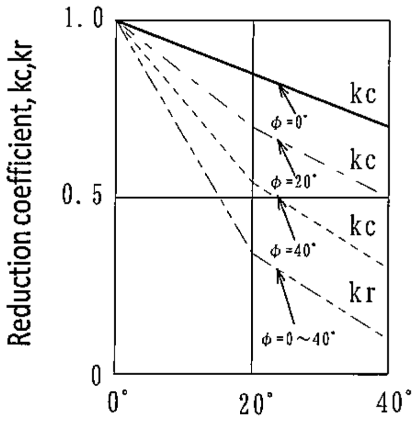
Angle of repose of soil

Soil	β′[Degree]	
	In water	In air
Sand	26	32
Sand mixing clay	18	37
Gravel	16	25
Gravel mixing clay	27	35
Gravel mixing sand and clay	18	35

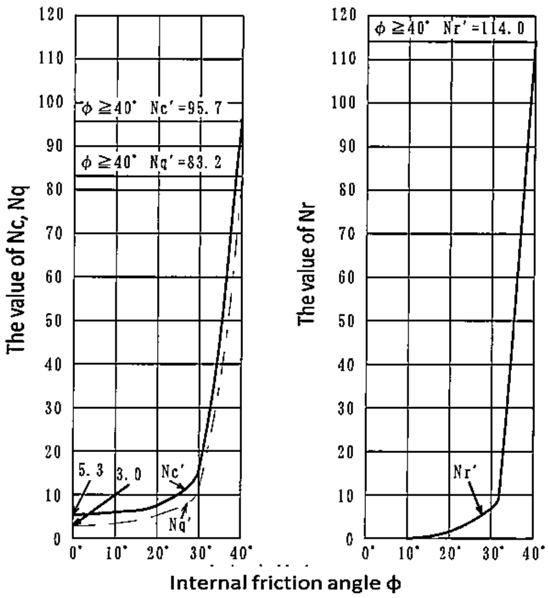
(Measured value by Seiichi Iiyoshi)

Average value of cohesion, C and internal friction angle, ϕ of soil

Soil	ϕ [Degree]	C[N／cm²]
Mudy sand	30	2.0
Well tight sand	34	5.0
Fluid clay	0	0.5
Well soft clay	2	1.0
Soft clay	4	2.0
Medium soft clacy	6	5.0
Tight clay	8	7.5



Reduction coefficient,Kc, kr, from the value of Nc′, Nr′ by B′



The relationship with internal friction angle,ϕ and Nc′, Nq′, Nr′

Ultimate bearing capacity in contact ground of cutting edge

Lot	Soil	ϕ (Degree)	C (kN/m²)	$\gamma_{1,2}$ (kN/m²)	B′ (m)	Df′ (m)	Nc′	Nr′	Nq′	β' (Degree)	kc	kr	qd (kN/m²)
①	Muddy sand	30	20.00	9.00	0.68	1.00	16.0	7.0	12.0	26	0.56	0.28	293.2
②	Muddy sand	30	20.00	9.00	0.84	1.30	16.0	7.0	12.0	26	0.56	0.28	327.0
③	Muddy sand	30	20.00	9.00	0.90	1.40	16.0	7.0	12.0	26	0.56	0.28	338.3
④	Muddy sand	30	20.00	9.00	0.95	1.50	16.0	7.0	12.0	26	0.56	0.28	349.6
⑤	Muddy sand	30	20.00	9.00	0.95	1.50	16.0	7.0	12.0	26	0.56	0.28	349.6
⑥	Tight sand	34	50.00	10.00	0.41	0.50	31.0	20.0	28.0	26	0.51	0.28	942.0
⑦	Tight sand	34	50.00	10.00	0.47	0.60	31.0	20.0	28.0	26	0.51	0.28	971.7

Resistance force on cutting edge

Lot	Soil	ϕ (Degree)	Df′ (m)	B′ (m)	qd (kN/m²)	A (m²)	Q (kN)
①	Muddy sand	30	1.00	0.68	293.2	21.4	6274.5
②	Muddy sand	30	1.30	0.84	327.0	26.0	8502.0
③	Muddy sand	30	1.40	0.90	338.3	27.7	9370.9
④	Muddy sand	30	1.50	0.95	349.6	29.1	10173.4
⑤	Muddy sand	30	1.50	0.95	349.6	29.1	10173.4
⑥	Tight sand	34	0.50	0.41	942.0	13.3	12528.6
⑦	Tight sand	34	0.60	0.47	971.7	15.1	14672.7

(5)Insertion pressure(P)

$P \geqq (U+F+Q)-Wc$

Lot	Load for sinking(kN)		Resistance force for sinking (kN)				Insertion pressure(kN)
	Depth (m)	Empty weight (Wc)	Buoyancy (U)	Skin friction (F)	Resistance force on cutting edge (Q)	Total (U+F+Q)	P≧(U+F+Q) −Wc
①	3.000	2260.0	8.4	2604.8	6274.5	8887.7	6627.7
②	8.400	6692.5	1583.4	3653.7	8502.0	13739.1	7046.6
③	13.800	11125.0	3356.2	5668.4	9370.9	18395.5	7270.5
④	19.200	15557.5	5129.0	8648.7	10173.4	23951.1	8393.6
⑤	24.600	19990.0	6901.8	12245.1	10173.4	29320.3	9330.3
⑥	30.000	24422.5	8674.6	15841.5	12528.6	37044.7	12622.2
⑦	32.500	25062.5	9495.3	17506.5	14672.7	41674.5	16612.0

(6)Examination for sinking

Examination for sinking of building the following lot after each lot immerses

Lot	Load in building next lot(kN)		Sinking resistance(kN)	Judgement
①	6692.5	<	8887.7	OK
②	11125.0	<	13739.1	OK
③	15557.5	<	18395.5	OK
④	19990.0	<	23951.1	OK
⑤	24422.5	<	29320.3	OK
⑥	25062.5	<	37044.7	OK
⑦	※			

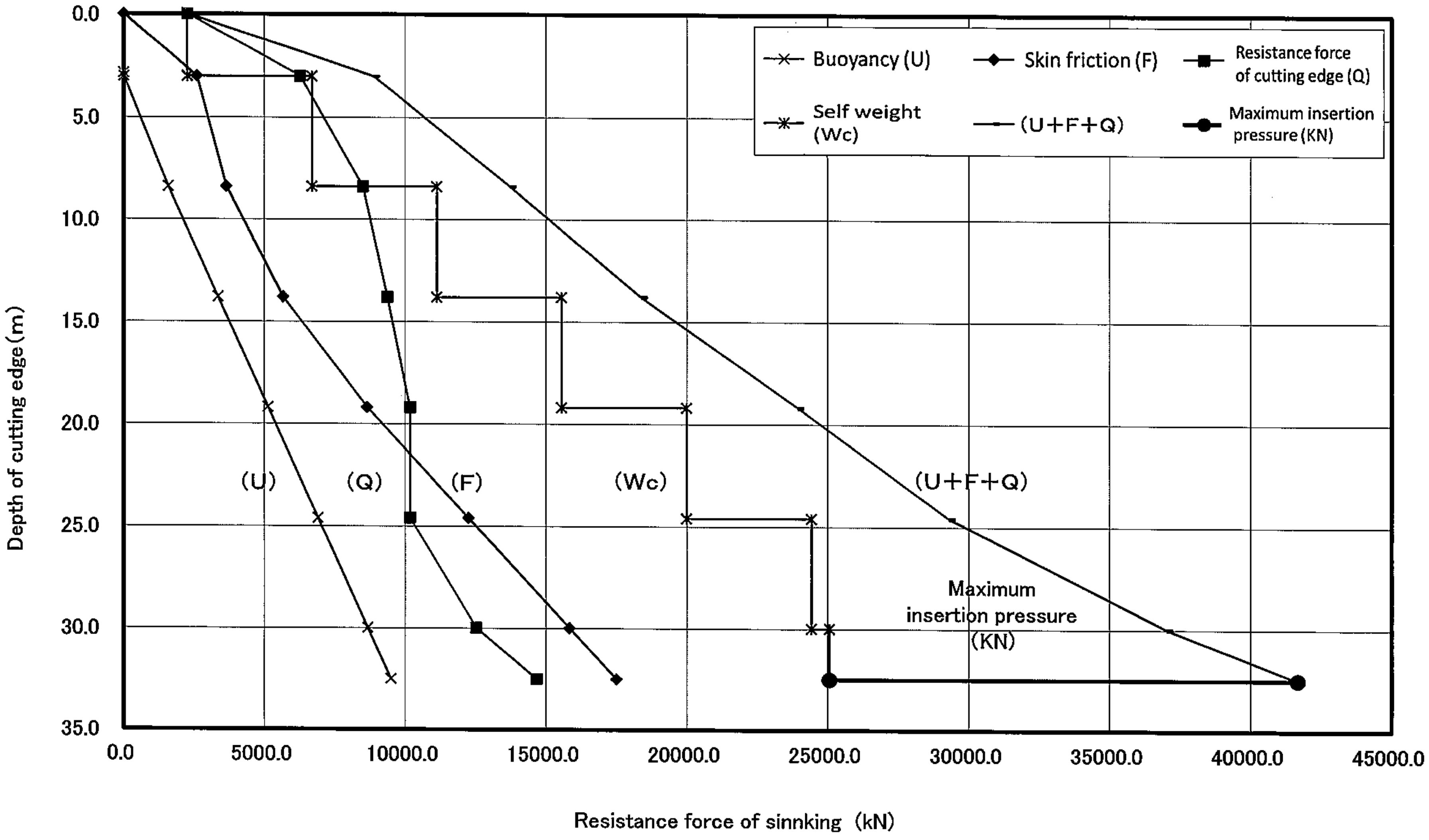
※In the last embedment of lot

In case of complete excavation of cutting edge part in the last embedment

Sinking resistance force＝ 9495.3(Buoyancy)＋17506.5(Skin friction)＋0(Resistance force on cutting edge)
＝ 27001.8 ＞ 25062.5(Load for sinking)・・・・・OK

Therefore, there is no problem if the part of cutting edge is completely excavated.

(6) Relationship diagram of theory of settlement



2. Analysis of the anchor for press fit

(1) The number of anchors and drawing force

If 8 anchors with the maximum pressure P≧16612.0 (kN) are laid and Caisson is pressed in, Drawing force (Pa) per one anchor shall be

$$Pa = \frac{16612.0}{8} = 2076.50 \fallingdotseq 2080 \ (kN／number)$$

(2) Steel wire of the anchor

JIS-G 3536

Nominal designation	Nominal cross sectional area (mm²)	Unit weight (kg/km)	Tension strength		Yield strength		Elongation %
			Tension load (kN)	Tensile stress (N/mm²)	Yield load (kN)	Yield stress (N/mm²)	
φ21.8 over 19	312.9	2,482	573	(1813)	495	(1568)	3.5

Pta = 0.65 × Tension load × 6(number) ---------------- Temporary anchor
= 0.65 × 573 ×6 = 2235 (kN) > Pa = 2080 (kN) ----- OK

(3) Embedment length of anchors (La)

$$La = \frac{Pa \cdot Fs}{\pi \cdot D \cdot \tau a}$$

To this, La : Embedment length (cm)
Pa : Drawing force of the anchor=2,080,000 (N)
Fs : Safety factor=1.5
D : Diameter of body of the anchor=13.5 (cm)
τ a : Frictional resistance of peripheral surface of body of the anchor (N／cm²)

τ a1 :	12.00 (N/cm²)	L1= 750	(cm)
τ a2 :	30.00 (N/cm²)	L2= 100	(cm)
τ a3 :	35.00 (N/cm²)	L3= 100	(cm)
τ a4 :	30.00 (N/cm²)	L4= 100	(cm)
τ a5 :	13.20 (N/cm²)	L5= 1000	(cm)
τ a6 :	35.00 (N/cm²)	L6= 950	(cm)
τ a7 :	10.80 (N/cm²)	L7= 600	(cm)
τ a8 :	35.00 (N/cm²)	L8= X	(cm)

※It is supposed that N value of sandy soil in the depth, (X) with 69.5m and deeper is 35 and over due to lack of information about soil boring log.

$$Pa \cdot Fs \leqq \pi \cdot D \quad (L1 \times \tau a1 + L2 \times \tau a2 + L3 \times \tau a3 + L4 \times \tau a4 + L5 \times \tau a5 + L6 \times \tau a6 + L7 \times \tau a7 + L8 \times \tau a8)$$

$$2{,}080{,}000 \times 1.5 \leqq \pi \times 13.5 \quad (750 \times 12.00 + 100 \times 30.00 + 100 \times 35.00 + 100 \times 30.00 + 1000 \times 13.20 + 950 \times 35.00 + 600 \times 10.80 + L8 \times 35.00)$$

L8 ≒ 61 (cm)

La = L1+L2+L3+L4+L5+L6+L7+L8

= 750+100+100+100+1000+950+600+61 = 3661 (cm) ≒ 37.0 (m)

9

Frictional resistance of peripheral surface of anchors

Type of ground			Frictional resistance (N/㎠)
Bedrock	Hard rock		150.0～250.0
	Soft rock		100.0～150.0
	Weathered rock		60.0～100.0
	Hard pan		60.0～120.0
Gravel	N value	10	10.0～ 20.0
		20	17.0～ 25.0
		30	25.0～ 35.0
		40	35.0～ 45.0
		50	45.0～ 70.0
Sand	N value	10	10.0～ 14.0
		20	18.0～ 22.0
		30	23.0～ 27.0
		40	29.0～ 35.0
		50	30.0～ 40.0
Cohesive soil	1.0c C= Cohesive C= (0.60～0.65) × N Value N/cm^2		

(4) Length of anchors

L = La + Lf

To this L: Length of anchors (m)

La: Embedment length = 37.0 (m)

Lf: Free length = 33.5 (m)

L = 37.0 + 33.5 = 70.5 (m)

(5) Examining adhesion between steel wires of the anchor and the bodies of anchors (cement base)

Pta = U・La・τ0

To this, Pta: Adhesion betwen steel wire and the body of anchors (N)

U : Perimeter of the steel wire = (6+π) × 2.18 = 19.93 (㎝)

La: Embedment length = 3700 (㎝)

τ0: Adhesive stress between steel wires and the bodies of the anchor = 100(N/㎠)

Pta = 19.93 × 3,700 × 100 = 7,374,100 (N) ＞ Pa = 2,080,000 (N) ――― OK

3. Structural Calculation Sheet for No. B Shaft

Contents

1. Design conditions

1-1. Structural type

Structural type : Structure of reinforced concrete
Foundation type : Open Caisson foundation

1-2. Load

1) Dead load

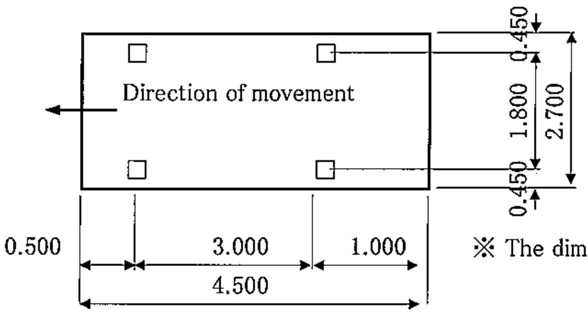
Material	Unit weight kN/m^3	Notes
Reinforced concrete	24.5	
Plain concrete	23.0	
Backfill soil (wet weight)	19.0	Internal frictional angle ϕ = 30.0 °
Backfill soil (submerged weight)	10.0	
Unit weight of water	10.0	

2) Vehicle load

If vehicle load is loaded, "load T-4" is considered.

The standard is shown in the following figure.

Gross weight W = 40.0 kN



※ The dimension of 6t vehicle is referred.

$$\text{Rear wheel:Pl1} = \frac{2\times\text{Load of rear wheel(kN)}}{\text{Occupied width of a set of T load(m)}}\times(1+\text{ impact factor})$$

$$= \frac{2 \times 15.0}{2.700}\times(1 + i) \quad \text{kN/m}$$

$$\text{Front Wheel:Pl2} = \frac{2\times\text{Load of front wheel(kN)}}{\text{Occupied width of a set of T load(m)}}\times(1+\text{ impact factor})$$

$$= \frac{2 \times 5.0}{2.700}\times(1 + i) \quad \text{kN/m}$$

To this

i : Coefficient of impact

Type of culvert	Earth covering(h)	Coefficient of impact
・Box culvert	h<4m	0.3
・Arch culvert		
・Portal culvert	4m≦h	0
・Corrugated metal culvert		
・Concrete pipe culvert	h<1.5m	0.5
・Ceramic pipe culvert	1.5m≦h<6.5m	0.65-0.1h
・Rigid polyvinyl chloride pipe culvert		
・Reinforced plastic composite pipe culvert	6.5m≦h	0

a) Vehicle load by live load which applies to top slab

i) Case of earth covering under 4m

$$\text{Rear wheel}: p_{l1} = \frac{P_{l1} \cdot \beta}{W_1} = \frac{P_{l1}}{2 \cdot h + 0.2} \text{ kN/m}^2$$

$$\text{Front wheel}: p_{l2} = \frac{P_{l2}}{W_1} = \frac{P_{l2}}{2 \cdot h + 0.2} \text{ kN/m}^2$$

To this

P_{l1} : Load of rear wheel per unit longitudianal length of calvert (kN/m)

P_{l2} : Load of front wheel per unit longitudianal length of calvert (kN/m)

W_1 : Distribution width of wheel load(m)

ii). Case of earth covering of 4 m and over

In case of earth covering of 4m and over, load, 10kN/m2 equally to top side of top slab as vertical live load is considered.

b). Horizontal load by live load which applies to the side of manhole

Load, 10 KN/m2 equally as live load of ground surface without considering impact is considered.

c). Sidewalk live load which applies to middle slab of manhole

Sidewalk live load, 5.0 kN/m2 as live load loading to middle slab is considered.

3) Earth pressure

a) At Ordinary condition

Horizontal earth pressure in an optinal depth is considered to be earth pressure at rest.

$$pa = k_0 \cdot \gamma \cdot h$$

To the above formula

pa : Earth pressure at rest(kN/m^2)

k_0 : Coefficient of earth pressure at rest (k_0= 0.5 is used)

γ : Unit volume weight of soil(kN/m^3)

h : Optional depth(m)

※When considering unit volume weight of soil in the calculation of earth pressure, earth pressure is generally seperated from water pressure.

b) At earthquale condition

Earth pressure in earthquake is considered to be affected by load from earth pressure at rest at ordinary cond including earth pressure calculated from response displacement method.

1-3. Soil condition

■ Location	Bor.No.B
■ Height of ground	E.L.+ 0.730 m
■ Groundwater level	E.L.+ -1.760 m
■ Basement level	E.L.+ -24.270 m

Elevation	Layer thickness	Sign	N value	γ	γ'	c	ϕ	E_0	α	$\alpha \cdot E_0$
m	m			kN/m^3	kN/m^3	kN/m^2	Degree	kN/m^2		kN/m^2
-3.270	4.000	Ac1	2.0	16.0	7.0	11.0	0.0	5,600	1	5,600
-12.270	9.000	Ac2	1.0	16.0	7.0	-	0.0	2,800	1	2,800
-14.270	2.000	Ac3	9.0	16.0	7.0	7.0	0.0	25,200	1	25,200
-24.270	10.000	Ac4	60.0	16.0	7.0	-	0.0	168,000	1	168,000
-26.270	2.000	Ac5	51.0	16.0	7.0	127.0	0.0	142,800	1	142,800
-49.270	23.000	Ac6	31.0	16.0	7.0	93.0	0.0	86,800	1	86,800

No	Modulus of deformation in each following testing methodology	α	
	$E_0(kN/m^2)$	Regular time	Earthquake
①	A half of modulus of deformation calculated from endurance curve of plate loading test by rigid disk of diameter with 0.3m.	1	1
②	Modulus of deformation measured inside borehole.	4	4
③	Modulus of deformation calculated from unconsolidated compression test and triaxial compression test of specimen.	4	4
④	Modulus of deformation estimated with E0=2800N by N value from standard penetration test.	1	1

1-4. Use material and allowable stress

1) Reinforced concrete

Unit: N/mm^2

Design strength		24.0
Compressive stress	Compressive stress due to bending	8.0
	Axial compressive stress	6.5
Shearing stress	In case of shearing stress burdened by only concrete (τ_{a1})	0.23
	In case of being burdened cooperated with diagonal tension bar (τ_{a2})	1.7
	Punching shear unit stress (τ_{a3})	0.90
Bonding sress	To deformed reinforce bars	1.6
Bearing stress		7.2

Note1. Punching shear unit stress does not consider extra according to combination of load.

Note2. If there is no haunch, allowable compressive stress due to bending of conrner is decreased to "3/4".

Elastic modulus	E =	2.5 × 10^7	kN/m^2
Linear expansion coefficient	T =	1.0 × 10^{-5}	$℃^{-1}$

If shear force is caused only by concerete, allowable shearing stress intensity τ a1 is corrected considering following influence.

① Influence of effective depth, d of member section

Correction coefficient, Ce related to effective depth, d of member section.

Effective depth, d	300 and fewer	1,000	3,000	5,000	10,000and over
C_e	1.4	1.0	0.7	0.6	0.5

② Influence of ration of axial stretched reinforcing bar, p_t

Correction coefficient, Cpt related to ration of axial stretched reinforcing bar, pt

Ration of axial stretched reinforcing bar, p_t (%)	0.1	0.2	0.3	0.5	1.0 and over
C_{pt}	0.7	0.9	1.0	1.2	1.5

③ If axial compressive force of member is large, correction coefficient, CN by axial compressive force in comp calculated from the following formula is multiplied by $\overline{\tau}$ a1.

$$C_N = 1 + \frac{M_0}{M}$$

To this

C_N : Correction coefficient by axial compressive force

M_0 : Bending moment N•mm with stress intensity of concrete with zero in the edge of tension member due to axial compressive force

$$= \frac{N}{Ac} \cdot \frac{Ic}{y}$$

M	:	Bending moment applying member section	N•mm
N	:	Axial stress in compression applying member section	N
Ic	:	Moment of inertia related to centroid axis of member section	mm^4
Ac	:	Sectional area of member	mm^2
y	:	Distance to the edge of tension member from centroid of sectional area of member	mm

2) Plain concrete

Unit:N/mm^2

Type of stress intensity	Allowable stress intensity	Design strength
		24.0
Compressive stress intensity	$\frac{\sigma ck}{4} \leqq 5.5$	5.5
Tensile stress intensity due to bending	$\frac{\sigma ck}{7} \leqq 0.3$	0.3
Bearing stress intensity	0.3 σ ck ≦ 6.0	6.0
Shearing unit stress	$\frac{\sigma ck}{100} + 0.15$ Note1)	0.39

Note1. Extra increase is not added according to combination of load

3) Reinforcing bar

Unit:N/mm^2

Variety of stress intensity and member / Variety of reinforcing bar			SD 345
Tensile stress intensity	1) In case that main load without live load and impact is applied(like beam member)		100
	Basic value in case that influence of collision load and earthquake is not included in the combination of load	2) General member	180
		3) Member installed in water level or under groundwater level	160
	Basic value in case that influence of collision load and earthquake is included in the combination of load	4) Axial reinforcing bar	200
		5) Other than that above	200
	6) Basic value in case of calculating the length of lap joint of reinforcing bar or fixing length		200
7) Compressive stress intensity			200

3) As for extra increase for allowable stress intensity

Extra increase of allowable tensile stress intensity is the following according to the combination of load.

Combinations of loads	Overdesign factor	Notes
Regular time	1.0	
Construction time	1.5	
Earthquake time(L1)	1.5	

1-5. Application specification and references

*1	Earthworks of road-guideline of culvert work	Corporate juridical person Japan Road Association
*2	Specification of highway bridge and the manual, I common version	Corporate juridical person Japan Road Association
*3	Specification of highway bridge and the manual, Ⅲconcrete bridge version	Corporate juridical person Japan Road Association
*4	Specification of highway bridge and the manual, ⅣSubstructure version	Corporate juridical person Japan Road Association
*5	Design manual of civil engineering(draft)-Civil engineering structure・Bridge version-	Ministry of Land, Transport and Touris
*6	Guideline and the manual for earthquake countermeasure of sewage facility	Corporate juridical person Japan Sewage Works Association
*7	Calculation examples for earthquake resistance of sewage facility	Corporate juridical person Japan Sewage Works Association
*8	Standard specification for tunnel [Open cut method version]・the manual	Japan Society of Civil Engineering
*9	Structural calculation criterion of reinforced concrete・the manual	Architectural Institute of Japan

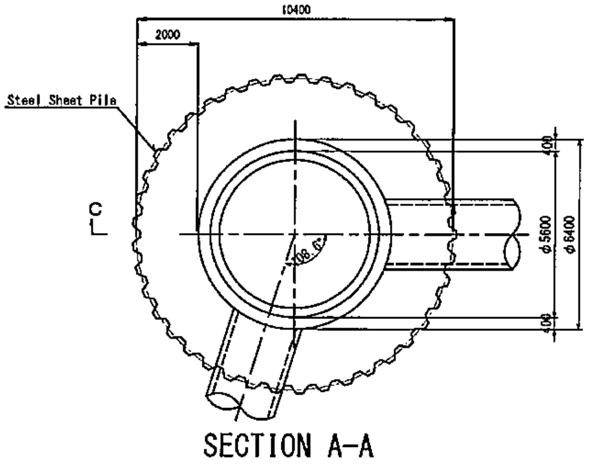
1-6. The others

- As for minimum reinforcement content

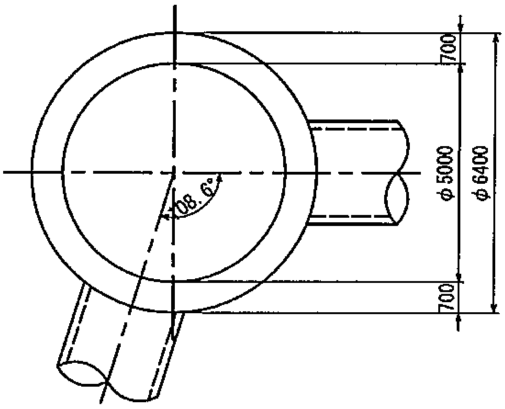
  Minimum reinforcement content is 0.2 and over of effective sectional area of member.

2. Structural drawing

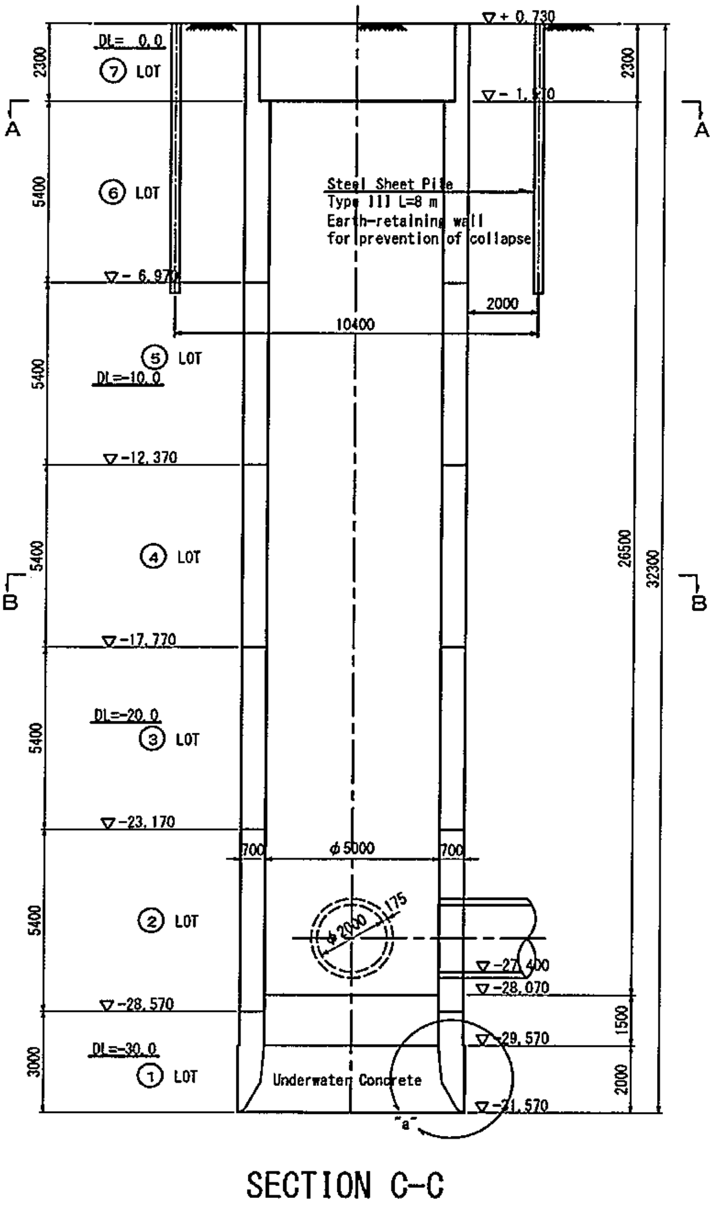
Section A-A



Section B-B



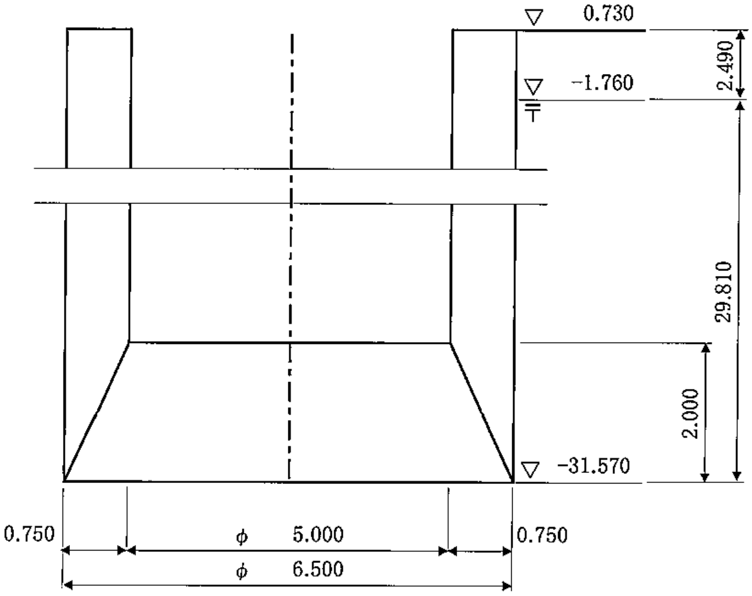
Section C-C



3. Stability computation

3-1. Design of bottom slab (underwater concrete)

Bottom slab (underwater concerete) is treated as plain concrete constructed in water.



1) Load calculation

As for design load, uplift pressure and self weight of bottom slab are considered.

Uplift pressure

w_u = 10.0 × 29.810 = 298.10 kN/m^2

Self weight of bottom slab of concrete

w_c = 23.0 × 2.000 = 46.00 kN/m^2

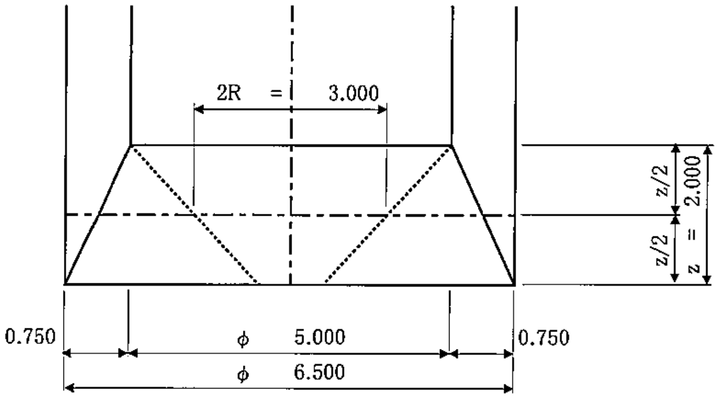
Design load

w = w_u - w_c

= 298.10 - 46.00 = 252.10 kN/m^2

2) Calculation of section force

As for cross sectional area, bottom slab is considered to be slab that the surrounding is simply supported.



Bending moment

$$\mathrm{Mmax} = (3 + \nu) \cdot \frac{w \cdot R^2}{16}$$

$$= (3 + \frac{1}{6}) \times \frac{252.10 \times 1.500^2}{16} = 112.26 \ \mathrm{kN \cdot m/m}$$

Shear force

$$S = \frac{w \cdot R}{2}$$

$$= \frac{252.10 \times 1.500}{2} = 189.08 \ \mathrm{kN/m}$$

3) Checking sectional area

	Bottom slab			
M kN·m	112.26			
N kN	0.00			
S kN	189.08			
b mm	1000			
h mm	2000			
Z mm^3	666,666,667			
A mm^2	2,000,000			
σc N/mm^2	0.2 < 8.25			
σt N/mm^2	0.2 < 0.45			
τ N/mm^2	0.09 < 0.39			
Judgement	OK			

Section modulus $Z = \frac{1}{6} b \cdot h^2$

Sectional area $A = b \cdot h$

3-2. Consideration for lift

1) Construction time

a) Load calculation

- Body part

Elevation	Height	External diameter	Internal diameter	Cross sectional area	Average cross section area	Volume
m	m	m	m	m^2	m^2	m^3
0.730	2.300	6.400	5.600	7.540	7.540	17.342
-1.570		6.400	5.600	7.540		
	28.000	6.400	5.000	12.535	12.535	350.979
-29.570		6.400	5.000	12.535		
	1.100	6.500	5.000	13.548	12.747	14.022
-30.670		6.500	5.200	11.946		
	0.800	6.500	5.200	11.946	7.952	6.362
-31.470		6.500	6.100	3.958		
	0.100	6.500	6.100	3.958	1.979	0.198
-31.570		6.500	6.500	0.000		
Total	32.300	-	-	-	-	388.902

- Bottom slab part (underwater concrete)

Elevation	Height	External diameter	Internal diameter	Cross sectional area	Average cross section area	Volume
m	m	m	m	m^2	m^2	m^3
-29.570	1.100	5.000	0.000	19.635	20.436	22.480
-30.670		5.200	0.000	21.237		
	0.800	5.200	0.000	21.237	25.231	20.185
-31.470		6.100	0.000	29.225		
	0.100	6.100	0.000	29.225	31.204	3.120
-31.570		6.500	0.000	33.183		
合計	2.000	-	-	-	-	45.785

- Total weight

$$\Sigma W = 24.5 \times 388.902 + 23.0 \times 45.785 = 10{,}581.14 \text{ kN}$$

- Buoyancy

$$W_u = 10.0 \times 29.810 \times \frac{\pi}{4} \times 6.500^2 = 9{,}891.87 \text{ kN}$$

b) Chekcking buoyancy

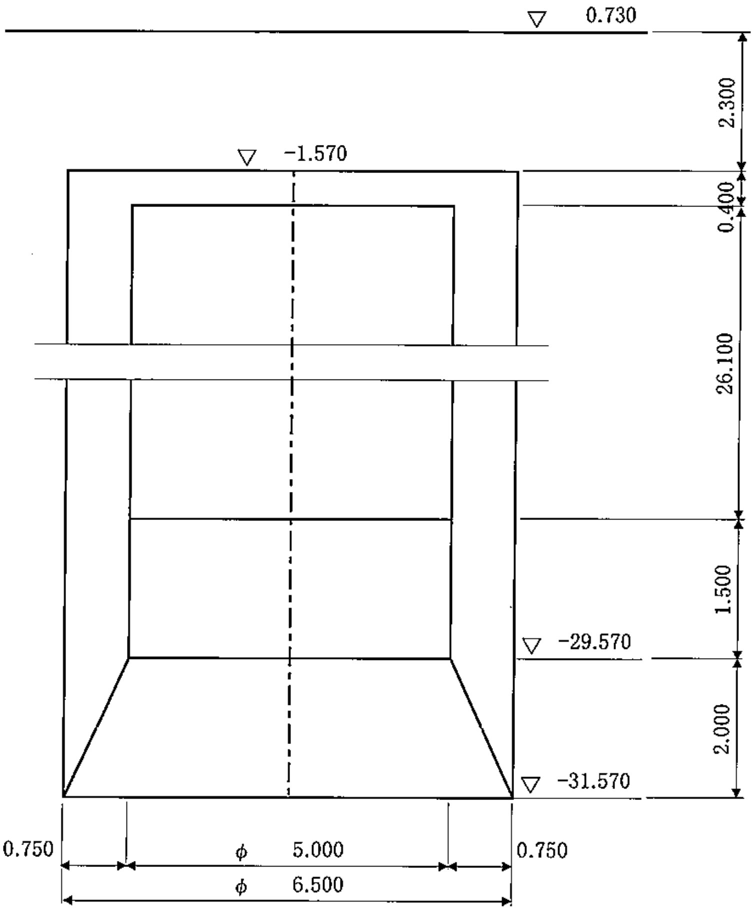
$$F = \frac{\Sigma W}{W_u}$$

$$= \frac{10{,}581.14}{9{,}891.87}$$

$$= 1.07 > Fs = 1.0 \text{ ------------------------------------ OK}$$

2) Completion time

a) Load calculation



Overburden load

$W_s = 19.0 \times 2.300 \times \frac{\pi}{4} \times 6.400^2 = 1{,}405.83$ kN

Self weight of top slab

$W_t = 24.5 \times 0.400 \times \frac{\pi}{4} \times 6.400^2 = 315.27$ kN

Self weight of lateral wall

$W_w = 24.5 \times \frac{\pi}{4} \times (6.400^2 - 5.000^2) \times 26.100 = 8{,}015.48$ kN

Self weight of bottom slab

$W_f = 24.5 \times 1.500 \times \frac{\pi}{4} \times 6.400^2 = 1{,}182.24$ kN

Cutting edge part (part of under undersurface of bottom slab)

$W_n = 24.5 \times (14.022 + 6.362 + 0.198) = 504.24$ kN

Self weight of medium slab

$W_m = 6 \times 24.5 \times 0.300 \times \frac{\pi}{4} \times 5.000^2 = 865.90$ kN

Bottom slab (underwater concrete)

$W = 23.0 \times 45.785 = 1{,}053.05$ kN

$\Sigma W = 13{,}342.01$ kN

- Buoyancy

$W_u = 9{,}891.87$ kN

b) Checking to buoyancy

$$F = \frac{\Sigma W}{W_u}$$

$$= \frac{13{,}342.01}{9{,}891.87}$$

$$= 1.35 > Fs = 1.2$$ -------------------------------------- OK

3-3. consideration for bearing capacity

1) Calculation for ultimate bearing capacity

$$q_d = \alpha \cdot c \cdot Nc + 1/2 \cdot \beta \cdot \gamma_1 \cdot B \cdot N_\gamma + \gamma_2 \cdot Df \cdot Nq$$

To this

q_d : Ultimate bearing capacity(kN/m^2)
c : Adhesive force intensity of ground under faoudation base(kN/m^2)
γ_1 : Unit volume weight of ground under foundation base(kN/m3)
γ_2 : Weight per unit volume of ground over foundation base(kN/m3)
α、β : Form coefficient indicated in a table

Form coefficient

Shape for load side of base	Shape like belt	Squuare, circle	Rectangle, oval
α	1.0	1.3	1 + 0.3 ·B/D
β	1.0	0.6	1 − 0.4 ·B/D

B : Base width(m)
Df : Effective depth of foundation(m)
Nc,Nr,Nq : Coefficient of bearing capacity shown in graph

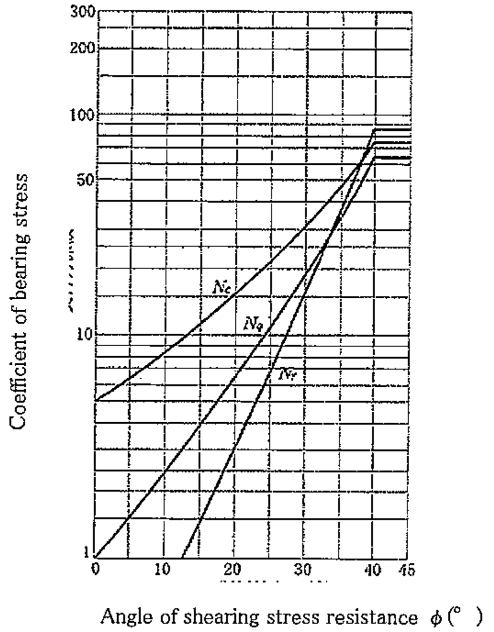


Figure 11.4.2 Figure for coefficient of bearing stress

$$q_d = 1.30 \times 127.0 \times 7.0 + \frac{1}{2} \times 0.60 \times 7.0 \times 6.500 \times 0.0 + 248.51 \times 1.0 = 1{,}404.21 \text{ kN/m}^2$$

c = 127.0 kN/m^2
γ_1 = 7.0 kN/m^3
α = 1.30
β = 0.60
B = 6.500 m
$\gamma_2 \cdot Df$ = 248.51 kN/m^2

Calculation for $\gamma_2 \cdot Df$

Soil	Elevation	Depth	Thickness of layer	γ	γ'	Vertical load	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^2	
Ac1	0.730	0.000	0.000	16.0	7.0	0.00	Ground level
	-1.760	2.490	2.490	16.0	7.0	39.84	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	50.41	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	50.41	-
	-12.270	13.000	9.000	16.0	7.0	113.41	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	113.41	-
	-14.270	15.000	2.000	16.0	7.0	127.41	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	127.41	-
	-24.270	25.000	10.000	16.0	7.0	197.41	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	197.41	-
	-26.270	27.000	2.000	16.0	7.0	211.41	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	211.41	-
	-31.570	32.300	5.300	16.0	7.0	248.51	Cutting edge

$N_c = 7.0 \quad N_q = 1.0 \quad N_r = 0.0$

2) Checking bearing strength

By consideration for uplift in completion time

ΣW = 13,342.01 kN

$$q = \frac{13{,}342.01}{\pi / 4 \times 6.500^2} + \overset{\text{Live load}}{10.00}$$

$$= 412.1 \text{ kN/m}^2 < q_a = \frac{1}{3} \times 1{,}404.21 = 468.1 \text{ kN/m}^2 \text{ ---------- OK}$$

4. Checking member in construction

4-1. Calculation of sidawall

As for checking lateral wall in construction, consideration for the case of occurrence of difference of head of water in working state and after work of sinking

- As for working state of sinking

  ① Active earth pressure adding hydrostatic pressure is acted into 4 directions. The acting directions are orthogonal direction towards lateral wall.

  ② A half of active earth pressure is acted into one direction as unbalanced load at the same time with ①. The acting direction is the direction with its decenterizing.

  Active earth pressure is evaluated by formula of Coulomb's earth pressure. However, if coefficient of active earth pressure is under 0.5, the coefficient is set with 0.5.

  Moreover, decrease of earth pressure by adhesion is not considered.

  ③ In case of open caisson, external pressure is not different as the case of pneumatic caisson. However, internal pressure considers hydrastatic stress having the difference between external hydrastatic stress and internal pressure with 3.0 m.

- In case of occurrence of difference of head of water after sinking work

  Stratified pressure including hydraostatic stress are acted into 4 directions in the situation of occurrence of difference of head of water between internal and external Caisson due to pump up after sinking. The acting directions are orthogonal direction towards lateral walls.

1) consideration in sinking working state

a) Load calculation

Calculation for coeffieicnet of active earth pressure

Coefficient of active earth pressure is calculated by the following formula. If the coefficient is under 0.5, the value is set with 0.5.

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

To the above formula

K_A : Coefficient of active earth pressure by Coulomb's earth pressure

ϕ : Angle of internal friction of soil (°)

α : Angle between ground surface and horizontal surface (°)

θ : Angle between rear side of wall and vertical plane (°)

δ : Wall friction angle between rear side of wall and ground (°) = 1/3 ϕ

Soil	ϕ	α	θ	δ	K_A		
	(°)	(°)	(°)	(°)	Calculated value	Minimum value	Adopted value
Ac1	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac2	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac3	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac4	0.0	0.0	0.0	0.0	1.000	0.5	1.000
Ac5	0.0	0.0	0.0	0.0	1.000	0.5	1.000

Calculation for earth pressure intensity

Active earth pressure

$p_a \ = K_A \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \ = \gamma_w \cdot \Sigma h_n$

To this

p_a : Active earth pressure

p_w : Hydrostatic stress (kN/m^2)

K_A : Coefficient of active earth pressure by Coulomb's earth pressure

q_0 : Vertical load (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stra (kN/m^3)

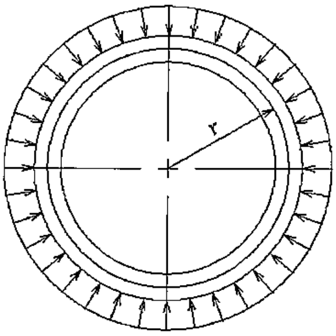
(in case of under groundwater level, submerged weight)

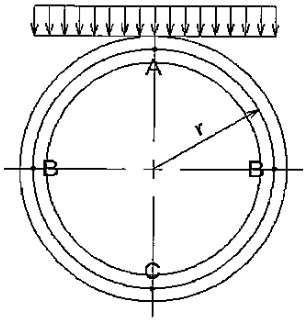
γ_w : Weight per unit volume of groundwate (kN/m^3)

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	1.000	10.00	0.00	Ground level
	-1.570	2.300	2.300	16.0	7.0	10.0	46.80	1.000	46.80	0.00	7Rsoffit
	-1.760	2.490	0.190	16.0	7.0	10.0	49.84	1.000	49.84	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	1.000	60.41	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	1.000	60.41	15.10	-
	-6.970	7.700	3.700	16.0	7.0	10.0	86.31	1.000	86.31	52.10	6Rsoffit
	-12.270	13.000	5.300	16.0	7.0	10.0	123.41	1.000	123.41	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	1.000	123.41	105.10	-
	-12.370	13.100	0.100	16.0	7.0	10.0	124.11	1.000	124.11	106.10	5Rsoffit
	-14.270	15.000	1.900	16.0	7.0	10.0	137.41	1.000	137.41	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	1.000	137.41	125.10	-
	-17.770	18.500	3.500	16.0	7.0	10.0	161.91	1.000	161.91	160.10	4Rsoffit
	-23.170	23.900	5.400	16.0	7.0	10.0	199.71	1.000	199.71	214.10	3Rsoffit
	-24.270	25.000	1.100	16.0	7.0	10.0	207.41	1.000	207.41	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	1.000	207.41	225.10	-
	-26.270	27.000	2.000	16.0	7.0	10.0	221.41	1.000	221.41	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	1.000	221.41	245.10	-
	-28.570	29.300	2.300	16.0	7.0	10.0	237.51	1.000	237.51	268.10	2Rsoffit
	-29.570	30.300	1.000	16.0	7.0	10.0	244.51	1.000	244.51	278.10	Undersurface of bottom slab

b) Calculation of sectional force





- In case of bearing even load from 4 directions
  (in case this, there is no bending moment)

Axial force

$N = 1.000 \cdot p \cdot r$

- In case of bearing unbalanced load from 1 direction

Bending moment

$M_A = 0.163 \cdot p' \cdot r^2$

$M_B = -0.125 \cdot p' \cdot r^2$

$M_C = 0.087 \cdot p' \cdot r^2$

Axial force

$N_A = 0.212 \cdot p' \cdot r$

$N_B = 1.000 \cdot p' \cdot r$

$N_C = -0.212 \cdot p' \cdot r$

Form and working load

Checking location		Internal diameter	Thickness of member	Shaft diameter of member	Radius of axis of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m	m	kN/m^2	kN/m^2	kN/m^2
1	6Rsoffit	5.000	0.700	5.700	2.850	86.31	30.00	43.16
2	5Rsoffit	5.000	0.700	5.700	2.850	124.11	30.00	62.06
3	4Rsoffit	5.000	0.700	5.700	2.850	161.91	30.00	80.96
4	3Rsoffit	5.000	0.700	5.700	2.850	199.71	30.00	99.86
5	2Rsoffit	5.000	0.700	5.700	2.850	237.51	30.00	118.76
6	Undersurface of bottom slab	5.000	0.700	5.700	2.850	244.51	30.00	122.26

※ Hydrostatic pressure in constructure

The difference of water level to inside caison is set with 3.0 m.

$p_w = 10.0 \times 3.000 = 30.00\ kN/m^2$

Calculation for sectional force

6Rsoffit		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN·m/m	kN/m
Bending moment	M_A	0.163	-	43.16	2.850	57.14	-
	M_B	-0.125	-	43.16	2.850	-43.82	-
	M_C	0.087	-	43.16	2.850	30.50	-
Axial force	N_A	0.212	116.31	43.16	2.850	-	357.56
	N_B	1.000	116.31	43.16	2.850	-	454.48
	N_C	-0.212	116.31	43.16	2.850	-	305.41

5Rsoffit		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN·m/m	kN/m
Bending moment	M_A	0.163	-	62.06	2.850	82.16	-
	M_B	-0.125	-	62.06	2.850	-63.01	-
	M_C	0.087	-	62.06	2.850	43.85	-
Axial force	N_A	0.212	154.11	62.06	2.850	-	476.71
	N_B	1.000	154.11	62.06	2.850	-	616.07
	N_C	-0.212	154.11	62.06	2.850	-	401.72

4Rsoffit		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN·m/m	kN/m
Bending moment	M_A	0.163	-	80.96	2.850	107.18	-
	M_B	-0.125	-	80.96	2.850	-82.19	-
	M_C	0.087	-	80.96	2.850	57.21	-
Axial force	N_A	0.212	191.91	80.96	2.850	-	595.86
	N_B	1.000	191.91	80.96	2.850	-	777.67
	N_C	-0.212	191.91	80.96	2.850	-	498.03

3Rsoffit		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN·m/m	kN/m
Bending moment	M_A	0.163	-	99.86	2.850	132.20	-
	M_B	-0.125	-	99.86	2.850	-101.38	-
	M_C	0.087	-	99.86	2.850	70.56	-
Axial force	N_A	0.212	229.71	99.86	2.850	-	715.01
	N_B	1.000	229.71	99.86	2.850	-	939.26
	N_C	-0.212	229.71	99.86	2.850	-	594.34

2Rsoffit		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN•m/m	kN/m
Bending moment	M_A	0.163	-	118.76	2.850	157.23	-
	M_B	-0.125	-	118.76	2.850	-120.57	-
	M_C	0.087	-	118.76	2.850	83.92	-
Axial force	N_A	0.212	267.51	118.76	2.850	-	834.16
	N_B	1.000	267.51	118.76	2.850	-	1,100.86
	N_C	-0.212	267.51	118.76	2.850	-	690.65

Undersurface of bottom slab		Coefficient	uniform load	Unbalanced load	Radius	M	N
			kN/m^2	kN/m^2	m	kN•m/m	kN/m
Bending moment	M_A	0.163	-	122.26	2.850	161.86	-
	M_B	-0.125	-	122.26	2.850	-124.13	-
	M_C	0.087	-	122.26	2.850	86.39	-
Axial force	N_A	0.212	274.51	122.26	2.850	-	856.22
	N_B	1.000	274.51	122.26	2.850	-	1,130.78
	N_C	-0.212	274.51	122.26	2.850	-	708.49

c) Checking section

- As for minimum amount of reinforcing bar

Minimum amount of reinforcing bar is 0.2 % and over of effective sectional area of member.

Member	b	h	d'	Formula							Arrangement of minimum reinforcing bar				
	mm	mm	mm	mm^2							mm^2				
Lateral wall	1000.0	700.0	100.0	1000.0	×	600.0	×	0.002	=	1,200.0	D	22	@	250	1,548.4

6Rsoffit		A point	B point	C point
		Inner surface	Exterior surface	Inner surface
M	kN･m	57.14	43.82	30.50
N	kN	357.56	454.48	305.41
b	mm	1000	1000	1000
h	mm	700	700	700
d	mm	600	600	600
d'	mm	100	100	100
As	cm^2	D 22 @ 250	D 22 @ 250	D 22 @ 250
		15.484	15.484	15.484
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00258	0.00258	0.00258
k		0.242	0.242	0.242
j		0.919	0.919	0.919
σ c	N/mm^2	1.2 < 12.0	1.1 < 12.0	0.8 < 12.0
σ s	N/mm^2	0.8 < 240	-14.9 < 240	-10.1 < 240
n		15	15	15

5Rsoffit		A point	B point	C point
		Inner surface	Exterior surface	Inner surface
M	kN･m	82.16	63.01	43.85
N	kN	476.71	616.07	401.72
b	mm	1000	1000	1000
h	mm	700	700	700
d	mm	600	600	600
d'	mm	100	100	100
As	cm^2	D 22 @ 250	D 22 @ 250	D 22 @ 250
		15.484	15.484	15.484
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00258	0.00258	0.00258
k		0.242	0.242	0.242
j		0.919	0.919	0.919
σ c	N/mm^2	1.8 < 12.0	1.6 < 12.0	1.1 < 12.0
σ s	N/mm^2	2.7 < 240	-20.6 < 240	-13.8 < 240
n		15	15	15

4Rsoffit		A point				B point				C point			
		Inner surface				Exterior surface				Inner surface			
M	kN•m	107.18				82.19				57.21			
N	kN	595.86				777.67				498.03			
b	mm	1000				1000				1000			
h	mm	700				700				700			
d	mm	600				600				600			
d'	mm	100				100				100			
As	cm^2	D	22	@	250	D	22	@	250	D	22	@	250
		15.484				15.484				15.484			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00258				0.00258				0.00258			
k		0.242				0.242				0.242			
j		0.919				0.919				0.919			
σ c	N/mm^2	2.3		<	12.0	2.0		<	12.0	1.4		<	12.0
σ s	N/mm^2	4.8		<	240	-26.4		<	240	-17.5		<	240
n		15				15				15			

3Rsoffit		A point				B point				C point			
		Inner surface				Exterior surface				Inner surface			
M	kN•m	132.20				101.38				70.56			
N	kN	715.01				939.26				594.34			
b	mm	1000				1000				1000			
h	mm	700				700				700			
d	mm	600				600				600			
d'	mm	100				100				100			
As	cm^2	D	22	@	250	D	22	@	250	D	22	@	250
		15.484				15.484				15.484			
As'	cm^2	D		@		D		@		D		@	
		0.000				0.000				0.000			
p		0.00258				0.00258				0.00258			
k		0.242				0.242				0.242			
j		0.919				0.919				0.919			
σ c	N/mm^2	2.8		<	12.0	2.5		<	12.0	1.7		<	12.0
σ s	N/mm^2	7.0		<	240	-32.1		<	240	-3.2		<	240
n		15				15				15			

2Rsoffit		A point	B point	C point
		Inner surface	Exterior surface	Inner surface
M	kN・m	157.23	120.57	83.92
N	kN	834.16	1,100.86	690.65
b	mm	1000	1000	1000
h	mm	700	700	700
d	mm	600	600	600
d'	mm	100	100	100
As	cm^2	D 22 @ 250	D 22 @ 250	D 22 @ 250
		15.484	15.484	15.484
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00258	0.00258	0.00258
k		0.242	0.242	0.242
j		0.919	0.919	0.919
σ c	N/mm^2	3.3 < 12.0	2.9 < 12.0	2.0 < 12.0
σ s	N/mm^2	9.2 < 240	-37.9 < 240	-3.4 < 240
n		15	15	15

Undersurface of bottom slab		A point	B point	C point
		Inner surface	Exterior surface	Inner surface
M	kN・m	161.86	124.13	86.39
N	kN	856.22	1,130.78	708.49
b	mm	1000	1000	1000
h	mm	700	700	700
d	mm	600	600	600
d'	mm	100	100	100
As	cm^2	D 22 @ 250	D 22 @ 250	D 22 @ 250
		15.484	15.484	15.484
As'	cm^2	D @	D @	D @
		0.000	0.000	0.000
p		0.00258	0.00258	0.00258
k		0.242	0.242	0.242
j		0.919	0.919	0.919
σ c	N/mm^2	3.4 < 12.0	3.0 < 12.0	2.1 < 12.0
σ s	N/mm^2	9.7 < 240	-39.0 < 240	-3.5 < 240
n		15	15	15

1) Consideration in case of occurence of difference of difference of head of water after sinking

a) Load calculation

Calculation of earth pressure intensity

Earth pressure at rest

$p_a \;\; = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \;\; = \gamma_w \cdot \Sigma h_n$

To the above formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Vertical load (kN/m^2)

= 10.0 kN/m^2

γ_n : Weight per unit volume of soil of each stratum (kN/m$^{3)}$

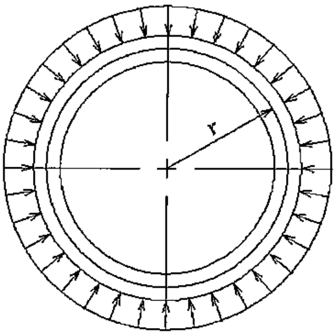
(in case under groundwater level, submerged weight)

γ_w : Weight per unit volume of groundwater (kN/m$^{3)}$

h_n : Thickness of each stratum (m)

Soil	Elevation	Depth	Thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.570	2.300	2.300	16.0	7.0	10.0	46.80	0.500	23.40	0.00	7Rsoffit
	-1.760	2.490	0.190	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	-
	-6.970	7.700	3.700	16.0	7.0	10.0	86.31	0.500	43.16	52.10	6Rsoffit
	-12.270	13.000	5.300	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-12.370	13.100	0.100	16.0	7.0	10.0	124.11	0.500	62.06	106.10	5Rsoffit
	-14.270	15.000	1.900	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-17.770	18.500	3.500	16.0	7.0	10.0	161.91	0.500	80.96	160.10	4Rsoffit
	-23.170	23.900	5.400	16.0	7.0	10.0	199.71	0.500	99.86	214.10	3Rsoffit
	-24.270	25.000	1.100	16.0	7.0	10.0	207.41	0.500	103.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	103.71	225.10	-
	-26.270	27.000	2.000	16.0	7.0	10.0	221.41	0.500	110.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	110.71	245.10	-
	-28.570	29.300	2.300	16.0	7.0	10.0	237.51	0.500	118.76	268.10	2Rsoffit
	-29.570	30.300	1.000	16.0	7.0	10.0	244.51	0.500	122.26	278.10	Undersurface of bottom slab

b) Calculation for sectional force



- In case of bearing equal load from 4 directions

  (In this case, bending moment does not occure)

  Axial force

  $N = 1.000 \cdot p \cdot r$

Form and working load

Checking locatior		Internal diameter	Member thickness	Diamter of center line of member	Radius of center line of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m	m	kN/m^2	kN/m^2	kN/m^2
1	6Rsoffit	5.000	0.700	5.700	2.850	43.16	52.10	0.00
2	5Rsoffit	5.000	0.700	5.700	2.850	62.06	106.10	0.00
3	4Rsoffit	5.000	0.700	5.700	2.850	80.96	160.10	0.00
4	3Rsoffit	5.000	0.700	5.700	2.850	99.86	214.10	0.00
5	2Rsoffit	5.000	0.700	5.700	2.850	118.76	268.10	0.00
6	Undersurface of bottom slab	5.000	0.700	5.700	2.850	122.26	278.10	0.00

Calculation for sectional force

Axial force	Uniform load	Radius	N
	kN/m^2	m	kN/m
6Rsoffit	95.26	2.850	271.48
5Rsoffit	168.16	2.850	479.24
4Rsoffit	241.06	2.850	687.01
3Rsoffit	313.96	2.850	894.77
2Rsoffit	386.86	2.850	1,102.54
Undersurface of bottom slab	400.36	2.850	1,141.01

c) Checking section

$$\sigma_c = \frac{N}{A}$$

To the above formula

σ_c : Compressive stress (N/mm^2)

A : Sectional area of member mm^2

Checking location		N	b	h	A	σ_c	σ_{ca}	Judgement
		kN/m	mm	mm	mm^2	N/mm^2	N/mm^2	
1	6Rsoffit	271.48	1,000	700	700,000	0.39	12.00	○
2	5Rsoffit	479.24	1,000	700	700,000	0.68	12.00	○
3	4Rsoffit	687.01	1,000	700	700,000	0.98	12.00	○
4	3Rsoffit	894.77	1,000	700	700,000	1.28	12.00	○
5	2Rsoffit	1,102.54	1,000	700	700,000	1.58	12.00	○
6	Undersurface of bottom slab	1,141.01	1,000	700	700,000	1.63	12.00	○

4-2. Calculation of cutting edge

i) Consideration of vertical direction

Design for cutting edge is for just before final settlement of Caisson. In the design, design load from outside considers earth pressure at rest plus hydrostatic pressure, while design load from inside considers hydrostatic pressure having the difference of head of water with 3.0 m to outside hydrostatic pressure.
In analytical model, span from cutting edge to bottom slab is regarded as cantilever. However, if there is no bottom slab, the span is set with 1.5 m.

a) Load calculation

Calculation of earth pressure intensity

Earth pressure at rest

$p_e = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

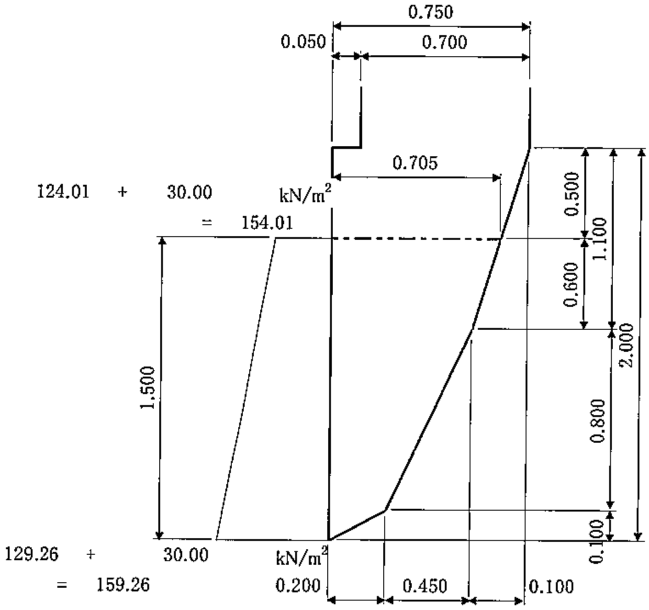
$p_w = \gamma_w \cdot \Sigma h_n$

To the above formula

p_0 : Earth pressure at rest
p_w : Hydrostatic pressure (kN/m^2)
K_0 : Coefficient of earth pressure at rest
q_0 : Load placed on the top (kN/m^2)
= 10.0 kN/m^2
γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$
(submerged weight in case under groundwater level)
γ_w : Unit volume weight of groundwater (kN/m$^{3)}$
h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.760	2.490	2.490	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	-
	-12.270	13.000	9.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-14.270	15.000	2.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-24.270	25.000	10.000	16.0	7.0	10.0	207.41	0.500	103.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	103.71	225.10	-
	-26.270	27.000	2.000	16.0	7.0	10.0	221.41	0.500	110.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	110.71	245.10	-
	-29.570	30.300	3.300	16.0	7.0	10.0	244.51	0.500	122.26	278.10	Undersurface of bottom slab
	-30.070	30.800	0.500	16.0	7.0	10.0	248.01	0.500	124.01	283.10	Supporting point of cutting edge
	-31.570	32.300	1.500	16.0	7.0	10.0	258.51	0.500	129.26	298.10	Cutting edge

b) Calculation for sectional force



Bending moment

$$M = (\frac{1}{6} \times 154.01 + \frac{1}{3} \times 159.26) \times 1.500^{2} = 177.19 \ \text{kN·m/m}$$

Shear force

$$S = \frac{1}{2} \times (154.01 + 159.26) \times 1.500 = 234.95 \ \text{kN/m}$$

c) Reviewing section

- As for minimum volume of reinforcing bar

Minimum volume of reinforcing bar is 0.2% and over of effective sectional area of member.

Membe	b	h	d'	Formula							Arrangement of minimum reinforcing bar				
	mm	mm	mm	mm^2							mm^2				
Lateral wall	1000.0	704.5	100.0	1000.0	×	604.5	×	0.002	=	1,209.1	D	19	@	200	1,432.5

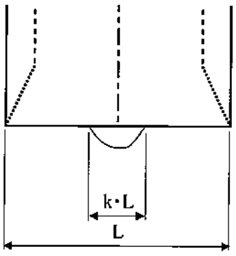
		Cutting edge			
		Exterior surface			
M	kN•m	177.19			
N	kN	0.00			
S	kN	234.95			
b	mm	1000			
h	mm	705			
d'	mm	100			
d	mm	605			
As	cm^2	D 22 @ 250			
		15.484			
p		0.00256			
k		0.241			
j		0.920			
σc	N/mm^2	4.4 < 12.0			
σs	N/mm^2	205.9 < 240			
τ	N/mm^2	0.39 < 0.40			
τ_{al}	N/mm^2	0.35			
C_c		1.226			
C_{pt}		0.956			
C_N		1.000			
n		15			

2) Consideration just after immersion of first lot

After assumption of condition of simple support partially without ground reaction just after sinking work of C and condition of supporting by cantilever of partial bottom slab, considereation is carried out.

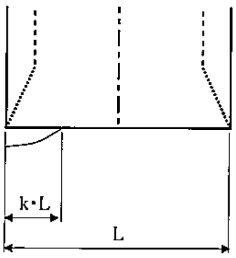
Case-1

Simple support



Case-2

Cantilever



L = 6.500 m

k = 1 / 4

k•L = 1.625 m

L = 6.500 m

k = 1 / 5

k•L = 1.300 m

a) Load calculation

Simple weight of first lot

Elevation	Height	External diameter	Internal diameter	Sectional area	Average sectional area	Volume
m	m	m	m	m^2	m^2	m^3
-28.570	1.000	6.400	5.000	12.535	12.535	12.535
-29.570		6.400	5.000	12.535		
	1.100	6.500	5.000	13.548	12.747	14.022
-30.670		6.500	5.200	11.946		
	0.800	6.500	5.200	11.946	7.952	6.362
-31.470		6.500	6.100	3.958		
	0.100	6.500	6.100	3.958	1.979	0.198
-31.570		6.500	6.500	0.000		
Total	3.000	-	-	-	-	33.116

W = 24.5 × 33.116 = 811.35 kN

Perimeter of first lot

U = π × 6.500 = 20.420 m

Design load

$$q = \frac{W}{U}$$

$$= \frac{811.35}{20.420} = 39.73 \text{ kN/m}$$

b) Calculation of section force

- Case-1 : Condition of simple supproting

Bending moment (tension of underside)

$$M = \frac{1}{8} \times 39.73 \times 1.625^{2} = 13.11 \text{ kN·m}$$

Shear force

$$S = \frac{1}{2} \times 39.73 \times 1.625 = 32.28 \text{ kN/m}$$

- Case-2 : Condition of canitilever supporting

Bending moment (Upper side of tension)

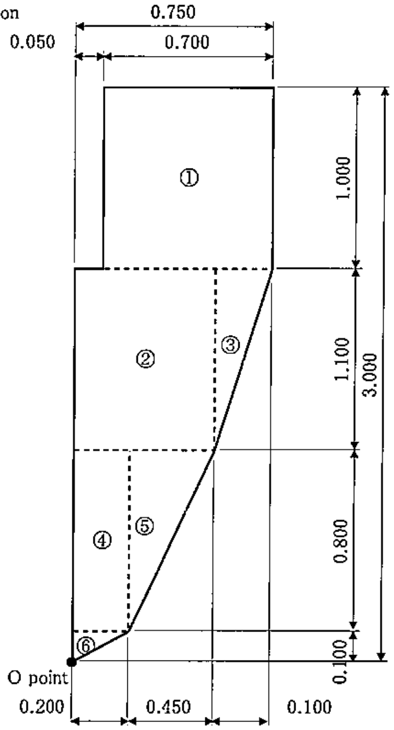
$$M = \frac{1}{2} \times 39.73 \times 1.300^{2} = 33.57 \text{ kN·m}$$

Shear force

$$S = 39.73 \times 1.300 = 51.65 \text{ kN/m}$$

c) Checking section

Various constant of section



	Formula					A	y	Ay	Ay^2	I
						m^2	m	m^3	m^4	m^4
1		0.700	×	1.000		0.700	2.500	1.750	4.375	0.058
2		0.650	×	1.100		0.715	1.450	1.037	1.503	0.072
3	1/2×	0.100	×	1.100		0.055	1.633	0.090	0.147	0.004
4		0.200	×	0.800		0.160	0.500	0.080	0.040	0.009
5	1/2×	0.450	×	0.800		0.180	0.633	0.114	0.072	0.006
6	1/2×	0.200	×	0.100		0.010	0.067	0.001	0.000	0.000
Total						1.820	1.688	3.071	6.137	0.149

Various constants of section in centroid axis

Geometrical accuracy moment of inertia

$$I = 6.137 + 0.149 - 1.820 \times 1.688^{2} = 1.104\ m^4$$

Modulus of section

$$Z_U = \frac{1.104}{3.000 - 1.688} = 0.841\ m^3$$

$$Z_L = \frac{1.104}{1.688} = 0.654\ m^3$$

Checking section

		Case-1			Case-2								
		Under side tension			Upper side tension								
M	kN•m	13.11			33.57								
N	kN	0.00			0.00								
S	kN	32.28			51.65								
Z_U	m^3	0.841			0.841								
Z_L	m^3	0.654			0.654								
A	m^2	1.820			1.820								
σ c	N/mm^2	0.02	<	8.25	0.05	<	8.25						
σ t	N/mm^2	0.02	<	0.45	0.04	<	0.45						
τ	N/mm^2	0.02	<	0.39	0.03	<	0.39						
Judgement		OK			OK								

4-3. Calculation for earth retaining wall

Design of earth retaining wall for temporaray work is carried out.

Load in that situation is considered to be active earth pressure plus hydrostatic pressure plus uneven earth pressure.

1) Calculation for load

Calculation for earth pressure intensity

Active earth pressure

$p_a \quad = K_A \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \quad = \gamma_w \cdot \Sigma h_n$

To the above formula

p_a : Active earth pressure

p_w : Hydrostatic pressure (kN/m^2)

K_A : Coefficient of active earth pressure by Coulomb's earth pressure

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

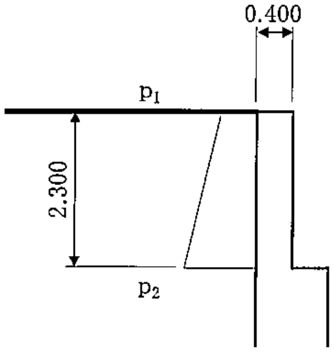
(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vercial load	Earth pressure coefficient	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	1.000	10.00	0.00	Ground level
	-1.570	2.300	2.300	16.0	7.0	10.0	46.80	1.000	46.80	0.00	7R soffit
	-1.760	2.490	0.190	16.0	7.0	10.0	49.84	1.000	49.84	0.00	Groundwater level

2) Calculation for sectional force



p_1 = 10.00 + 1/2 × 10.00 = 15.00 kN/m^2

p_1 = 46.80 + 1/2 × 46.80 = 70.20 kN/m^2

Bending moment

$$M = (\frac{1}{3} \times 15.00 + \frac{1}{6} \times 70.20) \times 2.300^2 = 88.34 \text{ kN·m/m}$$

Shear force

$$S = \frac{1}{2} \times (15.00 + 70.20) \times 2.300 = 97.98 \text{ kN/m}$$

3) Checking section

- As for minimu volueme of reinforcing bar

Minimum volume of reinforcing bar is set with 0.2 and over of effective sectional area of member.

Member	b	h	d'	Formula	Arrangement of minimum reinforcing bar	
	mm	mm	mm	mm^2	mm^2	
Lateral wall	1000.0	400.0	100.0	1000.0 × 300.0 × 0.002 = 600.0	D 16 @ 250	794.4

		Earth retaining wall			
		Exterior surface			
M	kN･m	88.34			
N	kN	0.00			
S	kN	97.98			
b	mm	1000			
h	mm	400			
d'	mm	100			
d	mm	300			
As	cm^2	D 22 @ 250			
		15.484			
p		0.00516			
k		0.324			
j		0.892			
σc	N/mm^2	6.8 < 12.0			
σs	N/mm^2	213.2 < 240			
τ	N/mm^2	0.33 < 0.58			
τ_{al}	N/mm^2	0.35			
C_e		1.400			
C_{pt}		1.210			
C_N		1.000			
n		15			

5. Checking member in regular time

5-1. Calculation for lateral wall

In regular time, only earth pressure at rest plus hydrostatic pressure is set as targes. The pressures are acted towards lateral with right angle wall from 4 directions.
Coefficient of earth pressure at rest adopts 0.5 without difference of sandy soil and cohesive soil. As for distribution of intensity of earth pressure at rest, if the depth is within 15m, the distribution is set as triangular distribution, while if the depth is over 15m, the distribution is considered to be same as intensity of earth

1) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$p_a = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w = \gamma_w \cdot \Sigma h_n$

To the above formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

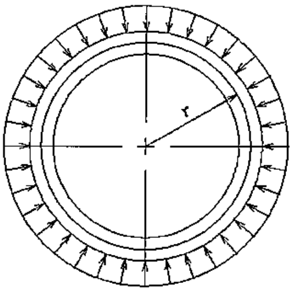
(In case under groundwater level, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness in each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Ac1	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.570	2.300	2.300	16.0	7.0	10.0	46.80	0.500	23.40	0.00	7R soffit
	-1.760	2.490	0.190	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	-
	-6.970	7.700	3.700	16.0	7.0	10.0	86.31	0.500	43.16	52.10	6R soffit
	-12.270	13.000	5.300	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-12.370	13.100	0.100	16.0	7.0	10.0	124.11	0.500	62.06	106.10	5R soffit
	-14.270	15.000	1.900	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	15m
	-17.770	18.500	3.500	16.0	7.0	10.0	161.91	0.500	68.71	160.10	4R soffit
	-23.170	23.900	5.400	16.0	7.0	10.0	199.71	0.500	68.71	214.10	3R soffit
	-24.270	25.000	1.100	16.0	7.0	10.0	207.41	0.500	68.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	-
	-26.270	27.000	2.000	16.0	7.0	10.0	221.41	0.500	68.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	68.71	245.10	-
	-28.570	29.300	2.300	16.0	7.0	10.0	237.51	0.500	68.71	268.10	2R soffit
	-29.570	30.300	1.000	16.0	7.0	10.0	244.51	0.500	68.71	278.10	Undersurface of bottom slab

2) Calculation for sectional force



- In case of receiving equal loads from 4 directions

  (In this case, there is no occurrence of bending momenet.)

  Axial force

  N = 1.000 · p · r

Form and working load

Checking location		Interior diameter	Thickness of member	Diamter of axis of member	Radius of axis of member	Active earth pressure	Hydrostatic pressure	Unbalanced load
		m	m	m	m	kN/m^2	kN/m^2	kN/m^2
1	6R soffit	5.000	0.700	5.700	2.850	43.16	52.10	0.00
2	5R soffit	5.000	0.700	5.700	2.850	62.06	106.10	0.00
3	4R soffit	5.000	0.700	5.700	2.850	68.71	160.10	0.00
4	3R soffit	5.000	0.700	5.700	2.850	68.71	214.10	0.00
5	2R soffit	5.000	0.700	5.700	2.850	68.71	268.10	0.00
6	Undersurface of bottom slab	5.000	0.700	5.700	2.850	68.71	278.10	0.00

Calculation for sectional force

Axial force	Uniform load	Radius	N
	kN/m^2	m	kN/m
6R soffit	95.26	2.850	271.48
5R soffit	168.16	2.850	479.24
4R soffit	228.81	2.850	652.09
3R soffit	282.81	2.850	805.99
2R soffit	336.81	2.850	959.89
Undersurface of bottom slab	346.81	2.850	988.39

c) Checking section

$$\sigma_c = \frac{N}{A}$$

to the above formula

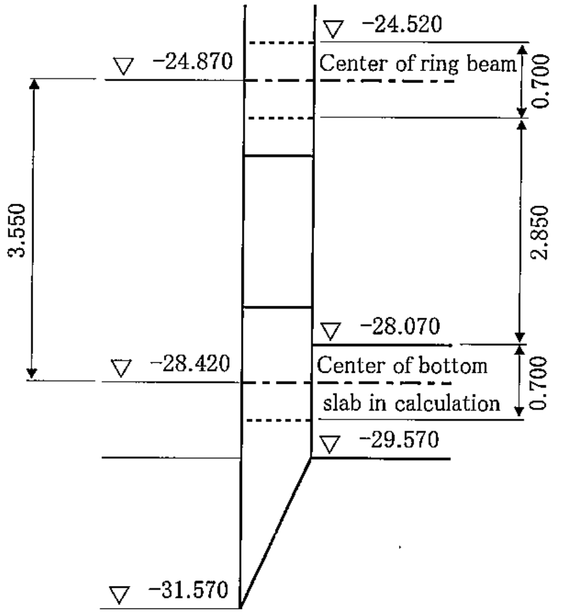
σ_c : Compressive stress (N/mm^2)

A : Sectional area of member mm^2

Checking location		N	b	h	A	σ_c	σ_{ca}	Judgement
		kN/m	mm	mm	mm^2	N/mm^2	N/mm^2	
1	6R soffit	271.48	1,000	700	700,000	0.39	8.00	○
2	5R soffit	479.24	1,000	700	700,000	0.68	8.00	○
3	4R soffit	652.09	1,000	700	700,000	0.93	8.00	○
4	3R soffit	805.99	1,000	700	700,000	1.15	8.00	○
5	2R soffit	959.89	1,000	700	700,000	1.37	8.00	○
6	Undersurface of bottom slab	988.39	1,000	700	700,000	1.41	8.00	○

5-2. Calculation for opening of lateral wall

1) Calculation for peripheral of opening of lateral wall (part of both ends fixed beam)



a) Calculation for load

Calclation for earth pressure intensity

Earth pressure at rest

p_a = $K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

p_w = $\gamma_w \cdot \Sigma h_n$

To the formula

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

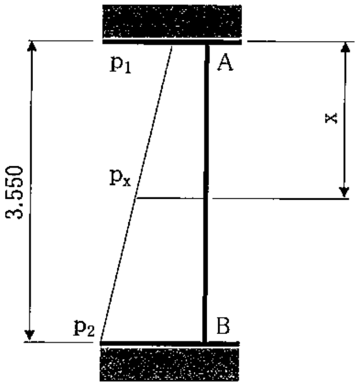
h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficint of earth pressure	Horizaontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.760	2.490	2.490	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	-
	-12.270	13.000	9.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-14.270	15.000	2.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	15m
	-24.270	25.000	10.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	-
	-24.870	25.600	0.600	16.0	7.0	10.0	211.61	0.500	68.71	231.10	Center of ring beam
	-26.270	27.000	1.400	16.0	7.0	10.0	221.41	0.500	68.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	68.71	245.10	-
	-28.420	29.150	2.150	16.0	7.0	10.0	236.46	0.500	68.71	266.60	Center of bottom slab
	-29.570	30.300	1.150	16.0	7.0	10.0	244.51	0.500	68.71	278.10	Undersurface of bottom slab

p_1 = 68.71 + 231.10 = 299.81 kN/m^2

p_2 = 68.71 + 266.60 = 335.31 kN/m^2

b) Calculation for sectional force



Bending moment of supporting point

$$M_A = \left(\frac{1}{20} \times 299.81 + \frac{1}{30} \times 335.31\right) \times 3.550^{2} = 329.77 \ \text{kN·m/m}$$

$$M_B = \left(\frac{1}{20} \times 335.31 + \frac{1}{30} \times 299.81\right) \times 3.550^{2} = 337.23 \ \text{kN·m/m}$$

Bending moment of span

$$p_x = p_1 + \frac{p_2 - p_1}{L} x$$

$$S_A = \frac{1}{2} \times (p_1 + p_x) \times x$$

From these

$$\frac{p_2 - p_1}{2 \cdot L} x^{2} + p_1 x - S_A = 0$$

Therefore,

$$x = 1.785 \ \text{m}$$

Load intensity

$$p_x = 299.81 + \frac{335.31 - 299.81}{3.550} \times 1.785 = 317.65 \ \text{kN/m}^2$$

$$M_{max} = - 329.77 - 551.06 \times 1.785$$

$$+ \left(\frac{1}{3} \times 299.81 + \frac{1}{6} \times 317.65\right) \times 1.785^{2} = 166.77 \ \text{kN·m/m}$$

Shear force

$$S_A = \left(\frac{7}{20} \times 299.81 + \frac{3}{20} \times 335.31\right) \times 3.550 = 551.06 \ \text{kN/m}$$

$$S_B = \left(\frac{7}{20} \times 335.31 + \frac{3}{20} \times 299.81\right) \times 3.550 = 576.26 \ \text{kN/m}$$

c) Checking section

		On supporting point	Span	Under supporting point	
		Exterior surface	Inner surface	Exterior surface	
M	kN·m	329.77	166.77	337.23	
N	kN	0.00	0.00	0.00	
S	kN	551.06	0.00	576.26	
b	mm	1000	1000	1000	
h	mm	700	700	700	
d'	mm	125.5	125.5	125.5	
d	mm	575	575	575	
As	cm^2	D 29 @ 125	D 25 @ 250	D 29 @ 125	
		51.392	20.268	51.392	
p		0.00895	0.00353	0.00895	
k		0.401	0.277	0.401	
j		0.866	0.908	0.866	
σc	N/mm^2	5.8 < 8.0	4.0 < 8.0	5.9 < 8.0	
σs	N/mm^2	128.9 < 160	157.8 < 160	131.8 < 160	
τ	N/mm^2	0.96 > 0.41	0.00 < 0.30	1.00 > 0.41	
τ_{al}	N/mm^2	0.23	0.23	0.23	
C_e		1.243	1.243	1.243	
C_{pt}		1.437	1.053	1.437	
C_N		1.000	1.000	1.000	
n		15	15	15	

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 340.26 \times 10^3 \times 250}{160 \times 574.5} \times 10^{-2}$$

$\fallingdotseq$ 10.64 cm^2/m < 4 Number D 19 (= 11.460 cm^2) are arranged.

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.41 \times 1000 \times 574.5 \times 10^{-3}$

$\fallingdotseq$ 236.00 kN

τa = 0.41 N/mm^2

b = 1000 mm

d = 575 mm

Shear force received by diagonal tension bar

$Sh = S - Sc$

= 576.26 − 236.00

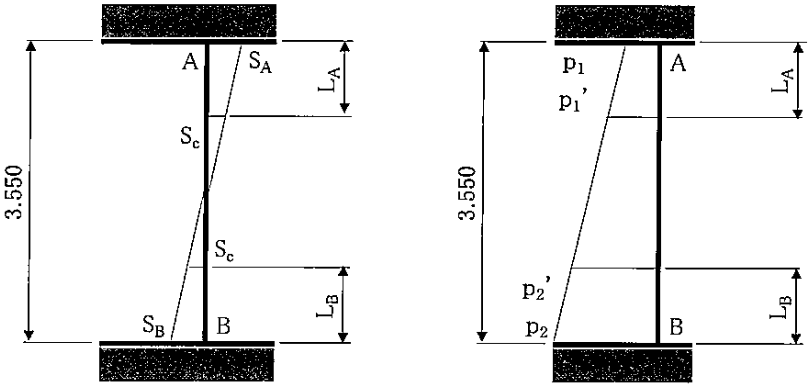
= 340.26 kN

S = 576.26 kN

a = 250 mm

σsa = 160 N/mm^2

Arrangement of sphere of diagonal tension bar



p_1 = 299.81 kN/m^2
p_2 = 335.31 kN/m^2

- Calculation for L_A

$$p_1' = p_1 + \frac{p_2 - p_1}{L} L_A$$

$$S_A = \frac{1}{2} \times (p_1 + p_1') \times L_A + S_c$$

From these

$$\frac{p_2 - p_1}{2 \cdot L} L_A{}^2 + p_1 L_A + S_c - S_A = 0$$

Therefore

L_A = 1.033 m

- Calculation for L_B

$$p_2' = p_2 - \frac{p_2 - p_1}{L} L_B$$

$$S_B = \frac{1}{2} \times (p_2' + p_2) \times L_B + S_c$$

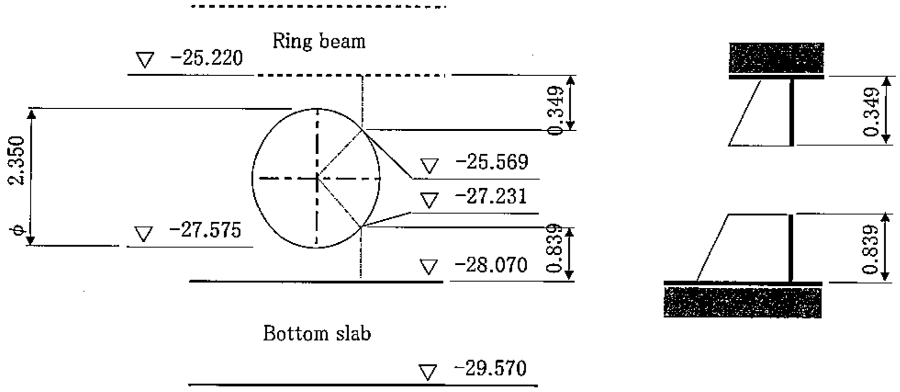
From these

$$-\frac{p_2 - p_1}{2 \cdot L} L_B{}^2 + p_2 L_B + S_c - S_B = 0$$

Therefore

L_A = 1.031 m

2) Calculatation for peripheral of opening of lateral wall(part of cantilever)



a) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$p_a \quad = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \quad = \gamma_w \cdot \Sigma h_n$

From this

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coefficient of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soil in each stratum (kN/m$^{3)}$

(In case under groundwater, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficint of earth pressure	Horizaontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.760	2.490	2.490	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	–
	-12.270	13.000	9.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-14.270	15.000	2.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	15m
	-24.270	25.000	10.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	-
	-25.220	25.950	0.950	16.0	7.0	10.0	214.06	0.500	68.71	234.60	Soffit of ring beam
	-25.569	26.299	0.349	16.0	7.0	10.0	216.50	0.500	68.71	238.09	Soffit of upper side of opening
	-26.270	27.000	0.701	16.0	7.0	10.0	221.41	0.500	68.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	68.71	245.10	-
	-27.231	27.961	0.961	16.0	7.0	10.0	228.14	0.500	68.71	254.71	Soffit of lower side of opening
	-28.070	28.800	0.839	16.0	7.0	10.0	234.01	0.500	68.71	263.10	Upper side of bottom slab

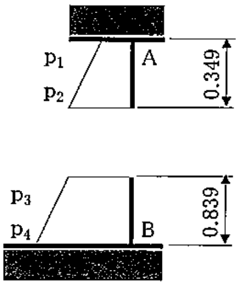
$p_1 = 68.71 + 234.60 = 303.31$ kN/m^2

$p_2 = 68.71 + 238.09 = 306.80$ kN/m^2

$p_3 = 68.71 + 254.71 = 323.41$ kN/m^2

$p_4 \quad = \quad 68.71 \quad + \quad 263.10 \qquad\qquad = \quad 331.81 \ \text{kN/m}^2$

b) Calculation for sectional force



Bending moment of supporting point

$$M_A \quad = \quad (\frac{1}{6} \times \ 303.31 \quad + \frac{1}{3} \times \ 306.80 \ ) \times \ 0.349^{\ 2} \qquad = \quad 18.63 \ \text{kN·m/m}$$

$$M_B \quad = \quad (\frac{1}{3} \times \ 323.41 \quad + \frac{1}{6} \times \ 331.81 \ ) \times \ 0.839^{\ 2} \qquad = \quad 114.85 \ \text{kN·m/m}$$

Shear force

$$S_A \quad = \frac{1}{2} \times (\ 303.31 \quad + \quad 306.80 \ ) \times \ 0.349 \qquad = \quad 106.51 \ \text{kN/m}$$

$$S_B \quad = \frac{1}{2} \times (\ 323.41 \quad + \quad 331.81 \ ) \times \ 0.839 \qquad = \quad 274.91 \ \text{kN/m}$$

c) Checking section

	On supporting point	Under supporting point		
	Exterior surface	Exterior surface		
M kN·m	18.63	114.85		
N kN	0.00	0.00		
S kN	106.51	274.91		
b mm	1000	1000		
h mm	700	700		
d' mm	130	130		
d mm	570	570		
As cm^2	D 19 @ 250	D 22 @ 250		
	11.460	15.484		
p	0.00201	0.00272		
k	0.217	0.248		
j	0.928	0.917		
σc N/mm^2	0.6 < 8.0	3.1 < 8.0		
σs N/mm^2	30.7 < 160	141.8 < 160		
τ N/mm^2	0.19 < 0.26	0.48 > 0.28		
τ_{a1} N/mm^2	0.23	0.23		
C_e	1.246	1.246		
C_{pt}	0.901	0.972		
C_N	1.000	1.000		
n	15	15		

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 116.23 \times 10^3 \times 250}{160 \times 570} \times 10^{-2}$$

$= 3.66\ cm^2/m$ < 4 Number D 13 (= 5.068 cm^2) is arranged

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.28 \times 1000 \times 570 \times 10^{-3}$

$= 158.68$ kN

$\tau a = 0.28\ N/mm^2$

$b = 1000$ mm

$d = 570$ mm

Shear force received by diagonal tension bar

$Sh = S - Sc$

$= 274.91 - 158.68$

$= 116.23$ kN

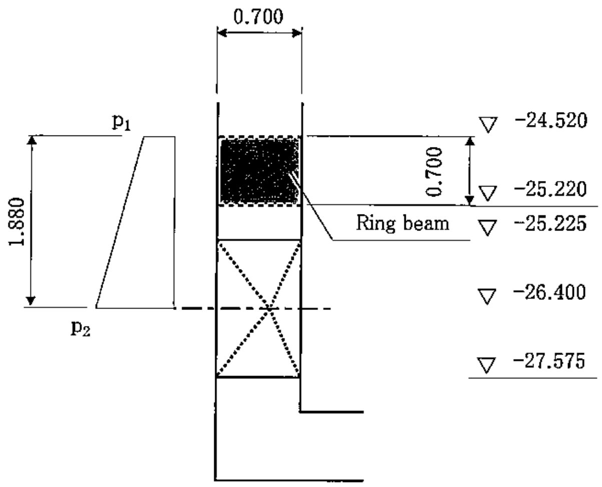
$S = 274.91$ kN

$a = 250$ mm

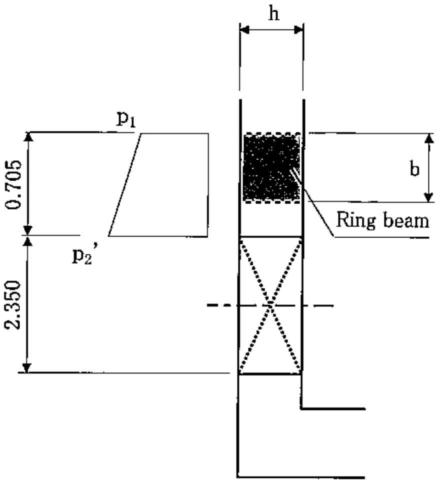
$\sigma sa = 160\ N/mm^2$

3) Calculation for ring beam

Load applied into ring beam



Location of edge of opening part



Location of center of opening part

a) Calculation for load

Calculation for earth pressure intensity

Earth pressure at rest

$p_a \quad = K_0 \cdot [q_0 + \Sigma(\gamma_n \cdot h_n)]$

Hydrostatic pressure

$p_w \quad = \gamma_w \cdot \Sigma h_n$

From this

p_0 : Earth pressure at rest

p_w : Hydrostatic pressure (kN/m^2)

K_0 : Coeffieicnet of earth pressure at rest

q_0 : Load placed on the top (kN/m^2)

= 10.0 kN/m^2

γ_n : Unit volume weight of soild of each stratum (kN/m$^{3)}$

(In case under groundwater level, submerged weight)

γ_w : Unit volume weight of groundwater (kN/m$^{3)}$

h_n : Layer thickness of each stratum (m)

Soil	Elevation	Depth	Layer thickness	γ	γ'	γ_w	Vertical load	Coefficient of earth pressure	Horizontal earth pressure	Hydrostatic pressure	Notes
	m	m	m	kN/m^3	kN/m^3	kN/m^3	kN/m^2		kN/m^2	kN/m^2	
Acl	0.730	0.000	0.000	16.0	7.0	10.0	10.00	0.500	5.00	0.00	Ground level
	-1.760	2.490	2.490	16.0	7.0	10.0	49.84	0.500	24.92	0.00	Groundwater level
	-3.270	4.000	1.510	16.0	7.0	10.0	60.41	0.500	30.21	15.10	Change point of stratum
Ac2	-3.270	4.000	0.000	16.0	7.0	10.0	60.41	0.500	30.21	15.10	-
	-12.270	13.000	9.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	Change point of stratum
Ac3	-12.270	13.000	0.000	16.0	7.0	10.0	123.41	0.500	61.71	105.10	-
	-14.270	15.000	2.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	Change point of stratum
Ac4	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	-
	-14.270	15.000	0.000	16.0	7.0	10.0	137.41	0.500	68.71	125.10	15m
	-24.270	25.000	10.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	Change point of stratum
Ac5	-24.270	25.000	0.000	16.0	7.0	10.0	207.41	0.500	68.71	225.10	-
	-24.520	25.250	0.250	16.0	7.0	10.0	209.16	0.500	68.71	227.60	Upper bed of ring beam
	-25.220	25.950	0.700	16.0	7.0	10.0	214.06	0.500	68.71	234.60	Soffit of ring beam
	-25.225	25.955	0.005	16.0	7.0	10.0	214.10	0.500	68.71	234.65	Upper bed of opening
	-26.270	27.000	1.045	16.0	7.0	10.0	221.41	0.500	68.71	245.10	Change point of stratum
Ac6	-26.270	27.000	0.000	16.0	7.0	10.0	221.41	0.500	68.71	245.10	-
	-26.400	27.130	0.130	16.0	7.0	10.0	222.32	0.500	68.71	246.40	Center of opening

$p_1 \quad = \quad 68.71 \quad + \quad 227.60 \quad = \quad 296.31$ kN/m^2

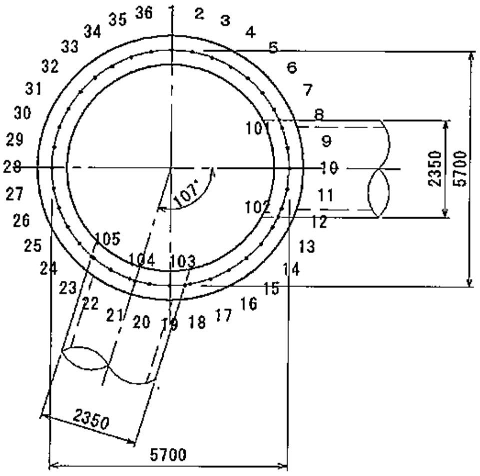
$p_2 \quad = \quad 68.71 \quad + \quad 246.40 \quad = \quad 315.11$ kN/m^2

$p_2' \quad = \quad 68.71 \quad + \quad 234.65 \quad = \quad 303.36$ kN/m^2

$$p_{w1} = \frac{1}{2} \times (\ 296.31 \ + \ 315.11 \ ) \times \ 1.880 \qquad = \ 574.73 \ \text{kN/m}$$

$$p_{w2} = \frac{1}{2} \times (\ 296.31 \ + \ 303.36 \ ) \times \ 0.705 \qquad = \ 211.38 \ \text{kN/m}$$

b) Skelton diagram



Coordinates of panel point

Panel point	Coordinates		Panel point	Coordinates		Panel point	Coordinates	
Number	x (m)	y (m)	Number	x (m)	y (m)	Number	x (m)	y (m)
1	0.000	2.850	19	0.000	-2.850	101	2.597	1.175
2	0.495	2.807	20	-0.495	-2.807	102	2.597	-1.175
3	0.975	2.678	21	-0.975	-2.678	103	0.365	-2.827
4	1.425	2.468	22	-1.425	-2.468	104	-0.833	-2.725
5	1.832	2.183	23	-1.832	-2.183	105	-1.883	-2.140
6	2.183	1.832	24	-2.183	-1.832	-	-	-
7	2.468	1.425	25	-2.468	-1.425	-	-	-
8	2.678	0.975	26	-2.678	-0.975	-	-	-
9	2.807	0.495	27	-2.807	-0.495	-	-	-
10	2.850	0.000	28	-2.850	0.000	-	-	-
11	2.807	-0.495	29	-2.807	0.495	-	-	-
12	2.678	-0.975	30	-2.678	0.975	-	-	-
13	2.468	-1.425	31	-2.468	1.425	-	-	-
14	2.183	-1.832	32	-2.183	1.832	-	-	-
15	1.832	-2.183	33	-1.832	2.183	-	-	-
16	1.425	-2.468	34	-1.425	2.468	-	-	-
17	0.975	-2.678	35	-0.975	2.678	-	-	-
18	0.495	-2.807	36	-0.495	2.807	-	-	-

Sectional area and moment of second order

	Width	Height	Sectional area	Moment of second order
	b	h	A	I
	m	m	m^2	m^4
Ring beam	0.700	0.700	0.490	0.020008

Sectional area $A = b \cdot h$

Second moment of area $I = \frac{1}{12} b \cdot h^3$

Calculation for coefficient of ground reaction

Coefficient of horizontal ground reaction

$$k_H = k_{H0} \cdot \left(\frac{B_H}{0.3} \right)^{-3/4}$$

$$k_{H0} = \frac{1}{0.3} \cdot \alpha \cdot E_0$$

$$\alpha = 1$$

$$E_0 = 142{,}800 \ \mathrm{kN/m^2}$$

$$B_H = \sqrt{A_H}$$

$$= \sqrt{0.8 \times 6.400 \times 30.100}$$

$$= 12.414 \ \mathrm{m} \geqq 10.0 \ \mathrm{m}$$

$$k_H = 476{,}000 \times \left(\frac{10.000}{0.3} \right)^{-3/4}$$

$$= 34{,}312 \ \mathrm{kN/m^3}$$

$$k_{H0} = \frac{1}{0.3} \times 1 \times 142{,}800$$

$$= 476{,}000 \ \mathrm{kN/m^3}$$

$$K_H = 34{,}312 \times 0.700$$

$$= 24{,}019 \ \mathrm{kN/m^2}$$

※ Only compression spring is valid.

c) Calculation for sectional force

Diagram of load

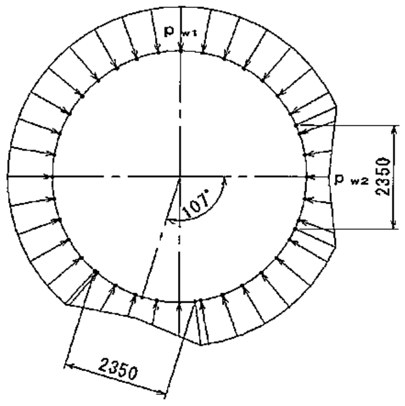
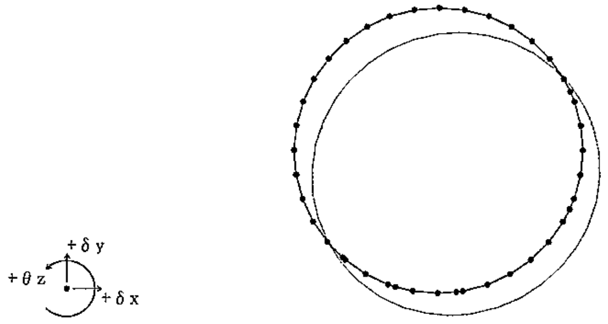
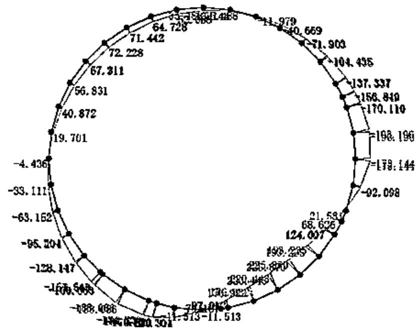


Diagram of transposition



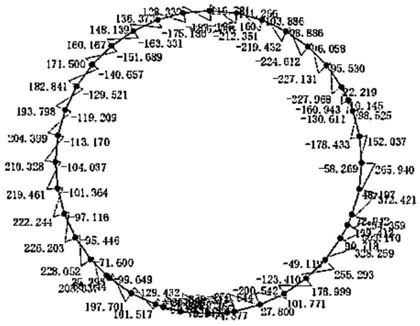
Stress diagram

Diagram Mz of sectional force



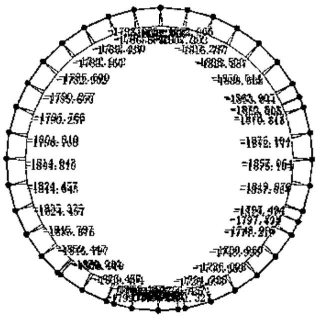
+M (i———j) +M

Diagram Sy of sectional force



+S ↑ i———j ↓ +S

Diagram Nx of sectional force



+N ← i———j → +N

c) Checking section

- Checking to bending

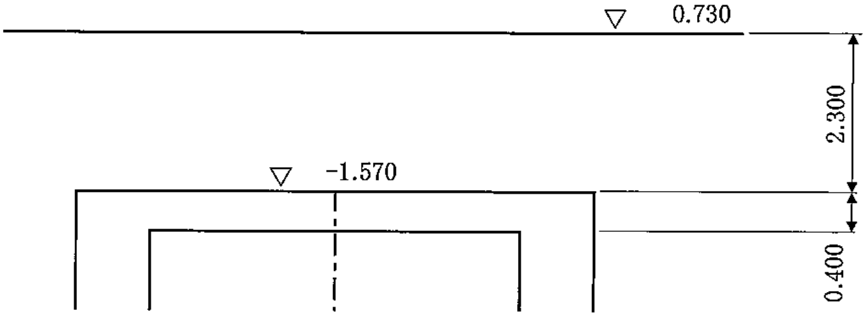
	Panel point 9	Panel point 15	Panel point 22	Panel point 33
	Exterior surface	Inner surface	Exterior surface	Inner surface
M kN·m	175.33	206.88	172.33	66.84
N kN	1,654.44	1,521.71	1,653.67	1,571.11
S kN	0.00	0.00	0.00	0.00
b mm	700	700	700	700
h mm	700	700	700	700
d' mm	100	100	100	100
d mm	600	600	600	600
As cm^2	3 Number D 22	3 Number D 22	3 Number D 22	3 Number D 22
	11.613	11.613	11.613	11.613
p	0.00277	0.00277	0.00277	0.00277
k	0.250	0.250	0.250	0.250
j	0.917	0.917	0.917	0.917
σ c N/mm^2	6.2 < 8.0	6.8 < 8.0	6.1 < 8.0	4.2 < 8.0
σ s N/mm^2	-92.5 < 160	-5.9 < 160	-91.8 < 160	-63.1 < 160
τ N/mm^2	0.00 < 0.55	0.00 < 0.51	0.00 < 0.55	0.00 < 0.55
τ_{al} N/mm^2	0.23	0.23	0.23	0.23
C_e	1.229	1.229	1.229	1.229
C_{pt}	0.977	0.977	0.977	0.977
C_N	2.000	1.858	2.000	2.000
n	15	15	15	15

- Checking to shear

		Panel point18						
		Inner surface						
M	kN·m	87.76						
N	kN	1,560.30						
S	kN	229.09						
b	mm	700						
h	mm	700						
d’	mm	100						
d	mm	600						
As	cm^2	3	Amount	D	22			
		11.613						
p		0.00277						
k		0.250						
j		0.917						
σ c	N/mm^2	4.5	<	8.0				
σ s	N/mm^2	-68.0	<	160				
τ	N/mm^2	0.55	<	0.55				
τ_{a1}	N/mm^2	0.23						
C_e		1.229						
C_{pt}		0.977						
C_N		2.000						
n		15						

5-3. Design of top slab

1) Calculation for load



Load of earth covering

w_s = 19.0 × 2.300 = 43.70 kN/m^2

Empty load of top slab

w_t = 24.5 × 0.400 = 9.80 kN/m^2

Live load

q = 10.00 kN/m^2

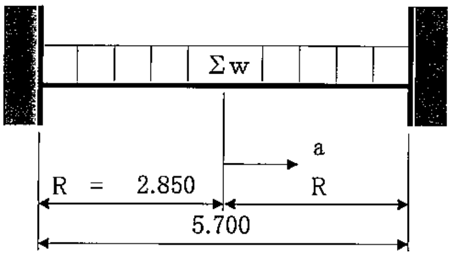
Σw = 63.50 kN/m^2

(Reference)

$$w_l = \frac{2 \times 15 \times (1 + 0.3) \times 1.0}{2.700 \times (2 \times 2.300 + 0.200)} = 3.01 \text{ kN/m}^2$$

2) Calculation for sectional force

Peripheral of circular plate is supposed to be fixed on lateral wall and sectional force is calculated.



Bending moment

$$Mr = \frac{1}{16} \cdot \Sigma w \cdot R^2 \cdot [(1 + \nu) - (3 + \nu) \cdot (\frac{a}{R})^2]$$

Shear force

$$Qr = \frac{1}{2} \cdot \Sigma w \cdot a$$

To the above formula

Mr : Bending moment applied to top slab(kN·m/m)

Qr : Shear force applied to top slab (kN/m)

Σw : Applied load (kN/m^2)

R : Radius of circular plate (m)

ν : Poisson's ration = 1 / 6

a : Distance from center of cirbular plate(m)

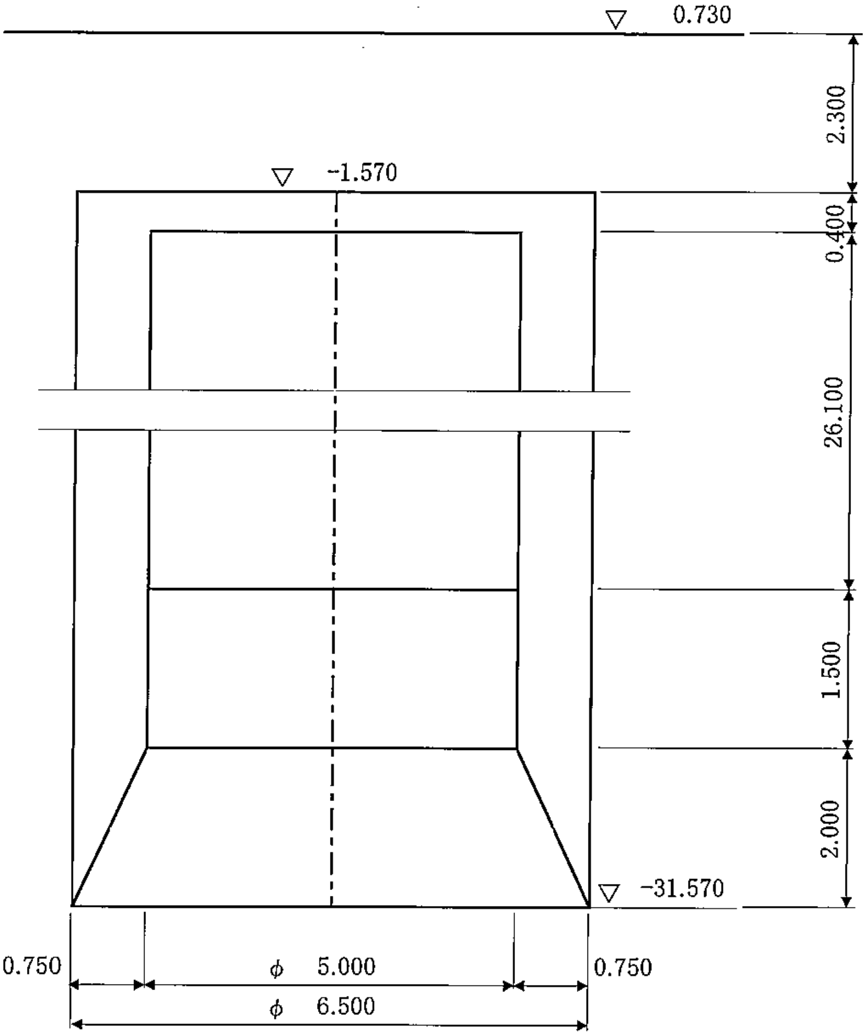
	R	a	Σw	Mr	Qr	Notes
	m	m	kN/m^2	kN·m/m	kN/m	
0	2.850	0.000	63.50	37.61	0.00	Center part
1	2.850	1.000	63.50	25.04	31.75	
2	2.850	1.730	63.50	-0.01	54.93	Inflection point
3	2.850	2.000	63.50	-12.66	63.50	
4	2.850	2.300	63.50	-28.87	73.03	Checking shear(H/2)
5	2.850	2.850	63.50	-64.47	90.49	Edge

3) Checking section

		Top slab															
		Edge				Center				Checking shear							
M	kN·m	64.47				37.61				28.87							
N	kN	0.00				0.00				0.00							
S	kN	0.00				0.00				73.03							
b	mm	1000				1000				1000							
h	mm	400				400				400							
d'	mm	100				100				100							
d	mm	300				300				300							
As	cm^2	D	22	@	##	D	19	@	250	D	22	@	250				
		15.484				11.460				15.484							
p		0.00516				0.00382				0.00516							
k		0.324				0.286				0.324							
j		0.892				0.905				0.892							
σc	N/mm^2	5.0	<	8.0		3.2	<	8.0		2.2	<	8.0					
σs	N/mm^2	155.6	<	160		120.9	<	160		69.7	<	160					
τ	N/mm^2	0.00	<	0.39		0.00	<	0.35		0.24	<	0.39					
τ_{al}	N/mm^2	0.23				0.23				0.23							
C_e		1.400				1.400				1.400							
C_{pt}		1.210				1.082				1.210							
C_N		1.000				1.000				1.000							
n		15				15				15							

5-4. Design of bottom slab

1) Calculation for load



Load of earth covering

w_s = 19.0 × 2.300 = 43.70 kN/m^2

Empty load of top slab

w_t = 24.5 × 0.400 = 9.80 kN/m^2

Live load

q = 10.00 kN/m^2

Empty load of lateral load

$$w_w = \frac{24.5 \times (6.400^2 - 5.000^2)}{6.500^2} \times 26.100 = 513.98 \text{ kN/m}^2$$

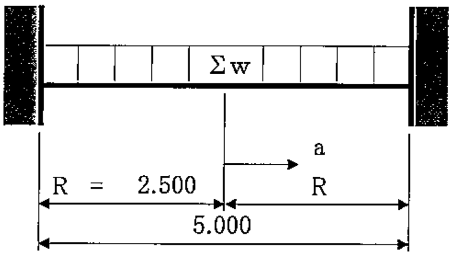
Empty load of medium slab

w_m = 11 × 24.5 × 0.300 = 80.85 kN/m^2

Σw = 658.33 kN/m^2

2) Calculation for sectional force

Circular plate, with the peripheral fixed on lateral wall is supposed and sectional force is calculated.



Bending moment

$$Mr = \frac{1}{16} \cdot \Sigma w \cdot R^{2} \cdot [(1 + \nu) - (3 + \nu) \cdot (\frac{a}{R})^{2}]$$

Shear force

$$Qr = \frac{1}{2} \cdot \Sigma w \cdot a$$

To the formula

Mr : Bending moment applied to bottom slab (kN•m/m)

Qr : Shear force applied to bottom slab(kN/m)

Σw : Applied load(kN/m^2)

R : Radius of circular plate (m)

ν : Poisson's ration = 1 / 6

a : Distance from the center of circular plates(m)

	R	a	Σw	Mr	Qr	Notes
	m	m	kN/m^2	kN•m/m	kN/m	
0	2.500	0.000	658.33	300.02	0.00	Center part
1	2.500	1.000	658.33	169.73	329.17	
2	2.500	1.517	658.33	0.17	499.34	Inflection point
3	2.500	2.000	658.33	-221.16	658.33	
4	2.500	1.750	658.33	-99.01	576.04	Checking shear (H/2)
5	2.500	2.500	658.33	-514.32	822.91	Edge

3) Checking section

	Bottom slab			
	Edge	Center part	Checking shear	
M kN·m	514.32	300.02	99.01	
N kN	0.00	0.00	0.00	
S kN	0.00	0.00	576.04	
b mm	1000	1000	1000	
h mm	1500	1500	1500	
d' mm	110	110	110	
d mm	1390	1390	1390	
As cm^2	D 29 @ ##	D 22 @ 250	D 29 @ 250	
	25.696	15.484	25.696	
p	0.00185	0.00111	0.00185	
k	0.209	0.167	0.209	
j	0.930	0.944	0.930	
σc N/mm^2	2.7 < 8.0	2.0 < 8.0	0.5 < 8.0	
σs N/mm^2	154.8 < 160	147.6 < 160	29.8 < 160	
τ N/mm^2	0.00 < 0.19	0.00 < 0.16	0.41 > 0.19	
τ_{a1} N/mm^2	0.23	0.23	0.23	
C_e	0.942	0.942	0.942	
C_{pt}	0.870	0.723	0.870	
C_N	1.000	1.000	1.000	
n	15	15	15	

※ Calculation for diagonal tension bar

$$Aw = \frac{1.15 \cdot Sh \cdot a}{\sigma sa \cdot d \cdot (\sin\theta + \cos\theta)}$$

$$= \frac{1.15 \times 314.25 \times 10^3 \times 250}{160 \times 1390} \times 10^{-2}$$

$= 4.06 \ cm^2/m \ < \ 4$ Number D 13 (= 5.068 cm^2) are arranged

Shear force received by concrete

$Sc = \tau a \cdot b \cdot d$

$= 0.19 \times 1000 \times 1390 \times 10^{-3}$

$= 261.79$ kN

$\tau a = 0.19 \ N/mm^2$

$b = 1000$ mm

$d = 1390$ mm

Shear force received by diagonal tension bar

$Sh = S - Sc$

$= 576.04 - 261.79$

$= 314.25$ kN

$S = 576.04$ kN

$a = 250$ mm

$\sigma sa = 160 \ N/mm^2$

Sphere of arrangement of diagonal tension bar

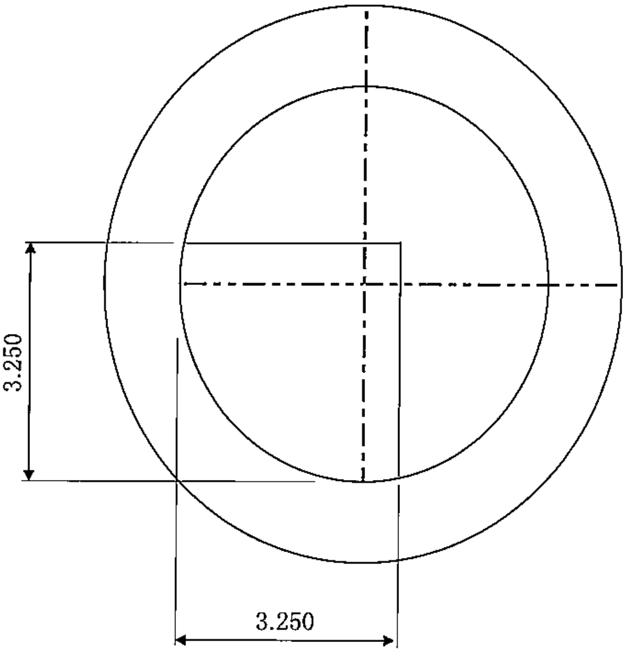
$$L = r - \frac{a}{S} \times S_c$$

$$= 2.500 - \frac{1.750}{576.04} \times 261.79$$

$= 1.705$ m

5-5. Design of medium slab

1) Calculation for from B1F to B5F



a). Calculation for load

Self weight of medium slab

w_s = 24.5 × 0.300 = 7.35 kN/m^2

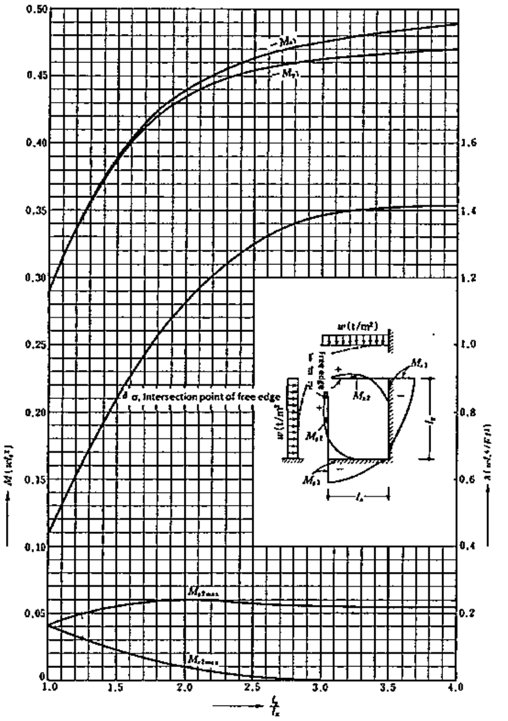
Sidewalk live load

q = 5.00 kN/m^2

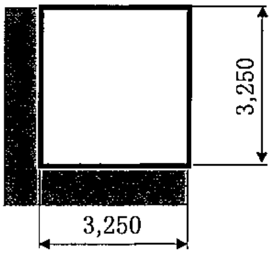
p = 12.35 kN/m^2

b). Calculation for stress

Plate of fixed 2 sides making axis of member length is considered.



Attached diagram 10.5 stress diagram of slab with two free next edges in uniform load and deformation σ1)(v=0) of intersection point of free edge



$$\frac{L_y}{L_x} = \frac{3.250}{3.250} = 1.00$$

From the left diagram

$-M_{x1} = 0.290$

$M_{x2max} = 0.041$

$-M_{y1} = 0.290$

$M_{y2max} = 0.041$

X direction (Direction for short span)

Bending moment of supporting point

$$M_{(supporting)} = 0.290 \times 12.35 \times 3.250^{2} = 37.83 \text{ kN·m/m}$$

Bending moment of span

$$M_{diameter} = 0.041 \times 12.35 \times 3.250^{2} = 5.35 \text{ kN·m/m}$$

Shear force

- $x = 0$

$$S = \frac{12.35 \times 3.250 \times 3.250}{3.250 + 3.250} = 20.07 \text{ kN/m}$$

- $x = 0.150$ m (located h/2 away from inside of lateral wall)

$$S = \frac{1 \times 20.07}{3.250} \times \left(\frac{3.250}{1} - 0.150 \right) = 19.14 \text{ kN/m}$$

Y direction (direction for long span)

Bending moment of supporting point

$$M_{supportin} = 0.290 \times 12.35 \times 3.250^{2} = 37.83 \text{ kN·m/m}$$

Bending moement of span

$$M_{diamete} = 0.041 \times 12.35 \times 3.250^{2} = 5.35 \text{ kN·m/m}$$

Shear force

- $x = 0$

$$S = \frac{12.35 \times 3.250 \times 3.250}{3.250 + 3.250} = 20.07 \text{ kN/m}$$

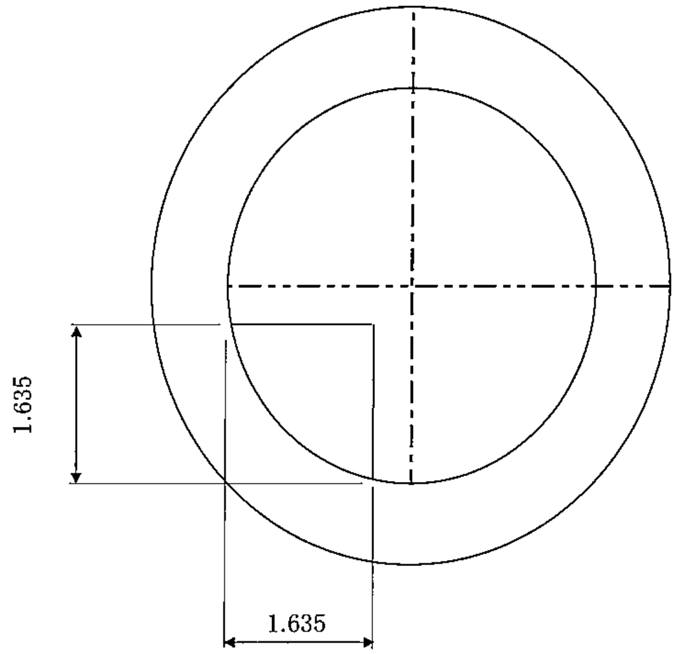
- $x = 0.150$ m (Loacated h/2 away from inside of lateral wall)

$$S = \frac{2 \times 20.07}{3.250} \times \left(\frac{3.250}{2} - 0.150 \right) = 18.22 \text{ kN/m}$$

c). Checking section

		X direction (direction for short span)		Y direction (direction for long span)	
		Supporting point	Span	Supporting point	Span
M	kN•m	37.83	5.35	37.83	5.35
N	kN	0.00	0.00	0.00	0.00
S	kN	19.14	0.00	18.22	0.00
b	mm	1000	1000	1000	1000
h	mm	300	300	300	300
d’	mm	100	100	100	100
d	mm	200	200	200	200
As	cm^2	D 22 @ 250	D 13 @ 250	D 22 @ 250	D 13 @ 250
		15.484	5.068	15.484	5.068
p		0.00774	0.00253	0.00774	0.00253
k		0.380	0.240	0.380	0.240
j		0.873	0.920	0.873	0.920
σ c	N/mm^2	5.7 < 8.0	1.2 < 8.0	5.7 < 8.0	1.2 < 8.0
σ s	N/mm^2	139.9 < 180	57.4 < 180	139.9 < 180	57.4 < 180
τ	N/mm^2	0.10 < 0.44	0.00 < 0.31	0.09 < 0.44	0.00 < 0.31
τ_{al}	N/mm^2	0.23	0.23	0.23	0.23
C_e		1.400	1.400	1.400	1.400
C_{pt}		1.365	0.953	1.365	0.953
C_N		1.000	1.000	1.000	1.000
n		15	15	15	15

2) Calculation for from BM2F to BM6F



a). Calculatioon for load

Self weight of medium slab

w_s = 24.5 × 0.300 = 7.35 kN/m^2

Sidewalk live load

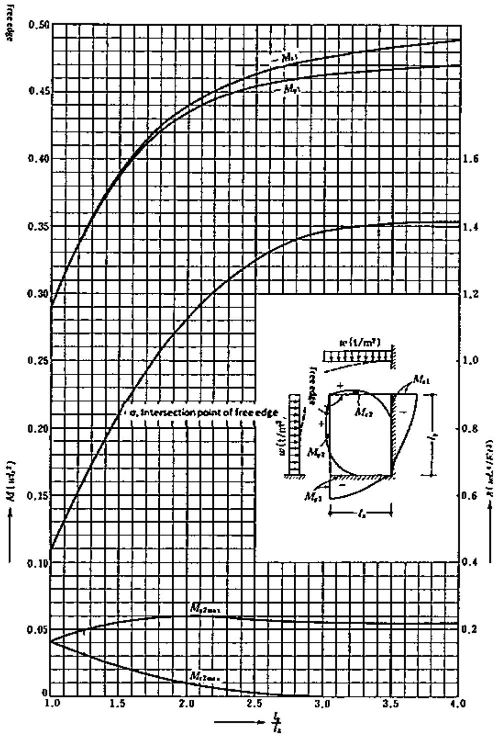
q = 5.00 kN/m^2

---

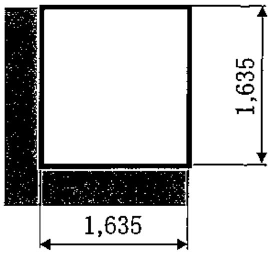
p = 12.35 kN/m^2

b). Calculation for stress

Plate of fixed 2 sides with making axis of member length is considered.



Attached diagram 10.5 stress diagram of slab with two free next edges in uniform load and deformation σ1)(ν=0) of intersection point of free edge



$$\frac{L_y}{L_x} = \frac{1.635}{1.635} = 1.00$$

From the left diagram

$-M_{x1} = 0.290$

$M_{x2max} = 0.041$

$-M_{yI} = 0.290$

$M_{y2max} = 0.041$

X direction (Direction for short span)

Bending moment of supporting point

$M_{supporti} = 0.290 \times 12.35 \times 1.635^{2} = 9.57$ kN·m/m

Bending moement of span

$M_{diameter} = 0.041 \times 12.35 \times 1.635^{2} = 1.35$ kN·m/m

Shear force

- x = 0

$S = \frac{12.35 \times 1.635 \times 1.635}{1.635 + 1.635} = 10.10$ kN/m

- x = 0.150 m (located h/2 away from inside of lateral wall)

$S = \frac{1 \times 10.10}{1.635} \times \left(\frac{1.635}{1} - 0.150 \right) = 9.17$ kN/m

Y direction (direction for long span)

Bending moment of supporting point

$M_{supporti} = 0.290 \times 12.35 \times 1.635^{2} = 9.57$ kN·m/m

Bending moement of span

$M_{diamete} = 0.041 \times 12.35 \times 1.635^{2} = 1.35$ kN·m/m

Shear force

- x = 0

$S = \frac{12.35 \times 1.635 \times 1.635}{1.635 + 1.635} = 10.10$ kN/m

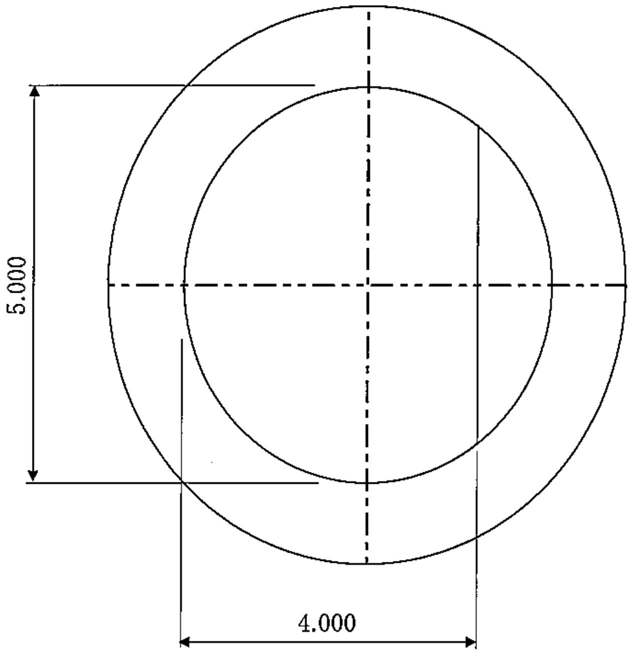
- x = 0.150 m (Loacated h/2 away from inside of lateral wall)

$S = \frac{2 \times 10.10}{1.635} \times \left(\frac{1.635}{2} - 0.150 \right) = 8.24$ kN/m

c). Checking section

		X direction (direction for short span)		Y direction (direction for long span)	
		Supporting point	Span	Supporting point	Span
M	kN・m	9.57	1.35	9.57	1.35
N	kN	0.00	0.00	0.00	0.00
S	kN	9.17	0.00	8.24	0.00
b	mm	1000	1000	1000	1000
h	mm	300	300	300	300
d'	mm	100	100	100	100
d	mm	200	200	200	200
As	cm^2	D 13 @ 250	D 13 @ 250	D 13 @ 250	D 13 @ 250
		5.068	5.068	5.068	5.068
p		0.00253	0.00253	0.00253	0.00253
k		0.240	0.240	0.240	0.240
j		0.920	0.920	0.920	0.920
σc	N/mm^2	2.2 < 8.0	0.3 < 8.0	2.2 < 8.0	0.3 < 8.0
σs	N/mm^2	102.7 < 180	14.5 < 180	102.7 < 180	14.5 < 180
τ	N/mm^2	0.05 < 0.31	0.00 < 0.31	0.04 < 0.31	0.00 < 0.31
τ_{al}	N/mm^2	0.23	0.23	0.23	0.23
C_e		1.400	1.400	1.400	1.400
C_{pt}		0.953	0.953	0.953	0.953
C_N		1.000	1.000	1.000	1.000
n		15	15	15	15

3) Calculation for B6F



a). Calculation for load

Self weight of medium slab

w_s = 24.5 × 0.300 = 7.35 kN/m^2

Sidewalk live load

q = 5.00 kN/m^2

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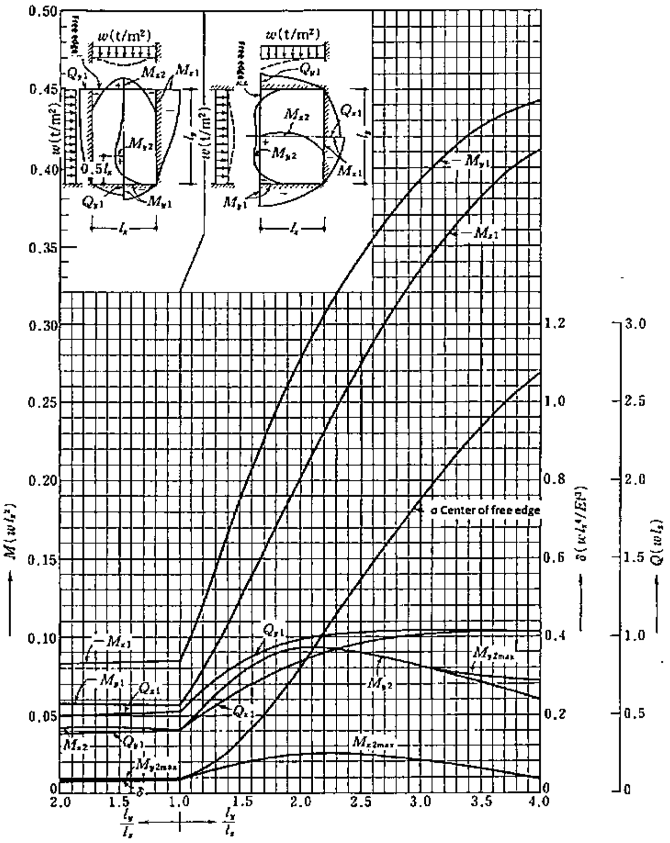
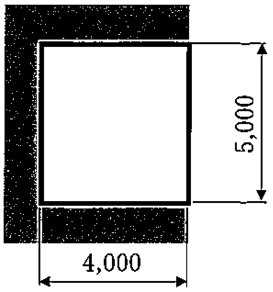
p = 12.35 kN/m^2

b). Calculation for stress

Plate of fixed 3 sides with making axis of member length is considered.





$$\frac{L_y}{L_x} = \frac{5.000}{4.000} = 1.25$$

From the left diagram

$-M_{x1} = 0.087$

$M_{x2} = 0.015$

$Q_{x1} = 0.22$

$-M_{y1} = 0.131$

$M_{y2max} = 0.059$

$Q_{yl} = 0.28$

Attached diagram 10.3 stress diagram of slab with three edge and 1 free edge and deformation σ 1(ν =0) of center of free edge in uniform load.

X direction (Direction for short span)

Bending moment of supporting point

$M_{supporti} = 0.087 \times 12.35 \times 4.000^2 = 17.19$ kN·m/m

Bending moement of span

$M_{diamete} = 0.015 \times 12.35 \times 4.000^2 = 2.96$ kN·m/m

Shear force

- $x = 0$

$S = 0.22 \times 12.35 \times 4.000 = 10.87$ kN/m

- $x = 0.150$ m (located h/2 away from inside of lateral wall)

$$S = \frac{1 \times 10.87}{4.000} \times \left(\frac{4.000}{1} - 0.150\right) = 10.46 \text{ kN/m}$$

Y direction (direction for long span)

Bending moment of supporting point

$M_{supporti} = 0.131 \times 12.35 \times 4.000^2 = 25.89$ kN·m/m

Bending moement of span

$M_{diamete} = 0.059 \times 12.35 \times 4.000^2 = 11.66$ kN·m/m

Shear force

- $x = 0$

$S = 0.28 \times 12.35 \times 4.000 = 13.83$ kN/m

- $x = 0.150$ m (located h/2 away from inside of lateral wall)

$$S = \frac{2 \times 13.83}{5.000} \times \left(\frac{5.000}{2} - 0.150\right) = 13.00 \text{ kN/m}$$

c). Checking section

		X direction (direction for short span)		Y direction (direction for long span)	
		Supporting point	Span	Supporting point	Span
M	kN•m	17.19	2.96	25.89	11.66
N	kN	0.00	0.00	0.00	0.00
S	kN	10.46	0.00	13.00	0.00
b	mm	1000	1000	1000	1000
h	mm	300	300	300	300
d'	mm	100	100	100	100
d	mm	200	200	200	200
As	cm^2	D 16 @ 250	D 13 @ 250	D 19 @ ##	D 13 @ 250
		5.617	3.584	8.103	3.584
p		0.00281	0.00179	0.00405	0.00179
k		0.251	0.207	0.293	0.207
j		0.916	0.931	0.902	0.931
σc	N/mm^2	3.7 < 8.0	0.8 < 8.0	4.9 < 8.0	3.0 < 8.0
σs	N/mm^2	167.0 < 180	44.4 < 180	177.0 < 180	174.7 < 180
τ	N/mm^2	0.05 < 0.32	0.00 < 0.28	0.07 < 0.36	0.00 < 0.28
τ_{a1}	N/mm^2	0.23	0.23	0.23	0.23
C_e		1.400	1.400	1.400	1.400
C_{pt}		0.981	0.858	1.105	0.858
C_N		1.000	1.000	1.000	1.000
n		15	15	15	15

※ The amount of reinforcing bars adopted a decreased value considering skew angle, 45° on arrangement of bar towards structure.

5-6. Design of stairs

a). Calculation for load

Self weight of slab

$w_s = 24.5 \times 0.300 \qquad = \quad 7.35 \text{ kN/m}^2$

Load on part of step

$$w_s = 24.5 \times \frac{1}{2} \times 0.200 \qquad = \quad 2.45 \text{ kN/m}^2$$

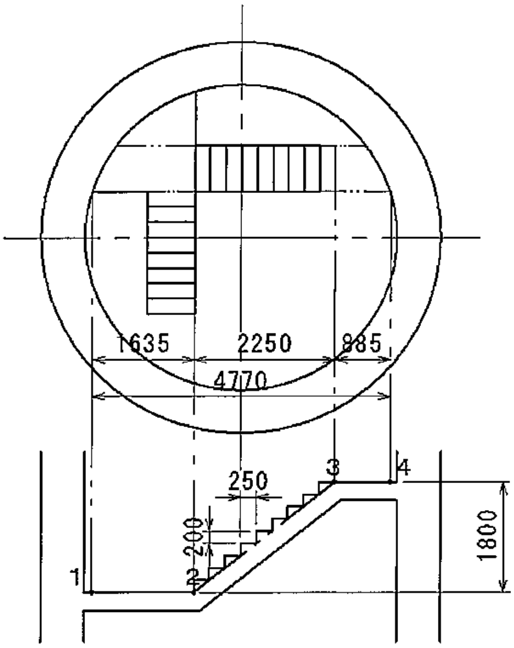
Sidewalk live load

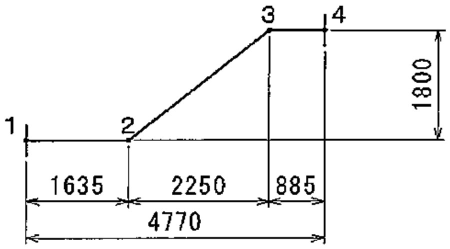
$q \qquad = \quad 5.00 \text{ kN/m}^2$

$p = 14.80 \text{ kN/m}^2$

b). Calculation for stress

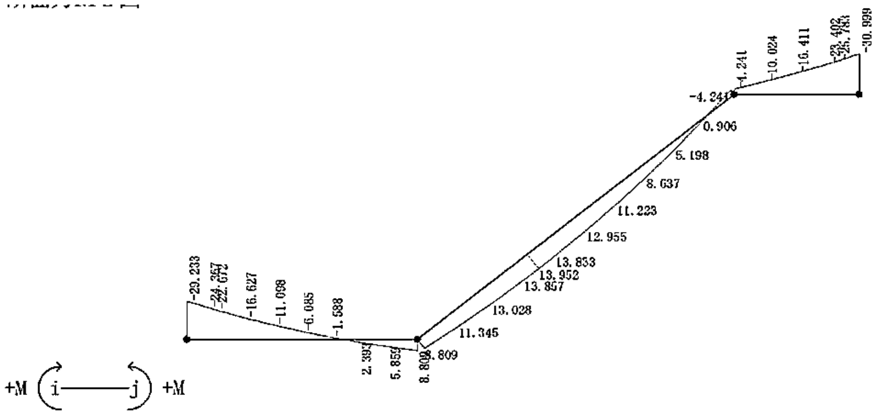
The model is set as beam with both ends built-in as shown in below diagram.



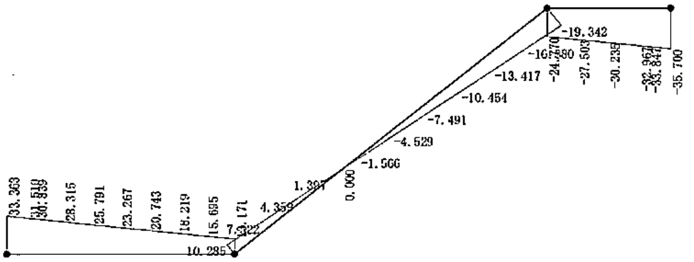


Stress diagram

Sy diagram for sectional force



Sy diagram for sectional force



+S ↑ i———j ↓ +S

c). Checking section

- Checking to bending

		Stairs			
		Supporting point	Span	Supporting point	
M	kN·m	29.23	13.95	31.00	
N	kN	0.00	0.00	0.00	
S	kN	0.00	0.00	0.00	
b	mm	1000	1000	1000	
h	mm	300	300	300	
d'	mm	#REF!	#REF!	#REF!	
d	mm	#REF!	#REF!	#REF!	
As	cm^2	D 19 @ 250	D 13 @ 250	D 19 @ 250	
		11.460	5.068	11.460	
p		#REF!	#REF!	#REF!	
k		#REF!	#REF!	#REF!	
j		#REF!	#REF!	#REF!	
σc	N/mm^2	##### ## 8.0	##### ## 8.0	##### ## 8.0	
σs	N/mm^2	##### ## 180	##### ## 180	##### ## 180	
τ	N/mm^2	##### ## #####	##### ## #####	##### ## #####	
τ_{a1}	N/mm^2	0.23	0.23	0.23	
C_e		#REF!	#REF!	#REF!	
C_{pt}		#REF!	#REF!	#REF!	
C_N		1.000	1.000	1.000	
n		15	15	15	

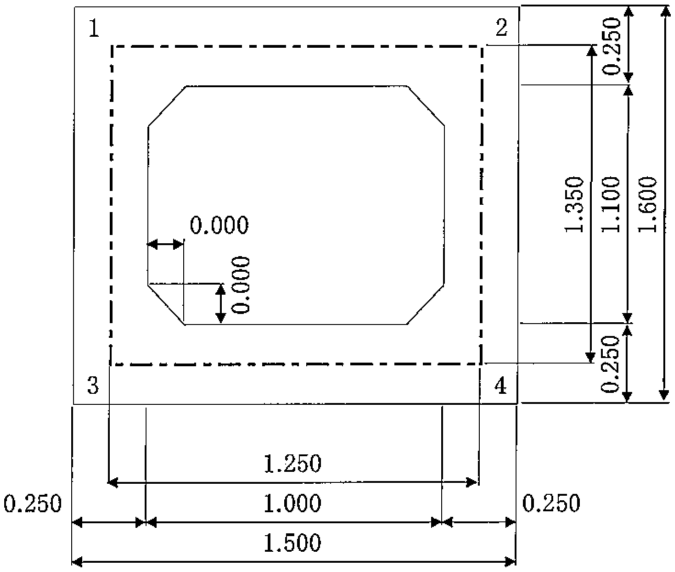
- Checking to shear

		Stairs									
		Supporting point1				Supporting point4					
M	kN・m	24.37				25.78					
N	kN	0.00				0.00					
S	kN	31.51				33.85					
b	mm	1000				1000					
h	mm	300				300					
d'	mm	100				100					
d	mm	200				200					
As	cm^2	D	19	@	250	D	19	@	250		
		11.460				11.460					
p		0.00573				0.00573					
k		0.337				0.337					
j		0.888				0.888					
σc	N/mm^2	4.1		<	8.0	4.3		<	8.0		
σs	N/mm^2	119.8		<	180	126.7		<	180		
τ	N/mm^2	0.16		<	0.40	0.17		<	0.40		
τ_{al}	N/mm^2	0.23				0.23					
C_e		1.400				1.400					
C_{pt}		1.244				1.244					
C_N		1.000				1.000					
n		15				15					

5-7. Calculation for cleaning hole

1). Skelton diagram

a). Skelton diagram



b). Secional area and second moment of area

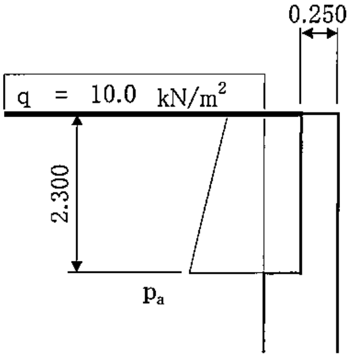
Sectional area A = 0.250 m^2/m

Moment of second orde $I = \frac{1}{12} \times 0.250^3$ = 0.001302 m^4/m

c). Point specified for calculation

Member	Point specified for calculation (m)							
	Front face of bearing	(Haunch)	Haunch	h/2	h/2	Haunch	(Haunch)	Front face of bearing
1-2	0.125	0.000	0.125	0.250	1.000	1.125	1.250	1.125
3-4	0.125	0.000	0.125	0.250	1.000	1.125	1.250	1.125
1-3	0.125	0.000	0.125	0.250	1.100	1.225	1.350	1.225
2-4	0.125	0.000	0.125	0.250	1.100	1.225	1.350	1.225

2). Calculation for load



Earth pressure

p_{a1} = 0.5 × 19.0 × 2.300 = 21.85 kN/m^2

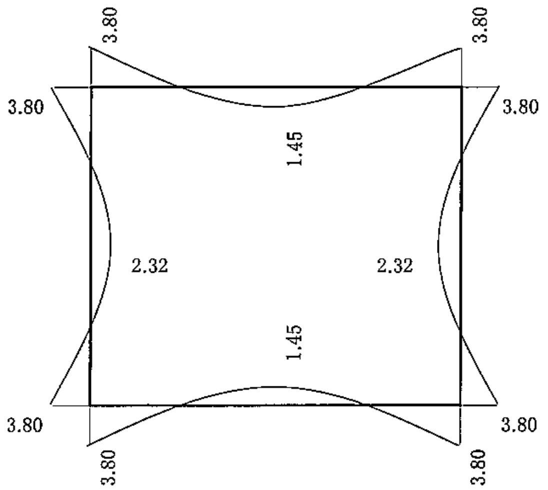
Earth pressure of live load

p_q = 0.5 × 10.0 = 5.00 kN/m^2

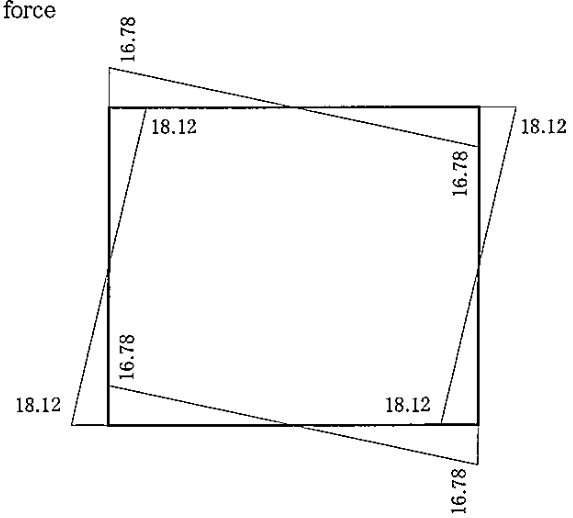
p_a = 26.85 kN/m^2

3). Calculation for load

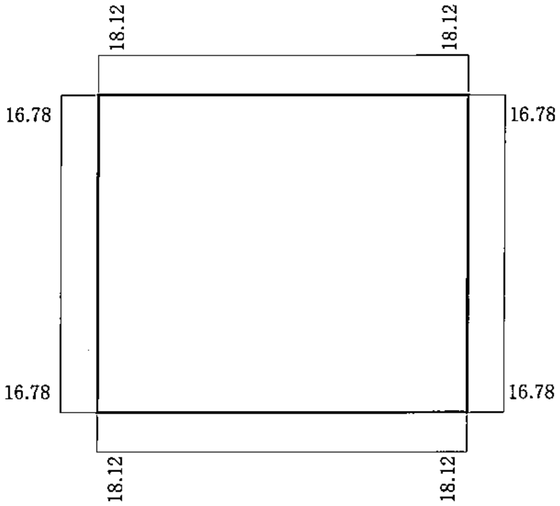
- Diagram of bending moment



- Diagram of shear force



- Diagram of axial force



4). Checkihg section

- Checkiong to bending

	1-2,3-4		1-3,2-4	
	Supporting point (Exterior surface)	Span (inner surface)	Supporting point (Exterior surface)	Span (inner surface)
M kN·m	3.80	1.45	3.80	2.32
N kN	18.12	18.12	16.78	16.78
b mm	1000	1000	1000	1000
h mm	250	250	250	250
d' mm	100	150	100	150
d mm	150	100	150	100
As cm^2	D 13 @ 250	D 13 @ 250	D 13 @ 250	D 13 @ 250
	5.068	5.068	5.068	5.068
p	0.00338	0.00507	0.00338	0.00507
k	0.272	0.321	0.272	0.321
j	0.909	0.893	0.909	0.893
σc N/mm^2	1.1 < 8.0	0.3 < 8.0	1.2 < 8.0	0.9 < 8.0
σs N/mm^2	28.2 < 160	-1.0 < 160	30.1 < 160	12.2 < 160
n	15	15	15	15

- Checking to shear force

	Long side		Shohrt side	
	Supporting point		Supporting point	
M kN·m	0.44		0.11	
N kN	18.12		16.78	
S kN	10.07		11.41	
b mm	1000		1000	
h mm	250		250	
d' mm	100		100	
d mm	150		150	
As cm^2	D 13 @ 250		D 13 @ 250	
	5.068		5.068	
p	0.00338		0.00338	
τ N/mm^2	0.07 < 0.67		0.08 < 0.67	
τ_{al} N/mm^2	0.23		0.23	
C_e	1.400		1.400	
C_{pt}	1.038		1.038	
C_N	2.000		2.000	

6. Checking section in earthquake (level 1)

6-1. Classification of ground on earthquake resistant design

Classification of ground on earthquake resistant design is classified by following table based on characteristic valu,TG calculated from the following formula in principle.

If ground level is same as faoundation bed level, I type ground is adopted.

$$T_G = 4 \sum_{i=1}^{n} \frac{Hi}{Vsi}$$

To this

T_G : Characteristic value of ground, (s)

Hi : Layer thickness of number i(m)

Vsi : Average shear wave velecity of leyer of number i(m/s). However, if there is no actual measurements, the value can be calculated from the following formula.

In case of cohesive soil $Vsi = 100Ni^{1/3}$ $(1 \leqq Ni \leqq 25)$

In case of sandy soil $Vsi = 80Ni^{1/3}$ $(1 \leqq Ni \leqq 50)$

Ni : Average N value of leyer of number I from standard penetation test

i : Number of layer of I number from ground level when classifying the layers into n layers from ground level to foundation bed.
Foundation bed level is considered to be upper surface of strata in case of cohesive soil having N value with 25 and over and in case of sandy soil having N value with 50 and more, or upper surface of starata of shear wave velocity with around 300m/s and over.

Classification of ground on earthquake resistant design

Classification of ground	Characteristic value of ground, T_G(s)
Class I	$T_G < 0.2$
Class II	$0.2 \leqq T_G < 0.6$
Class III	$0.6 \leqq T_G$

Judgement of classification of ground

Classification	Hi(m)	Name of layer		Ni	Vsi(m/s)	Hi/Vsi(s)
1	4.000	Ac1	Cohesive soil	2.0	126.0	0.03175
2	9.000	Ac2	Cohesive soil	1.0	100.0	0.09000
3	2.000	Ac3	Cohesive soil	9.0	208.0	0.00961
Σ	15.000	-		-	-	0.13136

$T_G = 4 \times 0.13136 = 0.52545$ s

$0.2 \leqq T_G = 0.525$ s < 0.6

∴ Judged as ground, class II

6-2. Response displacement of ground

In seismic calculation method by response displacement method, displacement amplitude of horizontal direction of ground in depth, z from ground level is calculated from the following formula.

$$U_h(z) = \frac{2}{\pi^2} \cdot Sv \cdot Ts \cdot \cos \frac{\pi z}{2 \cdot H}$$

To this

Uh(z) : Horizontal displacement amplitude in depth, z(m) from ground level (m)

Sv : Designed response velocity (m/s)

Ts : Natural period of subsurface ground(s)

$Ts = 1.25 \cdot T_G$

z : Depth of optional point (m)

H : Thickness of subsurface ground (m)

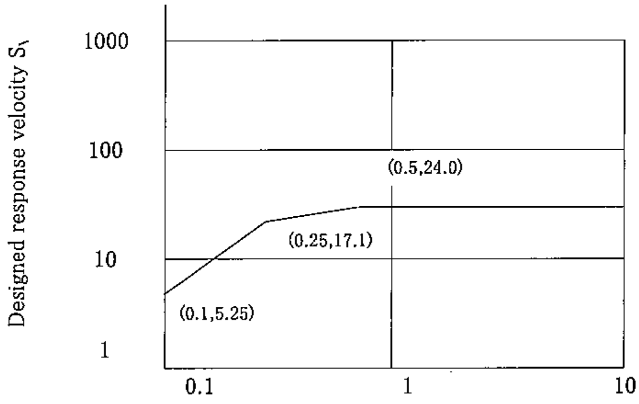
1). Natural period

$Ts = 1.25 \times 0.525$

$= 0.657$ s

$T_G = 0.525$ s

2). Designed response velocity



Natural period of subsurface of ground Ts (sec)

A Area (Area is regarded as maximum)

$0.50 < Ts = 0.657$ s

$Sv = 0.240$ (m/s)

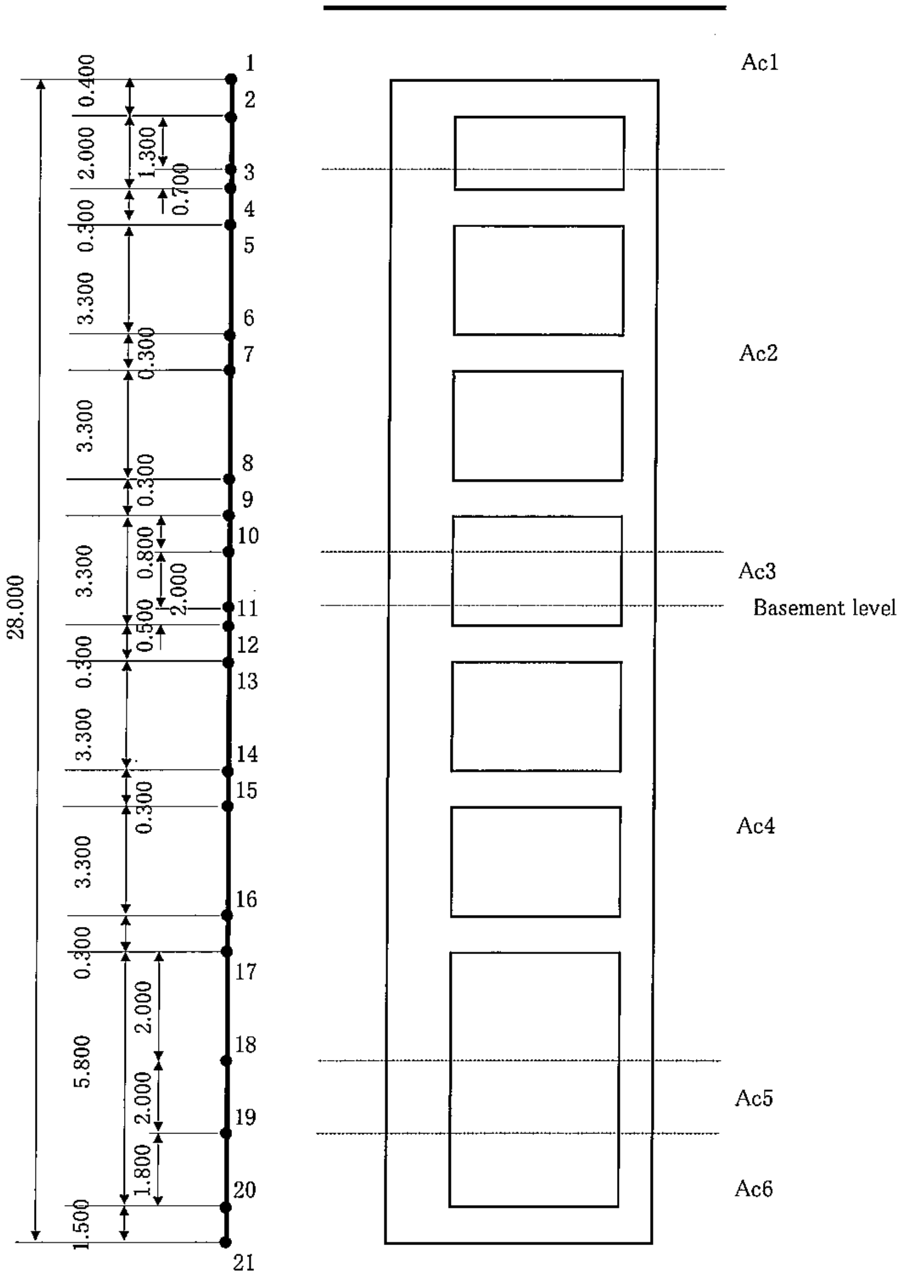
3). Calculation for displacement amplitude of ground

Panel point Number	Elevation (m)	Length of member (m)	Depth, z (m)	Uh(z) (m)	Notes	Sign
–	0.730	0.000	0.000	0.031944	Ground level	Ac1
1	−1.570	2.300	2.300	0.031022	Upside of upper slab	
2	−1.970	0.400	2.700	0.030675	Soffit of upper slab	
3	−3.270	1.300	4.000	0.029182	Change point of stratum	
–	−3.270	0.000	4.000	0.029182	–	Ac2
4	−3.970	0.700	4.700	0.028152	Upside of medium slab	
5	−4.270	0.300	5.000	0.027664	Soffit of medium slab	
6	−7.570	3.300	8.300	0.020618	Upside of medium slab	
7	−7.870	0.300	8.600	0.019842	Soffit of medium slab	
8	−11.170	3.300	11.900	0.010189	Upside of medium slab	
9	−11.470	0.300	12.200	0.009233	Soffit of medium slab	
10	−12.270	0.800	13.000	0.006641	Change point of stratum	
–	−12.270	0.000	13.000	0.006641	–	Ac3
11	−14.270	2.000	15.000	0.000000	Foundation bed level	
–	−14.270	0.000	15.000	0.000000	–	Ac4
12	−14.770	0.500	15.500	0.000000	Upside of medium slab	
13	−15.070	0.300	15.800	0.000000	Soffit of medium slab	
14	−18.370	3.300	19.100	0.000000	Upside of medium slab	
15	−18.670	0.300	19.400	0.000000	Soffit of medium slab	
16	−21.970	3.300	22.700	0.000000	Upside of medium slab	
17	−22.270	0.300	23.000	0.000000	Soffit of medium slab	
18	−24.270	2.000	25.000	0.000000	Change point of stratum	
–	−24.270	0.000	25.000	0.000000	–	Ac5
19	−26.270	2.000	27.000	0.000000	Change point of stratum	
–	−26.270	0.000	27.000	0.000000	–	Ac6
20	−28.070	1.800	28.800	0.000000	Upside of bottom slab	
21	−29.570	1.500	30.300	0.000000	Undersurface of bottom slab	

6-3. Calculation for vertical direction

1). Skelton diagram

a). Skelton diagram



b). Section modulus

Section	Outside dimension	Inside dimension	A	I
	D(m)	d(m)	(m^2)	(m^4)
1	6.400	0.000	32.170	82.355
2	6.400	5.000	12.535	51.675
3	6.400	0.000	32.170	82.355
4	6.400	5.000	12.535	51.675
5	6.400	0.000	32.170	82.355
6	6.400	5.000	12.535	51.675
7	6.400	0.000	32.170	82.355
8	6.400	5.000	12.535	51.675
9	6.400	0.000	32.170	82.355
10	6.400	5.000	12.535	51.675
11	6.400	0.000	32.170	82.355
12	6.400	5.000	12.535	51.675
13	6.400	0.000	32.170	82.355
14	6.400	5.000	12.535	51.675
15	6.400	0.000	32.170	82.355

c). Coefficient of ground reaction

i). Coefficient of horizontal ground reaction in lateral face

Coefficient of horizontal ground reaction is calculated from the following formula.

$$k_H = k_{H0} \left[\frac{B_h}{0.3} \right]^{-3/4}$$

To the above formula

k_H : Coefficient of horizontal ground reactionkN/m^3)

k_{H0} : Coefficient of horizontal ground reaction corresponding to the value of plate bearing test by rigid circular plate with diameter, 0.3m.(kN/m3)

$$k_{H0} = \frac{1}{0.3} \cdot \alpha \cdot E_0$$

α : Coefficient for estimation of coefficient of ground reaction

E_0 : Deformation modulus of ground(kN/m^2)

B_h : Converted loaded width of foundation(m)

$$B_h = \sqrt{Ah}$$

A_h : Horizontal loaded width(m)

The following table shows the result of calculation.

Member Number	Stratum Sign	Section Number	$\alpha \cdot E_0$ (kN/m^2)	k_{H0} (kN/m^3)	k_H (kN/m^3)	Loaded width B (m)	K_H=$k_H \cdot$B (kN/m^2)
1	Ac1	1	5,600	18,667	1,346	5.120	6,889
2	〃	2	5,600	18,667	1,346	5.120	6,889
3	Ac2	〃	2,800	9,333	673	5.120	3,445
4	〃	3	2,800	9,333	673	5.120	3,445
5	〃	4	2,800	9,333	673	5.120	3,445
6	〃	5	2,800	9,333	673	5.120	3,445
7	〃	6	2,800	9,333	673	5.120	3,445
8	〃	7	2,800	9,333	673	5.120	3,445
9	〃	8	2,800	9,333	673	5.120	3,445
10	Ac3	〃	25,200	84,000	6,055	5.120	31,002
11	Ac4	〃	168,000	560,000	40,367	5.120	206,680
12	〃	9	168,000	560,000	40,367	5.120	206,680
13	〃	10	168,000	560,000	40,367	5.120	206,680
14	〃	11	168,000	560,000	40,367	5.120	206,680
15	〃	12	168,000	560,000	40,367	5.120	206,680
16	〃	13	168,000	560,000	40,367	5.120	206,680
17	〃	14	168,000	560,000	40,367	5.120	206,680
18	Ac5	〃	142,800	476,000	34,312	5.120	175,678
19	Ac6	〃	86,800	289,333	20,856	5.120	106,785
20	〃	15	86,800	289,333	20,856	5.120	106,785

$B_h = \sqrt{143.360}$

$= 11.973$ m > 10.0 m

$A_h = 0.8 \times 6.400 \times 28.000$

$= 143.360$ m^2

$B_h = 10.000$ m Bh is calculated by the above formula.

ii). Coefficient of vertical ground reaction in underside.

Coefficient of vertical ground reaction is calculated from the following formula.

$$k_V = k_{V0} \left[\frac{B_v}{0.3} \right]^{-3/4}$$

To the above formula

k_V : Coefficient of vertical groudn reaction(kN/m^3)

k_{V0} : Coefficient of vertical ground reaction corresponding to the value of plate bearing test by rigid circular plate with diamter, 0.3m (kN/m3)

$$k_{V0} = \frac{1}{0.3} \cdot \alpha \cdot E_0$$

α : Coefficient for estimation of coefficient of ground reaction

E_0 : Deformation modulus of ground (kN/m^2)

B_v : Converted loaded width of foundation (m)

$B_v = \sqrt{Av}$

A_v : Vertical loaded width(m)

$$k_V = 289{,}333 \times \left[\frac{5.672}{0.3} \right]^{-3/4}$$

$$= 31{,}911 \ kN/m^3$$

$$k_{V0} = \frac{1}{0.3} \times 86{,}800$$

$$= 289{,}333 \ kN/m^3$$

$$B_v = \sqrt{32.170}$$

$$= 5.672 \ m < 10.0 \ m$$

$$A_V = \frac{\pi}{4} \times 6.400^{2}$$

$$= 32.170 \ m^2$$

B_v = 5.672 m Bv is calculated by the above formula

$$K_V = A_v \times k_V$$

$$= 32.170 \times 31{,}911$$

$$= 1{,}026{,}587 \ kN/m$$

iii). Shear spring coefficient in bottom face.

Shear spring coefficient is calculated from the following formula.

$$k_S = \lambda \cdot k_V$$

To the above formula

k_S : Shear spring coefficient in bottom face (kN/m^3)

λ : The ratio of shear spring coefficient, kS to coefficient of vertical ground reaction, kV in bottom face=0.3

k_V : Coefficient of vertical ground reaction in bottom face (kN/m^3)

$$k_S = 0.3 \times 31{,}911$$

$$= 9{,}573 \ kN/m^3$$

$$K_s = A_v \times k_s$$

$$= 32.170 \times 9{,}573$$

$$= 307{,}976 \ kN/m$$

iv). Rotary spring coefficient in bottom face

Rotary spring coefficient is calculated from the following formula.

$K_\theta = k_V \cdot I$

To the formula

K_θ : Rotary spring coefficient (kN·m/rad)
k_V : Coefficient of vertical ground reaction in bottom face (kN/m^3)
I : Second moment of area in bottom face of manhole (m^4)

$$K_\theta = 31{,}911 \times 82.355$$
$$= 2{,}628{,}061 \text{ kN}\cdot\text{m/rad}$$

$$I = \frac{\pi}{64} \times 6.400^{4}$$
$$= 82.355 \text{ m}^4$$

2). Calculation for load

Earth pressure from response displacement method based on the result of calculation for displacement amplitude is calculated.

Panel point	Uh(z)	Uh'(z) Uh(z)-Uh($_{hB}$)	K_H	Uh'(z)·K_H	Displacement of member	Displacement distance	Ground reaction	Stratum
Number	(m)	(m)	(kN/m^2)	(kN/m)	(m)	(m)	(kN/m)	Sign
1	0.031022	0.031022	6,889	213.72	0.0047652	0.026256	180.89	Ac1
2	0.030675	0.030675	6,889	211.33	0.0046661	0.026009	179.19	〃
3	0.029182	0.029182	6,889	201.04	0.0043443	0.024838	171.12	〃
-	0.029182	0.029182	3,445	100.52	0.0043443	0.024838	85.56	Ac2
4	0.028152	0.028152	3,445	96.97	0.0041711	0.023981	82.61	〃
5	0.027664	0.027664	3,445	95.29	0.0040969	0.023567	81.18	〃
6	0.020618	0.020618	3,445	71.02	0.0032587	0.017360	59.80	〃
7	0.019842	0.019842	3,445	68.35	0.0032126	0.016629	57.28	〃
8	0.010189	0.010189	3,445	35.10	0.0024230	0.007766	26.75	〃
9	0.009233	0.009233	3,445	31.80	0.0023528	0.006880	23.70	〃
10	0.006641	0.006641	3,445	22.88	0.0021673	0.004474	15.41	〃
-	0.006641	0.006641	31,002	205.90	0.0021673	0.004474	138.71	Ac3
11	0.000000	0.000000	31,002	0.00	0.0017165	0.001717	53.22	〃
-	0.000000	0.000000	206,680	0.00	0.0017165	0.001717	354.77	Ac4
12	0.000000	0.000000	206,680	0.00	0.0016071	0.001607	332.16	〃
13	0.000000	0.000000	206,680	0.00	0.0015421	0.001542	318.71	〃
14	0.000000	0.000000	206,680	0.00	0.0008612	0.000861	177.99	〃
15	0.000000	0.000000	206,680	0.00	0.0008025	0.000802	165.85	〃
16	0.000000	0.000000	206,680	0.00	0.0001867	0.000187	38.60	〃
17	0.000000	0.000000	206,680	0.00	0.0001332	0.000133	27.53	〃
18	0.000000	0.000000	206,680	0.00	-0.0002158	0.000216	44.61	〃
-	0.000000	0.000000	175,678	0.00	-0.0002158	0.000216	37.91	Ac5
19	0.000000	0.000000	175,678	0.00	-0.0005540	0.000554	97.32	〃
-	0.000000	0.000000	106,785	0.00	-0.0005540	0.000554	59.16	Ac6
20	0.000000	0.000000	106,785	0.00	-0.0008526	0.000853	91.05	〃
21	0.000000	0.000000	106,785	0.00	-0.0010996	0.001100	117.42	〃

Self weight and so on

Vertical load applying into upper side of manhole

Earth covering

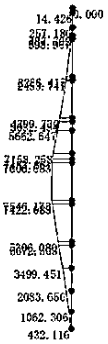
P = 19.0 × 2.300 × 32.170 = 1,405.83 kN

Empty weight

Section 1 w_1 = 24.5 × 32.170 = 788.16 kN/m
Section 2 w_2 = 24.5 × 12.535 = 307.11 kN/m
Section 3 w_3 = 24.5 × 32.170 = 788.16 kN/m
Section 4 w_4 = 24.5 × 12.535 = 307.11 kN/m
Section 5 w_5 = 24.5 × 32.170 = 788.16 kN/m
Section 6 w_6 = 24.5 × 12.535 = 307.11 kN/m
Section 7 w_7 = 24.5 × 32.170 = 788.16 kN/m
Section 8 w_8 = 24.5 × 12.535 = 307.11 kN/m
Section 9 w_9 = 24.5 × 32.170 = 788.16 kN/m
Section 10 w_{10} = 24.5 × 12.535 = 307.11 kN/m
Section 11 w_{11} = 24.5 × 32.170 = 788.16 kN/m
Section 12 w_{12} = 24.5 × 12.535 = 307.11 kN/m
Section 13 w_{13} = 24.5 × 32.170 = 788.16 kN/m
Section 14 w_{14} = 24.5 × 12.535 = 307.11 kN/m
Section 15 w_{15} = 24.5 × 32.170 = 788.16 kN/m

3). Calculation for stress

Diagram of sectional force, Mz



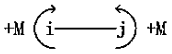
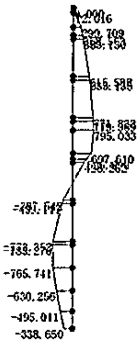
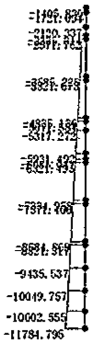


Diagram of sectional force, Sy



+S ↑ i———j ↓ +S

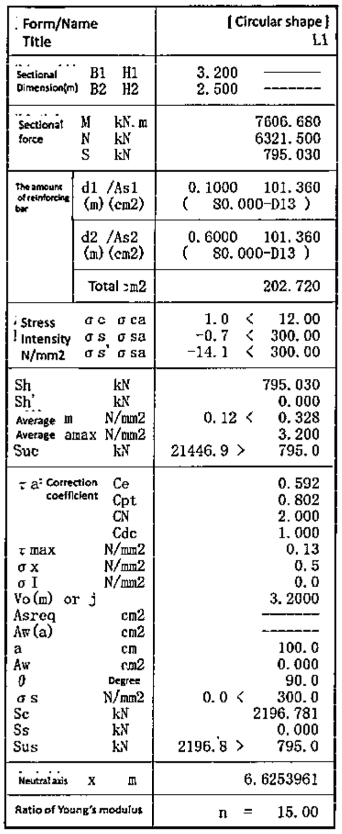
Diagram of sectional force, Nx



+N ← i———j → +N

4). Calculation for section

Calculation for stress intensity



Form/Name Title			[Circular shape] L1
Sectional Dimension(m)	B1 H1		3.200 ———
	B2 H2		2.500 -------
Sectional force	M	kN.m	7606.680
	N	kN	6321.500
	S	kN	795.030
The amount of reinforcing bar	d1 /As1 (m) (cm2)		0.1000 101.360 (80.000-D13)
	d2 /As2 (m) (cm2)		0.6000 101.360 (80.000-D13)
	Total cm2		202.720
Stress Intensity N/mm2	σc	σca	1.0 < 12.00
	σs	σsa	-0.7 < 300.00
	σs'	σsa	-14.1 < 300.00
Sh		kN	795.030
Sh'		kN	0.000
Average m		N/mm2	0.12 < 0.328
Average amax		N/mm2	3.200
Suc		kN	21446.9 > 795.0
τa: Correction coefficient	Ce		0.592
	Cpt		0.802
	CN		2.000
	Cdc		1.000
τmax		N/mm2	0.13
σx		N/mm2	0.5
σI		N/mm2	0.0
Vo(m) or j			3.2000
Asreq		cm2	———
Aw(a)		cm2	-------
a		cm	100.0
Aw		cm2	0.000
θ		Degree	90.0
σs		N/mm2	0.0 < 300.0
Sc		kN	2196.781
Ss		kN	0.000
Sus		kN	2196.8 > 795.0
Neutral axis	X	m	6.6253961
Ratio of Young's modulus			n = 15.00

X, Vo shows distance from extreme compression fiber

6-4. Calculation for horizontal direction

Calculation of load in order to calulate horizontal direction.

Panel point Number	Elevation	Depth	Earth pressure at rest	Water pressure	Earth pressure by displacement difference		Total	Location
	(m)	(m)	kN/m²	kN/m²	kN	kN/m²	kN/m²	
-	0.730	0.000	0.00	0.00	-	-	-	Ground level
1	-1.570	2.300	18.40	0.00	180.89	35.33	53.73	Upper side of upper slab
-	-1.760	2.490	19.92	0.00	-	-	-	Groundwater level
2	-1.970	2.700	20.66	2.10	179.19	35.00	57.75	Undersurface of upper slab
3	-3.270	4.000	25.21	15.10	171.12	33.42	73.73	Change point of stratum
-	-3.270	4.000	25.21	15.10	85.56	16.71	57.02	-
4	-3.970	4.700	27.66	22.10	82.61	16.13	65.89	Upper side of medium slab
5	-4.270	5.000	28.71	25.10	81.18	15.86	69.66	Undersurface of mnnedium slab
6	-7.570	8.300	40.26	58.10	59.80	11.68	110.03	Upper side of medium slab
7	-7.870	8.600	41.31	61.10	57.28	11.19	113.59	Undersurface of medium slab
8	-11.170	11.900	52.86	94.10	26.75	5.22	152.18	Upper side of medium slab
9	-11.470	12.200	53.91	97.10	23.70	4.63	155.63	Undersurface of medium
10	-12.270	13.000	56.71	105.10	15.41	3.01	164.82	Change point of stratum
-	-12.270	13.000	56.71	105.10	138.71	27.09	188.90	-
11	-14.270	15.000	63.71	125.10	53.22	10.39	199.20	Foundation bed level
-	-14.270	15.000	63.71	125.10	354.77	*69.29*	258.10	-
12	-14.770	15.500	63.71	130.10	332.16	64.87	258.68	Upper side of medium slab
13	-15.070	15.800	63.71	133.10	318.71	62.25	259.05	Undersurface of medium slab
14	-18.370	19.100	63.71	166.10	177.99	34.76	264.57	Upper side of medium slab
15	-18.670	19.400	63.71	169.10	165.85	32.39	265.20	Undersurace of medium slab
16	-21.970	22.700	63.71	202.10	38.60	7.54	273.34	Upper side of medium slab
17	-22.270	23.000	63.71	205.10	27.53	5.38	274.18	Undersurface of medium slab
18	-24.270	25.000	63.71	225.10	44.61	8.71	297.52	Change point of stratum
-	-24.270	25.000	63.71	225.10	37.91	7.41	296.21	-
19	-26.270	27.000	63.71	245.10	97.32	19.01	327.81	Change point of stratum
-	-26.270	27.000	63.71	245.10	59.16	11.55	320.36	-
20	-28.070	28.800	63.71	263.10	91.05	17.78	344.59	Upper side of bottom slab
21	-29.570	30.300	63.71	278.10	117.42	22.93	364.74	Undersurface of bottom slab

※ Location with earth pressure by displacement distance maximum is set as target.

Calculation in panel point 11

1). Calculation for load

Earth pressure at rest

$p_a = 63.71\ \mathrm{kN/m^2}$

Water pressure

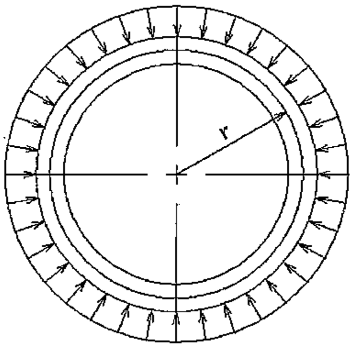
$p_w = 125.10\ \mathrm{kN/m^2}$

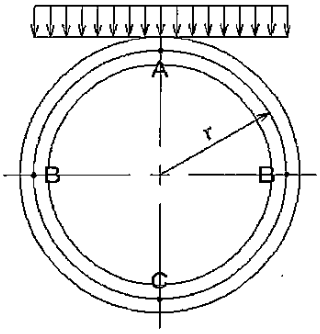
$p = 188.81\ \mathrm{kN/m^2}$

Response displacement earth pressure

$p_{ae} = 69.29\ \mathrm{kN/m^2}$

2). Calculation for stress





Inner diameter	Thickness of member	Diameter of member axis	Radius of member axis
m	m	m	m
5.000	0.700	5.700	2.850

Calculation for bending moment

$M_A = 0.163 \times 69.29 \times 2.850^{2} = 91.74\ \mathrm{kN \cdot m/m}$

$M_B = -\ 0.125 \times 69.29 \times 2.850^{2} = -70.35\ \mathrm{kN \cdot m/m}$

$M_C = 0.087 \times 69.29 \times 2.850^{2} = 48.97\ \mathrm{kN \cdot m/m}$

Calculation for axial force

$N_A = (\ 0.212 \times 69.29 + 188.81\ ) \times 2.850 = 579.96\ \mathrm{kN/m}$

$N_B = (\ 1.000 \times 69.29 + 188.81\ ) \times 2.850 = 735.57\ \mathrm{kN/m}$

$N_C = (-\ 0.212 \times 69.29 + 188.81\ ) \times 2.850 = 496.23\ \mathrm{kN/m}$

There is no occurrence of shear force.

3). Checking section

		A Point				B point				C point				
		Inner surface				Exterior surface				Inner surface				
M	kN·m	91.74				70.35				48.97				
N	kN	579.96				735.57				496.23				
b	mm	1000				1000				1000				
h	mm	700				700				700				
d	mm	600				600				600				
d'	mm	100				100				100				
As	cm^2	D	22	@	250	D	22	@	250	D	22	@	250	
		15.484				15.484				15.484				
As'	cm^2	D		@		D		@		D		@		
		0.000				0.000				0.000				
p		0.00258				0.00258				0.00258				
k		0.242				0.242				0.242				
j		0.919				0.919				0.919				
σc	N/mm^2	2.0		<	8.0	1.8		<	8.0	1.3		<	8.0	
σs	N/mm^2	1.1		<	300	-24.0		<	300	-16.4		<	300	
n		15				15				15				