C-10 Drainage, Erosion and Sediment Control Plan

Aim and Objective

This **C-10 Drainage, Erosion and Sediment Control Plan** (DESCP) sets out planning and control measures, monitoring and reporting to minimise erosion and prevent off-site sediment transport during project construction. The plan sets out:

- The principles of erosion and sediment control;
- Practical measures to minimise erosion and retain sediment on site, thereby helping to protect water quality, terrestrial and aquatic ecosystems and ecosystem services;
- Controls and monitoring to manage site drainage.

Summary of Impacts and Risks

Construction of the Project will result in extensive land disturbance, including removal of vegetation, reshaping topography, blasting and tunnelling as well as stockpiling of topsoil, spoil and aggregate. Such activities can generate sediment and make the remaining soil vulnerable to erosion, resulting in sedimentation and airborne dust. Measures to address erosion and sediment control shall prioritise preventative rather than treatment measures. To develop effective erosion controls it is necessary to obtain information on the erosion potential of the site where soil disturbance is planned. This includes soil type and structure, vegetative cover, topography, climate (rainfall and wind), and the nature of land-clearing.

C-10 DESCP covers erosion and sediment control, including from roadside drainage. It excludes the design of cross-drainage beneath roads which is covered in **C-8 Watercourse Crossing Management Plan. P-2 Biodiversity Management Plan** presents mitigation of impacts to critical and natural habitat.

Mitigation and Management Actions

#	Issue or Risk	Action	Timing / Frequency	Responsibility
C-10-1.	Design review	• The overall Project design shall be reviewed to ensure to ensure long-term stability of final project landforms. • Erosion and sediment control will be designed and installed based on site conditions, including but not limited to treatment area, soil type, slope, and sensitivity of the receiving environment. A combination of structural and non-structural approaches will be used to minimize sediment mobilization: – **Non-structural approaches:** minimising exposed areas and site clearance; timing and staging of works; limiting work on steep slopes (>15°) where possible; storage of materials in stable stockpiles; setback distances from watercourses; rapidly stabilising and revegetating exposed soil. – **Structural approaches:** sediment retention ponds; primary and secondary devices; decanting earth bunds; sumps; silt fences; silt socks; wheel wash areas. – **Water management controls:** Separation of 'clean' and 'dirty' water through diversion channels and bunds; contour drains (cut-offs); check dams / sumps; pipe-drop structures and flumes; concreting / surface protection; surface roughening; – **Coagulant and flocculant treatments:** to increase settlement of fines prior to discharge. • Design specifications for erosion and sediment controls shall be included in the contractor's detailed design submission. These shall be submitted by the contractor for engineering review and clearance by the OE prior to commencement of land disturbing activities. At minimum, these design specifications will include: – Sediment basin dimensions and capacity based on **Annex C-10-I Calculation Sheet for Sediment Basin Surface Area** – Culvert dimensions and capacity – Diversion drain cross-section and grade – Delineation of disturbed areas – Details of implementation activities and design details for the measures (e.g. timing/sequencing of works, sediment basin volume calculations, temporary measures, stabilization/revegetation works). • Erosion and sediment control devices shall be located away from community water sources (refer **C-7 Water Supply Replacement Plan**).	Pre-construction	HEC Design Team THL/OE (review)
C-10-2.	Site preparation	• Site preparation, including installation of sediment control devices will proceed in accordance with **C-3 Forest Clearance Plan** and **C-9 Spoil and Topsoil Management Plan**. Earthworks will be strictly undertaken within marked areas, avoiding vegetation and soil disturbance beyond these designated sites.	Vegetation clearance and earthworks	HEC Construction Manager
C-10-3.	Erosion control and drainage	• Adequate permanent and temporary drainage shall be installed to control 'clean' and 'dirty' water, including diverting runoff around sites and controlling it through and off site. Specific measures shall include: – Diversion of clean water around sites via cut-off drains (earth or vegetated spoon drains or diversion banks) installed above disturbed areas. – Installation of temporary spoon drains and diversion bunds across large construction areas subject to earthworks or ongoing disturbance, to break the site into sub-drainage areas, reducing overland runoff distances/volumes/velocities. – Installation of drains to as low a grade as possible to minimise flow velocities, scouring and erosion. – Installation of permanent lined drains as soon as possible, with outlet protection to prevent scouring. – Construction of roadside drains and road camber to direct runoff into side drains and safely convey it to crossroad drains; – Drains to discharge into stable vegetated areas or natural drainage lines wherever possible; – Road sections with a high risk of erosion (e.g. steeper grades) sealed; – Installation of retaining structures (rock walls, gabions, etc) to stabilize cuttings and embankments; and – Use of temporary erosion control measures (e.g. temporary diversion banks; plastic sheeting on exposed batters). – Existing stable drainage lines to be used for trunk drainage where possible. • Erosion control on steep slopes subject to gully erosion will be carried out by appropriate erosion control good practices. These measures rely on: – Minimising stormwater volumes through peripheral drainage. – Run-off control, including regular cut-off drains, crossroad drainage, sealed roadside drains, berms and flumes. – For areas of earthworks, design slopes with appropriate grade, compaction and benching – Rapid stabilisation by revegetation and engineering methods. • Cross road drainage will be the minimum grade and length possible and will be designed in accordance with **C-8 Watercourse Crossing Management Plan**. • Drainage lines shall be protected by marking off these areas from adjacent construction sites to ensure machinery and vehicles do not enter these areas. • All vehicles and machinery shall be restricted to designated accessways and areas to prevent additional ground disturbance.	Throughout construction	HEC Construction Manager

#	Issue or Risk	Action	Timing / Frequency	Responsibility
C-10-4.	Sediment control	• Sediment controls shall be installed across construction sites to remove sediment before runoff discharges from the site, based on the principle of dividing the catchment into manageable areas (rather than relying on a single trap at the bottom of the site that may fail). • All runoff from project work sites shall be directed into sediment traps or basins to remove coarse and suspended sediment from runoff before it is discharged. • Sediment basin volume and dimensions will be designed based on catchment area, estimated storm discharge (e.g. two-year return period) and soil particle size, in accordance with good practice. • Sediment basins will be fitted with a runoff overflow pipe and spillway that either discharges into a secondary basin (as needed) or into a stable natural watercourse or broad open area. • Sediment basins shall be accessible by machinery so they can be cleaned. • Where significant sediment volumes may be generated basins shall have primary and secondary treatment to settle out sediment before discharge. • Sediment basins, traps and fences shall be cleaned of sediment when 50% of their available capacity is full. • Removed sediment will either be deposited into an adjacent trench to dry or moved to spoil disposal sites.	Throughout construction	HEC Construction Manager
C-10-5.	Stockpiling of materials	• Topsoil and spoil shall be stockpiled in accordance with **C-9 Spoil and Topsoil Management Plan**. • Erodible construction material (sand, soil, etc) will be stockpiled: – Within Core Land or the Lot 1 right of way – On flat and lower slope land (<15°) – At least 20 m from drainage lines or streams, and outside the normal Tina River flood zone (1:5 year flood), with the exception of temporary storage of aggregate at river extraction sites. – On sites already devoid of trees. – Not directly upslope of houses and other structures.	Throughout construction	HEC Construction Manager
C-10-6.	Temporary facilities	• Erosion and sediment controls will be installed at all temporary and permanent site facilities, in accordance with this plan. • Process waters generated at concrete batch plants will be treated and reused, including removing suspended solids and neutralising pH. Ingress of stormwater into the process water treatment stream will be minimised as much as possible. Polymer flocculants are proposed to be used. • Process waters used in the crusher plant will be treated and reused. Polymer flocculants are proposed to be used.	Throughout construction	HEC Construction Manager
C-10-7.	Main dam works	• The river diversion and associated in-river works shall proceed in accordance with **Annex C-10-III River Diversion Construction Method** and will: – Be undertaken in the dry season to the extent possible, when the river is at its lowest – Use bunding, sheetpiles or similar to separate the zone of work from the active river channel. – Minimise tracking of machinery vehicles through flowing water. • Where community water sources are potentially impacted, replacement supplies will be provided in accordance with **C-7 Water Supply Replacement Plan.**	Throughout construction	HEC Construction Manager
C-10-8.	Tunnelling	• Surface water and groundwater from the tunnel shall be diverted or treated to remove sediment prior to discharge. • Spoil and wastewater generated during tunnelling shall be disposed of in accordance with **C-9 Spoil and Topsoil Management Plan** and **P-12 Waste Management and Point Source Pollution Plan.**	During tunnelling	HEC Construction Manager
C-10-9.	Site rehabilitation	• Cutting and filling will be completed to final cross-sections at each site/discrete section as soon as possible, ideally in one continuous activity, to create the final landform and enable site stabilisation works to be undertaken as early as possible. • Rehabilitation of temporary facilities is to be undertaken in accordance with the requirements of **C-4 Post Construction Rehabilitation and Revegetation Plan.**	During and post-earthworks	HEC Construction Manager HEC HSE Manager

Monitoring Requirements

#	Title	Description	Target / Performance Indicator	Timing / Frequency	Responsibility
C-10-A.	Assessment and approval of controls	THL will inspect each site to be cleared prior to the commencement of any clearing. They will approve and provide signoff on HEC's IR002 (Site Specific ESMP Assessment Pre-Clearing and Grubbing) for sites that have been clearly marked out in accordance with: • The Construction drawings. • The **C-3 Forest Clearance Plan**; • The correct installation of initial erosion and sediment controls before stripping and grubbing commences.	All controls installed to required standard	Once following installation of controls	THL
C-10-B.	Erosion and sediment control maintenance	Inspection of erosion and sediment controls weekly as part of regular monitoring, and after each major rainfall event, with maintenance conducted. IR-005 Inspection Report (ESMP Controls, refer to **P-1**) form will be used by HEC. Sediment storage capacity in basins will be documented each week and after each major rainfall event. Where installed erosion and sediment controls are deemed inadequate, modifications to these controls or additional controls shall be designed and installed.	No FOR A or B, NCRs or grievances raised in relation to sediment discharges / water quality. No exceedances in monitored water quality parameters (TSS).	Part of weekly site inspections + inspections following each major rainfall event	HEC Construction Manager HEC HSE Manager Subcontractors.
C-10-C.	Water quality monitoring	Any discharges to the environment will comply with water quality targets in **M-1 Suspended Sediment Monitoring Plan** and **M-2 Water Quality Monitoring Plan** (WQMP). Water quality monitoring will be undertaken as detailed in **M-1 Suspended Sediment Monitoring Plan** and **M-2 Water Quality Monitoring Plan** (WQMP).	Refer details in **M-2 Water Quality Monitoring Plan**		

Supporting Documents		
Annex	**Name**	**Description**
C-10-I.	Calculation Sheet for Sediment Basin Surface Area	Calculation of surface area for sediment basins.
C-10-II.	Approved Erosion and Sediment Control Plans for Access Roads	Design plans approved by OE.
C-10-III.	River Diversion Construction Method	Description of sequencing river diversion for main works construction.

ANNEX C-10-I CALCULATION SHEET FOR SEDIMENT BASIN SURFACE AREA

▣ Calculation of Surface Area for Sediment Basin

1. Rainfall runoff Calculation

○ Formula

□ Q = C × I × A × 1/360

Q; Discharge (㎥/sec) C; Runoff coefficient

I; Rainfall Intensity (mm/hr) A; Catchment aeaa (ha)

1) Runoff Coefficient (C)

○ Runoff coefficient is applied as per access road design as follows.

Lot 1 : 0.5

Lot 2 : 0.8

Lot 3 : 0.8

2) Rainfall Intensity (I)

○ Rainfall intensity applied is 19.45mm/hr for 10yr, 6hour rain event.

3) Calculation of discharge (Q)

LOT	No.	Location	C	I (mm/hr)	A (ha)	Q (㎥/s)	Remark
LOT 1	01	0+000.00	0.50	19.450408	2.07	0.0559	
LOT 1	02	0+000.00	0.50	19.450408	2.86	0.0773	
LOT 1	03	2+730.00	0.50	19.450408	1.56	0.0421	
LOT 1	04	3+740.00	0.50	19.450408	1.56	0.0421	
LOT 1	05	3+740.00	0.50	19.450408	1.13	0.0305	
LOT 1	06	4+000.00	0.50	19.450408	0.75	0.0203	
LOT 1	07	5+140.00	0.50	19.450408	0.50	0.0135	
LOT 1	08	5+140.00	0.50	19.450408	0.52	0.0140	
LOT 1	09	5+760.00	0.50	19.450408	0.91	0.0246	
LOT 1	10	5+760.00	0.50	19.450408	0.92	0.0249	
LOT 1	11	7+440.00	0.50	19.450408	2.42	0.0654	
LOT 1	12	8+230.00	0.50	19.450408	0.48	0.0130	
LOT 1	13	8+280.00	0.50	19.450408	0.47	0.0127	
LOT 1	14	8+840.00	0.50	19.450408	0.22	0.0059	
LOT 1	15	8+840.00	0.50	19.450408	0.24	0.0065	
LOT 1	16	9+110.00	0.50	19.450408	0.21	0.0057	
LOT 1	17	9+400.00	0.50	19.450408	0.97	0.0262	
LOT 1	18	9+700.00	0.50	19.450408	0.12	0.0032	
LOT 1	19	9+860.00	0.50	19.450408	0.67	0.0181	
LOT 1	20	9+860.00	0.50	19.450408	0.58	0.0157	
LOT 1	21	10+750.00	0.50	19.450408	0.90	0.0243	

LOT 1	22	10+830.00	0.50	19.450408	0.48	0.0130	
LOT 1	23	11+160.00	0.50	19.450408	0.44	0.0119	
LOT 1	24	11+500.00	0.50	19.450408	0.50	0.0135	
LOT 1	25	11+540.00	0.50	19.450408	0.43	0.0116	
LOT 1	26	12+140.00	0.50	19.450408	0.86	0.0232	
LOT 1	27	12+760.00	0.50	19.450408	0.40	0.0108	
LOT 1	28	12+920.00	0.50	19.450408	0.36	0.0097	
LOT 1	29	13+000.00	0.50	19.450408	0.74	0.0200	
LOT 2-1	30	0+030.00	0.80	19.450408	0.72	0.0311	
LOT 2-1	31	0+150.00	0.80	19.450408	1.09	0.0471	
LOT 2-1	32	1+190.00	0.80	19.450408	0.27	0.0117	
LOT 2-1	33	1+230.00	0.80	19.450408	0.20	0.0086	
LOT 2-1	34	1+380.00	0.80	19.450408	0.22	0.0095	
LOT 2-1	35	1+590.00	0.80	19.450408	0.05	0.0022	
LOT 2-1	36	1+840.00	0.80	19.450408	0.24	0.0104	
LOT 2-1	37	1+850.00	0.80	19.450408	0.40	0.0173	
LOT 2-1	38	2+070.00	0.80	19.450408	0.21	0.0091	
LOT 2-1	39	0+070.00	0.80	19.450408	0.26	0.0112	
LOT 2-1	40	2+340.00	0.80	19.450408	0.29	0.0125	
LOT 2-1	41	2+350.00	0.80	19.450408	0.13	0.0056	
LOT 2-2	42	0+600.00	0.80	19.450408	1.11	0.0480	
LOT 2-2	43	0+730.00	0.80	19.450408	0.91	0.0393	
LOT 2-2	44	1+350.00	0.80	19.450408	0.89	0.0385	
LOT 2-2	45	1+400.00	0.80	19.450408	1.32	0.0571	
LOT 2-2	46	1+760.00	0.80	19.450408	0.13	0.0056	
LOT 2-2	47	1+770.00	0.80	19.450408	0.69	0.0298	
LOT 2-2	48	1+960.00	0.80	19.450408	0.05	0.0022	
LOT 2-2	49	1+980.00	0.80	19.450408	0.26	0.0112	
LOT 2-3	50	2+360.00	0.80	19.450408	0.25	0.0108	
LOT 2-2	51	2+430.00	0.80	19.450408	3.07	0.1327	
LOT 2-3	52	2+460.00	0.80	19.450408	0.05	0.0022	
LOT 2-3	53	2+690.00	0.80	19.450408	1.52	0.0657	
LOT 3-1	54	0+120.00	0.80	19.450408	0.21	0.0091	
LOT 3-1	55	0+260.00	0.80	19.450408	0.42	0.0182	
LOT 3-1	56	0+490.00	0.80	19.450408	0.21	0.0091	
LOT 3-1	57	0+500.00	0.80	19.450408	0.44	0.0190	
LOT 3-1	58	0+860.00	0.80	19.450408	0.39	0.0169	
LOT 3-2	59	0+010.00	0.80	19.450408	0.59	0.0255	

LOT 3-1	60	1+040.00	0.80	19.450408	0.13	0.0056	
LOT 3-1	61	1+240.00	0.80	19.450408	0.48	0.0207	
LOT 3-1	62	1+400.00	0.80	19.450408	0.26	0.0112	
LOT 3-1	63	1+480.00	0.80	19.450408	0.53	0.0229	

2. Calculation of water surface area

1) Design particle size and sedimentaion velocity

○ Design particle size applied is 0.05mm of coarse silt.

○ Settling velocity applied is 0.00189m/s based on the Goldman et at. (1986).

2) Calculation of surface area required

○ Formula

□ $A_s = 1.2Q/V_s$

A_s; Minimum surface area (㎡) Q; Discharge (㎥/sec)

V_s; Settling velocity (m/s)

3) Surface area required

○ Depth applied is 1.0m and basin length is a minimum of twice the basin width.

LOT	No.	Location	Q (㎥/sec)	Vs (m/s)	A (㎡)	A (㎡) required	Remark
LOT 1	01	0+000.00	0.06	0.001890	29.59	35.5	
LOT 1	02	0+000.00	0.08	0.001890	40.88	49.1	
LOT 1	03	2+730.00	0.04	0.001890	22.30	26.8	
LOT 1	04	3+740.00	0.04	0.001890	22.30	26.8	
LOT 1	05	3+740.00	0.03	0.001890	16.15	19.4	
LOT 1	06	4+000.00	0.02	0.001890	10.72	12.9	
LOT 1	07	5+140.00	0.01	0.001890	7.15	8.6	
LOT 1	08	5+140.00	0.01	0.001890	7.43	8.9	
LOT 1	09	5+760.00	0.02	0.001890	13.01	15.6	
LOT 1	10	5+760.00	0.02	0.001890	13.15	15.8	
LOT 1	11	7+440.00	0.07	0.001890	34.59	41.5	
LOT 1	12	8+230.00	0.01	0.001890	6.86	8.2	
LOT 1	13	8+280.00	0.01	0.001890	6.72	8.1	
LOT 1	14	8+840.00	0.01	0.001890	3.14	3.8	
LOT 1	15	8+840.00	0.01	0.001890	3.43	4.1	
LOT 1	16	9+110.00	0.01	0.001890	3.00	3.6	
LOT 1	17	9+400.00	0.03	0.001890	13.87	16.6	
LOT 1	18	9+700.00	0.00	0.001890	1.72	2.1	
LOT 1	19	9+860.00	0.02	0.001890	9.58	11.5	
LOT 1	20	9+860.00	0.02	0.001890	8.29	9.9	

LOT 1	21	10+750.00	0.02	0.001890	12.87	15.4	
LOT 1	22	10+830.00	0.01	0.001890	6.86	8.2	
LOT 1	23	11+160.00	0.01	0.001890	6.29	7.5	
LOT 1	24	11+500.00	0.01	0.001890	7.15	8.6	
LOT 1	25	11+540.00	0.01	0.001890	6.15	7.4	
LOT 1	26	12+140.00	0.02	0.001890	12.29	14.8	
LOT 1	27	12+760.00	0.01	0.001890	5.72	6.9	
LOT 1	28	12+920.00	0.01	0.001890	5.15	6.2	
LOT 1	29	13+000.00	0.02	0.001890	10.58	12.7	
LOT 2-1	30	0+030.00	0.03	0.001890	16.47	19.8	
LOT 2-1	31	0+150.00	0.05	0.001890	24.93	29.9	
LOT 2-1	32	1+190.00	0.01	0.001890	6.18	7.4	
LOT 2-1	33	1+230.00	0.01	0.001890	4.57	5.5	
LOT 2-1	34	1+380.00	0.01	0.001890	5.03	6.0	
LOT 2-1	35	1+590.00	0.00	0.001890	1.14	1.4	
LOT 2-1	36	1+840.00	0.01	0.001890	5.49	6.6	
LOT 2-1	37	1+850.00	0.02	0.001890	9.15	11.0	
LOT 2-1	38	2+070.00	0.01	0.001890	4.80	5.8	
LOT 2-1	39	0+070.00	0.01	0.001890	5.95	7.1	
LOT 2-1	40	2+340.00	0.01	0.001890	6.63	8.0	
LOT 2-1	41	2+350.00	0.01	0.001890	2.97	3.6	
LOT 2-2	42	0+600.00	0.05	0.001890	25.39	30.5	
LOT 2-2	43	0+730.00	0.04	0.001890	20.81	25.0	
LOT 2-2	44	1+350.00	0.04	0.001890	20.36	24.4	
LOT 2-2	45	1+400.00	0.06	0.001890	30.19	36.2	
LOT 2-2	46	1+760.00	0.01	0.001890	2.97	3.6	
LOT 2-2	47	1+770.00	0.03	0.001890	15.78	18.9	
LOT 2-2	48	1+960.00	0.00	0.001890	1.14	1.4	
LOT 2-2	49	1+980.00	0.01	0.001890	5.95	7.1	
LOT 2-3	50	2+360.00	0.01	0.001890	5.72	6.9	
LOT 2-2	51	2+430.00	0.13	0.001890	70.22	84.3	
LOT 2-3	52	2+460.00	0.00	0.001890	1.14	1.4	
LOT 2-3	53	2+690.00	0.07	0.001890	34.77	41.7	
LOT 3-1	54	0+120.00	0.01	0.001890	4.80	5.8	
LOT 3-1	55	0+260.00	0.02	0.001890	9.61	11.5	
LOT 3-1	56	0+490.00	0.01	0.001890	4.80	5.8	
LOT 3-1	57	0+500.00	0.02	0.001890	10.06	12.1	
LOT 3-1	58	0+860.00	0.02	0.001890	8.92	10.7	

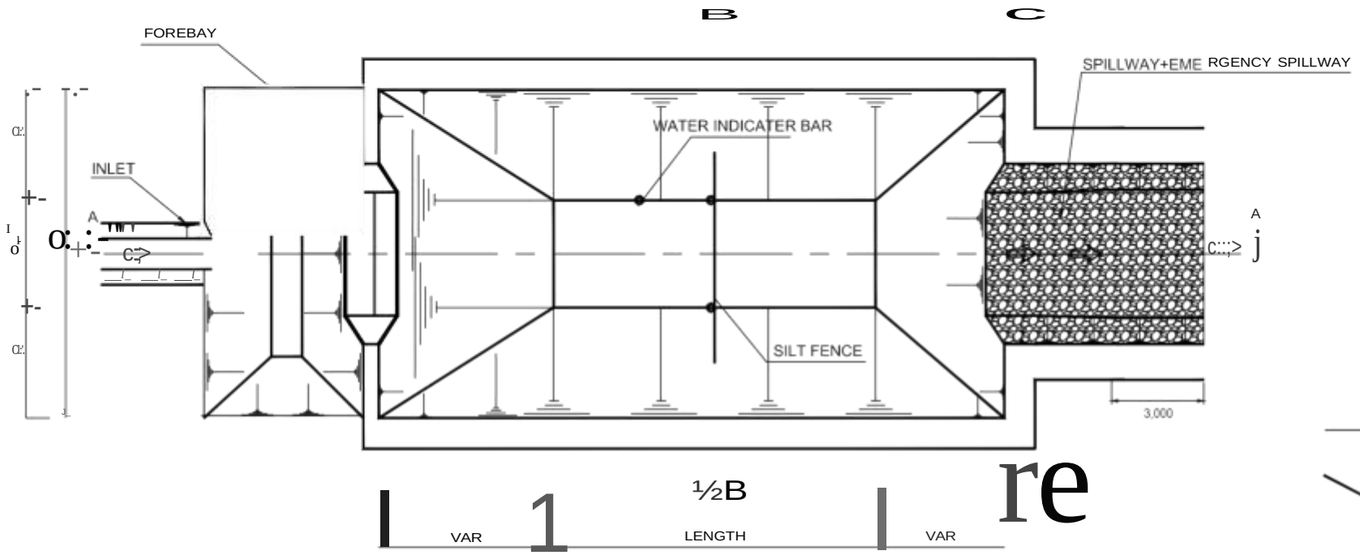
LOT 3-2	59	0+010.00	0.03	0.001890	13.49	16.2	
LOT 3-1	60	1+040.00	0.01	0.001890	2.97	3.6	
LOT 3-1	61	1+240.00	0.02	0.001890	10.98	13.2	
LOT 3-1	62	1+400.00	0.01	0.001890	5.95	7.1	
LOT 3-1	63	1+480.00	0.02	0.001890	12.12	14.5	

ANNEX C-10-II APPROVED EROSION AND SEDIMENT CONTROL PLANS FOR ACCESS ROAD

(ACCESS ROAD LOT 1) SEDIMENT BASIN DETAILS

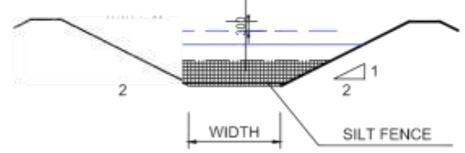
GENENAL DRAWINGS

S=NONE

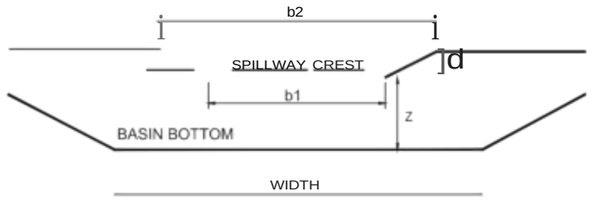


GEOTEXTILE SPECIFICATION

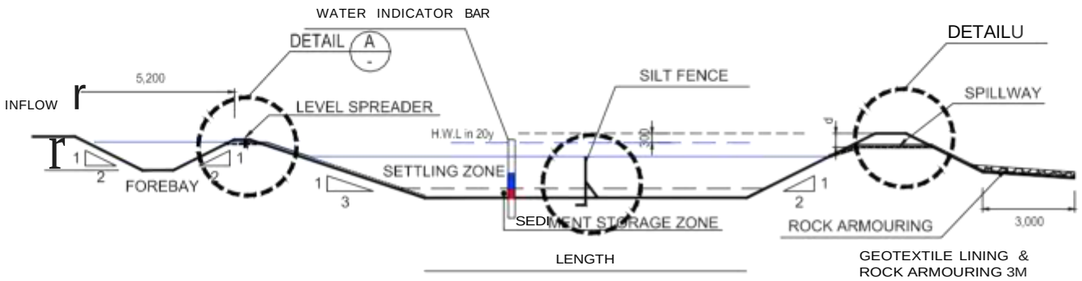
- Tensile Streangth at break 2: 50kn/m(EN ISO 10319)
- Weathering Resistance 2: 40%(EN ISO 12224)
- Apparent opening size ≤: 0.4mm(EN ISO 12956)



SECTION B - B



SECTION C-C



GEOTEXTILE LINING & ROCK ARMOURING 3M AWAY FROM TOE OF FILL.

SECTION A-A

LEVEL SPREADER

THE LEVEL SPREADER ON THE FOREBAY IS TO BE CONSTRUCTED 100MM ABOVE THE OUTLET SPILLWAY

GEOTEXTILE WRAPPED AROUNID LEVEL SPREADER AND C NCRETE HAUNCHING

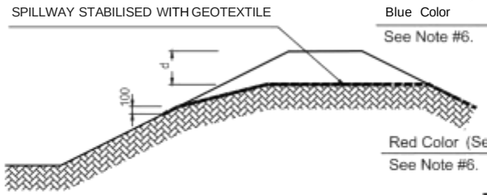
NON-WOVEN GEOTEXTILE

NON_-WOVERN GEOTEXTILE SPECIFICATION

- Tensile Streangth 17kn/m(ASTM D 4595)
- Tensile Elongation 50%(ASTM D 4595)
- Tear Streangth 400N(ASTM D 4533)
- Apparent opening size 0.1mm(ASTM D 4751)

A DETAIL

SPILLWAY STABILISED WITH GEOTEXTILE



B DETAIL

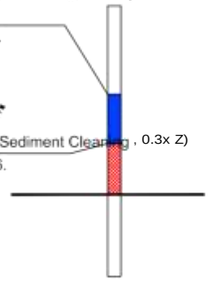
(Dewatering, 0.5x Z)

Blue Color

See Note #6.

Red Color (Sediment Cleaning, 0.3x Z)

See Note #6.



C WATER INDICATOR BAR

SPECIFICATION OF SEDIMENT BASIN

(UNITS: m)

Catchment No.	Location	SEDIMENT BASIN DIMENSIONS				SPILLWAY DIMENSIONS			
		Depth (m)	Width (m)	Length (m)	Volume (m³)	Z (m)	b1 (m)	b2 (m)	d (m)
1	0+000.00	2.0	4.5	13.5	354.5	2.00	2.25	3.35	0.55
2	0+000.00	2.0	6.0	18.0	500.0	2.00	3.00	4.10	0.55
3	2+730.00	1.5	4.5	13.5	210.9	1.50	2.25	3.25	0.50
4	2+740.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
5	3+740.00	1.5	3.5	10.5	155.8	1.50	1.75	2.75	0.50
6	4+000.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
7	5+140.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
8	5+140.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
9	5+760.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
10	5+760.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
11	7+460.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
12	7+460.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
13	7+480.00	1.0	3.0	9.0	62.5	1.00	1.50	2.40	0.45
14	7+480.00	1.0	3.0	9.0	62.5	1.00	1.50	2.40	0.45
15	8+230.00	1.0	2.5	7.5	50.0	1.00	1.25	2.15	0.45
16	8+280.00	1.0	2.5	7.5	50.0	1.00	1.25	2.15	0.45
20	9+400.00	1.5	3.0	9.0	131.6	1.50	1.50	2.50	0.50
22	9+860.00	1.0	3.5	10.5	76.5	1.00	1.75	2.65	0.45
23	9+860.00	1.0	3.0	9.0	62.5	1.00	1.50	2.40	0.45
24	9+960.00	2.0	7.5	22.5	672.5	2.00	3.75	4.75	0.50
25	10+750.00	1.5	3.0	9.0	131.6	1.50	1.50	2.50	0.50
26	10+830.00	1.0	2.5	7.5	50.0	1.00	1.25	2.15	0.45
27	11+160.00	1.0	2.5	7.5	50.0	1.00	1.25	2.15	0.45
28	11+500.00	1.0	2.5	7.5	50.0	1.00	1.25	2.15	0.45
29	11+540.00	1.0	2.0	6.0	39.0	1.00	1.00	1.90	0.45
31	12+760.00	1.0	3.5	10.5	76.5	1.00	1.75	2.65	0.45
32	12+980.00	1.0	3.5	10.5	76.5	1.00	1.75	2.65	0.45

NOTES

1. DE-WATERING SYSTEM IS NOT MANDATORY FOR TYPE-B SEDIMENT BASIN GENERALLY. HOWEVER PUMP OR SIPHON CAN BE USED FOR DE-WATERING SYSTEM.
2. PRIOR TO THE DISCHARGE OF WATER FROM A SEDIMENT BASIN, SPECIFIED WATER QUALITY OBJECTIVES ARE WATER PH AND NTU.
   ACCORDING TO IECA AUSTRALIA, DISCHARGE WATER QUALITY STANDARD ARE ;
   1) WATER PH IN THE RANGE 6.5 TO 8.5
   2) 90 PERCENTILE NTU READING NOT EXCEEDING 100, AND 50 PERCENTILE NTU READING NOT EXCEEDING 60.
3. DE-SILTING MARKER POST SHALL BE INSTALLED IN THE BASIN TO INDICATE THE TOP OF THE SEDIMENT STORAGE ZONE. THE BASIN SHALL BE DE-SILTED IF THE NEXT STORM IS LIKELY TO CAUSE THE SETTLED SEDIMENT TO RISE ABOVE THE MARKER POINT, OR IF THE SETTLES SEDIMENT HAS EXCEEDED 90% OF THE NOMINATED SEDIMENT STORAGE VOLUME.
4. REMOVED SEDIMENT WILL BE MOVED TO AN ADJACENT SPOIL DISPOSAL SITE TO DRY, AND DISPOSAL FINALLY.
5. THE SHAPE OF THE THE SEDIMENT BASINS CAN BE CHANGED IN CONSULTATION WITH THE EMPLOYER DEPENDING ON THE SITE CONDITIONS.
6. IN WATER INDICATOR, THE RED COLOR OF THE WATER LEVEL SIGN INDICATES THE LIMIT HEIGHT OF THE SEDIMENT CLEANING (0.3xZ) AND THE BLUE COLOR INDICATES THE LIMIT HEIGHT OF THE DEWATERING(0.5xZ)."

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
6	Aii2'i6	ISSUED FOR CONSTRUCTION					
6	FEB09 2021	ISSUED FOR CONSTRUCTION					
A	JUL 17 2020	ISSUED FOR CONSTRUCTION					
6	APR10 2020	ISSUED FOR CONSTRUCTION					

CLIENT: r.u.Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: bydrogel

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

TITLE: (ACCESS ROAD LOT 1) SEDIMENT BASIN DETAILS

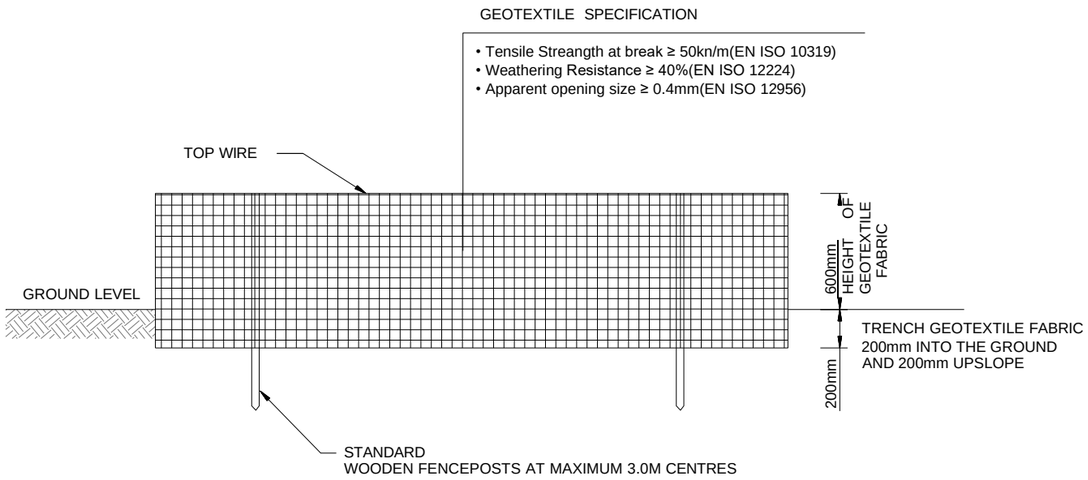
GENENAL DRAWINGS

DATE	SCALE	DRAWING NO.
APR 06,2021	S=NONE	E-PR-CVR 1-02-84400

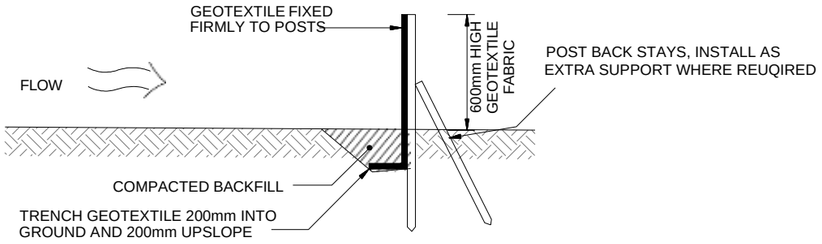
A3 (420X297mm)

(ACCESS ROAD) SILT FENCE

S=NONE

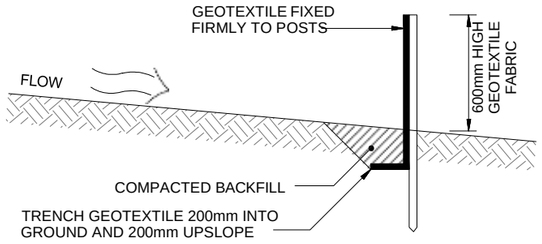


ELEVATION - SILT FENCE



TYPICAL SECTION -SILT FENCE

at sediment basin



TYPICAL SECTION - SILT FENCE

except for sediment basin

SILT FENCE DESIGN CRITERIA TABLE

Slope steepness	Slope length(m)(maximum)	Spacing of returns(m)	Silt fence length(m) (maximum)
Flatter than 2%	Unlimited	N/A	Unlimited
2~10%	40	60	300
10~20%	30	50	230
20~33%	20	40	150
33~50%	15	30	75
>50%	6	20	40

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
0	APR 06 2021	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

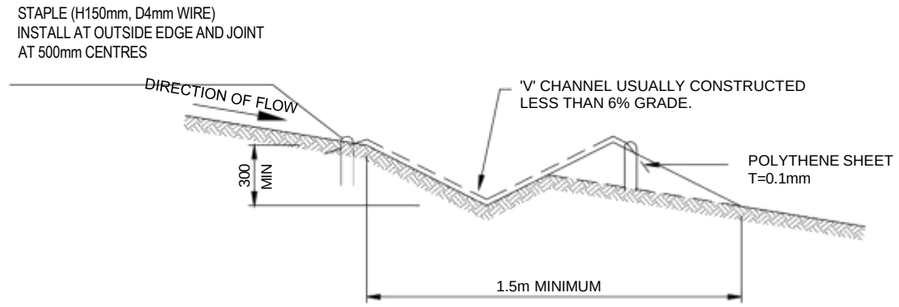
TITLE: (ACCESS ROAD) SILT FENCE

DATE	SCALE	DRAWING NO.	REV.
APR 06, 2021	S=NONE	E-PR-CVR1-D2-84420	0

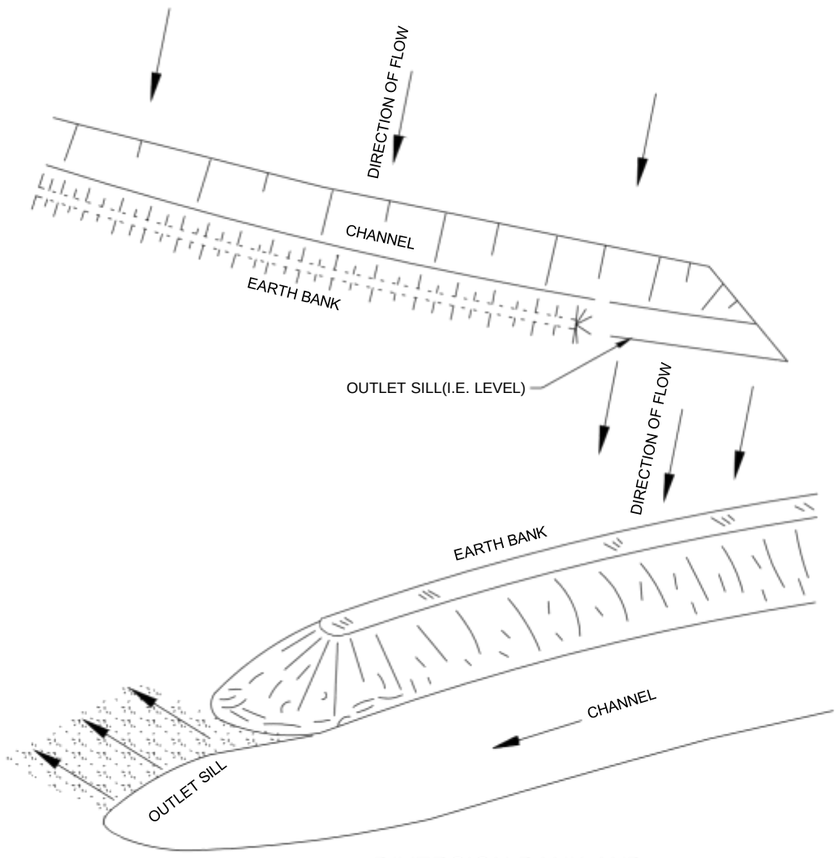
A3 (420X297mm)

(ACCESS ROAD) CLEAN WATER DIVERSION CHANNEL

S=NONE



TYPICAL CROSS SECTION OF
TEMPORARY DIVERSION BANK



DIVERSION BANK AND
SPREADING SYSTEM OUTLET

NOTES

TEMPORARY DIVERSION BANKS

1. LOCATIONS SELECTED CONTINUALLY DURING CONSTRUCTION TO PROTECT WORKS (I.E. MAY REGULARLY CHANGE).
2. CHANNEL PROTECTION TO BE CONSIDERED IF GREATER THAN 6% GRADE.
3. BANKS TO BE ADEQUATELY COMPACTED TO PREVENT FAILURE.
4. SPACING BETWEEN BANKS TO BE DEPENDENT UPON SLOPE AND SOIL TYPE.
5. OUTLETS FROM BANKS TO DISCHARGE ONTO A STABLE AREA (E.G. ROCKS, NATURAL UNDISTURBED GROUND, TIMBER WINDROW, SEDIMENT TRAP, GEOTEXTILE BATTER DRAIN). BANK OUTLETS TO EXTEND PAST DISTURBED AREAS AND NOT BE TOO SHORT.
6. BANKS TO BE INSPECTED AFTER STORM EVENTS AND REPAIRED AS REQUIRED.

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
0	APR 06 2021	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

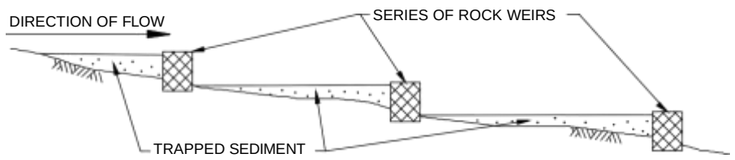
TITLE: (ACCESS ROAD) CLEAN WATER DIVERSION CHANNEL

DATE	SCALE	DRAWING NO.	REV.
APR 06, 2021	S=NONE	E-PR-CVR1-D2-84410	0

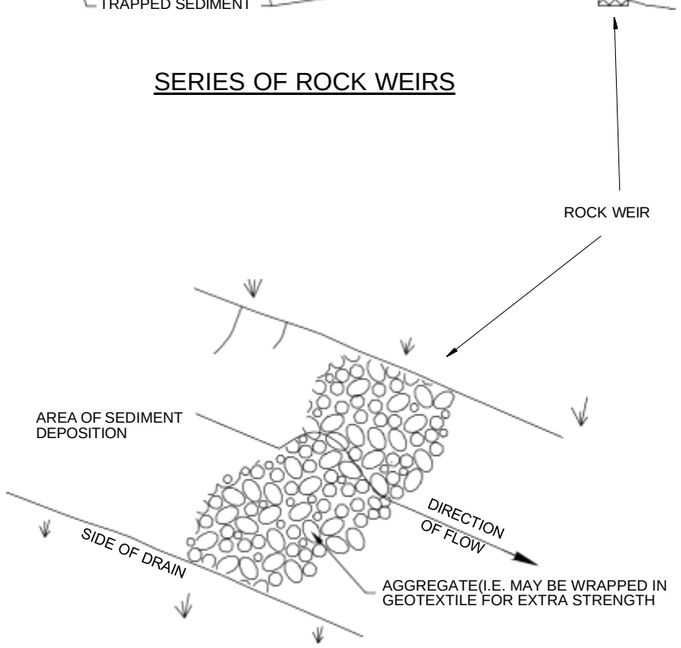
A3 (420X297mm)

(ACCESS ROAD) ROCK WEIR (CHECK DAM) OR IN-STREAM WEIRS

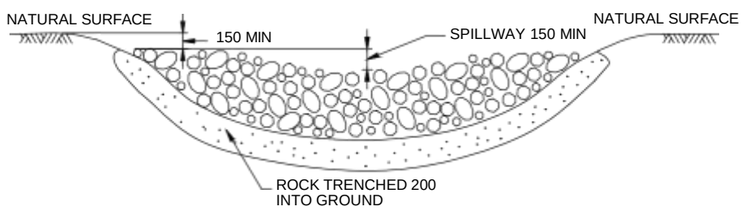
S=NONE



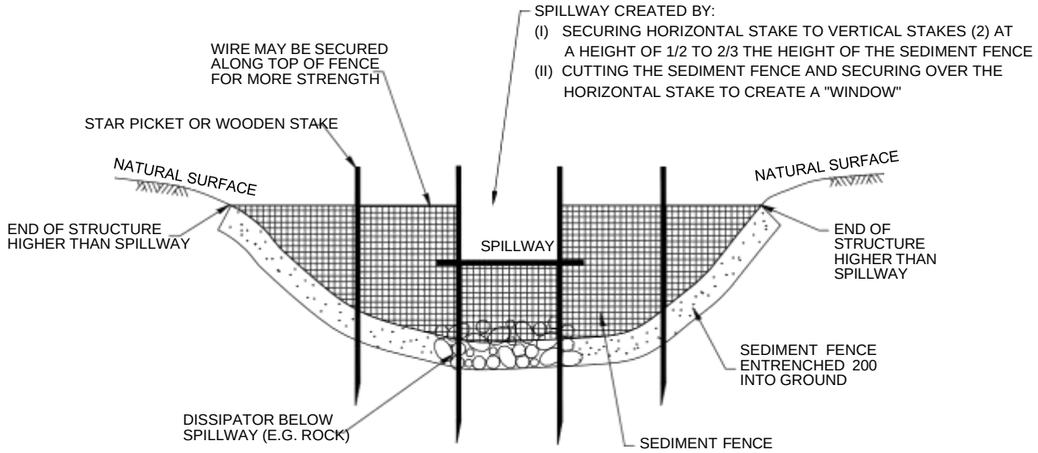
SERIES OF ROCK WEIRS



AGGREGATE ROCK WEIR



CROSS SECTION OF AGGREGATE ROCK WEIR



CROSS SECTION OF SEDIMENT FENCE ROCK WEIR

NOTES

1. ROCK WEIRS MAY BE CONSTRUCTED OF A VARIETY OF MATERIALS (E.G. STRAW BALES, SEDIMENT FENCE, ROCK & GEOTEXTILE, SHEET PILING, ETC.)
2. ROCK WEIRS TO BE TRENCHED 200 mm INTO GROUND SURFACE ON BASE AND SIDES AND SECURELY BACKFILLED.
3. SPILLWAY TO BE OVER INVERT OF DRAIN WITH DISCHARGE NOT PERMITTED TO FLOW AROUND ENDS.
4. SPILLWAY TO BE LESS THAN 1 METER ABOVE INVERT OF DRAIN.
5. SOME FORM OF DISSIPATION MAY BE REQUIRED BELOW SPILLWAYS OF ROCK WEIRS (E.G. ROCK, SAND BAGS).
6. ROCK WEIRS TO BE INSPECTED AFTER STORM EVENTS AND REPAIRED AS REQUIRED.
7. THE ROCK WEIRS MAY BE PLACED IN SERIES DOWN THE CHANNEL AND USED DURING CONSTRUCTION TO REDUCE THE VELOCITY IN DITCHES OR CHANNELS

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
B	APR 08 2021	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYDROSI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

TITLE: (ACCESS ROAD) ROCK WEIR (CHECK DAM) OR IN-STREAM WEIRS

DATE	SCALE	DRAWING NO.	REV.
APR 08, 2021	S=NONE	E-PR-CVR1-D2-84430	0

A3 (420X297mm)

TINA RIVER HYDROPOWER DEVELOPMENT PROJECT(TRHDP)

(ACCESS ROAD) SEDIMENT BASIN

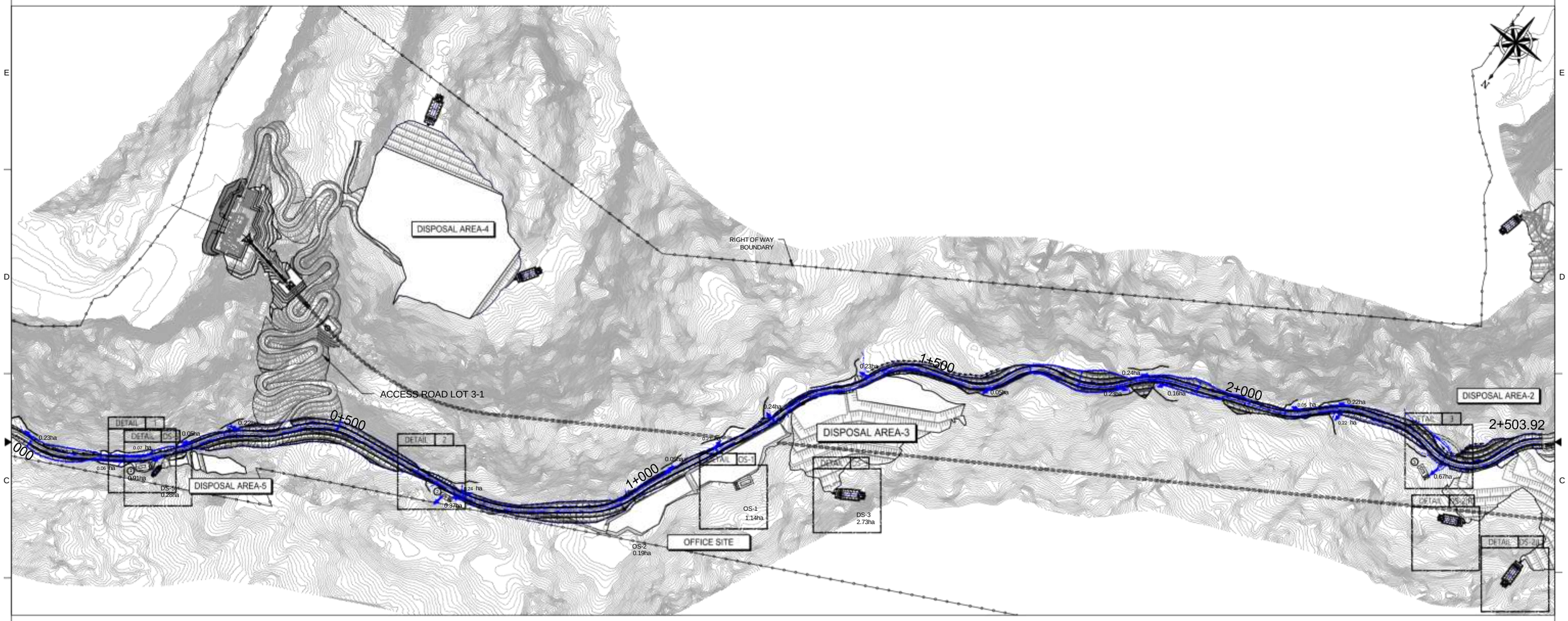
- LOT 2 & 3

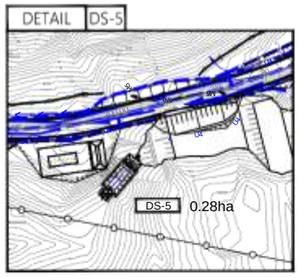
10-NOV-2021

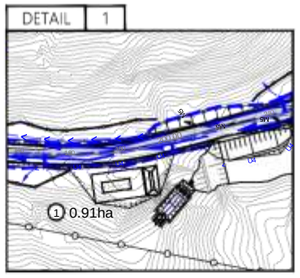
Tina Hydropower Limited

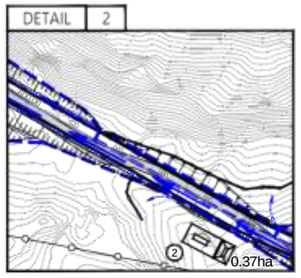
(ACCESS ROAD LOT 2-1) SEDIMENT BASIN PLAN

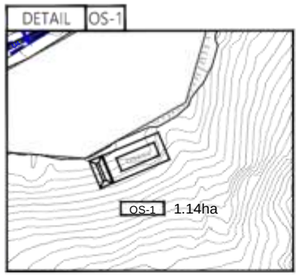
AS SHOWN

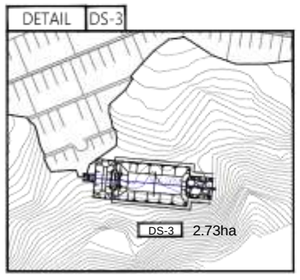


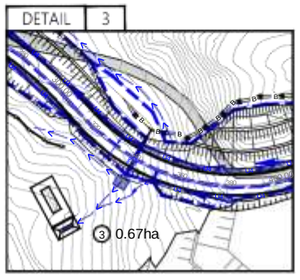


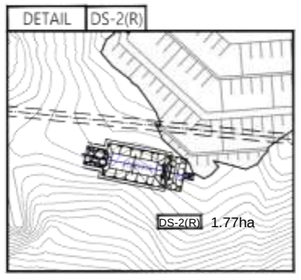


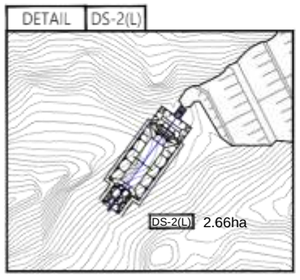












LEGEND

	Water Flow
	Sediment Basin
	Silt Fence

CATCHMENT AREA

NO.	LOCATION	CATCHMENT AREA(ha)	20-year flow (Q20,m³/s)	SEDIMENT BASIN SIZE			REMARKS
				Depth (m)	Width (m)	Length (m)	
DS-5	0+200.00	0.28	0.071	0.8	3.1	9.2	Disposal Area-5
1	0+180.00	0.91	0.312	1.8	4.0	11.0	LOT 2-1
2	0+690.00	0.37	0.137	1.5	2.0	6.0	〃
OS-1	1+250.00	1.14	0.389	1.5	5.5	16.5	Office site-1
DS-3	1+340.00	2.73	0.514	2.5	7.1	21.2	Disposal Area-3
3	2+340.00	0.67	0.235	1.5	3.5	10.0	LOT 2-1
DS-2(R)	2+380.00	1.77	0.377	2.7	6.0	18.1	Disposal Area-2
DS-2(L)	2+503.92	2.66	0.553	3.0	6.8	20.3	〃

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
3	NOV 10 2021	ISSUED FOR CONSTRUCTION					
2	SEP 21 2020	ISSUED FOR CONSTRUCTION					
1	JUL 18 2020	ISSUED FOR CONSTRUCTION					
0	JUN 25 2020	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

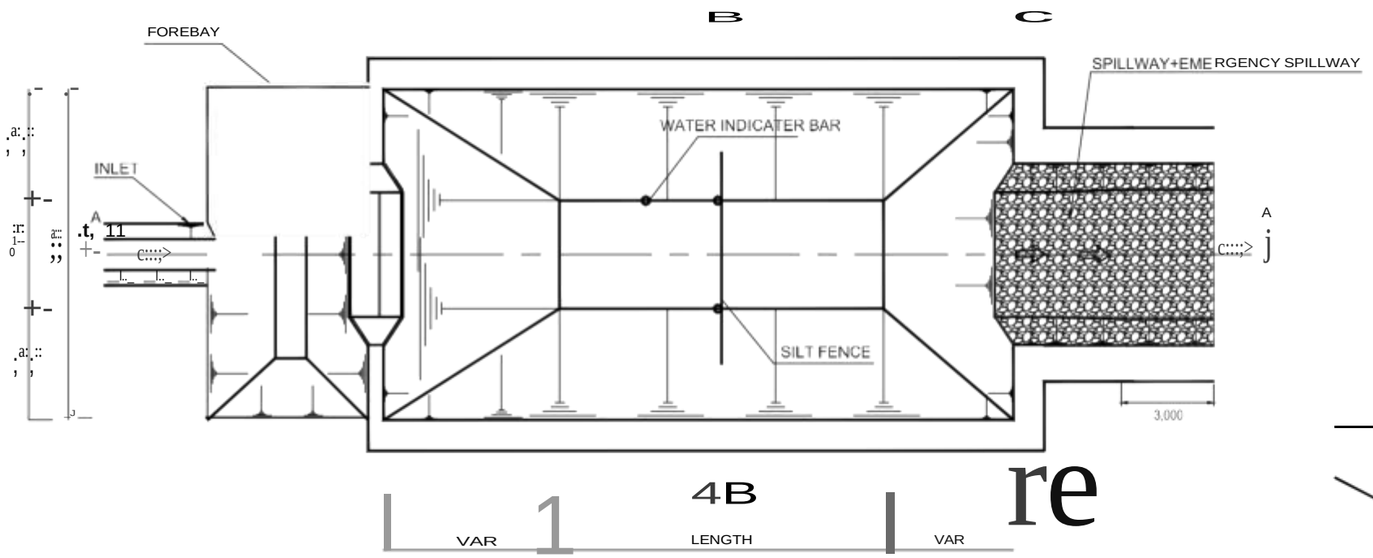
TITLE: (ACCESS ROAD LOT 2-1) SEDIMENT BASIN PLAN

DATE	SCALE	DRAWING NO.	REV.
NOV 10, 2021	AS SHOWN	E-PR-CVR2-D2-84090	3

A3 (420X297mm)

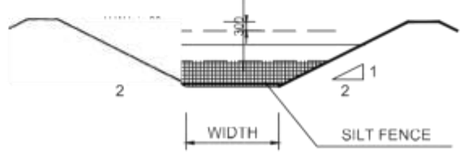
(ACCESS ROAD LOT 2-1) SEDIMENT BASIN DETAIL

S=NONE

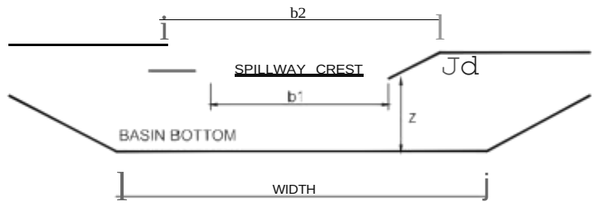


GEOTEXTILE SPECIFICATION

- Tensile Streangth at break;?:50kn/m(EN ISO 10319)
- Weathering Resistance ;?: 40%(EN ISO 12224)
- Apparent opening size :S: 0.4mm(EN ISO 12956)



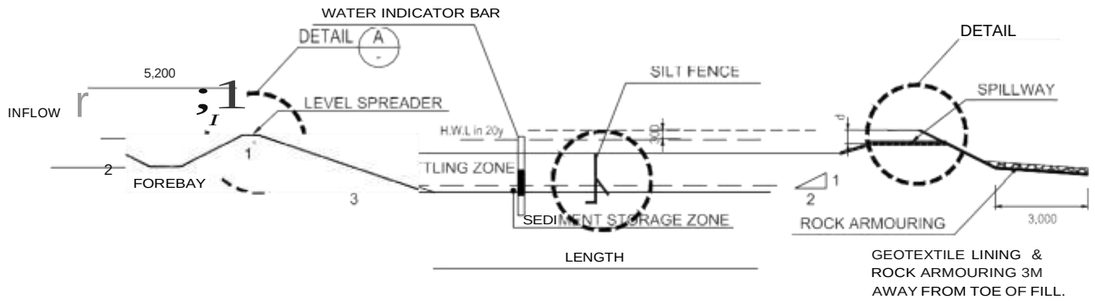
SECTION B - B



SECTION C-C

NOTES

1. DE-WATERING SYSTEM IS NOT MANDATORY FOR TYPE-B SEDIMENT BASIN GENERALLY. HOWEVER PUMP OR SIPHON CAN BE USED FOR DE-WATERING SYSTEM.
2. PRIOR TO THE DISCHARGE OF WATER FROM A SEDIMENT BASIN, SPECIFIED WATER QUALITY OBJECTIVES ARE WATER PH AND NTU.
   ACCORDING TO IECA AUSTRALIA, DISCHARGE WATER QUALITY STANDARD ARE :
   1) WATER PH IN THE RANGE 6.5 TO 8.5
   2) 90 PERCENTILE NTU READING NOT EXCEEDING 100, AND 50 PERCENTILE NTU READING NOT EXCEEDING 60.
3. DE-SILTING MARKER POST SHALL BE INSTALLED IN THE BASIN TO INDICATE THE TOP OF THE SEDIMENT STORAGE ZONE. THE BASIN SHALL BE DE-SILTED IF THE NEXT STORM IS LIKELY TO CAUSE THE SETTLED SEDIMENT TO RISE ABOVE THE MARKER POINT, OR IF THE SETTLES SEDIMENT HAS EXCEEDED 90% OF THE NOMINATED SEDIMENT STORAGE VOLUME.
4. REMOVED SEDIMENT WILL BE MOVED TO AN ADJACENT SPOIL DISPOSAL SITE TO DRY, AND DISPOSAL FINALLY.
5. THE SHAPE OF THE THE SEDIMENT BASINS CAN BE CHANGED IN CONSULTATION WITH THE EMPLOYER DEPENDING ON THE SITE CONDITIONS.
6. IN WATER INDICATOR, THE RED COLOR OF THE WATER LEVEL SIGN INDICATES THE LIMIT HEIGHT OF THE SEDIMENT CLEANING (0.3xZ) AND THE BLUE COLOR INDICATES THE LIMIT HEIGHT OF THE DEWATERING(0.5xZ).*



SECTION A-A

SPECIFICATION OF SEDIMENT BASIN

(UNITS: m)

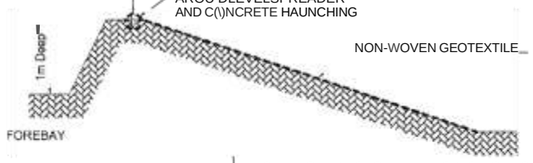
Catchment No.	Location	SEDIMENT BASIN DIMENSIONS				SPILLWAY DIMENSIONS			
		Depth (m)	Width (m)	Length (m)	Volume (m³)	Z (m)	b1 (m)	b2 (m)	d (m)
01	0+180.00	1.8	4.0	11.0	241.2	1.80	2.00	3.00	0.50
02	0+690.00	1.5	2.0	6.0	90.0	1.50	1.00	2.00	0.50
03	2+340.00	1.5	3.5	10.0	150.9	1.50	1.75	2.75	0.50

LEVEL SPREADER

THE LEVEL SPREADER ON THE FOREBAY IS TO BE CONSTRUCTED 100MM ABOVE THE OUTLET SPILLWAY

GEOTEXTILE WRAPPED AROU DLEVELSPREADER AND C(\)NCRETE HAUNCHING

NON-WOVEN GEOTEXTILE

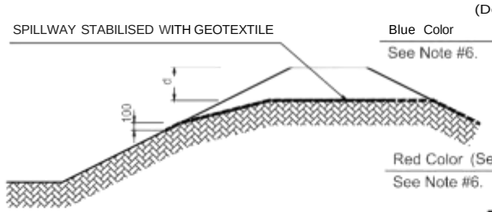


NON_-WOVERN GEOTEXTILE SPECIFICATION

- Tensile Streangth 17kn/m(ASTM D 4595)
- Tensile Elongation 50%(ASTM D 4595)
- Tear Streangth 400N(ASTM D 4533)
- Apparent opening size 0.1mm(ASTM D 4751)

A DETAIL

SPILLWAY STABILISED WITH GEOTEXTILE



B DETAIL

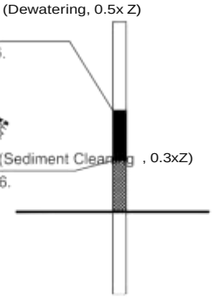
(Dewatering, 0.5x Z)

Blue Color

See Note #6.

Red Color (Sediment Cleaning , 0.3xZ)

See Note #6.



C WATER INDICATOR BAR

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
D.							
D.							
fil	NOV10 2021	ISSUED FOR CONSTRUCTION					
fil	JUL18 2020	ISSUED FOR CONSTRUCTION					
	26	ISSUED FOR CONSTRUCTION					

CLIENT: TinaHydro)Xlwer Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUDRI

DESIGNER: Dongju Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

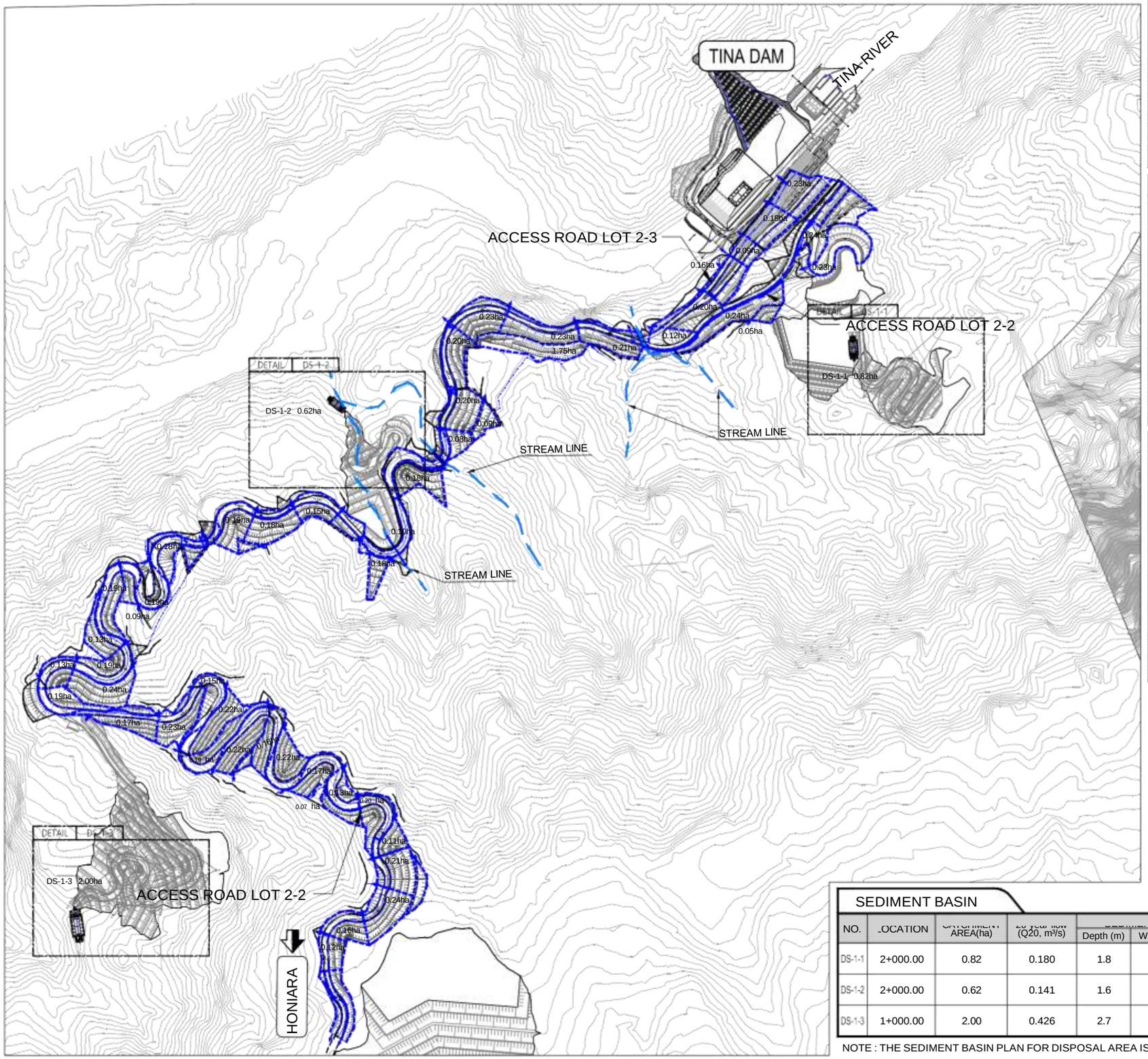
TITLE: (ACCESS ROAD LOT 2-1) SEDIMENT BASIN DETAIL

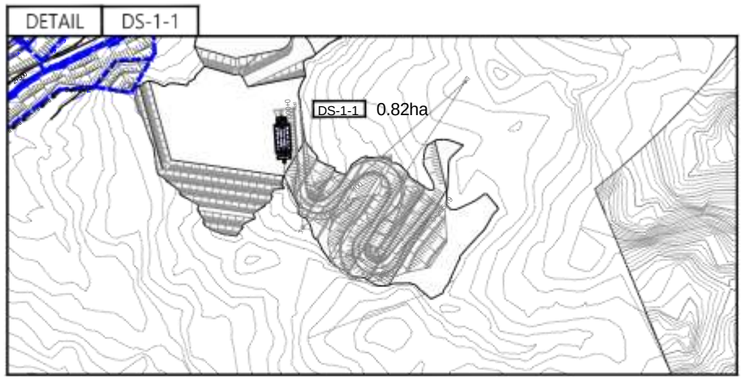
DATE	SCALE	DRAWING NO
NOV 10, 2021	S=NONE	E-PR-CVR2-D2-84100

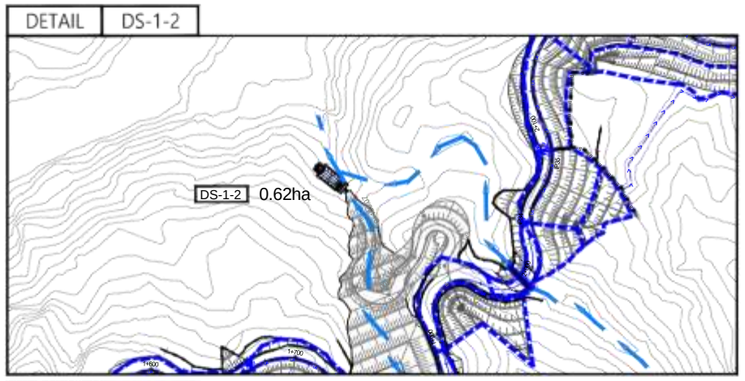
A3 (420X297mm)

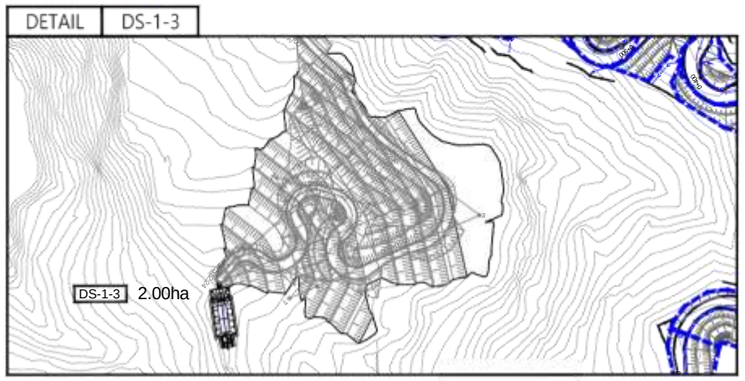
(ACCESS ROAD LOT 2-2) SEDIMENT BASIN PLAN

AS SHOWN









LEGEND

	Water Flow
	Sediment Basin
	Silt Fence

SEDIMENT BASIN

NO.	LOCATION	CATCHMENT AREA(ha)	20 Year flow (Q20, m³/s)	SEDIMENT BASIN SIZE			REMARKS
				Depth (m)	Width (m)	Length (m)	
DS-1-1	2+000.00	0.82	0.180	1.8	4.2	12.7	DISPOSAL AREA-1-1
DS-1-2	2+000.00	0.62	0.141	1.6	3.7	11.1	DISPOSAL AREA-1-2
DS-1-3	1+000.00	2.00	0.426	2.7	5.9	17.7	DISPOSAL AREA-1-3

NOTE : THE SEDIMENT BASIN PLAN FOR DISPOSAL AREA IS ONGOING

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
3	NOV 10 2021	ISSUED FOR CONSTRUCTION					
2	SEP 21 2020	ISSUED FOR CONSTRUCTION					
1	JUL 18 2020	ISSUED FOR CONSTRUCTION					
0	JUN 25 2020	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

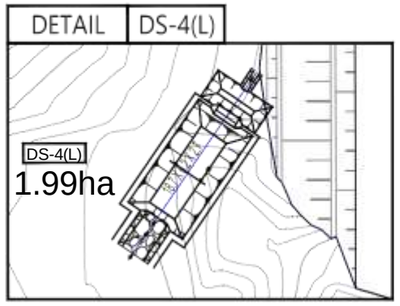
TITLE: (ACCESS ROAD LOT 2-2) SEDIMENT BASIN PLAN

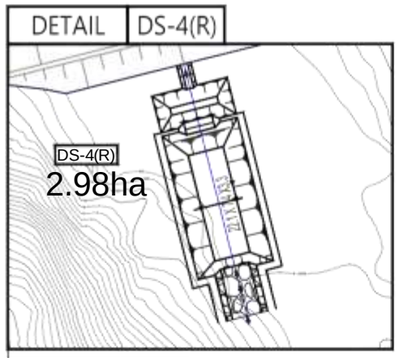
DATE	SCALE	DRAWING NO.	REV.
NOV 10, 2021	AS SHOWN	E-PR-CVR3-D2-84100	3

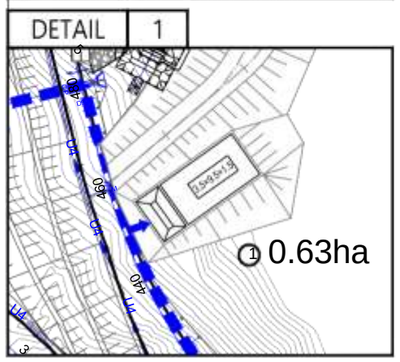
A3 (420X297mm)

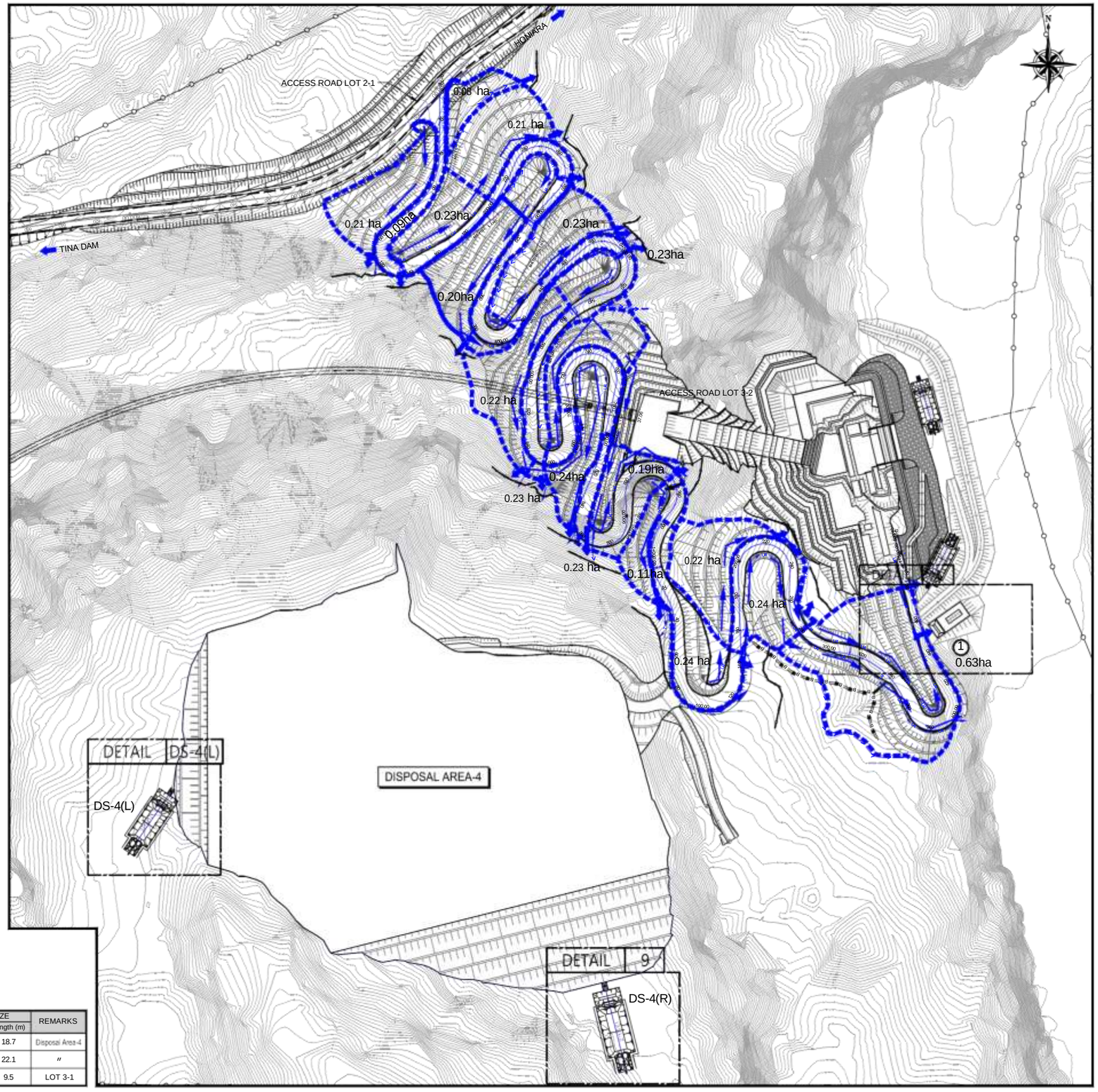
(ACCESS ROAD LOT 3-1) SEDIMENT BASIN PLAN

AS SHOWN









LEGEND

	Water Flow
	Sediment Basin
	Silt Fence

SEDIMENT BASIN

NO.	LOCATION	CATCHMENT AREA(ha)	20-year flow (Q20,m³/s)	SEDIMENT BASIN SIZE			REMARKS
				Depth (m)	Width (m)	Length (m)	
DS-4(L)	1+080.00	1.99	0.447	2.7	6.2	18.7	Disposal Area-4
DS-4(R)	1+080.00	2.98	0.643	3.3	7.4	22.1	〃
1	1+490.00	0.63	0.221	1.5	3.5	9.5	LOT 3-1

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
3	NOV 10 2021	ISSUED FOR CONSTRUCTION					
2	SEP 21 2020	ISSUED FOR CONSTRUCTION					
1	JUL 18 2020	ISSUED FOR CONSTRUCTION					
0	JUN 25 2020	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

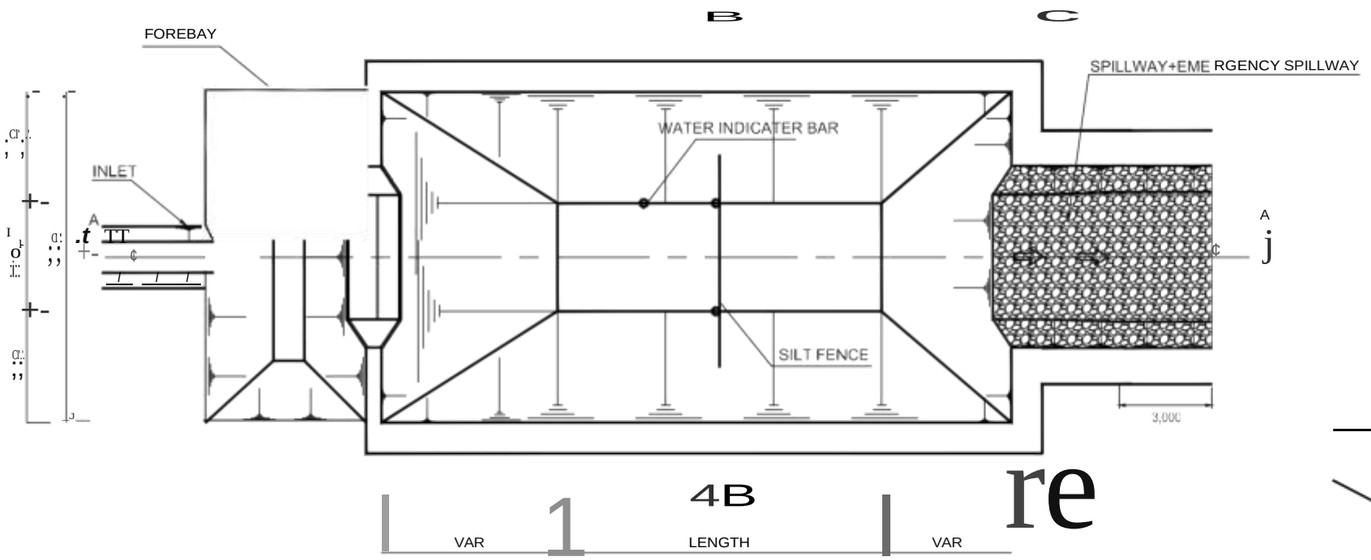
TITLE: (ACCESS ROAD LOT 3-1) SEDIMENT BASIN PLAN

DATE	SCALE	DRAWING NO.	REV.
NOV 10, 2021	AS SHOWN	E-PR-CVR5-D2-84080	3

A3 (420X297mm)

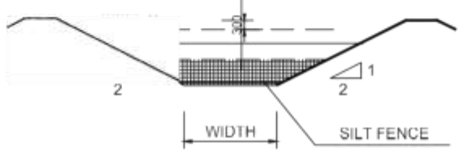
(ACCESS ROAD LOT 3-1) SEDIMENT BASIN DETAIL

S=NONE

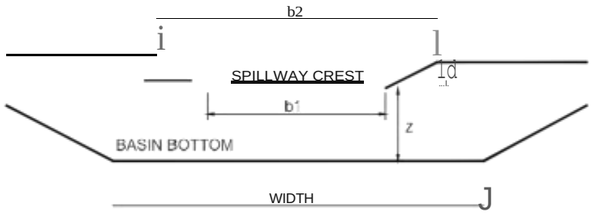


GEOTEXTILE SPECIFICATION

- Tensile Streangth at break?. 50kn/m(EN ISO 10319)
- Weathering Resistance ?. 40%(EN ISO 12224)
- Apparent opening size ⊂⊂; 0.4mm(EN ISO 12956)



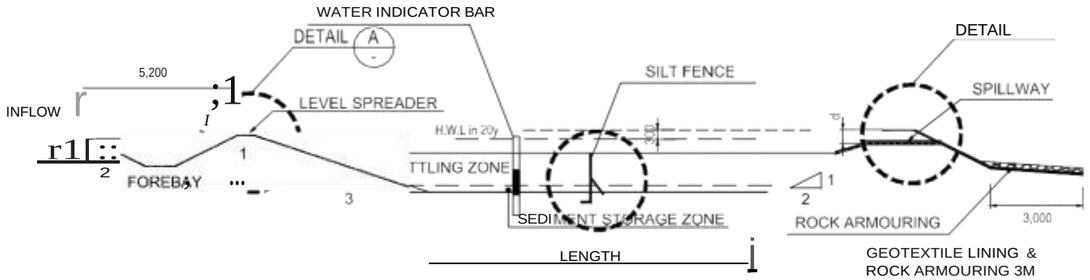
SECTION B - B



SECTION C-C

NOTES

1. DE-WATERING SYSTEM IS NOT MANDATORY FOR TYPE-B SEDIMENT BASIN GENERALLY. HOWEVER PUMP OR SIPHON CAN BE USED FOR DE-WATERING SYSTEM.
2. PRIOR TO THE DISCHARGE OF WATER FROM A SEDIMENT BASIN, SPECIFIED WATER QUALITY OBJECTIVES ARE WATER PH AND NTU.
   ACCORDING TO IECA AUSTRALIA, DISCHARGE WATER QUALITY STANDARD ARE :
   1) WATER PH IN THE RANGE 6.5 TO 8.5
   2) 90 PERCENTILE NTU READING NOT EXCEEDING 100, AND 50 PERCENTILE NTU READING NOT EXCEEDING 60.
3. DE-SILTING MARKER POST SHALL BE INSTALLED IN THE BASIN TO INDICATE THE TOP OF THE SEDIMENT STORAGE ZONE. THE BASIN SHALL BE DE-SILTED IF THE NEXT STORM IS LIKELY TO CAUSE THE SETTLED SEDIMENT TO RISE ABOVE THE MARKER POINT, OR IF THE SETTLES SEDIMENT HAS EXCEEDED 90% OF THE NOMINATED SEDIMENT STORAGE VOLUME.
4. REMOVED SEDIMENT WILL BE MOVED TO AN ADJACENT SPOIL DISPOSAL SITE TO DRY, AND DISPOSAL FINALLY.
5. THE SHAPE OF THE THE SEDIMENT BASINS CAN BE CHANGED IN CONSULTATION WITH THE EMPLOYER DEPENDING ON THE SITE CONDITIONS.
6. IN WATER INDICATOR, THE RED COLOR OF THE WATER LEVEL SIGN INDICATES THE LIMIT HEIGHT OF THE SEDIMENT CLEANING (0.3xZ) AND THE BLUE COLOR INDICATES THE LIMIT HEIGHT OF THE DEWATERING(0.5xZ).*



GEOTEXTILE LINING & ROCK ARMOURING 3M AWAY FROM TOE OF FILL.

SECTION A-A

SPECIFICATION OF SEDIMENT BASIN

(UNITS: m)

Catchment No.	Location	SEDIMENT BASIN DIMENSIONS				SPILLWAY DIMENSIONS			
		Depth (m)	Width (m)	Length (m)	Volume (㎥)	Z (m)	b1 (m)	b2 (m)	d (m)
01	1+490.00	1.5	3.5	9.5	146.1	1.50	1.75	2.65	0.45

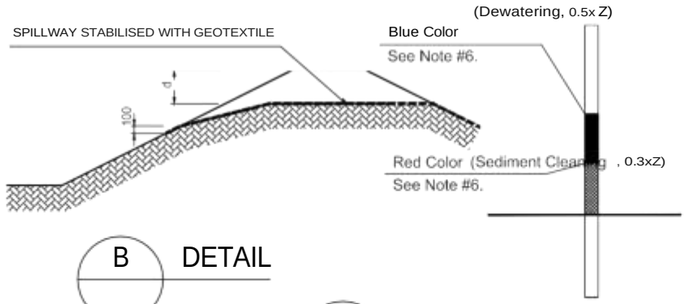
LEVEL SPREADER

THE LEVEL SPREADER ON THE FOREBAY IS TO BE CONSTRUCTED 100MM ABOVE THE OUTLET SPILLWAY

NON_-WOVERN GEOTEXTILE SPECIFICATION

- Tensile Streangth 17kn/m(ASTM D 4595)
- Tensile Elongation 50%(ASTM D 4595)
- Tear Streangth 400N(ASTM D 4533)
- Apparent opening size 0.1mm(ASTM D 4751)

A DETAIL



B DETAIL

C WATER INDICATOR BAR

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
D.							
D.							
fil	NOV10 2021	ISSUED FOR CONSTRUCTION					
fil	JUL18 2020	ISSUED FOR CONSTRUCTION					
	J 26	ISSUED FOR CONSTRUCTION					

CLIENT: TinaHydro)Xlwer Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYDRAI

DESIGNER: Dongju Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

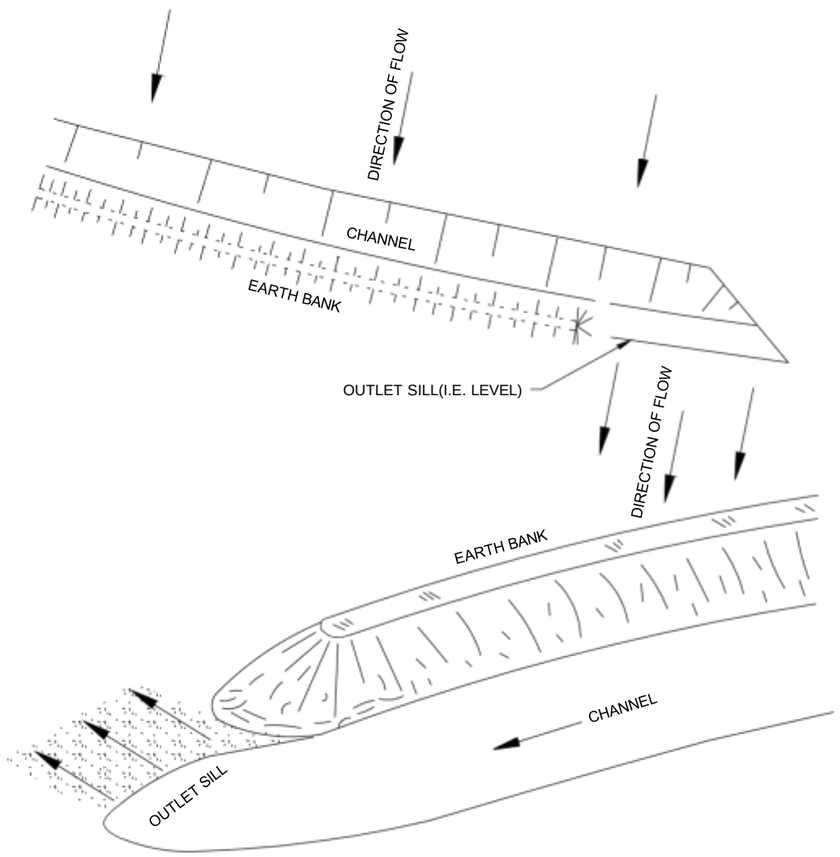
TITLE: (ACCESS ROAD LOT 3-1) SEDIMENT BASIN DETAIL

DATE	SCALE	DRAWING NO
NOV 10, 2021	S=NONE	E-PR-CVR5-D2-84090

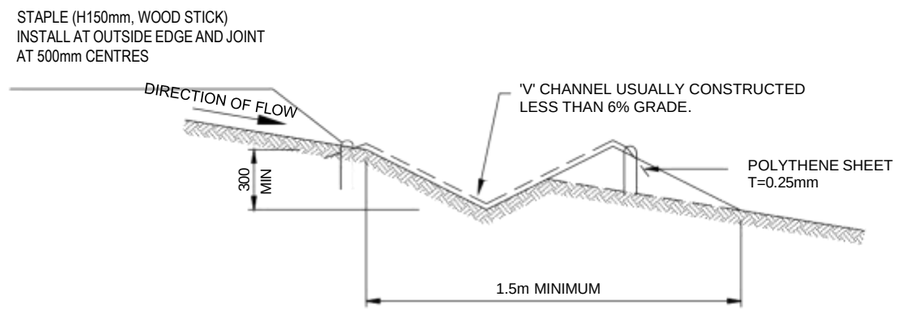
A3 (420X297mm)

(ACCESS ROAD) CLEAN WATER DIVERSION CHANNEL&OUTLET SPREADING SYSTEMS

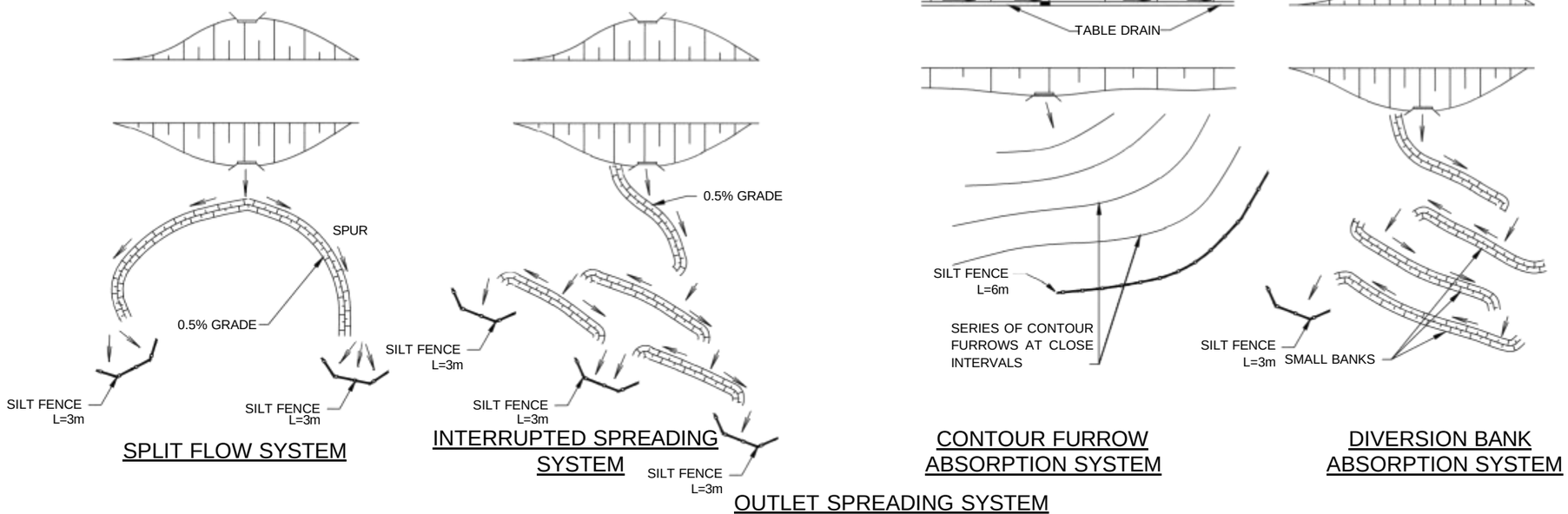
S=NONE



DIVERSION BANK AND SPREADING SYSTEM OUTLET



TYPICAL CROSS SECTION OF TEMPORARY DIVERSION BANK



SPLIT FLOW SYSTEM

INTERRUPTED SPREADING SYSTEM

CONTOUR FURROW ABSORPTION SYSTEM

DIVERSION BANK ABSORPTION SYSTEM

OUTLET SPREADING SYSTEM

NOTES

TEMPORARY DIVERSION BANKS

1. LOCATIONS SELECTED CONTINUALLY DURING CONSTRUCTION TO PROTECT WORKS (I.E. MAY REGULARLY CHANGE).
2. CHANNEL PROTECTION TO BE CONSIDERED IF GREATER THAN 6% GRADE.
3. BANKS TO BE ADEQUATELY COMPACTED TO PREVENT FAILURE.
4. SPACING BETWEEN BANKS TO BE DEPENDENT UPON SLOPE AND SOIL TYPE.
5. OUTLETS FROM BANKS TO DISCHARGE ONTO A STABLE AREA (E.G. ROCKS, NATURAL UNDISTURBED GROUND, TIMBER WINDROW, SEDIMENT TRAP, GEOTEXTILE BATTER DRAIN). BANK OUTLETS TO EXTEND PAST DISTURBED AREAS AND NOT BE TOO SHORT.
6. BANKS TO BE INSPECTED AFTER STORM EVENTS AND REPAIRED AS REQUIRED.
7. ACCORDING TO MONITORING RESULT, THE LENGTH, HEIGHT AND CAPACITY OF SILT FENCE MIGHT BE LONGER AND INCREASED THAN TYPICAL DRAWING.

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
0	NOV.10 2021	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

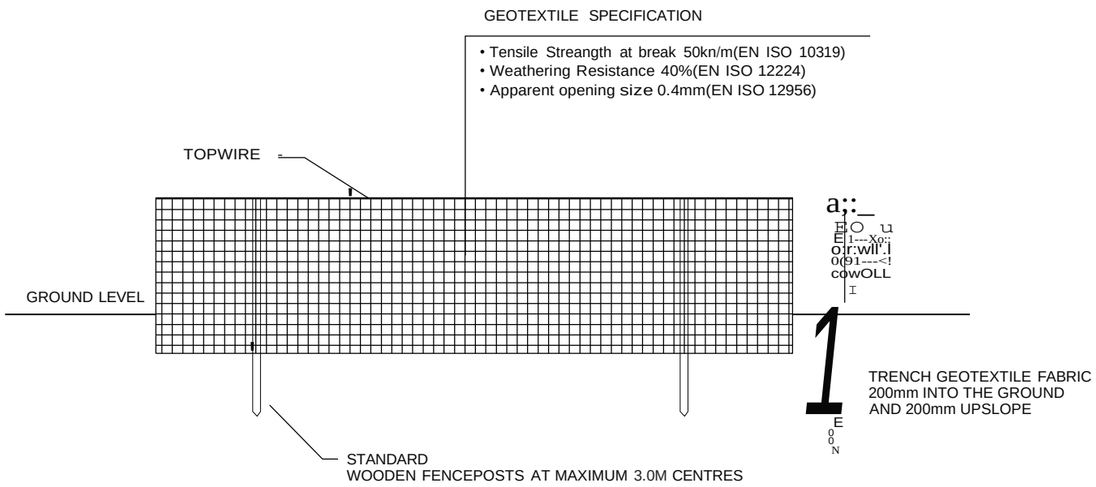
TITLE: (ACCESS ROAD) CLEAN WATER DIVERSION CHANNEL &OUTLET SPREADING SYSTEMS

DATE	SCALE	DRAWING NO.	REV.
NOV 10, 2021	S=NONE	E-PR-CVR2-D2-84110	0

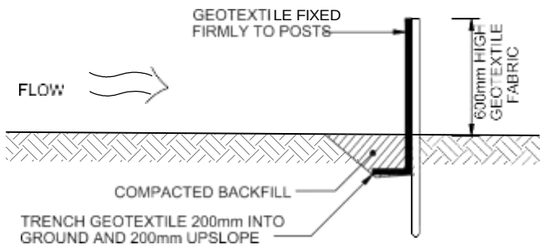
A3 (420X297mm)

(ACCESS ROAD) SILT FENCE

S=NONE

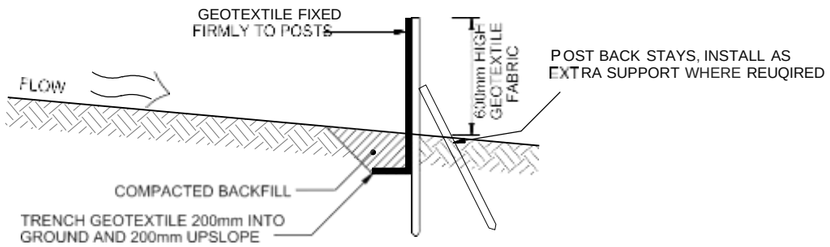


ELEVATION - SILT FENCE



TYPICAL SECTION -SILT FENCE

at sediment basin



TYPICAL SECTION - SILT FENCE

except for sediment basin

SILT FENCE DESIGN CRITERIA TABLE

Slope steepness	Slope length(m)(maximum)	Spacing of returns(m)	Silt fence length(m) (maximum)
Flatter than 2%	Unlimited	N/A	Unlimited
2-10%	40	60	300
10-20%	30	50	230
20-33%	20	40	150
33-50%	15	30	75
>50%	6	20	40

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
D.							
D.							
D.							
D.							
/'- L.!DI.	NOV10 2021	ISSUED FOR CONSTRUCTION					

CLIENT: TinaHydro)Xlwer Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYUNDAI

DESIGNER: Dongfu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

TITLE: (ACCESS ROAD) SILT FENCE

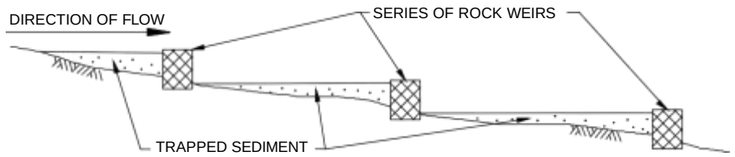
DATE	SCALE	DRAWING NO
NOV 10, 2021	S=NONE	E-PR-CVR2-D2-84120

0

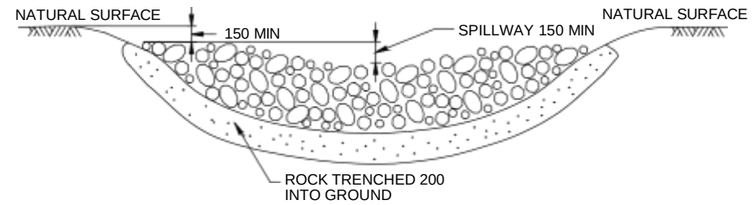
A3 (420X297mm)

(ACCESS ROAD) ROCK WEIR (CHECK DAM) OR IN-STREAM WEIRS

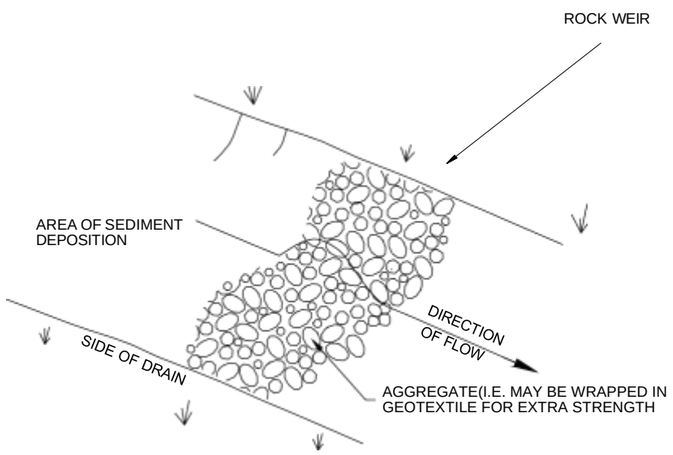
S=NONE



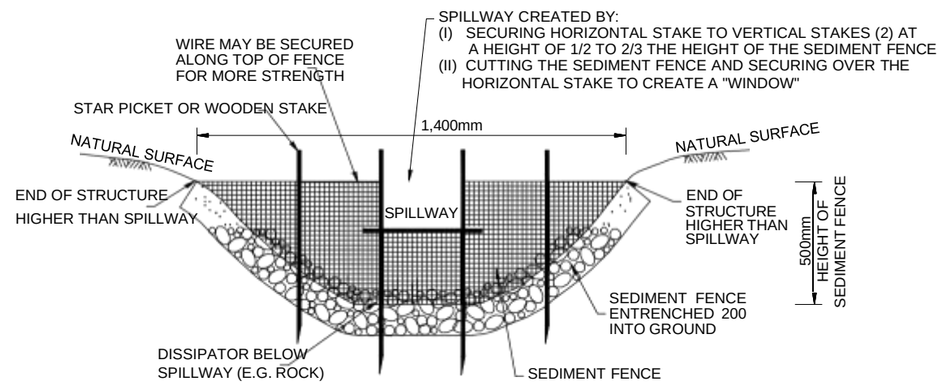
SERIES OF ROCK WEIRS



CROSS SECTION OF AGGREGATE ROCK WEIR



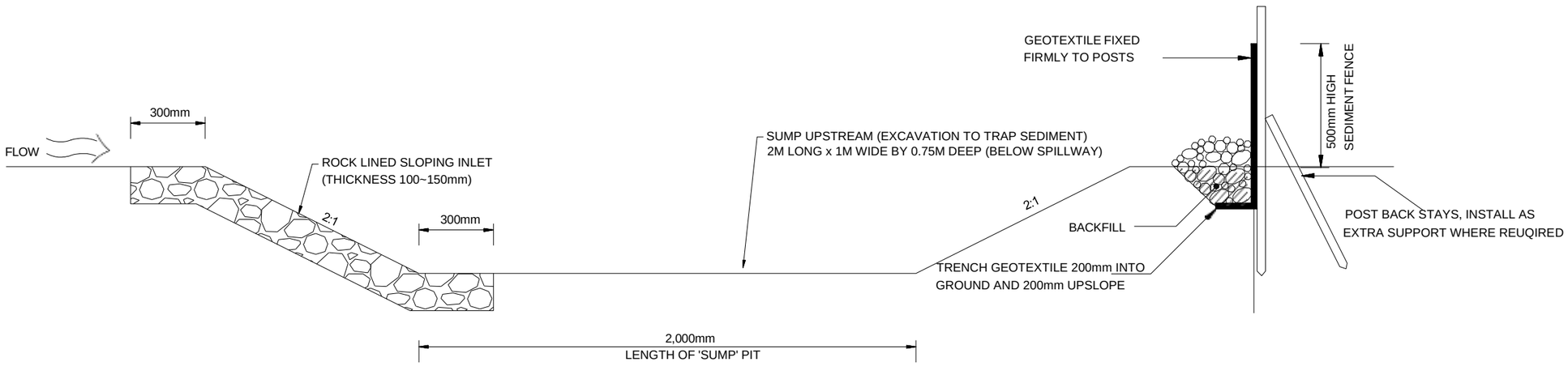
AGGREGATE ROCK WEIR



CROSS SECTION OF SEDIMENT FENCE ROCK WEIR

NOTES

1. ROCK WEIRS MAY BE CONSTRUCTED OF A VARIETY OF MATERIALS (E.G. STRAW BALES, SEDIMENT FENCE, ROCK & GEOTEXTILE, SHEET PILING, ETC.)
2. ROCK WEIRS TO BE TRENCHED 200 mm INTO GROUND SURFACE ON BASE AND SIDES AND SECURELY BACKFILLED.
3. SPILLWAY TO BE OVER INVERT OF DRAIN WITH DISCHARGE NOT PERMITTED TO FLOW AROUND ENDS.
4. SPILLWAY TO BE LESS THAN 1 METER ABOVE INVERT OF DRAIN.
5. SOME FORM OF DISSIPATION MAY BE REQUIRED BELOW SPILLWAYS OF ROCK WEIRS (E.G. ROCK, SAND BAGS).
6. ROCK WEIRS TO BE INSPECTED AFTER STORM EVENTS AND REPAIRED AS REQUIRED.
7. THE ROCK WEIRS MAY BE PLACED IN SERIES DOWN THE CHANNEL AND USED DURING CONSTRUCTION TO REDUCE THE VELOCITY IN DITCHES OR CHANNELS
8. AGGREGATE SIZE :100mm ~ 150mm
9. SILT FENCE ROCK WEIR WILL BE INSTALLED AS A LAST STRUCTURE PLACED ON CONCENTRATED FLOWS FROM SITE DRAINAGE CHANNEL, OR WHERE FLOWS MAY EXIT THE SITE AND ENTER NATURAL FLOW PATH
10. THE ROCK WEIR WILL BE SPACED AT 20M INTERVAL UNDER 15% FLOW DEGREE, AND 15M INTERVAL IN STEEPER AREA MORE THAN 15% WITH TEMPORARY ROCK LINING(BOTTOM).



SEDIMENT FENCE ROCK WEIR

REV.	DATE	DESCRIPTION	DRAWN	DESIGN	CHECK	REVIEW	APRVD
0	NOV 10 2021	ISSUED FOR CONSTRUCTION					

CLIENT: Tina Hydropower Limited

OWNER'S ENGINEER: Stantec

CONTRACTOR: HYDROSEL

DESIGNER: Dongbu Engineering

SUB-CONTRACTOR:

PROJ. NAME: TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

TITLE: (ACCESS ROAD) ROCK WEIR (CHECK DAM) OR IN-STREAM WEIRS

DATE	SCALE	DRAWING NO.	REV.
NOV 10 2021	S=NONE	E-PR-CVR2-D2-84130	0

A3 (420X297mm)

PROJECT : TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

CALCULATION
(ACCESS ROAD LOT 1)

DOCUMENT No.: E-PR-CVGD-C2-00010

EMPLOYER : TINA HYDROPOWER LIMITED (THL)

EPC CONTRACTOR : HYUNDAI ENGINEERING CO., LTD. (HEC)

ISSUE STATUS

1	17-JUL-2020	ISSUED FOR CONSTRUCTION	H.K CHO		H.K YU		T.H. KIM		J.K. LEE	
0	10-APR-2020	ISSUED FOR CONSTRUCTION	H.K CHO		H.K YU		T.H. KIM		J.K. LEE	
REV. No.	DATE	DESCRIPTION	PREPARED		CHECKED		REVIEWED		APPROVED	

Tina River Hydropower Development P oject (TRHDP)

CALCULATION
(ACCESS ROAD LOT 1)

17 - JUL -2020

Tina Hydropower Limited

HYUNDAI ENGINEERING CO., LTD

Content

1. BOX CULVERT

(LOT 1)

1.1 Box Culvert 1 (STA.0+012.00)

2@1.5×0.90

FH=0.42 m [SI UNIT]

1.1.1 Design Conditions

1) General Items

(1) Type of Culvert : 2 Box

(2) Width (w) : 2 @ 1.5 m

(3) Height (h) : 0.90 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength : fc' = 32 MPa

¤ Modulus of Elasticity : Ec = 26587 MPa

(2) Reinforcement bar

⊳ Yield Strength : fy = 420 MPa

⊲ Modulus of Elasticity : Es = 200000 MPa

3) Material weight

(1) Reinforced Concrete : ωc = 25.00 kN/m³

(2) plain concrete : γcn = 23.50 kN/m³

(3) Pavement : γasp = 23.00 kN/m³

(4) Subterranean : γw = 10.00 kN/m³

4) Soil

(1) Wet Unit Weight : γt = 19.00 kN/m³

(2) Submerged Unit Weight : γsub = 10.00 kN/m³

(3) angle of internal friction : φ = 28.00 °

(4) coefficient of earth pressure atrest : Ko = 1-sinΦ = 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

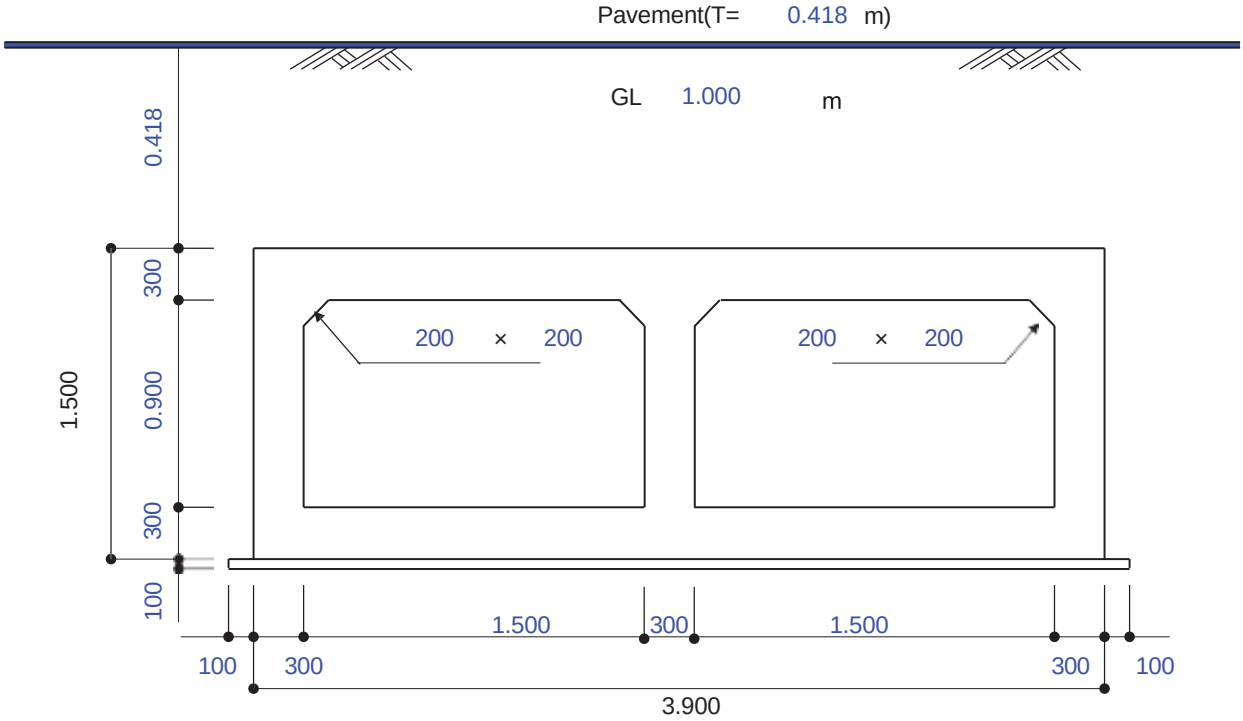
· SAP2000 (Structure Analysis Program)

8) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.1.2 Section Assumption



1.1.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.418 × 3.900 × 23.0	37.495
Vertical earth pressure (EV)	No exist ground water	0.000 × 3.900 × 19.0	0.000
	Exist ground water	(0.000 × 19.0 + 0.000 × 10.0) × 3.900	0.000
Ground Water(WA')		0.000 × 3.900 × 10.0	0.000
Sub Total		Surcharge Load for Bouyancy Check	37.495
Slab(DC)	Top	0.300 × 3.900 × 25.0	29.250
	Bottom	0.300 × 3.900 × 25.0	29.250
Wall(DC)	Left	0.300 × 0.900 × 25.0	6.750
	Right	0.300 × 0.900 × 25.0	6.750
	Inner	0.300 × 0.900 × 25.0	6.750
Hunch(DC)		0.200 × 0.200 / 2 × 25.0 × 4 EA	2.000
Sub Total		Surcharge Load for Bouyancy Check	80.750

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.000 m)

- Total Load for Bouyancy Check : 118.245 kN
- Uplife force : 3.900 ×(1.918 - 1.000)× 10.0 kN/□ = 35.802 kN
- Safety factor = 1.25

□ F.S = 118.245 / 35.802 = 3.303 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL- 0.000 m)

- Total Load for Bouyancy Check :

80.750 +(0.418 × 3.900 × 10.000 kN/□) = 97.052 kN

- Uplife force : 3.900 × 1.500 × 10.000 kN/□ = 58.500 kN
- Safety factor = 1.1

□ F.S = 97.052 / 58.500 = 1.659 > 1.1 - O.K

Ā Securing safety at all ground water levels

3) Allowable vertical bearing capacity check

(1) Load

- Dead load

  - Self weight of Structure = 80.750 / 3.900 = 20.705 kN/m²

  - Vertical earth pressure = 37.495 / 3.900 = 9.614 kN/m² (No exist ground water)

- Live load = 79.702 kN/m² (Refer to 1.1.4.2)

- Water load in Culvrt = 0.900 ɤ 10.000 = 9.000 kN/m²

(2) Allowable vertical bearing capacity

- Q_{max} = 119.021 kN/m²

- Q_a = 291.670 kN/m² (Refer to Geotechnic Report)

□ Q_a = 291.670 kN/□ > Q_{max} = 119.021 kN/□ - O.K

1.1.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

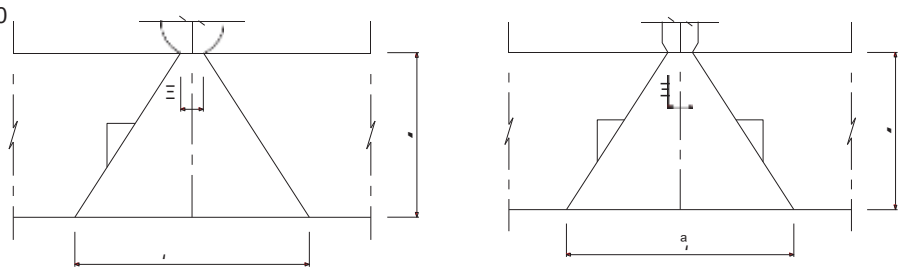
TYPE	Depth (m)	Unit weight (kN/ᵤ)	Load (kN/m²)	
Pavement	0.418	23.000	1.000 × 0.418 × 23.000 =	9.614
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
□	0.418		Psv =	9.614 kN/m²

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/ᵤ)	Load (kN/m²)	
Pavement	0.418	23.000	1.000 × 0.418 × 23.000 =	9.614
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
	0.000	10.000	1.000 × 0.000 × 10.000 =	0.000
□	0.418		Psvh =	9.614 kN/m²

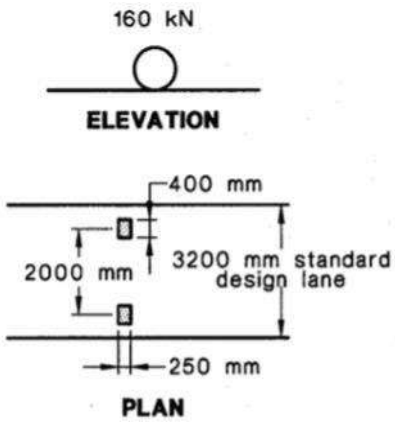
2) Live Load

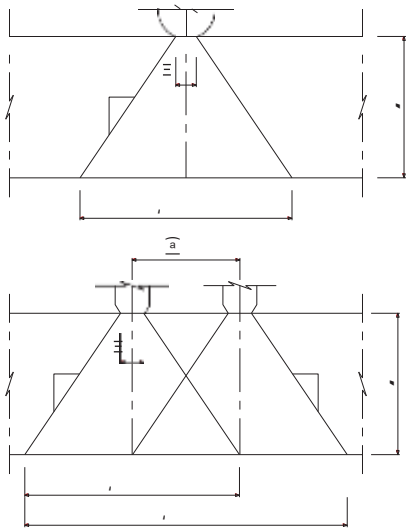
(1) W80



$$P_{vl} = \frac{80}{(0.25+2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.836) \times (0.4 + 0.836)} = 59.599 \text{ kN/m}^2$$

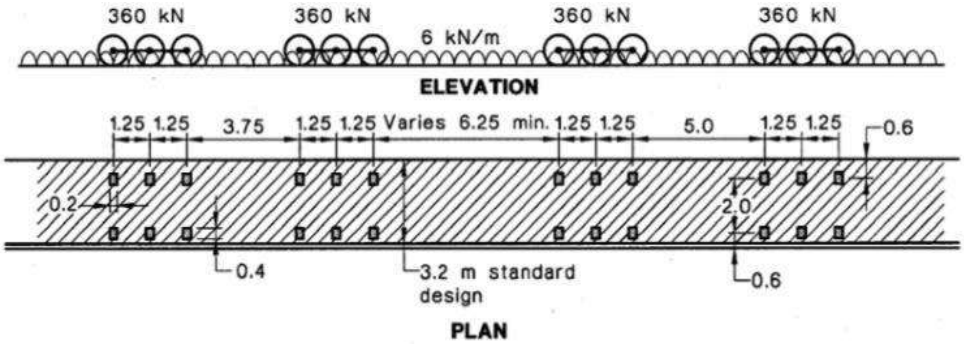
(2) A160



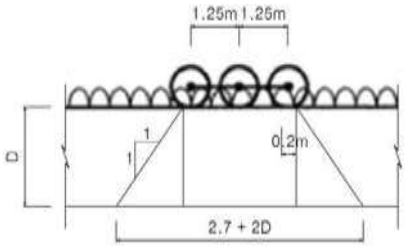


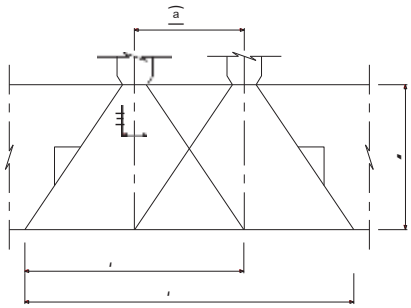
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.836) \times (0.4 + 0.836)} = 59.599 \text{ kN/m}^2$$

(3) M1600



- Axle group



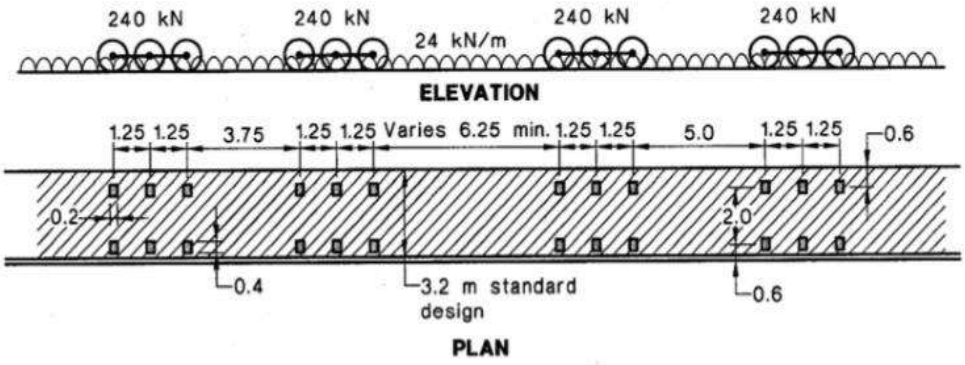


$$P_{v1} = \frac{60}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{60}{(0.2 + 0.836) \times (0.4 + 0.836)} = 46.857 \text{ kN/m}^2$$

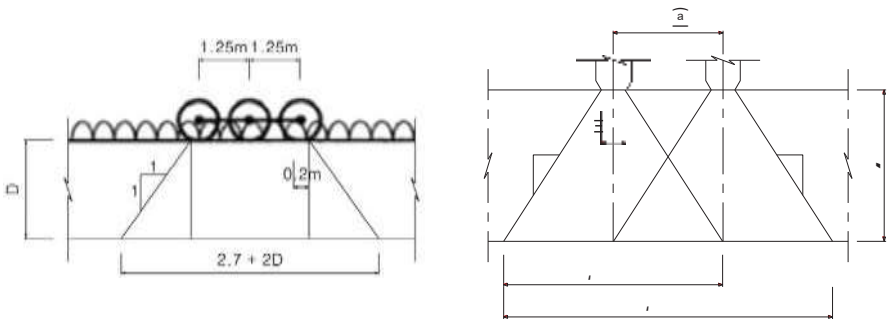
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 46.857 + 1.875 = 48.732 kN/m²

(4) S1600



- Axle group

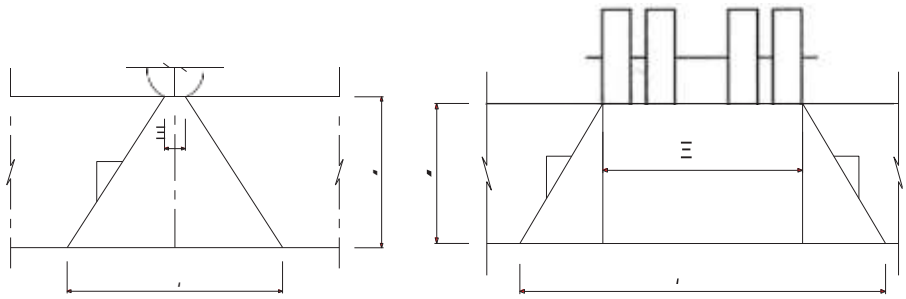


$$P_{v1} = \frac{40}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{40}{(0.2 + 0.836) \times (0.4 + 0.836)} = 31.238 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 31.238 + 7.500 = 38.738 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2\times0.418) \times (1.4 + 2\times0.418)} = 53.961 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	59.599	0.34	79.702
A160	59.599	0.34	79.702
M1600	48.732	0.26	61.314
S1600	38.738	0.00	38.738
HLP	53.961	0.10	59.357

□ P_{vl} = 79.702 kN/m² = 79.702 kN/m²

(7) Live Load Surcharge

□ P_{vlh} = 79.702 kN/m² × 0.500 = 39.851 kN/m²

3) Lateral Earth Pressure

↳ coefficient of earth pressure at rest : Ko = 1 - sin30 = 0.500

- No exist ground water

P_{sh} = k_o × γ_t × H

P_{sh1} = 0.500 x (23 x 0.418 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000) = 4.807 kN/m²

P_{sh2} = 4.807 + 0.500 × 19.0 × 0.150 = 6.232 kN/m²

P_{sh3} = 6.232 + 0.500 × 19.0 × 0.350 = 9.557 kN/m²

P_{sh4} = 9.557 + 0.500 × 19.0 × 0.350 = 12.882 kN/m²

P_{sh5} = 12.882 + 0.500 × 19.0 × 0.350 = 16.207 kN/m²

P_{sh6} = 16.207 + 0.500 × 19.0 × 0.150 = 17.632 kN/m²

- Exist ground water

$P_{sh'}$ = k_o × (γ_t × H_1 + γ_{sub} × H_2)

$P_{sh1'}$ = 0.500 x (23 x 0.418 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000 + 10 x 0.000) = 4.807 kN/m²

$P_{sh2'}$ = 4.807 + 0.500 × 19.0 × 0.150 = 6.232 kN/m²

$P_{sh3'}$ = 6.232 + 0.500 × 19.0 × 0.350 = 9.557 kN/m²

$P_{sh4'}$ = 9.557 + 0.500 × 10.0 × 0.350 = 11.307 kN/m²

$P_{sh5'}$ = 11.307 + 0.500 × 10.0 × 0.350 = 13.057 kN/m²

$P_{sh6'}$ = 13.057 + 0.500 × 10.0 × 0.150 = 13.807 kN/m²

4) Ground Water Load

1) Horizontal ground Water Pressure

$P_{wh} = \gamma_w \times H_2$

P_{wh1} = 10.0 × 0.000 = 0.000 kN/m²

P_{wh2} = 0.000 + 0.0 × 0.150 = 0.000 kN/m²

P_{wh3} = 0.000 + 0.0 × 0.350 = 0.000 kN/m²

P_{wh4} = 0.000 + 10.0 × 0.350 = 3.500 kN/m²

P_{wh5} = 3.500 + 10.0 × 0.350 = 7.000 kN/m²

P_{wh6} = 7.000 + 10.0 × 0.150 = 8.500 kN/m²

2) Vertical ground Water Pressure

- Top Slab : P_{wv1} = 10.0 × 0.000 = 0.000 kN/m²

-Bottom Slab : P_{wv2} = 10.0 × 1.500 = 15.000 kN/m²

5) Load Combination

(1) Ultimate Load

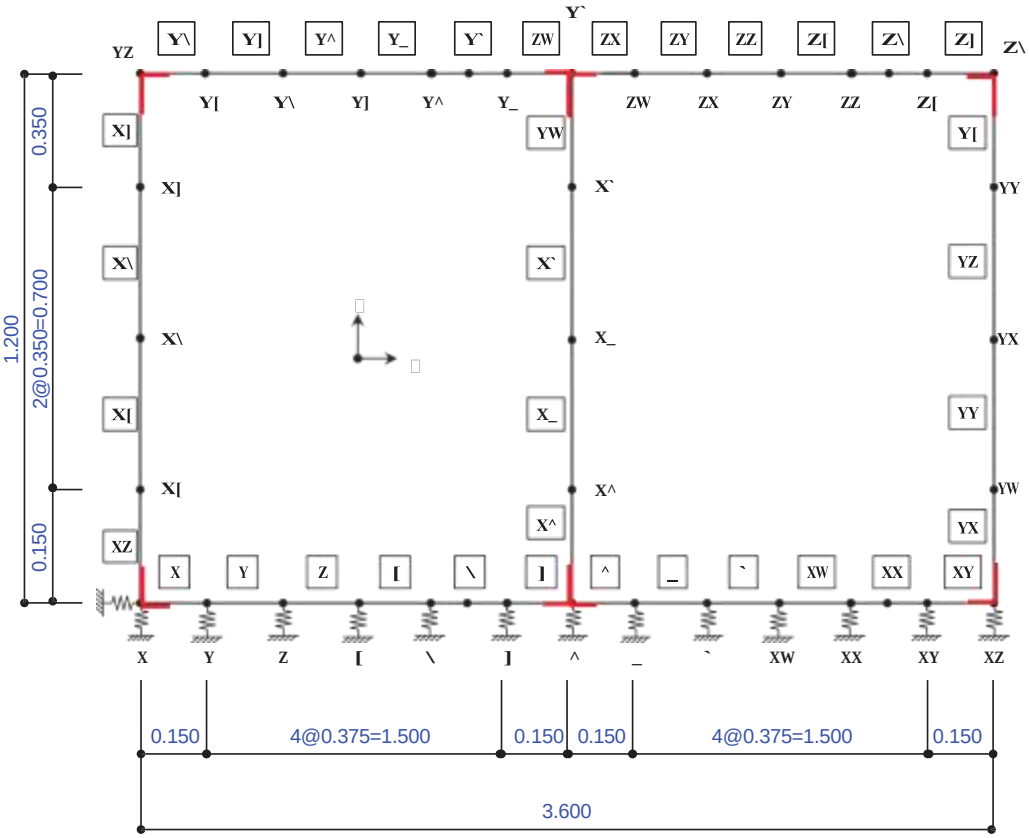
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.60		1.60	1.60	1.60			
COMB 3	1.20	1.60		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.60	1.60	1.60		1.60	1.60	1.60
COMB 7	1.20		1.60	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	1.000		1.00	1.00	1.00			
SCOMB 2	1.00	1.000		1.00	1.00	0.56			
SCOMB 3	1.00	1.000				0.00			
SCOMB 4	1.00		1.000	1.00	1.00		1.00	1.00	1.000
SCOMB 5	1.00		1.000	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		1.000				0.00	0.00	0.00

1.1.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	Z	Section	Node	X	Z	Section
1	0.150	0.150	Bottom Slab	19	1.950	1.000	Middle Wall
2	0.300	0.150		20	3.750	0.300	Right Wall
3	0.675	0.150		21	3.750	0.650	
4	1.050	0.150		22	3.750	1.000	
5	1.425	0.150		23	0.150	1.350	Top Slab
6	1.800	0.150		24	0.500	1.350	
7	1.950	0.150		25	0.775	1.350	
8	2.100	0.150		26	1.050	1.350	
9	2.475	0.150		27	1.325	1.350	
10	2.850	0.150		28	1.600	1.350	
11	3.225	0.150		29	1.950	1.350	
12	3.600	0.150		30	2.300	1.350	
13	3.750	0.150		31	2.575	1.350	
14	0.150	0.300	Left Wall	32	2.850	1.350	
15	0.150	0.650		33	3.125	1.350	
16	0.150	1.000		34	3.400	1.350	
17	1.950	0.300	Middle Wall	35	3.750	1.350	
18	1.950	0.650					

(2) Section

NO.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~5 , 8~11	Bottom Slab
2	0.300	1.000	0.300	0.002250	14~15	Left Wall
3	0.300	1.000	0.300	0.002250	18~19	Middle Wall
4	0.300	1.000	0.300	0.002250	22~23	Right Wall
5	0.300	1.000	0.300	0.002250	26~29 , 32~35	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

kvo = 1/0.3 ×α×Eo

Eo : the modulus of subgrade elasticity (kN/m²)

α : correction factor for calculating Eo

Eo = 7000 kN/m² (Refer to Geotechnic Report)

α = 4

Kvo = 1/0.3 × α × Eo = 1/0.3 × 4 × 7000 = 93333.333 kN/

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{3.90 \times 3.90} = 3.900$ m²

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$= 93333.333 \times (3.900 / 0.3)^{-3/4} = 13632.6$ kN/

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 13	13632.600	0.2250	1.0000	0.2250	3067.3
2, 12	13632.600	0.2625	1.0000	0.2625	3578.6
3~5, 9~11	13632.600	0.3750	1.0000	0.3750	5112.2
6, 8	13632.600	0.2625	1.0000	0.2625	3578.6
7	13632.600	0.1500	1.0000	0.1500	2044.9

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

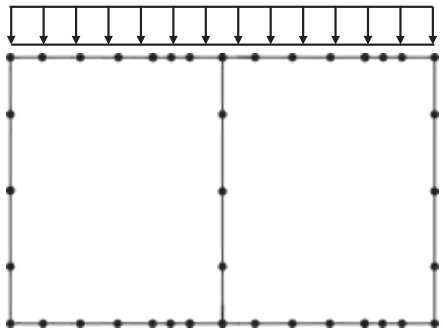
3) Loading

(1) LOAD-1 : Self weight - Automatic consideration in program

(2) LOAD-2,3 : Vertical earth pressure

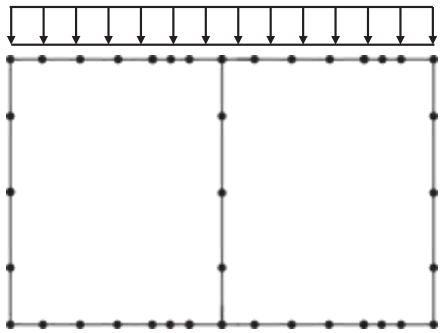
Psvh= 9.614 kN/m² (Exist ground water)

Psv= 9.614 kN/m² (No exist ground water)



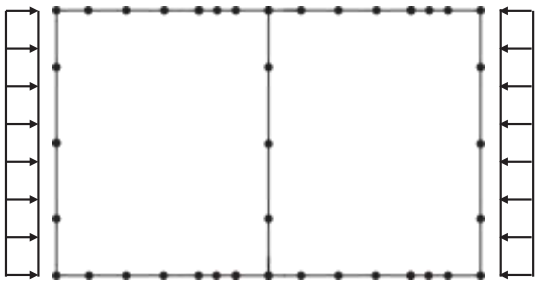
(3) LOAD-4 : Live Load

Pvl= 79.702 kN/m²



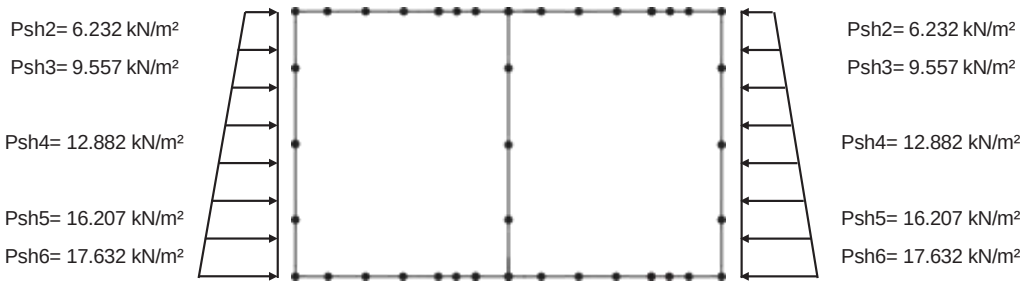
(4) LOAD-5 : Live Load Surcharge

Pvlh= 39.851 kN/m²

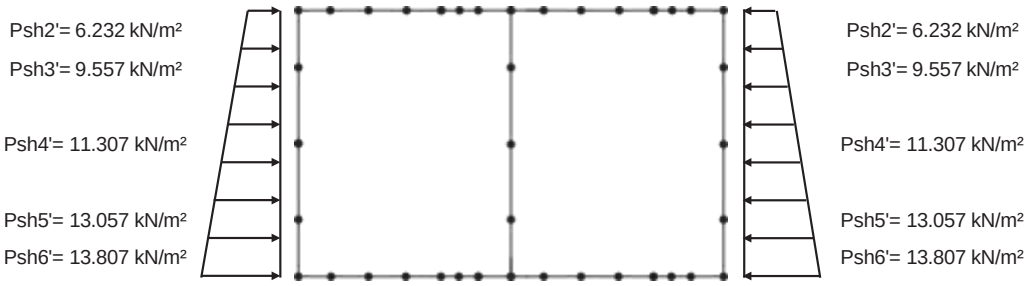


Pvlh= 39.851 kN/m²

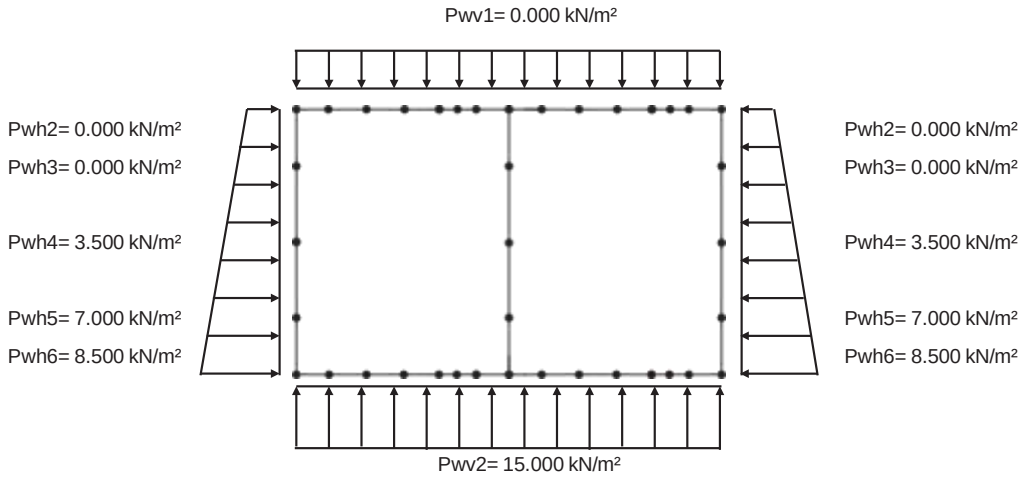
(5) LOAD-6 : Horizontal Earth Pressure (No Ground Water)



(6) LOAD-7 : Horizontal Earth Pressure (Ground Water)

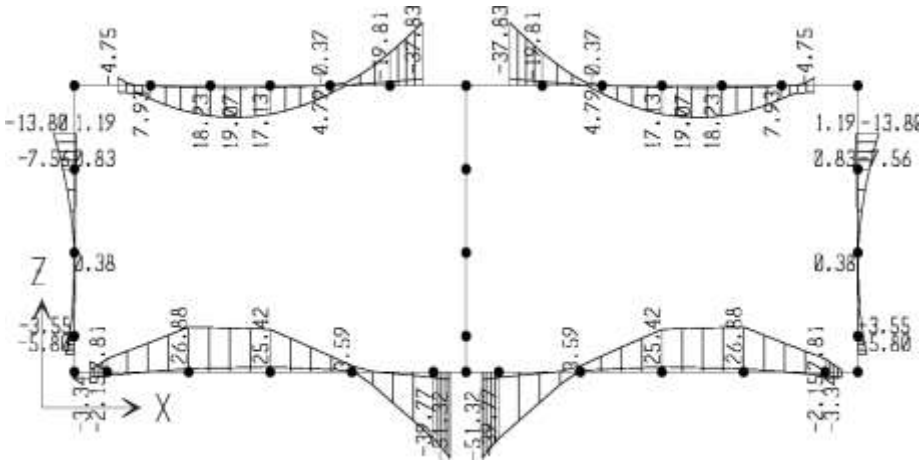


(7) LOAD-8 : Ground Water Pressure

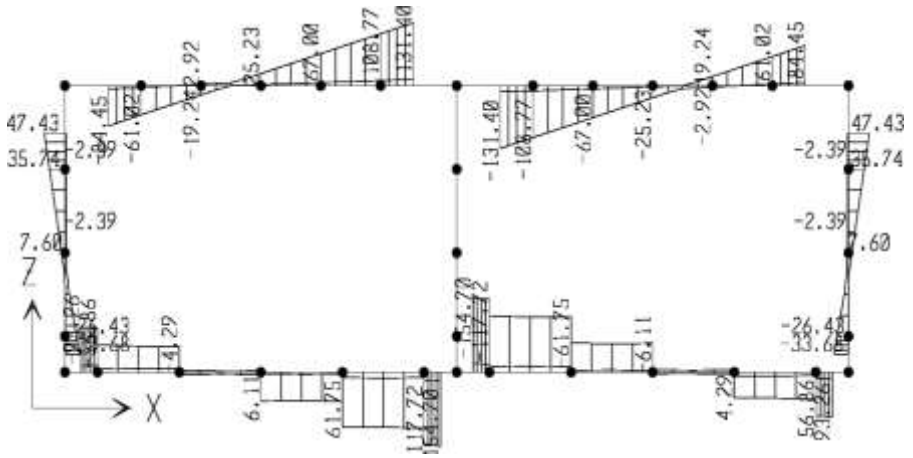


1.1.6 Summary of Analysis Results

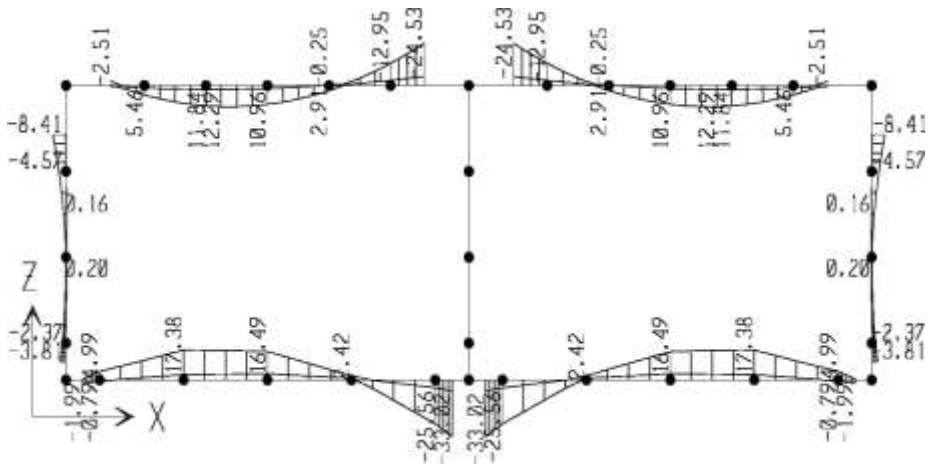
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	ØMn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	4.754	84.452	2.513	367	310.5	74.556	D13 @ 200	15.68
	Middle(+)	19.071	0.000	12.290	300	213.5	50.907	D13 @ 200	2.67
	End of Middle Wall(-)	37.824	131.395	24.522	367	310.5	74.556	D13 @ 200	1.97
Wall	Top(-)	13.801	47.429	8.412	367	310.5	74.556	D13 @ 200	5.40
	Middle(+)	0.836	0.000	0.201	300	213.5	25.740	D13 @ 400	30.79
	Middle(-)	2.528	0.000	1.506	300	243.5	29.397	D13 @ 400	11.63
	Bottom(-)	5.802	26.424	3.808	300	243.5	58.221	D13 @ 200	10.03
Middle Wall	Top & Bottom(-)	0.000	0.000	0.000	300	213.5	25.740	D13 @ 400	-
Bottom Slab	End of the point(-)	3.342	56.890	1.989	300	243.5	58.221	D13 @ 200	17.42
	Middle(+)	26.896	0.000	17.388	300	213.5	50.907	D13 @ 200	1.89
	End of Middle Wall(-)	51.342	117.760	33.033	300	243.5	73.622	D13+D16 @ 400	1.43

1.1.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm			
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm			
M_u	=	4.754	kN·m	V_u	=	84.452	kN	M_o	=		2.513	kN·m			

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 40.546 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎡ (5 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	7715.9	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1045.5	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00017	㎡	$A_{s,4/3req}$	=	54.1	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00017	㎡	$A_{s,min}$	=	54.1	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00208	㎡	$A_{s,min}$	=	645.0	㎡

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 74.556 kN·m > Mu = 4.754 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 84.452 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 2.513 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 0.332 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 2.513 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 13.287 MPa

fst = fs x (H - d' - χ) / (d - χ) = 13 x (367 - 57 - 0) / (311 - 52) = 13.29 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 13.29) - 2.5 x 50.00 = 7882.99 ㎜

300 x (280 / fs) = 300 x (280 / 13.29) = 6322.10 ㎜

Sa = 6322.10 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (6322.10 mm) Ā O.K

2) Top Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	19.071	kN·m	V_u	=	0.000	kN	M_o	=		12.290	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 238.366 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00149	㎟	$A_{s,4/3req}$	=	317.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00149	㎟	$A_{s,min}$	=	317.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 19.071 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 12.290 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 2.930 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 12.290 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 95.523 MPa

fst = fs x (H - d' - χ) / (d - χ) = 96 x (300 - 87 - 3) / (214 - 42) = 95.52 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 95.52) - 2.5 x 80.00 = 913.87 ㅡ

300 x (280 / fs) = 300 x (280 / 95.52) = 879.37 ㅡ

Sa = 879.37 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (879.37 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 3.900 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 3.9 / 28 × (0.43 + 420 / 700)

= 0.143 m < H = 0.300 m Ā O.K

3. Top Slab - At the end of middle Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm		
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm		
M_u	=	37.824	kN·m	V_u	=	131.395	kN	M_o	=		24.522	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 324.888 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00140	㎟	$A_{s,4/3req}$	=	433.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00140	㎟	$A_{s,min}$	=	433.2	㎟
P_{use}	=	A_s / (B·d)	=	0.00208	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 37.824 kN·m

Ā O.K

ⴰ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 131.395 kN

Ā No shear reinforcement is required

(2) Crack Check

ⴰ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 24.522 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 3.236 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 24.522 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 129.635 MPa

fst = fs x (H - d' - χ) / (d - χ) = 130 x (367 - 57 - 3) / (311 - 52) = 129.63 MPa

ⴰ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 129.63) - 2.5 x 50.00 = 695.77 ㎜

300 x (280 / fs) = 300 x (280 / 129.63) = 647.97 ㎜

Sa = 647.97 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (647.97 mm) Ā O.K

4. Wall - Top

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm		
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm		
M_u	=	13.801	kN·m	V_u	=	47.429	kN	M_o	=		8.412	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 117.936 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00051	㎟	$A_{s,4/3req}$	=	157.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00051	㎟	$A_{s,min}$	=	157.2	㎟
P_{use}	=	A_s / (B·d)	=	0.00208	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 13.801 kN·m

Ā O.K

ㅇ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 47.429 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅇ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 8.412 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 1.110 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 8.412 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 44.472 MPa

fst = fs x (H - d' - χ) / (d - χ) = 44 x (367 - 57 - 1) / (311 - 52) = 44.47 MPa

ㅇ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 44.47) - 2.5 x 50.00 = 2267.51 ㎜

300 x (280 / fs) = 300 x (280 / 44.47) = 1888.82 ㎜

Sa = 1888.82 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1888.82 mm) Ā O.K

5. Wall - Middle(In)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	0.836	kN·m	V_u	=	0.000	kN	M_o	=		0.201	kN·m	

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 5.726) / 5.726

= 0.1089

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 10.362 ㎠

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎠ (3 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00006	㎠	$A_{s,4/3req}$	=	13.8	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00006	㎠	$A_{s,min}$	=	13.8	㎠
P_{use}	=	A_s / (B·d)	=	0.00151	㎠	$A_{s,min}$	=	322.5	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

ØMn = 0.9 × As × fy × (d - a/2) = 25.740 kN·m > Mu = 0.836 kN·m

Ā O.K

ⱨ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⱨ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × {1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 322.50)]

= 30.711 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.201 / [1000 × 30.711 x (213.5 - 30.711 / 3)] × 10^6

= 0.064 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.201 / [322.500 x (213.5 - 30.711 / 3)] × 10^6

= 3.062 MPa

fst = fs x (H - d' - χ) / (d - χ) = 3 x (300 - 87 - 0) / (214 - 31) = 3.06 MPa

ⱨ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 3.06) - 2.5 x 80.00 = 3.5E+04 ⌟

300 x (280 / fs) = 300 x (280 / 3.06) = 2.7E+04 ⌟

Sa = 2.74E+04 ⌟ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (2.7E+04 mm) Ā O.K

6. Wall - Middle(Out)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	2.528	kN·m	V_u	=	0.000	kN	M_o	=		1.506	kN·m		

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 5.726) / 5.726

= 0.1246

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 27.494 ㎡

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎡ (3 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00015	㎡	$A_{s,4/3req}$	=	36.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00015	㎡	$A_{s,min}$	=	36.7	㎟
P_{use}	=	A_s / (B·d)	=	0.00132	㎡	$A_{s,min}$	=	322.5	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

ØMn = 0.9 × As × fy × (d - a/2) = 29.397 kN·m > Mu = 2.528 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 322.50)]

= 32.960 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 1.506 / [1000 × 32.960 x (243.5 - 32.960 / 3)] × 10^6

= 0.393 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 1.506 / [322.500 x (243.5 - 32.960 / 3)] × 10^6

= 20.078 MPa

fst = fs x (H - d' - χ) / (d - χ) = 20 x (300 - 57 - 0) / (244 - 33) = 20.08 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 20.08) - 2.5 x 50.00 = 5174.43 ㎜

300 x (280 / fs) = 300 x (280 / 20.08) = 4183.76 ㎜

Sa = 4183.76 ㎜ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (4183.76 mm) Ā O.K

7. Wall - Bottom

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	5.802	kN·m	V_u	=	26.424	kN	M_o	=		3.808	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 63.160 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎡ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	6051.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	819.9	㎡
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00035	㎡	$A_{s,4/3req}$	=	84.2	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00035	㎡	$A_{s,min}$	=	84.2	㎡
P_{use}	=	A_s / (B·d)	=	0.00265	㎡	$A_{s,min}$	=	645.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 5.802 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 26.424 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 3.808 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 0.737 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 3.808 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 25.844 MPa

fst = fs x (H - d' - χ) / (d - χ) = 26 x (300 - 57 - 1) / (244 - 45) = 25.84 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 25.84) - 2.5 x 50.00 = 3992.05 ㎜

300 x (280 / fs) = 300 x (280 / 25.84) = 3250.31 ㎜

Sa = 3250.31 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (3250.31 mm) Ā O.K

8. Bottom Slab - At the end of the point

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	3.342	kN·m	V_u	=	56.890	kN	M_o	=		1.989	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 36.349 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎡ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00020	㎡	$A_{s,4/3req}$	=	48.5	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00020	㎡	$A_{s,min}$	=	48.5	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎡	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 3.342 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 172.181 kN > Vu = 56.890 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = $- nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

= $- 8 \times 645.00 / 1000 + 8 \times 645.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 243.5 / (8 \times 645.00)]}$

= 45.234 ㎜

f_c = $2 \times M_o / [B \times \chi \times (d - \chi/3)]$

= $2.0 \times 1.989 / [1000 \times 45.234 \times (243.5 - 45.234 / 3)] \times 10^6$

= 0.385 MPa

f_s = $M_o / [A_s \times (d - \chi/3)]$

= $1.989 / [645.000 \times (243.5 - 45.234 / 3)] \times 10^6$

= 13.497 MPa

fst = fs x (H - d' - χ) / (d - χ) = 13 x (300 - 57 - 0) / (244 - 45) = 13.50 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 13.50) - 2.5 x 50.00 = 7758.32 ㎜

300 x (280 / fs) = 300 x (280 / 13.50) = 6223.68 ㎜

Sa = 6223.68 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (6223.68 mm) Ā O.K

9. Bottom Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm		
M_u	=	26.896	kN·m	V_u	=	0.000	kN	M_o	=		17.388	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 337.384 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00211	㎟	$A_{s,4/3req}$	=	449.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00211	㎟	$A_{s,min}$	=	449.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 26.896 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 17.388 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 4.145 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 17.388 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 135.139 MPa

fst = fs x (H - d' - χ) / (d - χ) = 135 x (300 - 87 - 4) / (214 - 42) = 135.14 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 135.14) - 2.5 x 80.00 = 587.34 ㎜

300 x (280 / fs) = 300 x (280 / 135.14) = 621.58 ㎜

Sa = 587.34 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (587.34 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 3.900 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 3.9 / 28 × (0.43 + 420 / 700)

= 0.143 m < H = 0.300 m Ā O.K

10. Bottom Slab - At the end of middle Wall

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	51.342	kN·m	V_u	=	117.760	kN	M_o	=		33.033	kN·m		

- Check of Strength reduction factor (Φ)

a = 11.958

Because T = C , *c* = 11.958 / β1 = 11.958 / 0.821 = 14.558 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 14.558) / 14.558

= 0.0472

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 568.031 ㎟

Use As = D 13 @ 400 + D 16 @ 400 = 820.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00311	㎟	$A_{s,4/3req}$	=	757.4	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00311	㎟	$A_{s,min}$	=	757.4	㎟
P_{use}	=	A_s / (B·d)	=	0.00337	㎟	$A_{s,min}$	=	820.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 11.958 mm

ØMn = 0.9 × As × fy × (d - a/2) = 73.622 kN·m > Mu = 51.342 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 117.760 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 820.00 / 1000 + 8 × 820.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 820.00)]

= 50.341 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 33.033 / [1000 × 50.341 x (243.5 - 50.341 / 3)] × 10^6

= 5.788 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 33.033 / [820.000 x (243.5 - 50.341 / 3)] × 10^6

= 177.680 MPa

fst = fs x (H - d' - χ) / (d - χ) = 178 x (300 - 57 - 6) / (244 - 50) = 177.68 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 177.68) - 2.5 x 50.00 = 473.83 ㎜

300 x (280 / fs) = 300 x (280 / 177.68) = 472.76 ㎜

Sa = 472.76 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (472.76 mm) Ā O.K

11. Middle Wall - Top & Bottom

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	0.000	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 5.726) / 5.726

= 0.1089

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 0.000 ㎠

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎠ (3 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00000	㎠	$A_{s,4/3req}$	=	0.0	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00000	㎠	$A_{s,min}$	=	0.0	㎠
P_{use}	=	A_s / (B·d)	=	0.00151	㎠	$A_{s,min}$	=	322.5	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

ØMn = 0.9 × As × fy × (d - a/2) = 25.740 kN·m > Mu = 0.000 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 103.827 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 322.50)]

= 30.711 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.000 / [1000 × 30.711 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.000 / [322.500 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

fst = fs x (H - d' - χ) / (d - χ) = 0 x (300 - 87 - 0) / (214 - 31) = 0.00 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 3.58E+15 ㎜

300 x (280 / fs) = 300 x (280 / 0.00) = 2.82E+15 ㎜

Sa = 2.82335E+15 ㎜ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (2.82335E+15 mm) Ā O.K

1.1.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	H	13@	200	=	645.0	㎟			
Compression side	H	13@	200	=	645.0	㎟			
				□	=	1290.0	㎟	> 540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Wall (H = 300 mm)

· $A_{s,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
				□	=	1290.0	㎟	> 540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

3) Bottom Slab (H = 300 mm)

· $A_{s,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
				□	=	1290.0	㎟	> 540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

4) Middle Wall (H = 300 mm)

· $A_{s,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

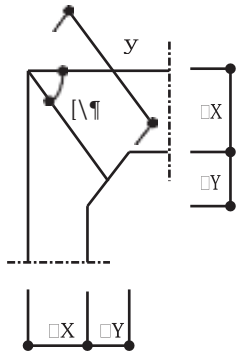
· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
				□	=	1290.0	㎟	> 540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

1.1.9 Corner Design

1) Top slab Cheak



a1	=	300	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 8.412 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 565.7 \text{ mm}$$

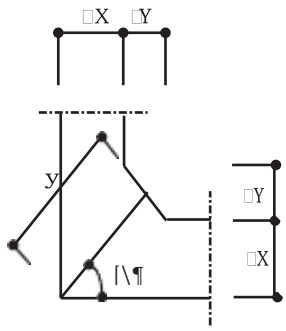
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 8.412 \times 10^{\wedge}6}{565.7^2 \times 1000} = 0.131 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.131 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	300	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm

M_o = 3.808 kN·m

$R = \square(a1^2 + a2^2) = 424.3$ mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 3.808 \times 10^{\wedge}6}{424.3^2 \times 1000} = 0.106 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.106 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

1.2 Box Culvert 2 (STA.1+400.00 Right)

1@1.2×1.2

FH=0.35 m [SI UNIT]

1.2.1 Design Conditions

1) General Items

(1) Type of Culvert : 1 Box

(2) Width (w) : 1 @ 1.2 m

(3) Height (h) : 1.20 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength : fc' = 32 MPa

¤ Modulus of Elasticity : Ec = 26587 MPa

(2) Reinforcement bar

⊳ Yield Strength : fy = 420 MPa

⊲ Modulus of Elasticity : Es = 200000 MPa

3) Material weight

(1) Reinforced Concrete : ωc = 25.00 kN/m³

(2) plain concrete : γcn = 23.50 kN/m³

(3) Pavement : γasp = 23.00 kN/m³

(4) Subterranean : γw = 10.00 kN/m³

4) Soil

(1) Wet Unit Weight : γt = 19.00 kN/m³

(2) Submerged Unit Weight : γsub = 10.00 kN/m³

(3) angle of internal friction : φ = 28.00 °

(4) coefficient of earth pressure atrest : Ko = 1-sinΦ = 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

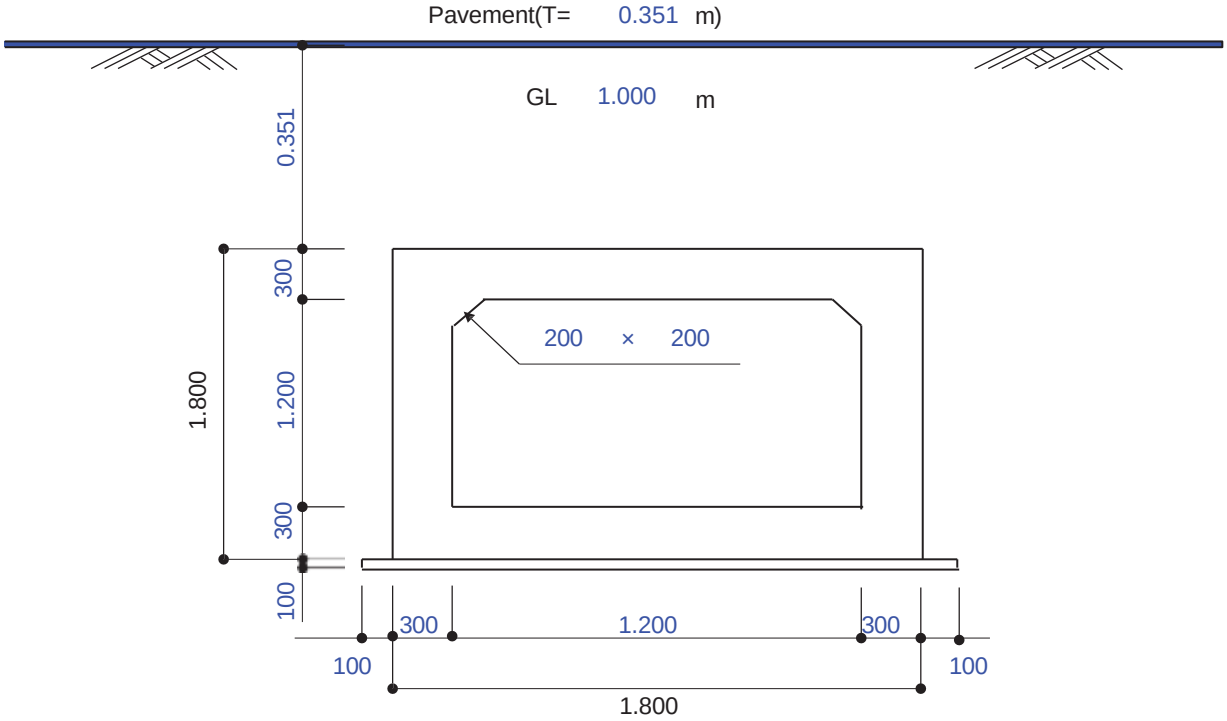
· SAP2000 (Structure Analysis Program)

8) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.2.2 Section Assumption



1.2.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.351 × 1.800 × 23.0	14.531
Vertical earth pressure (EV)	No exist ground water	0.000 × 1.800 × 19.0	0.000
	Exist ground water	(0.000 × 19.0 + 0.000 × 10.0) × 1.800	0.000
Ground Water(WA')		0.000 × 1.800 × 10.0	0.000
Sub Total		Surcharge Load for Bouyancy Check	14.531
Slab(DC)	Top	0.300 × 1.800 × 25.0	13.500
	Bottom	0.300 × 1.800 × 25.0	13.500
Wall(DC)	Left	0.300 × 1.200 × 25.0	9.000
	Inner	0.000 × 1.200 × 25.0	0.000
	Right	0.300 × 1.200 × 25.0	9.000
Hunch(DC)		0.200 × 0.200 / 2 × 25.0 × 2 EA	1.000
Sub Total		Surcharge Load for Bouyancy Check	46.000

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.000 m)

- Total Load for Bouyancy Check : 60.531 kN
- Uplife force : 1.800 ×(2.151 - 1.000)× 10.0 kN/□ = 20.718 kN
- Safety factor = 1.25

□ F.S = 60.531 / 20.718 = 2.922 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL 0.000 m)

- Total Load for Bouyancy Check :

46.000 +(0.351 × 1.800 × 10.000 kN/□) = 52.318 kN

- Uplife force : 1.800 × 1.800 × 10.000 kN/□ = 32.400 kN
- Safety factor = 1.1

□ F.S = 52.318 / 32.400 = 1.615 > 1.1 - O.K

Ā Securing safety at all ground water levels

3) Allowable vertical bearing capacity check

(1) Load

- Dead load
  - Self weight of Structure = 46.000 / 1.800 = 25.556 kN/m²
  - Vertical earth pressure = 14.531 / 1.800 = 8.073 kN/m² (No exist ground water)
- Live load = 102.743 kN/m² (Refer to 1.1.4.2)
- Water load in Culvrt = 1.200 ๑ 10.000 = 12.000 kN/m²

(2) Allowable vertical bearing capacity

- Q_{max} = 148.371 kN/m²
- Q_a = 304.170 kN/m² (Refer to Geotechnic Report)

□ Q_a = 304.170 kN/□ > Q_{max} = 148.371 kN/□ - O.K

1.2.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

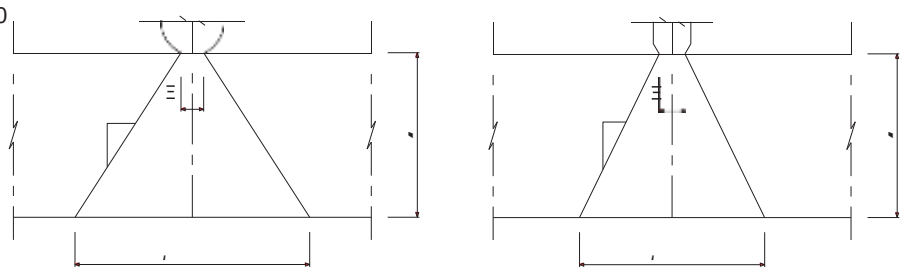
TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m²)	
Pavement	0.351	23.000	1.000 × 0.351 × 23.000 =	8.073
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
□	0.351		Psv =	8.073 kN/m²

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m²)	
Pavement	0.351	23.000	1.000 × 0.351 × 23.000 =	8.073
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
	0.000	10.000	1.000 × 0.000 × 10.000 =	0.000
□	0.351		Psvh =	8.073 kN/m²

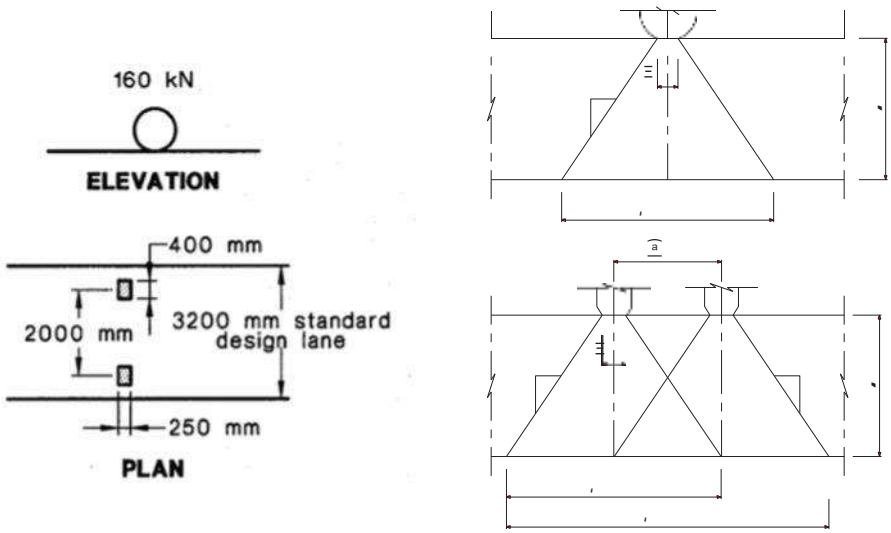
2) Live Load

(1) W80



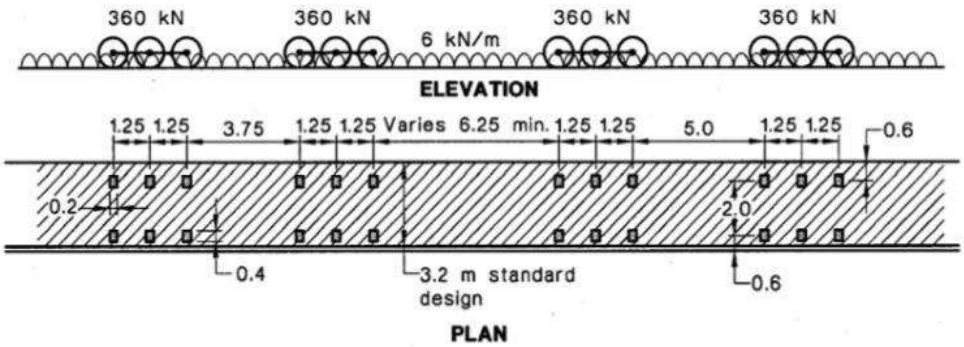
$$P_{vl} = \frac{80}{(0.25+2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.702) \times (0.4 + 0.702)} = 76.256 \text{ kN/m}^2$$

(2) A160

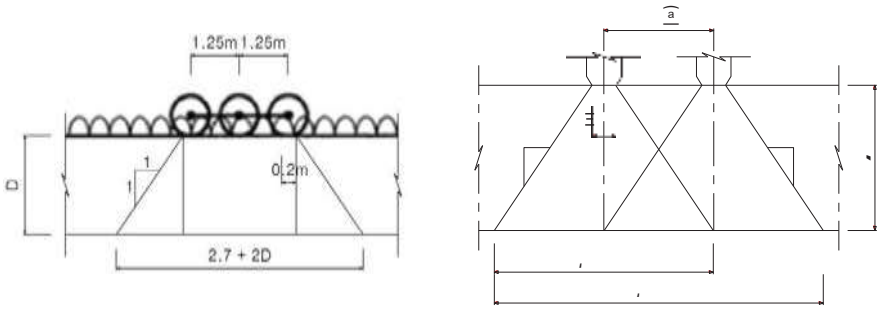


$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.702) \times (0.4 + 0.702)} = 76.256 \text{ kN/m}^2$$

(3) M1600



- Axle group

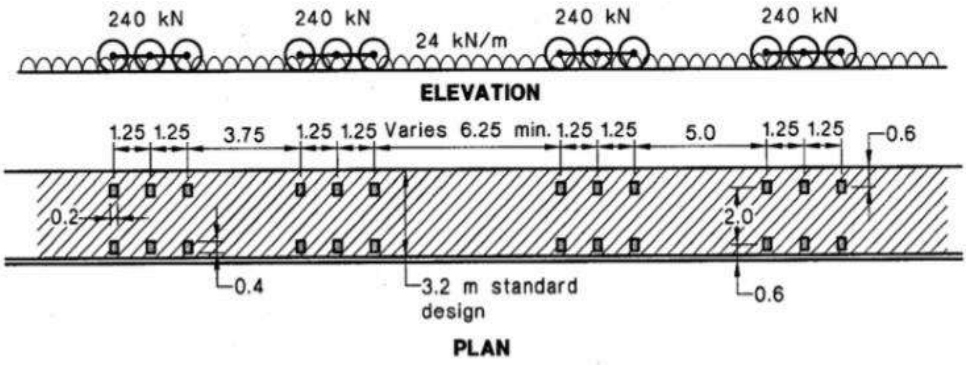


$$P_{v1} = \frac{60}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{60}{(0.2 + 0.702) \times (0.4 + 0.702)} = 60.362 \text{ kN/m}^2$$

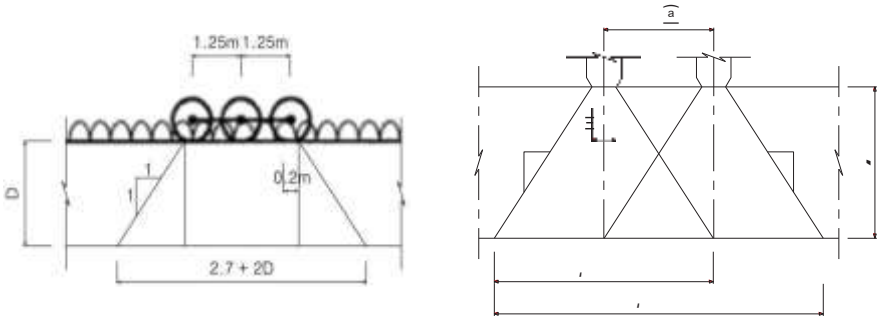
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 60.362 + 1.875 = 62.237 kN/m²

(4) S1600



- Axle group

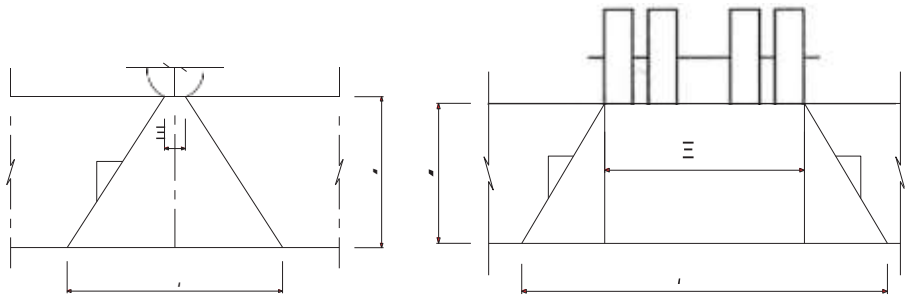


$$P_{v1} = \frac{40}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{40}{(0.2 + 0.702) \times (0.4 + 0.702)} = 40.241 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 40.241 + 7.500 = 47.741 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2\times0.351) \times (1.4 + 2\times0.351)} = 65.928 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	76.256	0.35	102.743
A160	76.256	0.35	102.743
M1600	62.237	0.26	78.723
S1600	47.741	0.00	47.741
HLP	65.928	0.10	72.521

□ P_{vl} = 102.743 kN/m² = 102.743 kN/m²

(7) Live Load Surcharge

□ P_{vlh} = 102.743 kN/m² × 0.500 = 51.371 kN/m²

3) Lateral Earth Pressure

coefficient of earth pressure at rest : Ko = 1 - sin30 = 0.500

- No exist ground water

P_{sh} = k_o × γ_t × H

P_{sh1} = 0.500 x (23 x 0.351 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000) = 4.037 kN/m²

P_{sh2} = 4.037 + 0.500 × 19.0 × 0.150 = 5.462 kN/m²

P_{sh3} = 5.462 + 0.500 × 19.0 × 0.350 = 8.787 kN/m²

P_{sh4} = 8.787 + 0.500 × 19.0 × 0.250 = 11.162 kN/m²

P_{sh5} = 11.162 + 0.500 × 19.0 × 0.250 = 13.537 kN/m²

P_{sh6} = 13.537 + 0.500 × 19.0 × 0.250 = 15.912 kN/m²

P_{sh7} = 15.912 + 0.500 × 19.0 × 0.250 = 18.287 kN/m²

P_{sh8} = 18.287 + 0.500 × 19.0 × 0.150 = 19.712 kN/m²

- Exist ground water

$P_{sh'}$ = k_o × (γ_t × H_1 + γ_{sub} × H_2)

$P_{sh1'}$ = 0.500 x (23 x 0.351 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000 + 10 x 0.000) = 4.037 kN/m²

$P_{sh2'}$ = 4.037 + 0.500 × 19.0 × 0.150 = 5.462 kN/m²

$P_{sh3'}$ = 5.462 + 0.500 × 19.0 × 0.350 = 8.787 kN/m²

$P_{sh4'}$ = 8.787 + 0.500 × 10.0 × 0.250 = 10.037 kN/m²

$P_{sh5'}$ = 10.037 + 0.500 × 10.0 × 0.250 = 11.287 kN/m²

$P_{sh6'}$ = 11.287 + 0.500 × 10.0 × 0.250 = 12.537 kN/m²

$P_{sh7'}$ = 12.537 + 0.500 × 10.0 × 0.250 = 13.787 kN/m²

$P_{sh8'}$ = 13.787 + 0.500 × 10.0 × 0.150 = 14.537 kN/m²

4) Ground Water Load

(1) Horizontal ground Water Pressure

$P_{wh} = \gamma_w \times H_2$

P_{wh1} = 10.0 × 0.000 = 0.000 kN/m²

P_{wh2} = 0.000 + 0.0 × 0.150 = 0.000 kN/m²

P_{wh3} = 0.000 + 0.0 × 0.350 = 0.000 kN/m²

P_{wh4} = 0.000 + 10.0 × 0.250 = 2.500 kN/m²

P_{wh5} = 2.500 + 10.0 × 0.250 = 5.000 kN/m²

P_{wh6} = 5.000 + 10.0 × 0.250 = 7.500 kN/m²

P_{wh7} = 7.500 + 10.0 × 0.250 = 10.000 kN/m²

P_{wh8} = 10.000 + 10.0 × 0.150 = 11.500 kN/m²

(2) Vertical ground Water Pressure

- Top Slab : P_{wv1} = 10.0 × 0.000 = 0.000 kN/m²

-Bottom Slab : P_{wv2} = 10.0 × 1.800 = 18.000 kN/m²

5) Load Combination

(1) Ultimate Load

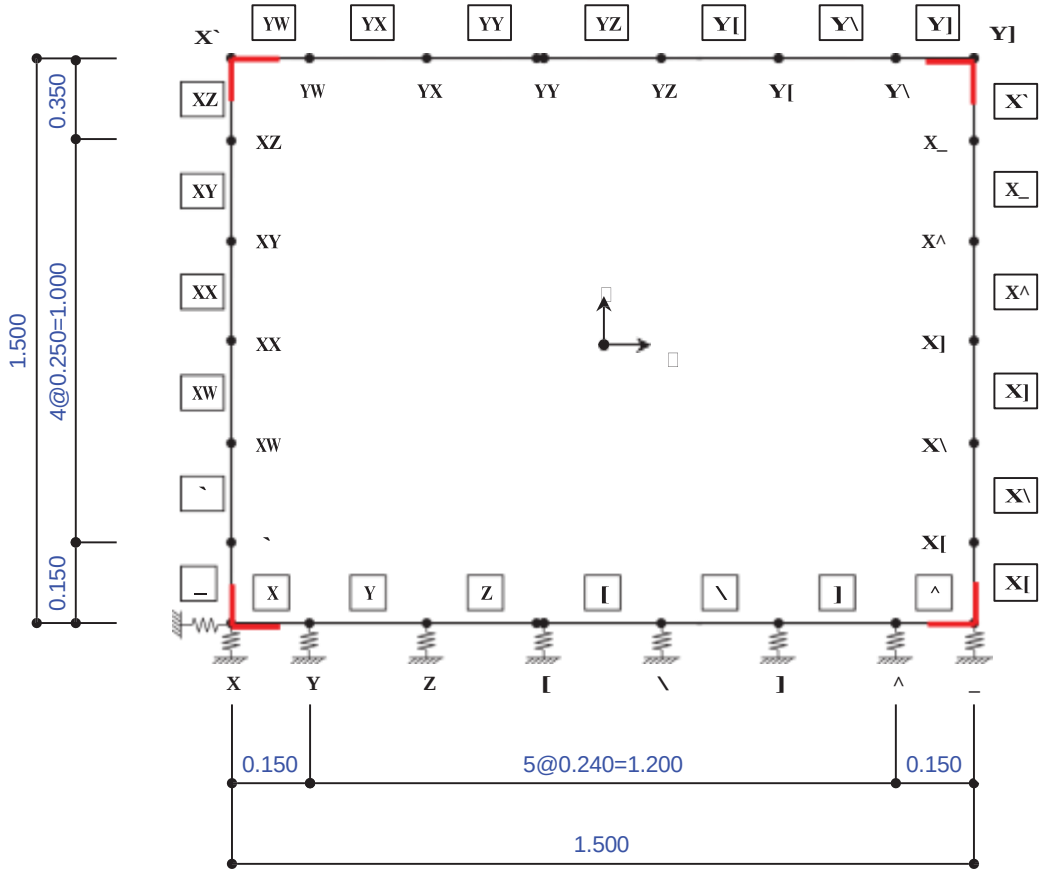
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.60		1.60	1.60	1.60			
COMB 3	1.20	1.60		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.60	1.60	1.60		1.60	1.60	1.60
COMB 7	1.20		1.60	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	1.000		1.00	1.00	1.00			
SCOMB 2	1.00	1.000		1.00	1.00	0.56			
SCOMB 3	1.00	1.000				0.00			
SCOMB 4	1.00		1.000	1.00	1.00		1.00	1.00	1.000
SCOMB 5	1.00		1.000	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		1.000				0.00	0.00	0.00

1.2.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	Z	Section	Node	X	Z	Section
1	0.150	0.150	Bottom Slab	14	1.650	0.300	Right Wall
2	0.300	0.150		15	1.650	0.550	
3	0.540	0.150		16	1.650	0.800	
4	0.780	0.150		17	1.650	1.050	
5	1.020	0.150		18	1.650	1.300	
6	1.260	0.150		19	0.150	1.650	Top Slab
7	1.500	0.150		20	0.500	1.650	
8	1.650	0.150		21	0.660	1.650	
9	0.150	0.300	Left Wall	22	0.820	1.650	
10	0.150	0.550		23	0.980	1.650	
11	0.150	0.800		24	1.140	1.650	
12	0.150	1.050		25	1.300	1.650	
13	0.150	1.300		26	1.650	1.650	

(2) Section

NO.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~6	Bottom Slab
2	0.300	1.000	0.300	0.002250	9~12	Left Wall
3	0.300	1.000	0.300	0.002250	15~18	Right Wall
4	0.300	1.000	0.300	0.002250	21~25	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$kvo = 1/0.3 \times \alpha \times Eo$

Eo : the modulus of subgrade elasticity (kN/m²)

α : correction factor for calculating Eo

Eo = 7000 kN/m² (Refer to Geotechnic Report)

α = 4

Kvo = 1/0.3 × α × Eo = 1/0.3 × 4 7000 = 93333 kN/

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{1.80 \times 1.80} = 1.800$ m²

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$= 93333.333 \times (1.800 / 0.3)^{-3/4} = 24345.8$ kN/

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 8	24345.800	0.2250	1.0000	0.2250	5477.8
2, 7	24345.800	0.1950	1.0000	0.1950	4747.4
3 ~ 6	24345.800	0.2400	1.0000	0.2400	5843.0

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

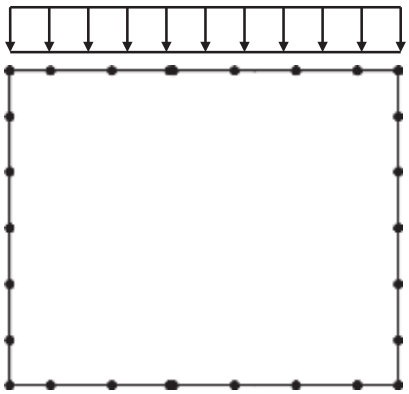
3) Loading

(1) LOAD-1 : Self weight - Automatic consideration in program

(2) LOAD-2,3 : Vertical earth pressure

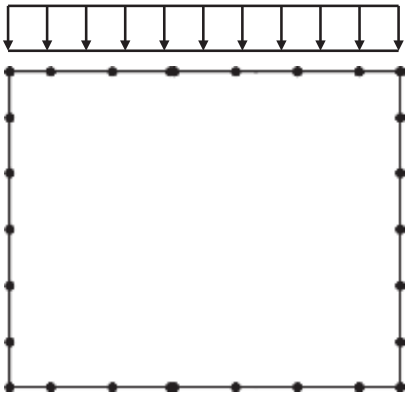
Psvh= 8.073 kN/m² (Exist ground water)

Psv= 8.073 kN/m² (No exist ground water)



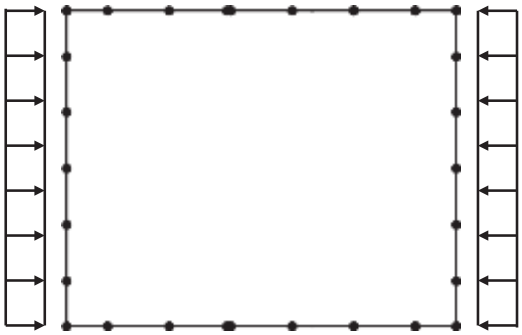
(3) LOAD-4 : Live Load

Pvl= 102.743 kN/m²



(4) LOAD-5 : Live Load Surcharge

Pvlh= 51.371 kN/m²

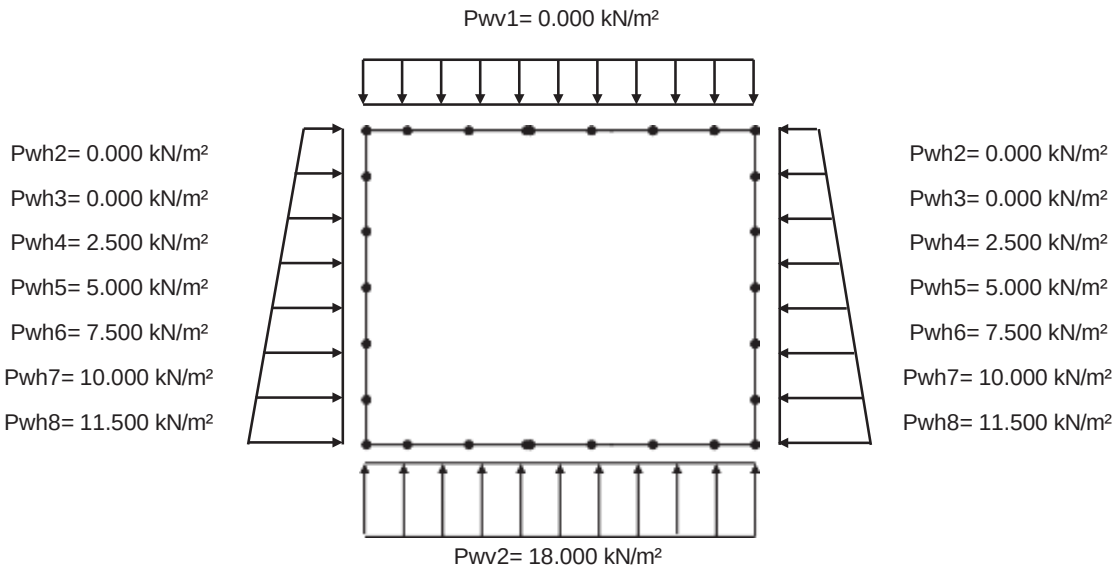


Pvlh= 51.371 kN/m²

(5) LOAD-6 : Horizontal Earth Pressure (No Ground Water)

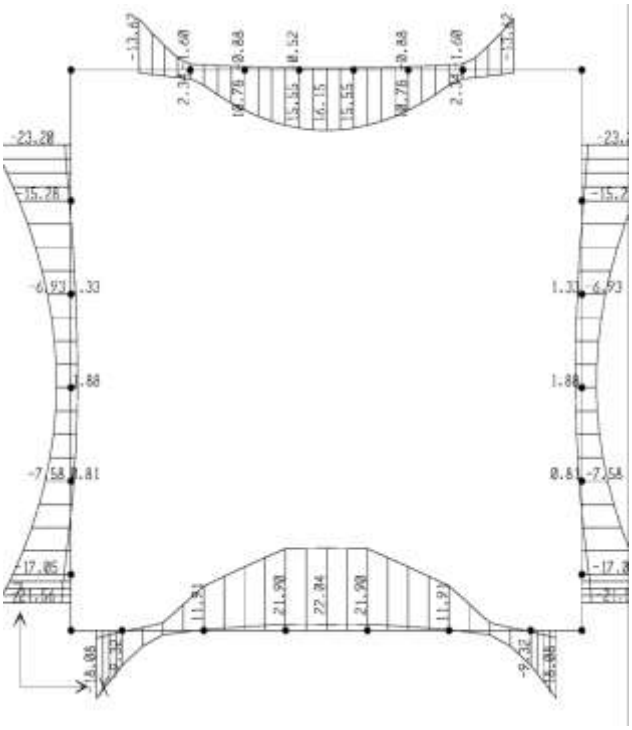
(6) LOAD-7 : Horizontal Earth Pressure (Ground Water)

(7) LOAD-8 : Ground Water Pressure

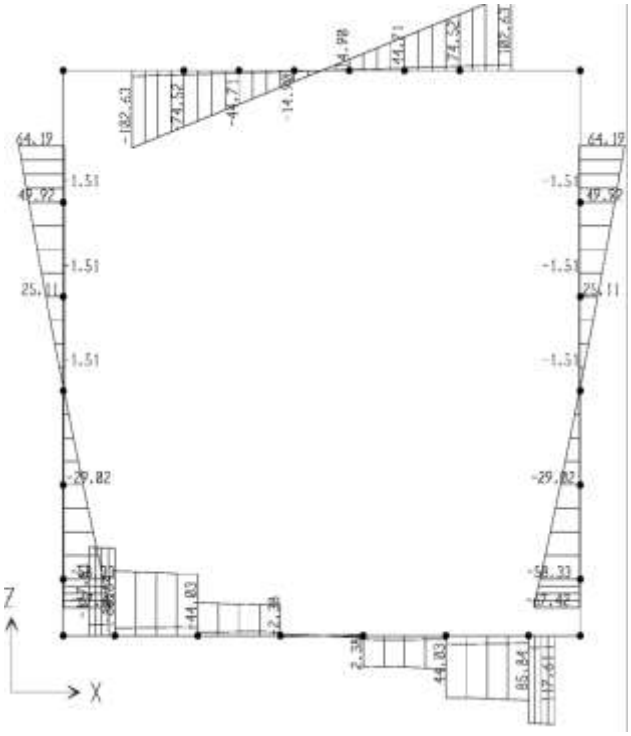


1.2.6 Summary of Analysis Results

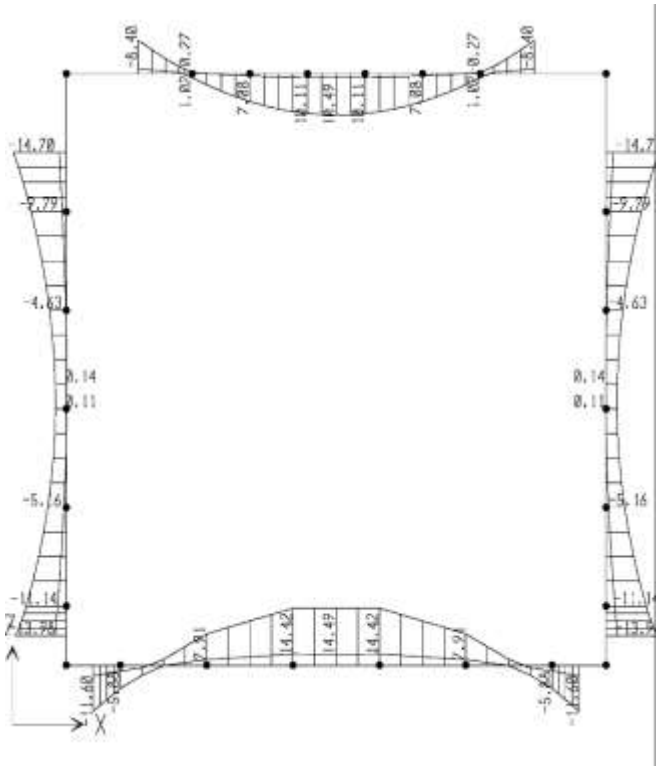
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	ØMn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	5.48
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	3.15
Wall	Top(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	3.21
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	25.740	D13 @ 400	13.69
	Middle(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	3.41
	Bottom(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	2.70
Bottom Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	3.22
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	2.31

1.2.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm			
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm			
M_u	=	13.616	kN·m	V_u	=	102.629	kN	M_o	=		8.400	kN·m			

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_ТΦГТУКУΦУZТ НПΣ И□_

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 116.347 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎡ (5 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	7715.9	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1045.5	㎡
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00050	㎡	$A_{s,4/3req}$	=	155.1	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00050	㎡	$A_{s,min}$	=	155.1	㎡
P_{use}	=	A_s / (B·d)	=	0.00208	㎡	$A_{s,min}$	=	645.0	㎡

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 13.616 kN·m

Ā O.K

ⱨ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 102.629 kN

Ā No shear reinforcement is required

(2) Crack Check

ⱨ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 8.400 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 1.108 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 8.400 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 44.408 MPa

fst = fs x (H - d' - χ) / (d - χ) = 44 x (367 - 57 - 1) / (311 - 52) = 44.41 MPa

ⱨ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 44.41) - 2.5 x 50.00 = 2270.96 ㎜

300 x (280 / fs) = 300 x (280 / 44.41) = 1891.54 ㎜

Sa = 1891.54 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1891.54 mm) Ā O.K

2) Top Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	16.151	kN·m	V_u	=	0.000	kN	M_o	=		10.489	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ㅇ ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ ㅇ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 201.592 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00126	㎟	$A_{s,4/3req}$	=	268.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00126	㎟	$A_{s,min}$	=	268.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 16.151 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 10.489 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 2.500 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 10.489 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 81.524 MPa

fst = fs x (H - d' - χ) / (d - χ) = 82 x (300 - 87 - 3) / (214 - 42) = 81.52 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 81.52) - 2.5 x 80.00 = 1105.13 ㅡ

300 x (280 / fs) = 300 x (280 / 81.52) = 1030.37 ㅡ

Sa = 1030.37 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1030.37 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.800 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 1.8 / 28 × (0.43 + 420 / 700)

= 0.066 m < H = 0.300 m Ā O.K

3) Wall - Top

(1) Section Design

ㅎ. Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm		
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm		
M_u	=	23.202	kN·m	V_u	=	64.182	kN	M_o	=		14.699	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 198.668 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅎ. Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00085	㎟	$A_{s,4/3req}$	=	264.9	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00085	㎟	$A_{s,min}$	=	264.9	㎟
P_{use}	=	A_s / (B·d)	=	0.00208	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅎ. Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 23.202 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 64.182 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 14.699 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 1.940 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 14.699 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 77.704 MPa

fst = fs x (H - d' - χ) / (d - χ) = 78 x (367 - 57 - 2) / (311 - 52) = 77.70 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 77.70) - 2.5 x 50.00 = 1244.29 ㎜

300 x (280 / fs) = 300 x (280 / 77.70) = 1081.02 ㎜

Sa = 1081.02 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1081.02 mm) Ā O.K

4) Wall - Middle(In)

(1) Section Design

ㅎ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	1.880	kN·m	V_u	=	0.000	kN	M_o	=		0.135	kN·m	

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 5.726) / 5.726

= 0.1089

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 23.310 ㎡

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎡ (3 ea/m)

ㅎ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	5305.5	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	718.9	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00015	㎡	$A_{s,4/3req}$	=	31.1	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00015	㎡	$A_{s,min}$	=	31.1	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00151	㎡	$A_{s,min}$	=	322.5	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅎ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 25.740 kN·m > Mu = 1.880 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 322.50)]

= 30.711 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.135 / [1000 × 30.711 x (213.5 - 30.711 / 3)] × 10^6

= 0.043 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.135 / [322.500 x (213.5 - 30.711 / 3)] × 10^6

= 2.059 MPa

fst = fs x (H - d' - χ) / (d - χ) = 2 x (300 - 87 - 0) / (214 - 31) = 2.06 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 2.06) - 2.5 x 80.00 = 5.1E+04 ㎜

300 x (280 / fs) = 300 x (280 / 2.06) = 4.1E+04 ㎜

Sa = 4.08E+04 ㎜ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (4.1E+04 mm) Ā O.K

5) Wall - Middle(Out)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	17.059	kN·m	V_u	=	0.000	kN	M_o	=		11.148	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 186.440 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00102	㎟	$A_{s,4/3req}$	=	248.6	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00102	㎟	$A_{s,min}$	=	248.6	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 17.059 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.148 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 2.158 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.148 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 75.664 MPa

fst = fs x (H - d' - χ) / (d - χ) = 76 x (300 - 57 - 2) / (244 - 45) = 75.66 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 75.66) - 2.5 x 50.00 = 1281.22 ㅡ

300 x (280 / fs) = 300 x (280 / 75.66) = 1110.17 ㅡ

Sa = 1110.17 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1110.17 mm) Ā O.K

6) Wall - Bottom

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	21.567	kN·m	V_u	=	58.338	kN	M_o	=	13.984	kN·m			

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ㅇ ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ ㅇ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 236.087 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00129	㎟	$A_{s,4/3req}$	=	314.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00129	㎟	$A_{s,min}$	=	314.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 21.567 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 58.338 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 13.984 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 2.707 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 13.984 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 94.917 MPa

fst = fs x (H - d' - χ) / (d - χ) = 95 x (300 - 57 - 3) / (244 - 45) = 94.92 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 94.92) - 2.5 x 50.00 = 995.98 ㎜

300 x (280 / fs) = 300 x (280 / 94.92) = 884.98 ㎜

Sa = 884.98 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (884.98 mm) Ā O.K

7) Bottom Slab - At the end of the point

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	18.091	kN·m	V_u	=	85.876	kN	M_o	=		11.604	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 197.788 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00108	㎟	$A_{s,4/3req}$	=	263.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00108	㎟	$A_{s,min}$	=	263.7	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 58.221 kN·m > Mu = 18.091 kN·m

Ā O.K

ᴖ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 85.876 kN

Ā No shear reinforcement is required

(2) Crack Check

ᴖ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ˩

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.604 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 2.246 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.604 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 78.760 MPa

fst = fs x (H - d' - χ) / (d - χ) = 79 x (300 - 57 - 2) / (244 - 45) = 78.76 MPa

ᴖ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ˩

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 78.76) - 2.5 x 50.00 = 1225.95 ˩

300 x (280 / fs) = 300 x (280 / 78.76) = 1066.54 ˩

Sa = 1066.54 ˩ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1066.54 mm) Ā O.K

8) Bottom Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm		
M_u	=	22.055	kN·m	V_u	=	0.000	kN	M_o	=		14.499	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИД_Т ТФГТУКУФУZТ НПΣ ИД_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 276.039 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00172	㎟	$A_{s,4/3req}$	=	368.1	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00172	㎟	$A_{s,min}$	=	368.1	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 22.055 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 14.499 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 3.456 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 14.499 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 112.689 MPa

fst = fs x (H - d' - χ) / (d - χ) = 113 x (300 - 87 - 3) / (214 - 42) = 112.69 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 112.69) - 2.5 x 80.00 = 744.19 ㎜

300 x (280 / fs) = 300 x (280 / 112.69) = 745.41 ㎜

Sa = 744.19 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (744.19 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.800 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 1.8 / 28 × (0.43 + 420 / 700)

= 0.066 m < H = 0.300 m Ā O.K

1.2.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
			□	=	1290.0	㎟	>	540.0	㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Wall (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
			□	=	1290.0	㎟	>	540.0	㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

3) Bottom Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

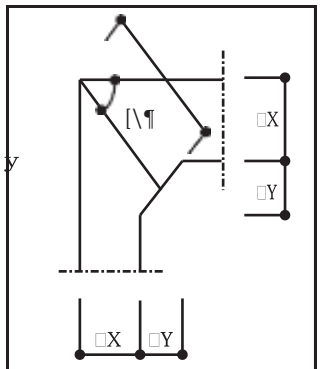
· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
			□	=	1290.0	㎟	>	540.0	㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

1.2.9 Corner Design

1) Top slab Cheak



a1	=	300	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 14.699 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 565.7 \text{ mm}$$

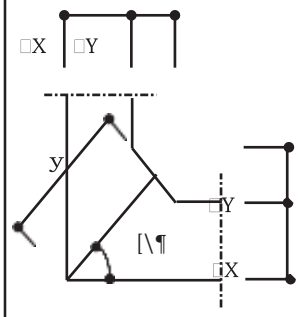
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 14.699 \times 10^{\wedge}6}{565.7^2 \times 1000} = 0.230 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.230 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	300	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm

M_o = 13.984 kN·m

R = □$(a1^2 + a2^2)$ = 424.3 mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 13.984 \times 10^{\wedge}6}{424.3^2 \times 1000} = 0.388 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.388 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

1.3 Box Culvert 2 (STA.1+400.00 Left)

1@0.9×0.6

FH=0.43 m [SI UNIT]

1.3.1 Design Conditions

1) General Items

(1) Type of Culvert : 1 Box

(2) Width (w) : 1 @ 0.9 m

(3) Height (h) : 0.60 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength : fc' = 32 MPa

¤ Modulus of Elasticity : Ec = 26587 MPa

(2) Reinforcement bar

⊳ Yield Strength : fy = 420 MPa

⊲ Modulus of Elasticity : Es = 200000 MPa

3) Material weight

(1) Reinforced Concrete : ωc = 25.00 kN/m³

(2) plain concrete : γcn = 23.50 kN/m³

(3) Pavement : γasp = 23.00 kN/m³

(4) Subterranean : γw = 10.00 kN/m³

4) Soil

(1) Wet Unit Weight : γt = 19.00 kN/m³

(2) Submerged Unit Weight : γsub = 10.00 kN/m³

(3) angle of internal friction : φ = 28.00 °

(4) coefficient of earth pressure atrest : Ko = 1-sinΦ = 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

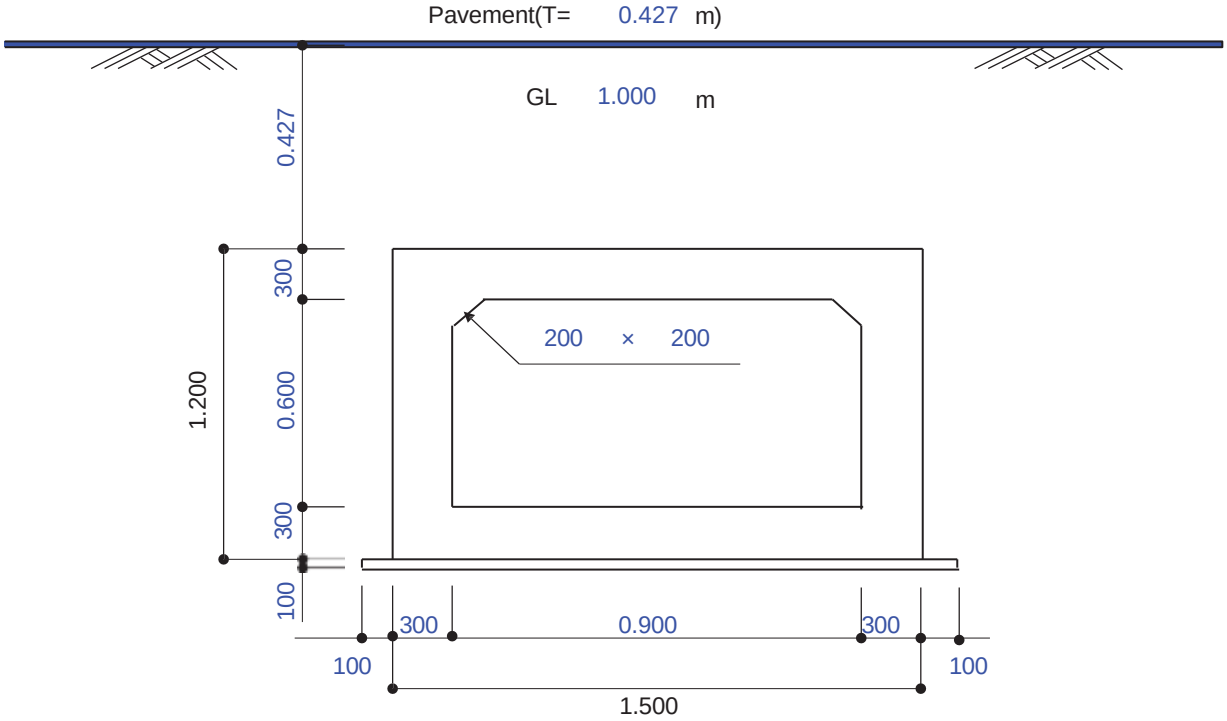
· SAP2000 (Structure Analysis Program)

8) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.3.2 Section Assumption



1.3.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.427 × 1.500 × 23.0	14.732
Vertical earth pressure (EV)	No exist ground water	0.000 × 1.500 × 19.0	0.000
	Exist ground water	(0.000 × 19.0 + 0.000 × 10.0) × 1.500	0.000
Ground Water(WA')		0.000 × 1.500 × 10.0	0.000
Sub Total		Surcharge Load for Bouyancy Check	14.732
Slab(DC)	Top	0.300 × 1.500 × 25.0	11.250
	Bottom	0.300 × 1.500 × 25.0	11.250
Wall(DC)	Left	0.300 × 0.600 × 25.0	4.500
	Inner	0.000 × 0.600 × 25.0	0.000
	Right	0.300 × 0.600 × 25.0	4.500
Hunch(DC)		0.200 × 0.200 / 2 × 25.0 × 2 EA	1.000
Sub Total		Surcharge Load for Bouyancy Check	32.500

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.000 m)

- Total Load for Bouyancy Check : 47.232 kN
- Uplife force : 1.500 ×(1.627 - 1.000)× 10.0 kN/□ = 9.405 kN
- Safety factor = 1.25

□ F.S = 47.232 / 9.405 = 5.022 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL 0.000 m)

- Total Load for Bouyancy Check :

32.500 +(0.427 × 1.500 × 10.000 kN/□) = 38.905 kN

- Uplife force : 1.500 × 1.200 × 10.000 kN/□ = 18.000 kN
- Safety factor = 1.1

□ F.S = 38.905 / 18.000 = 2.161 > 1.1 - O.K

Ā Securing safety at all ground water levels

3) Allowable vertical bearing capacity check

(1) Load

- Dead load
  - Self weight of Structure = 32.500 / 1.500 = 21.667 kN/m²
  - Vertical earth pressure = 14.732 / 1.500 = 9.821 kN/m² (No exist ground water)
- Live load = 77.199 kN/m² (Refer to 1.1.4.2)
- Water load in Culvrt = 0.600 ๑ 10.000 = 6.000 kN/m²

(2) Allowable vertical bearing capacity

- Q_{max} = 114.687 kN/m²
- Q_a = 279.170 kN/m² (Refer to Geotechnic Report)

□ Q_a = 279.170 kN/□ > Q_{max} = 114.687 kN/□ - O.K

1.3.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

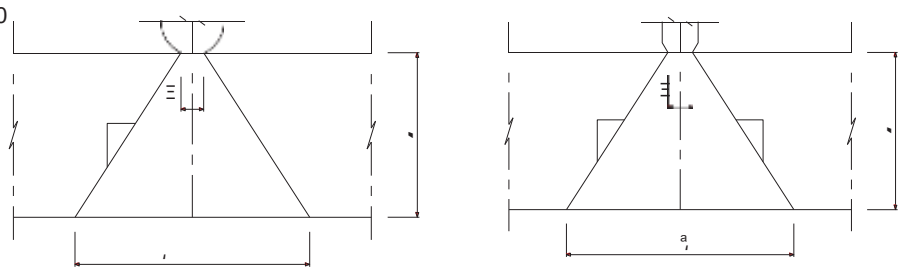
TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m²)	
Pavement	0.427	23.000	1.000 × 0.427 × 23.000 =	9.821
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
□	0.427		Psv =	9.821 kN/m²

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m²)	
Pavement	0.427	23.000	1.000 × 0.427 × 23.000 =	9.821
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
	0.000	10.000	1.000 × 0.000 × 10.000 =	0.000
□	0.427		Psvh =	9.821 kN/m²

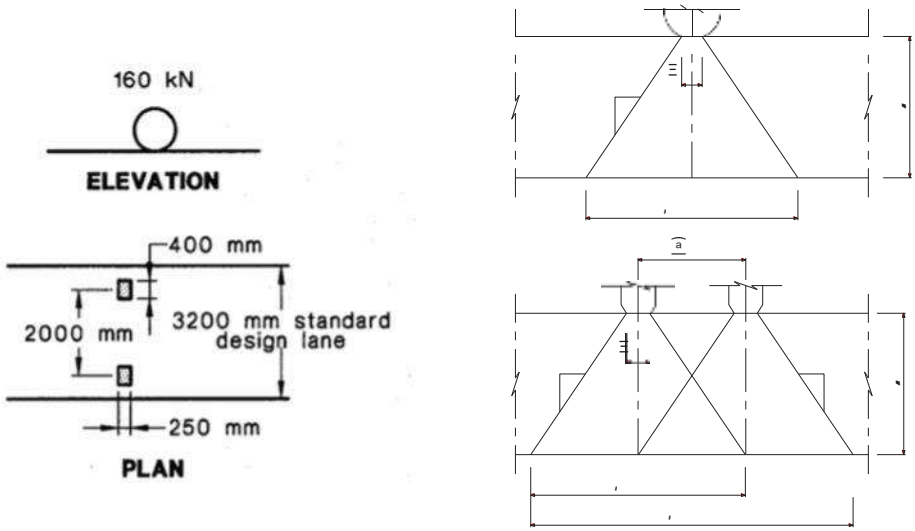
2) Live Load

(1) W80



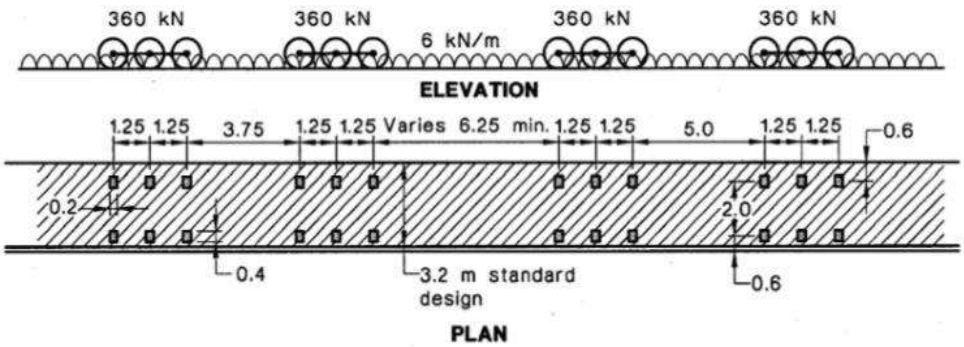
$$P_{vl} = \frac{80}{(0.25+2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.854) \times (0.4 + 0.854)} = 57.786 \text{ kN/m}^2$$

(2) A160

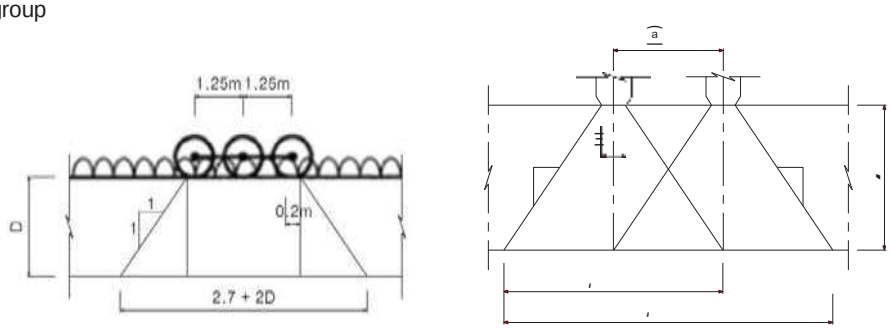


$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.854) \times (0.4 + 0.854)} = 57.786 \text{ kN/m}^2$$

(3) M1600



- Axle group

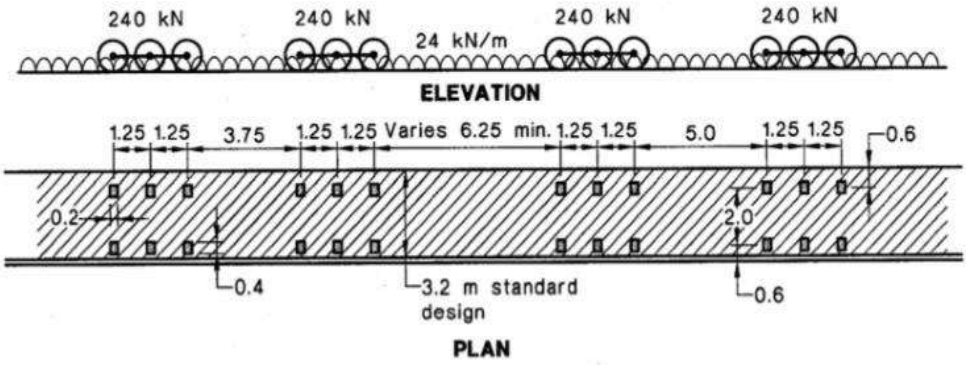


$$P_{v1} = \frac{60}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{60}{(0.2 + 0.854) \times (0.4 + 0.854)} = 45.396 \text{ kN/m}^2$$

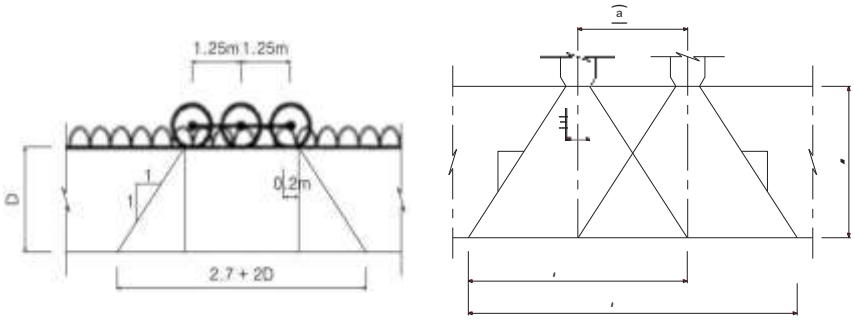
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 45.396 + 1.875 = 47.271 kN/m²

(4) S1600



- Axle group

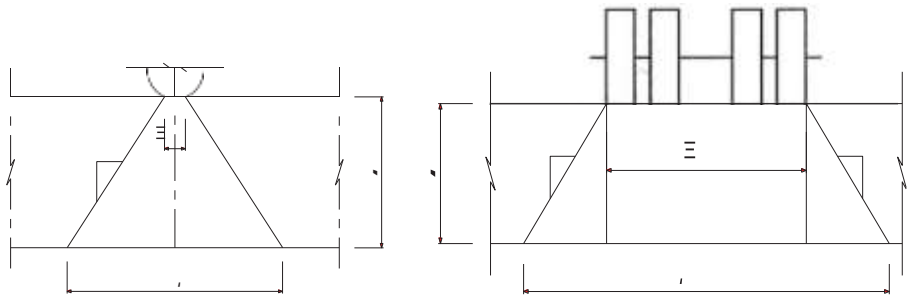


$$P_{v1} = \frac{40}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{40}{(0.2 + 0.854) \times (0.4 + 0.854)} = 30.264 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 30.264 + 7.500 = 37.764 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2\times0.427) \times (1.4 + 2\times0.427)} = 52.616 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	57.786	0.34	77.199
A160	57.786	0.34	77.199
M1600	47.271	0.26	59.433
S1600	37.764	0.00	37.764
HLP	52.616	0.10	57.877

□ P_{vl} = 77.199 kN/m² = 77.199 kN/m²

(7) Live Load Surcharge

□ P_{vlh} = 77.199 kN/m² × 0.500 = 38.600 kN/m²

3) Lateral Earth Pressure

coefficient of earth pressure at rest : Ko = 1 - sin30 = 0.500

- No exist ground water

P_{sh} = k_o × γ_t × H

P_{sh1} = 0.500 x (23 x 0.427 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000) = 4.911 kN/m²

P_{sh2} = 4.911 + 0.500 × 19.0 × 0.150 = 6.336 kN/m²

P_{sh3} = 6.336 + 0.500 × 19.0 × 0.350 = 9.661 kN/m²

P_{sh4} = 9.661 + 0.500 × 19.0 × 0.100 = 10.611 kN/m²

P_{sh5} = 10.611 + 0.500 × 19.0 × 0.100 = 11.561 kN/m²

P_{sh6} = 11.561 + 0.500 × 19.0 × 0.100 = 12.511 kN/m²

P_{sh7} = 12.511 + 0.500 × 19.0 × 0.100 = 13.461 kN/m²

P_{sh8} = 13.461 + 0.500 × 19.0 × 0.150 = 14.886 kN/m²

- Exist ground water

$P_{sh'}$ = k_o × (γ_t × H_1 + γ_{sub} × H_2)

$P_{sh1'}$ = 0.500 x (23 x 0.427 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000 + 10 x 0.000) = 4.911 kN/m²

$P_{sh2'}$ = 4.911 + 0.500 × 19.0 × 0.150 = 6.336 kN/m²

$P_{sh3'}$ = 6.336 + 0.500 × 19.0 × 0.350 = 9.661 kN/m²

$P_{sh4'}$ = 9.661 + 0.500 × 10.0 × 0.100 = 10.161 kN/m²

$P_{sh5'}$ = 10.161 + 0.500 × 10.0 × 0.100 = 10.661 kN/m²

$P_{sh6'}$ = 10.661 + 0.500 × 10.0 × 0.100 = 11.161 kN/m²

$P_{sh7'}$ = 11.161 + 0.500 × 10.0 × 0.100 = 11.661 kN/m²

$P_{sh8'}$ = 11.661 + 0.500 × 10.0 × 0.150 = 12.411 kN/m²

4) Ground Water Load

(1) Horizontal ground Water Pressure

$P_{wh} = \gamma_w \times H_2$

P_{wh1} = 10.0 × 0.000 = 0.000 kN/m²

P_{wh2} = 0.000 + 0.0 × 0.150 = 0.000 kN/m²

P_{wh3} = 0.000 + 0.0 × 0.350 = 0.000 kN/m²

P_{wh4} = 0.000 + 10.0 × 0.100 = 1.000 kN/m²

P_{wh5} = 1.000 + 10.0 × 0.100 = 2.000 kN/m²

P_{wh6} = 2.000 + 10.0 × 0.100 = 3.000 kN/m²

P_{wh7} = 3.000 + 10.0 × 0.100 = 4.000 kN/m²

P_{wh8} = 4.000 + 10.0 × 0.150 = 5.500 kN/m²

(2) Vertical ground Water Pressure

- Top Slab : P_{wv1} = 10.0 × 0.000 = 0.000 kN/m²

-Bottom Slab : P_{wv2} = 10.0 × 1.200 = 12.000 kN/m²

5) Load Combination

(1) Ultimate Load

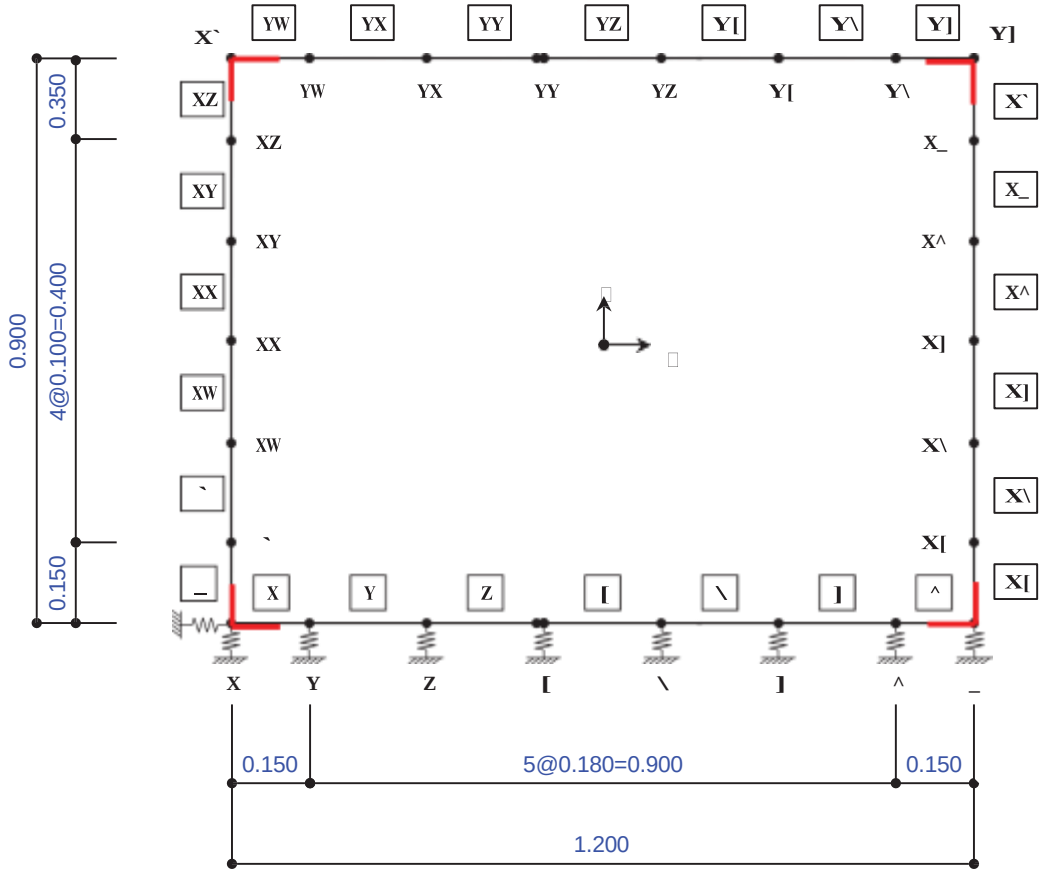
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.60		1.60	1.60	1.60			
COMB 3	1.20	1.60		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.60	1.60	1.60		1.60	1.60	1.60
COMB 7	1.20		1.60	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	1.000		1.00	1.00	1.00			
SCOMB 2	1.00	1.000		1.00	1.00	0.56			
SCOMB 3	1.00	1.000				0.00			
SCOMB 4	1.00		1.000	1.00	1.00		1.00	1.00	1.000
SCOMB 5	1.00		1.000	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		1.000				0.00	0.00	0.00

1.3.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	Z	Section	Node	X	Z	Section
1	0.150	0.150	Bottom Slab	14	1.350	0.300	Right Wall
2	0.300	0.150		15	1.350	0.400	
3	0.480	0.150		16	1.350	0.500	
4	0.660	0.150		17	1.350	0.600	
5	0.840	0.150		18	1.350	0.700	
6	1.020	0.150		19	0.150	1.050	Top Slab
7	1.200	0.150		20	0.500	1.050	
8	1.350	0.150		21	0.600	1.050	
9	0.150	0.300	Left Wall	22	0.700	1.050	
10	0.150	0.400		23	0.800	1.050	
11	0.150	0.500		24	0.900	1.050	
12	0.150	0.600		25	1.000	1.050	
13	0.150	0.700		26	1.350	1.050	

(2) Section

NO.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~6	Bottom Slab
2	0.300	1.000	0.300	0.002250	9~12	Left Wall
3	0.300	1.000	0.300	0.002250	15~18	Right Wall
4	0.300	1.000	0.300	0.002250	21~25	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$kvo = 1/0.3 \times \alpha \times Eo$

Eo : the modulus of subgrade elasticity (kN/m²)

α : correction factor for calculating Eo

Eo = 7000 kN/m² (Refer to Geotechnic Report)

α = 4

Kvo = 1/0.3 × α × Eo = 1/0.3 × 4 7000 = 93333 kN/

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{1.50 \times 1.50} = 1.500$ m²

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$= 93333.333 \times (1.500 / 0.3)^{-3/4} = 27913.2$ kN/

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 8	27913.200	0.2250	1.0000	0.2250	6280.5
2, 7	27913.200	0.1650	1.0000	0.1650	4605.7
3 ~ 6	27913.200	0.1800	1.0000	0.1800	5024.4

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

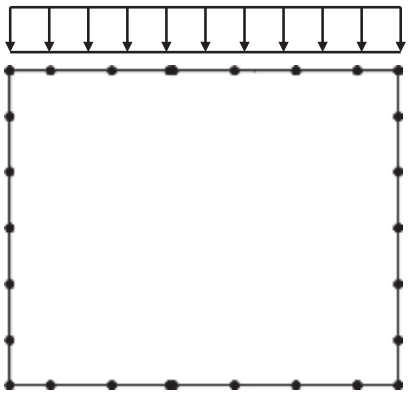
3) Loading

(1) LOAD-1 : Self weight - Automatic consideration in program

(2) LOAD-2,3 : Vertical earth pressure

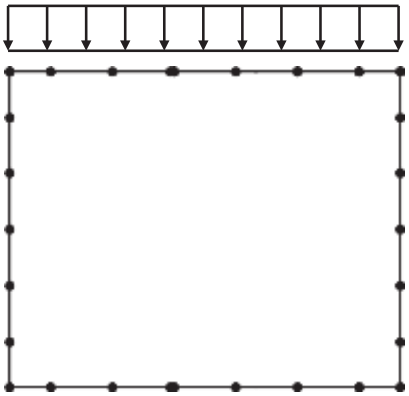
Psvh= 9.821 kN/m² (Exist ground water)

Psv= 9.821 kN/m² (No exist ground water)



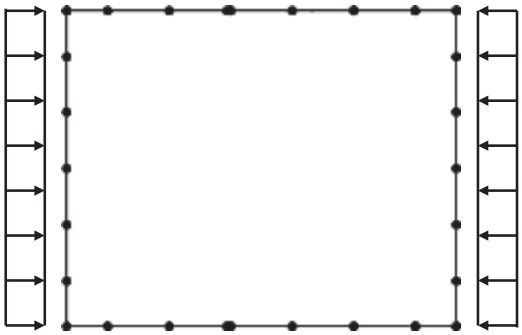
(3) LOAD-4 : Live Load

Pvl= 77.199 kN/m²



(4) LOAD-5 : Live Load Surcharge

Pvlh= 38.600 kN/m²

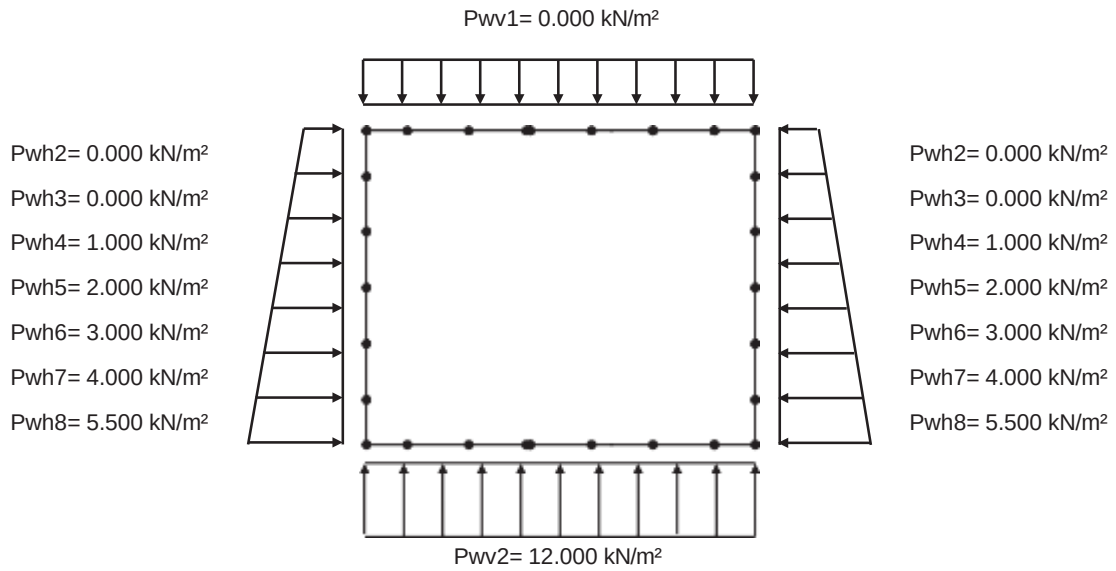


Pvlh= 38.600 kN/m²

(5) LOAD-6 : Horizontal Earth Pressure (No Ground Water)

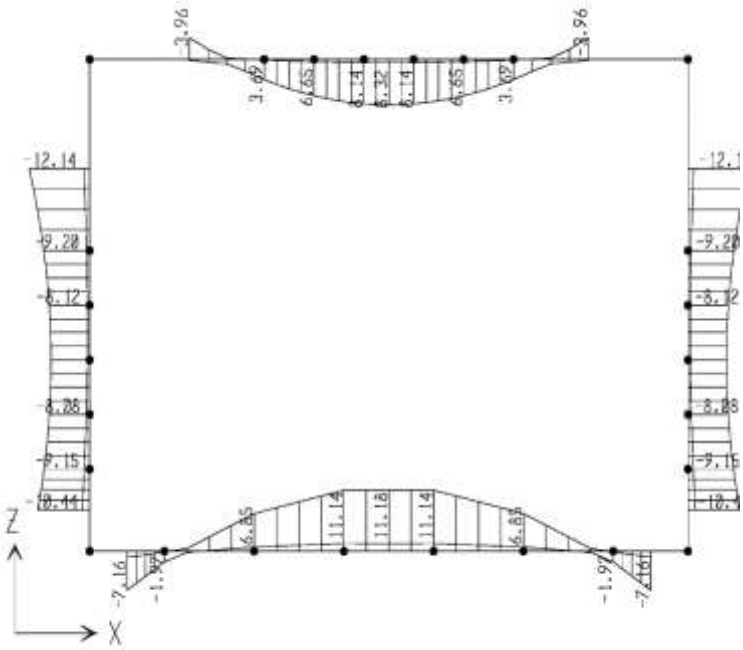
(6) LOAD-7 : Horizontal Earth Pressure (Ground Water)

(7) LOAD-8 : Ground Water Pressure

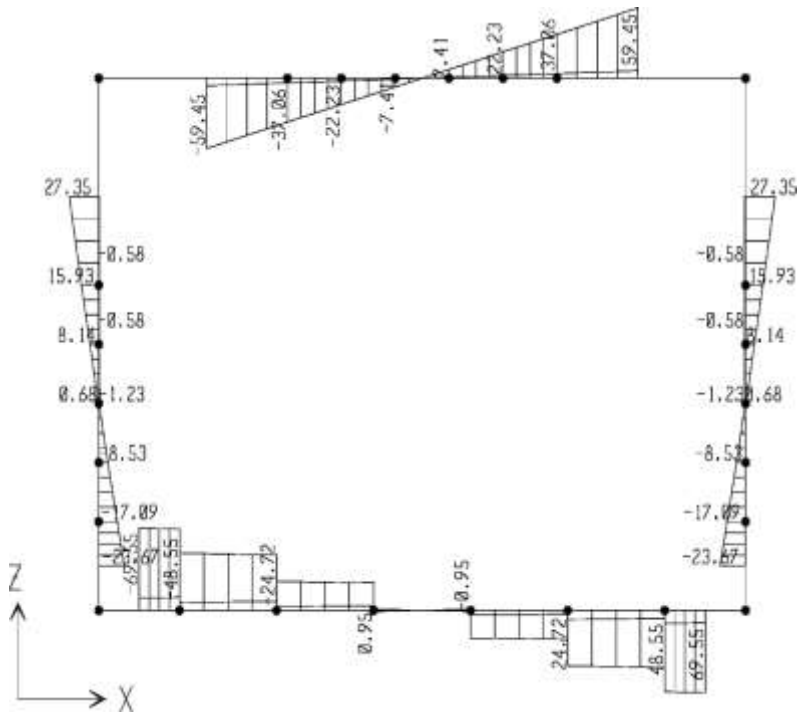


1.3.6 Summary of Analysis Results

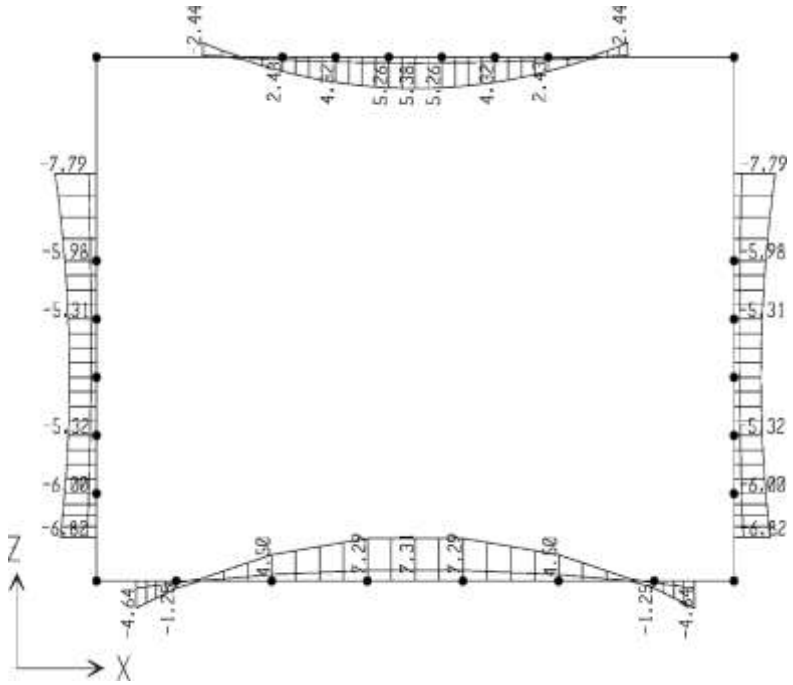
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	ØMn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	18.85
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	6.12
Wall	Top(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	6.14
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	25.740	D13 @ 400	-
	Middle(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	6.32
	Bottom(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	5.58
Bottom Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	8.13
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	4.55

1.3.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm		
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm		
M_u	=	3.955	kN·m	V_u	=	59.454	kN	M_o	=		2.442	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 33.721 ㎠

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎠ (5 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	7715.9	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	1045.5	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00014	㎠	$A_{s,4/3req}$	=	45.0	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00014	㎠	$A_{s,min}$	=	45.0	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00208	㎠	$A_{s,min}$	=	645.0	㎠

∠ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 74.556 kN·m > Mu = 3.955 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 59.454 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 2.442 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 0.322 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 2.442 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 12.912 MPa

fst = fs x (H - d' - χ) / (d - χ) = 13 x (367 - 57 - 0) / (311 - 52) = 12.91 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 12.91) - 2.5 x 50.00 = 8115.65 ㎜

300 x (280 / fs) = 300 x (280 / 12.91) = 6505.78 ㎜

Sa = 6505.78 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (6505.78 mm) Ā O.K

2) Top Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	8.322	kN·m	V_u	=	0.000	kN	M_o	=		5.383	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 103.510 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00065	㎟	$A_{s,4/3req}$	=	138.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00065	㎟	$A_{s,min}$	=	138.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 8.322 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 5.383 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 1.283 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 5.383 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 41.834 MPa

fst = fs x (H - d' - χ) / (d - χ) = 42 x (300 - 87 - 1) / (214 - 42) = 41.83 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 41.83) - 2.5 x 80.00 = 2343.40 ㅡ

300 x (280 / fs) = 300 x (280 / 41.83) = 2007.95 ㅡ

Sa = 2007.95 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (2007.95 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.500 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 1.5 / 28 × (0.43 + 420 / 700)

= 0.055 m < H = 0.300 m Ā O.K

3) Wall - Top

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm			
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm			
M_u	=	12.136	kN·m	V_u	=	27.342	kN	M_o	=		7.786	kN·m			

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИΠ_Ţ ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ ИΠ_Ţ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 103.665 ㎠

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎠ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	7715.9	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	1045.5	㎠
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00045	㎠	$A_{s,4/3req}$	=	138.2	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00045	㎠	$A_{s,min}$	=	138.2	㎠
P_{use}	=	A_s / (B·d)	=	0.00208	㎠	$A_{s,min}$	=	645.0	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 12.136 kN·m

Ā O.K

ⓒ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 27.342 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓒ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 7.786 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 1.027 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 7.786 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 41.158 MPa

fst = fs x (H - d' - χ) / (d - χ) = 41 x (367 - 57 - 1) / (311 - 52) = 41.16 MPa

ⓒ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 41.16) - 2.5 x 50.00 = 2460.15 ㎜

300 x (280 / fs) = 300 x (280 / 41.16) = 2040.91 ㎜

Sa = 2040.91 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (2040.91 mm) Ā O.K

4) Wall - Middle(In)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	0.000	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 5.726) / 5.726

= 0.1089

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИΠ_Π ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ ИΠ_Π

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$ □ Req. As = 0.000 ㎠

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎠ (3 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00000	㎠	$A_{s,4/3req}$	=	0.0	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00000	㎠	$A_{s,min}$	=	0.0	㎠
P_{use}	=	A_s / (B·d)	=	0.00151	㎠	$A_{s,min}$	=	322.5	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

ØMn = 0.9 × As × fy × (d - a/2) = 25.740 kN·m > Mu = 0.000 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × {1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 322.50)]

= 30.711 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.000 / [1000 × 30.711 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.000 / [322.500 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

fst = fs x (H - d' - χ) / (d - χ) = 0 x (300 - 87 - 0) / (214 - 31) = 0.00 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 7.0E+08 ⌟

300 x (280 / fs) = 300 x (280 / 0.00) = 5.5E+08 ⌟

Sa = 5.51E+08 ⌟ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (5.5E+08 mm) Ā O.K

5) Wall - Middle(Out)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	9.205	kN·m	V_u	=	0.000	kN	M_o	=		6.001	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ㅇ ㅌㅍㄱㅌㅋㅋㅍㅋㅈㅌ ㅎㅁㅅ ㅇ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 100.330 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00055	㎟	$A_{s,4/3req}$	=	133.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00055	㎟	$A_{s,min}$	=	133.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 9.205 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 6.001 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 1.162 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 6.001 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 40.731 MPa

fst = fs x (H - d' - χ) / (d - χ) = 41 x (300 - 57 - 1) / (244 - 45) = 40.73 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 40.73) - 2.5 x 50.00 = 2487.25 ㎜

300 x (280 / fs) = 300 x (280 / 40.73) = 2062.30 ㎜

Sa = 2062.30 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (2062.30 mm) Ā O.K

6) Wall - Bottom

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	10.441	kN·m	V_u	=	17.099	kN	M_o	=		6.818	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 113.847 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00062	㎟	$A_{s,4/3req}$	=	151.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00062	㎟	$A_{s,min}$	=	151.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 10.441 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 17.099 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × {1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 6.818 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 1.320 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 6.818 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 46.278 MPa

fst = fs x (H - d' - χ) / (d - χ) = 46 x (300 - 57 - 1) / (244 - 45) = 46.28 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 46.28) - 2.5 x 50.00 = 2174.14 ㎜

300 x (280 / fs) = 300 x (280 / 46.28) = 1815.11 ㎜

Sa = 1815.11 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1815.11 mm) Ā O.K

7) Bottom Slab - At the end of the point

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	7.161	kN·m	V_u	=	48.568	kN	M_o	=		4.642	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 77.992 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎡ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	6051.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	819.9	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00043	㎡	$A_{s,4/3req}$	=	104.0	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00043	㎡	$A_{s,min}$	=	104.0	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00265	㎡	$A_{s,min}$	=	645.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 58.221 kN·m > Mu = 7.161 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 48.568 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × {1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 4.642 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 0.898 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 4.642 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 31.505 MPa

fst = fs x (H - d' - χ) / (d - χ) = 32 x (300 - 57 - 1) / (244 - 45) = 31.51 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 31.51) - 2.5 x 50.00 = 3252.23 ㎜

300 x (280 / fs) = 300 x (280 / 31.51) = 2666.24 ㎜

Sa = 2666.24 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (2666.24 mm) Ā O.K

8) Bottom Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	11.182	kN·m	V_u	=	0.000	kN	M_o	=		7.315	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИД_Т ТФГТУКУФУZТ НПΣ ИД_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 139.264 ㎡

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00087	㎟	$A_{s,4/3req}$	=	185.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00087	㎟	$A_{s,min}$	=	185.7	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 11.182 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 7.315 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 1.744 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 7.315 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 56.855 MPa

fst = fs x (H - d' - χ) / (d - χ) = 57 x (300 - 87 - 2) / (214 - 42) = 56.86 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 56.86) - 2.5 x 80.00 = 1671.42 ㎜

300 x (280 / fs) = 300 x (280 / 56.86) = 1477.43 ㎜

Sa = 1477.43 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1477.43 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.500 m
- Thickness : H = 0.300 m

ⓐ T_{min} = L / 28 × (0.43 + fy / 700) = 1.5 / 28 × (0.43 + 420 / 700)

= 0.055 m < H = 0.300 m Ā O.K

1.3.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Wall (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

3) Bottom Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

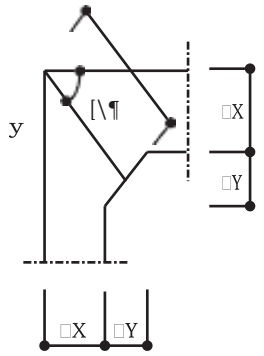
· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

1.3.9 Corner Design

1) Top slab Cheak



a1	=	300	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 7.786 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 565.7 \text{ mm}$$

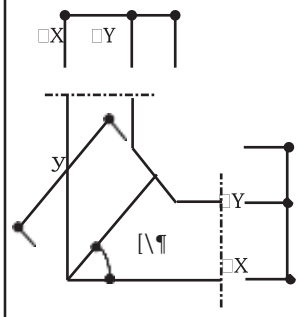
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 7.786 \times 10^{\wedge}6}{565.7^2 \times 1000} = 0.122 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.122 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	300	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm

M_o = 6.818 kN·m

R = □($a1^2$ + $a2^2$) = 424.3 mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 6.818 \times 10^{\wedge}6}{424.3^2 \times 1000} = 0.189 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.189 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

1.4 Box Culvert 4 (STA.3+300.00)

1@1.2×0.9

FH=0.39 m [SI UNIT]

1.4.1 Design Conditions

1) General Items

(1) Type of Culvert : 1 Box

(2) Width (w) : 1 @ 1.2 m

(3) Height (h) : 0.90 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength : fc' = 32 MPa

¤ Modulus of Elasticity : Ec = 26587 MPa

(2) Reinforcement bar

⊳ Yield Strength : fy = 420 MPa

⊲ Modulus of Elasticity : Es = 200000 MPa

3) Material weight

(1) Reinforced Concrete : ωc = 25.00 kN/m³

(2) plain concrete : γcn = 23.50 kN/m³

(3) Pavement : γasp = 23.00 kN/m³

(4) Subterranean : γw = 10.00 kN/m³

4) Soil

(1) Wet Unit Weight : γt = 19.00 kN/m³

(2) Submerged Unit Weight : γsub = 10.00 kN/m³

(3) angle of internal friction : φ = 28.00 °

(4) coefficient of earth pressure atrest : Ko = 1-sinΦ = 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

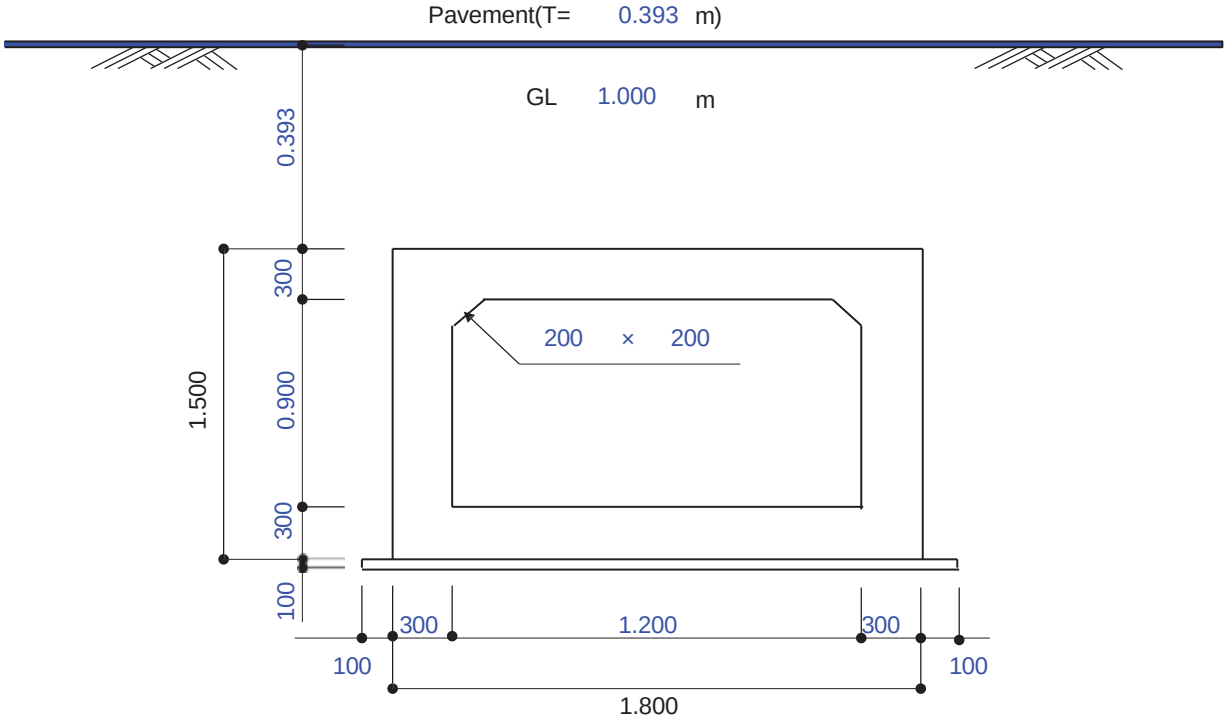
· SAP2000 (Structure Analysis Program)

8) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.4.2 Section Assumption



1.4.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.393 × 1.800 × 23.0	16.270
Vertical earth pressure (EV)	No exist ground water	0.000 × 1.800 × 19.0	0.000
	Exist ground water	(0.000 × 19.0 + 0.000 × 10.0) × 1.800	0.000
Ground Water(WA')		0.000 × 1.800 × 10.0	0.000
Sub Total		Surcharge Load for Bouyancy Check	16.270
Slab(DC)	Top	0.300 × 1.800 × 25.0	13.500
	Bottom	0.300 × 1.800 × 25.0	13.500
Wall(DC)	Left	0.300 × 0.900 × 25.0	6.750
	Inner	0.000 × 0.900 × 25.0	0.000
	Right	0.300 × 0.900 × 25.0	6.750
Hunch(DC)		0.200 × 0.200 / 2 × 25.0 × 2 EA	1.000
Sub Total		Surcharge Load for Bouyancy Check	41.500

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.000 m)

- Total Load for Bouyancy Check : 57.770 kN
- Uplife force : 1.800 ×(1.893 - 1.000)× 10.0 kN/□ = 16.074 kN
- Safety factor = 1.25

□ F.S = 57.770 / 16.074 = 3.594 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL 0.000 m)

- Total Load for Bouyancy Check :

41.500 +(0.393 × 1.800 × 10.000 kN/□) = 48.574 kN

- Uplife force : 1.800 × 1.500 × 10.000 kN/□ = 27.000 kN
- Safety factor = 1.1

□ F.S = 48.574 / 27.000 = 1.799 > 1.1 - O.K

Ā Securing safety at all ground water levels

3) Allowable vertical bearing capacity check

(1) Load

- Dead load
  - Self weight of Structure = 41.500 / 1.800 = 23.056 kN/m²
  - Vertical earth pressure = 16.270 / 1.800 = 9.039 kN/m² (No exist ground water)
- Live load = 87.315 kN/m² (Refer to 1.1.4.2)
- Water load in Culvrt = 0.900 ๑ 10.000 = 9.000 kN/m²

(2) Allowable vertical bearing capacity

- Q_{max} = 128.410 kN/m²
- Q_a = 291.670 kN/m² (Refer to Geotechnic Report)

□ Q_a = 291.670 kN/□ > Q_{max} = 128.410 kN/□ - O.K

1.4.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

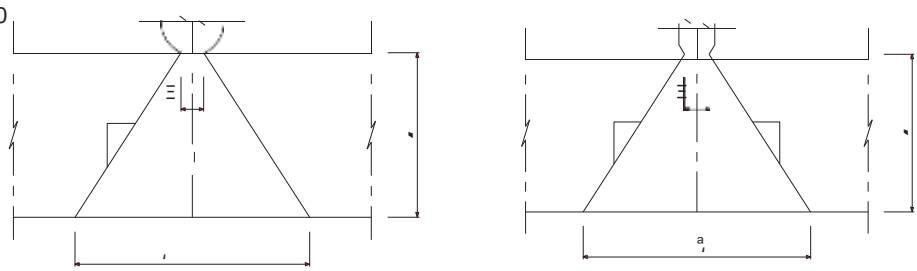
TYPE	Depth (m)	Unit weight (kN/)	Load (kN/m²)	
Pavement	0.393	23.000	1.000 × 0.393 × 23.000 =	9.039
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
□	0.393		Psv =	9.039 kN/m²

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/)	Load (kN/m²)	
Pavement	0.393	23.000	1.000 × 0.393 × 23.000 =	9.039
Vertical earth pressure	0.000	19.000	1.000 × 0.000 × 19.000 =	0.000
	0.000	10.000	1.000 × 0.000 × 10.000 =	0.000
□	0.393		Psvh =	9.039 kN/m²

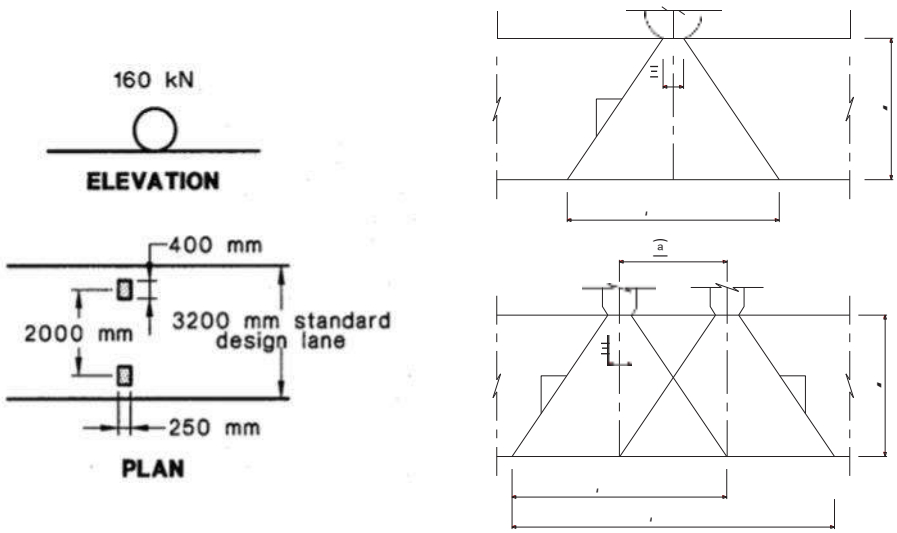
2) Live Load

(1) W80



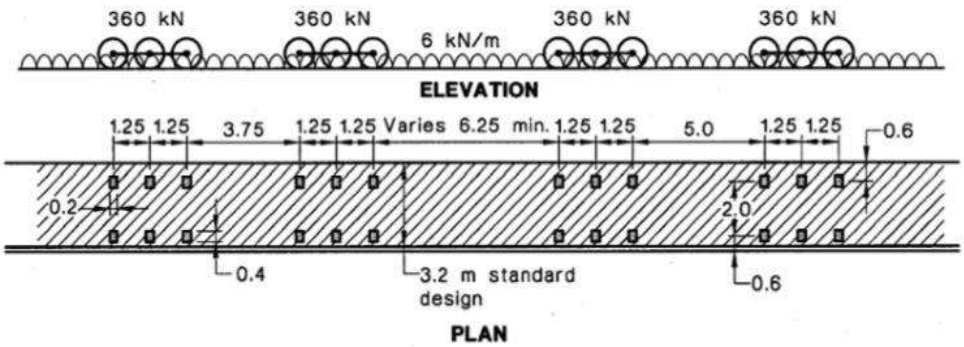
$$P_{vl} = \frac{80}{(0.25+2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.786) \times (0.4 + 0.786)} = 65.110 \text{ kN/m}^2$$

(2) A160

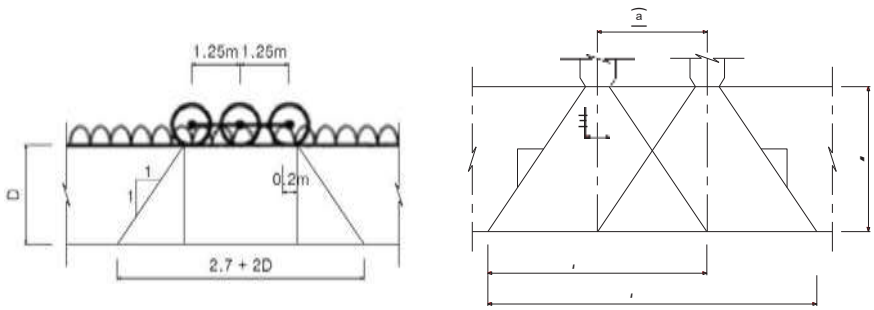


$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 0.786) \times (0.4 + 0.786)} = 65.110 \text{ kN/m}^2$$

(3) M1600



- Axle group

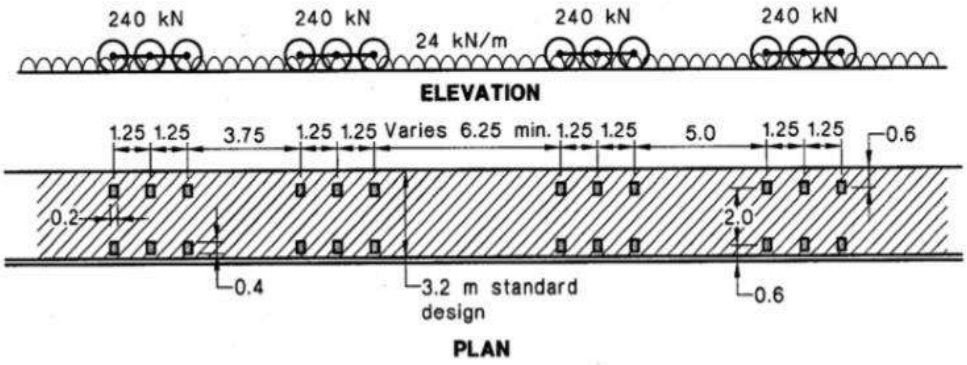


$$P_{v1} = \frac{60}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{60}{(0.2 + 0.786) \times (0.4 + 0.786)} = 51.309 \text{ kN/m}^2$$

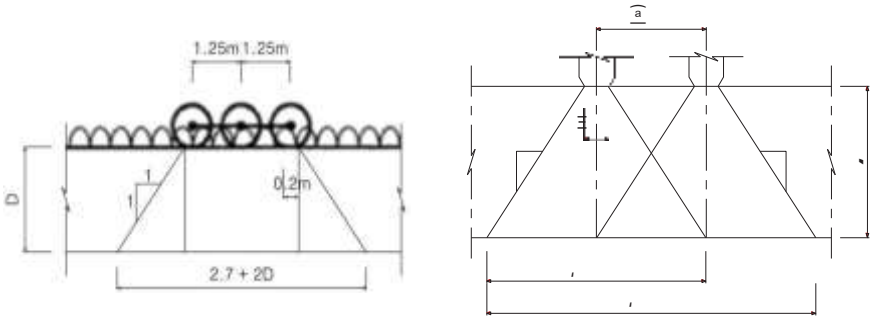
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 51.309 + 1.875 = 53.184 kN/m²

(4) S1600



- Axle group

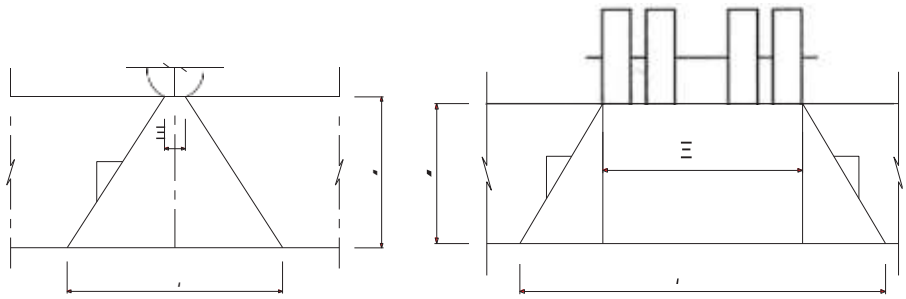


$$P_{v1} = \frac{40}{(0.2 + 2D) \times (0.4 + 2D)} = \frac{40}{(0.2 + 0.786) \times (0.4 + 0.786)} = 34.206 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 34.206 + 7.500 = 41.706 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2\times0.393) \times (1.4 + 2\times0.393)} = 57.994 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	65.110	0.34	87.315
A160	65.110	0.34	87.315
M1600	53.184	0.26	67.049
S1600	41.706	0.00	41.706
HLP	57.994	0.10	63.793

□ P_{vl} = 87.315 kN/m² = 87.315 kN/m²

(7) Live Load Surcharge

□ P_{vlh} = 87.315 kN/m² × 0.500 = 43.658 kN/m²

3) Lateral Earth Pressure

- coefficient of earth pressure at rest : Ko = 1 - sin30 = 0.500

- No exist ground water

P_{sh} = k_o × γ_t × H

P_{sh1} = 0.500 x (23 x 0.393 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000) = 4.520 kN/m²

P_{sh2} = 4.520 + 0.500 × 19.0 × 0.150 = 5.945 kN/m²

P_{sh3} = 5.945 + 0.500 × 19.0 × 0.350 = 9.270 kN/m²

P_{sh4} = 9.270 + 0.500 × 19.0 × 0.175 = 10.933 kN/m²

P_{sh5} = 10.933 + 0.500 × 19.0 × 0.175 = 12.596 kN/m²

P_{sh6} = 12.596 + 0.500 × 19.0 × 0.175 = 14.259 kN/m²

P_{sh7} = 14.259 + 0.500 × 19.0 × 0.175 = 15.922 kN/m²

P_{sh8} = 15.922 + 0.500 × 19.0 × 0.150 = 17.347 kN/m²

- Exist ground water

$P_{sh'}$ = k_o × (γ_t × H_1 + γ_{sub} × H_2)

$P_{sh1'}$ = 0.500 x (23 x 0.393 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.000 + 10 x 0.000) = 4.520 kN/m²

$P_{sh2'}$ = 4.520 + 0.500 × 19.0 × 0.150 = 5.945 kN/m²

$P_{sh3'}$ = 5.945 + 0.500 × 19.0 × 0.350 = 9.270 kN/m²

$P_{sh4'}$ = 9.270 + 0.500 × 10.0 × 0.175 = 10.145 kN/m²

$P_{sh5'}$ = 10.145 + 0.500 × 10.0 × 0.175 = 11.020 kN/m²

$P_{sh6'}$ = 11.020 + 0.500 × 10.0 × 0.175 = 11.895 kN/m²

$P_{sh7'}$ = 11.895 + 0.500 × 10.0 × 0.175 = 12.770 kN/m²

$P_{sh8'}$ = 12.770 + 0.500 × 10.0 × 0.150 = 13.520 kN/m²

4) Ground Water Load

(1) Horizontal ground Water Pressure

$P_{wh} = \gamma_w \times H_2$

P_{wh1} = 10.0 × 0.000 = 0.000 kN/m²

P_{wh2} = 0.000 + 0.0 × 0.150 = 0.000 kN/m²

P_{wh3} = 0.000 + 0.0 × 0.350 = 0.000 kN/m²

P_{wh4} = 0.000 + 10.0 × 0.175 = 1.750 kN/m²

P_{wh5} = 1.750 + 10.0 × 0.175 = 3.500 kN/m²

P_{wh6} = 3.500 + 10.0 × 0.175 = 5.250 kN/m²

P_{wh7} = 5.250 + 10.0 × 0.175 = 7.000 kN/m²

P_{wh8} = 7.000 + 10.0 × 0.150 = 8.500 kN/m²

(2) Vertical ground Water Pressure

- Top Slab : P_{wv1} = 10.0 × 0.000 = 0.000 kN/m²

-Bottom Slab : P_{wv2} = 10.0 × 1.500 = 15.000 kN/m²

5) Load Combination

(1) Ultimate Load

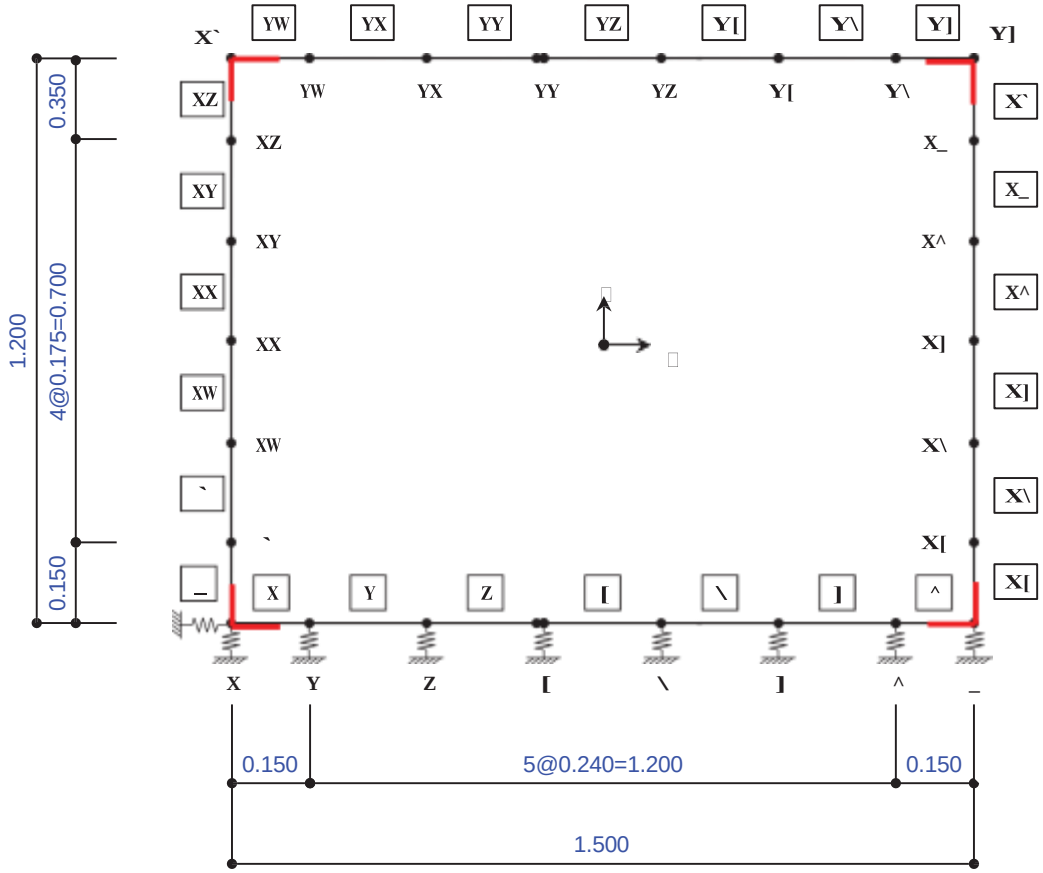
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.60		1.60	1.60	1.60			
COMB 3	1.20	1.60		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.60	1.60	1.60		1.60	1.60	1.60
COMB 7	1.20		1.60	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	1.000		1.00	1.00	1.00			
SCOMB 2	1.00	1.000		1.00	1.00	0.56			
SCOMB 3	1.00	1.000				0.00			
SCOMB 4	1.00		1.000	1.00	1.00		1.00	1.00	1.000
SCOMB 5	1.00		1.000	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		1.000				0.00	0.00	0.00

1.4.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	Z	Section	Node	X	Z	Section
1	0.150	0.150	Bottom Slab	14	1.650	0.300	Right Wall
2	0.300	0.150		15	1.650	0.475	
3	0.540	0.150		16	1.650	0.650	
4	0.780	0.150		17	1.650	0.825	
5	1.020	0.150		18	1.650	1.000	
6	1.260	0.150		19	0.150	1.350	Top Slab
7	1.500	0.150		20	0.500	1.350	
8	1.650	0.150		21	0.660	1.350	
9	0.150	0.300	Left Wall	22	0.820	1.350	
10	0.150	0.475		23	0.980	1.350	
11	0.150	0.650		24	1.140	1.350	
12	0.150	0.825		25	1.300	1.350	
13	0.150	1.000		26	1.650	1.350	

(2) Section

NO.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~6	Bottom Slab
2	0.300	1.000	0.300	0.002250	9~12	Left Wall
3	0.300	1.000	0.300	0.002250	15~18	Right Wall
4	0.300	1.000	0.300	0.002250	21~25	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$kvo = 1/0.3 \times \alpha \times Eo$

Eo : the modulus of subgrade elasticity (kN/m²)

α : correction factor for calculating Eo

Eo = 7000 kN/m² (Refer to Geotechnic Report)

α = 4

$Kvo = 1/0.3 \times \alpha \times Eo = 1/0.3 \times 4 \quad 7000 = 93333$ kN/

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{1.80 \times 1.80} = 1.800$ m²

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$= 93333.333 \times (1.800 / 0.3)^{-3/4} = 24345.8$ kN/

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 8	24345.800	0.2250	1.0000	0.2250	5477.8
2, 7	24345.800	0.1950	1.0000	0.1950	4747.4
3 ~ 6	24345.800	0.2400	1.0000	0.2400	5843.0

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

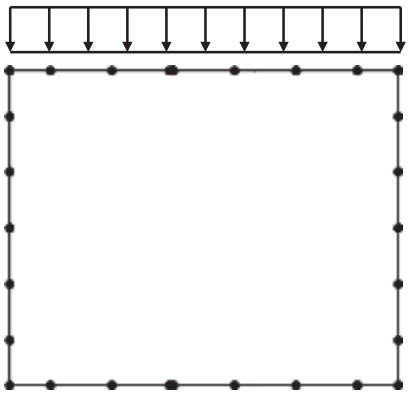
3) Loading

(1) LOAD-1 : Self weight - Automatic consideration in program

(2) LOAD-2,3 : Vertical earth pressure

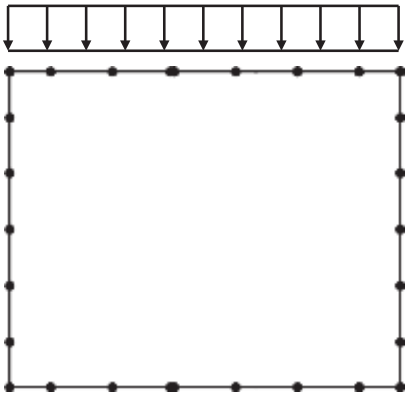
Psvh= 9.039 kN/m² (Exist ground water)

Psv= 9.039 kN/m² (No exist ground water)



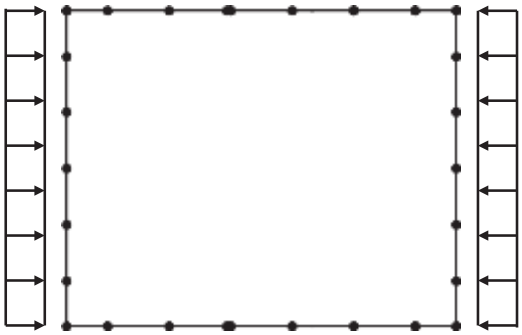
(3) LOAD-4 : Live Load

Pvl= 87.315 kN/m²



(4) LOAD-5 : Live Load Surcharge

Pvlh= 43.658 kN/m²

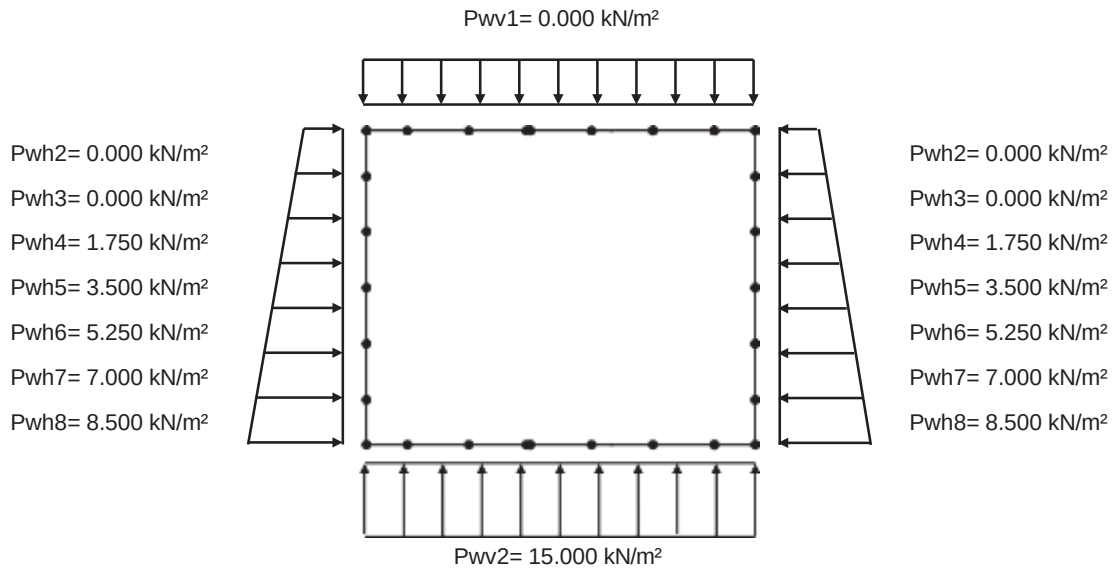


Pvlh= 43.658 kN/m²

(5) LOAD-6 : Horizontal Earth Pressure (No Ground Water)

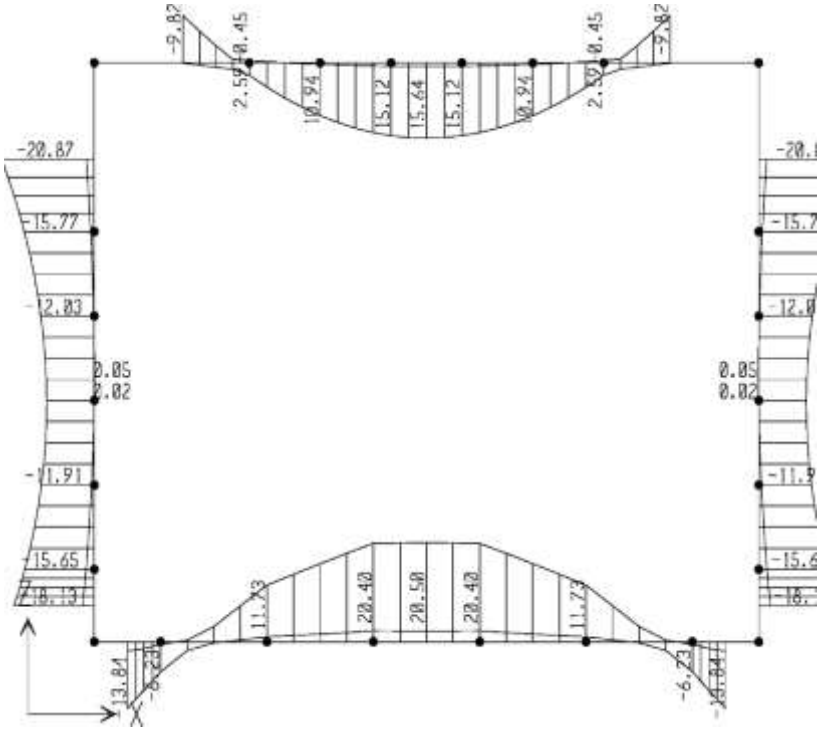
(6) LOAD-7 : Horizontal Earth Pressure (Ground Water)

(7) LOAD-8 : Ground Water Pressure

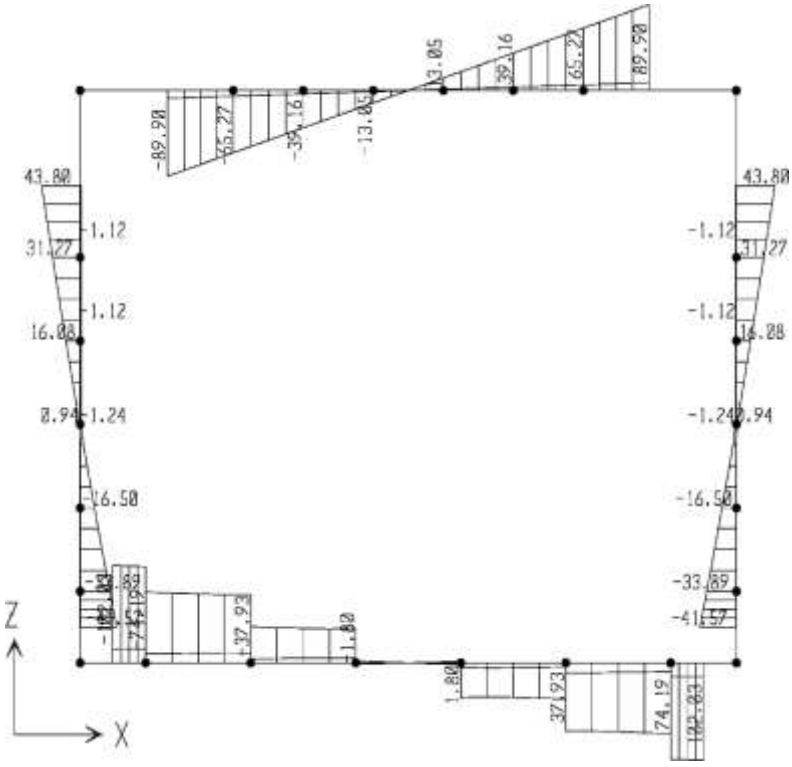


1.4.6 Summary of Analysis Results

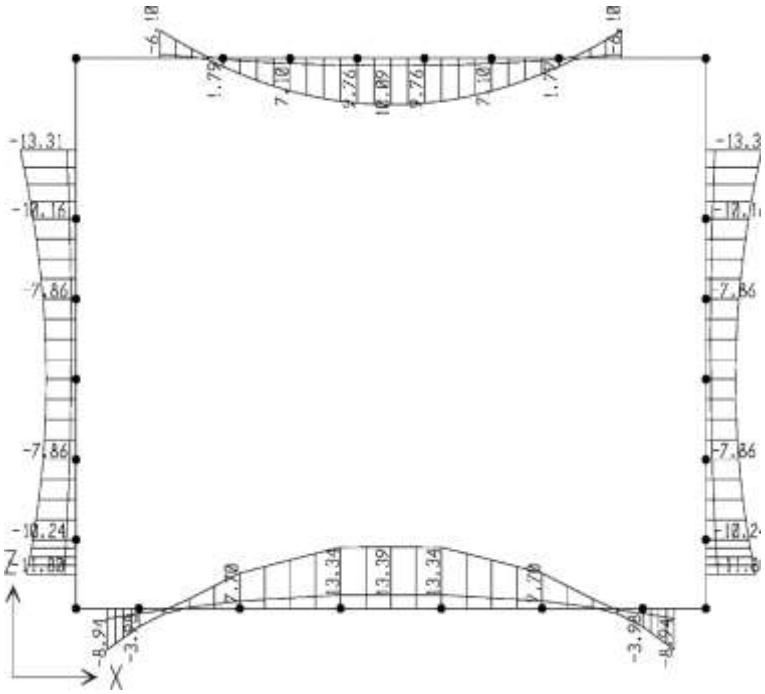
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	ØMn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	7.60
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	3.25
Wall	Top(-)	[illegible]	[illegible]	[illegible]	367	310.5	74.556	D13 @ 200	3.57
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	25.740	D13 @ 400	537.33
	Middle(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	3.69
	Bottom(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	3.21
Bottom Slab	End of the point(-)	[illegible]	[illegible]	[illegible]	300	243.5	58.221	D13 @ 200	4.20
	Middle(+)	[illegible]	[illegible]	[illegible]	300	213.5	50.907	D13 @ 200	2.48

1.4.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm	
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm	
M_u	=	9.813	kN·m	V_u	=	89.903	kN	M_o	=		6.100	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 83.784 ㎠

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎠ (5 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	7715.9	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	1045.5	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00036	㎠	$A_{s,4/3req}$	=	111.7	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00036	㎠	$A_{s,min}$	=	111.7	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00208	㎠	$A_{s,min}$	=	645.0	㎠

∠ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 74.556 kN·m > Mu = 9.813 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 89.903 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 6.100 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 0.805 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 6.100 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 32.245 MPa

fst = fs x (H - d' - χ) / (d - χ) = 32 x (367 - 57 - 1) / (311 - 52) = 32.25 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 32.25) - 2.5 x 50.00 = 3174.72 ㎜

300 x (280 / fs) = 300 x (280 / 32.25) = 2605.04 ㎜

Sa = 2605.04 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (2605.04 mm) Ā O.K

2) Top Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	15.643	kN·m	V_u	=	0.000	kN	M_o	=		10.096	kN·m	

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 195.210 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00122	㎟	$A_{s,4/3req}$	=	260.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00122	㎟	$A_{s,min}$	=	260.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 15.643 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 10.096 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 2.407 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 10.096 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 78.467 MPa

fst = fs x (H - d' - χ) / (d - χ) = 78 x (300 - 87 - 2) / (214 - 42) = 78.47 MPa

ㅿ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 78.47) - 2.5 x 80.00 = 1155.98 ㅡ

300 x (280 / fs) = 300 x (280 / 78.47) = 1070.51 ㅡ

Sa = 1070.51 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1070.51 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.800 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 1.8 / 28 × (0.43 + 420 / 700)

= 0.066 m < H = 0.300 m Ā O.K

3) Wall - Top

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm			
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm			
M_u	=	20.872	kN·m	V_u	=	43.793	kN	M_o	=	13.313	kN·m				

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 11.451) / 11.451

= 0.0783

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 178.629 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00077	㎟	$A_{s,4/3req}$	=	238.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00077	㎟	$A_{s,min}$	=	238.2	㎟
P_{use}	=	A_s / (B·d)	=	0.00208	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 74.556 kN·m > Mu = 20.872 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 43.793 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 645.00)]

= 51.682 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 13.313 / [1000 × 51.682 x (310.5 - 51.682 / 3)] × 10^6

= 1.757 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 13.313 / [645.000 x (310.5 - 51.682 / 3)] × 10^6

= 70.377 MPa

fst = fs x (H - d' - χ) / (d - χ) = 70 x (367 - 57 - 2) / (311 - 52) = 70.38 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 70.38) - 2.5 x 50.00 = 1386.86 ㎜

300 x (280 / fs) = 300 x (280 / 70.38) = 1193.58 ㎜

Sa = 1193.58 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1193.58 mm) Ā O.K

4) Wall - Middle(In)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	0.048	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (Φ)

a = 4.703

Because T = C , *c* = 4.703 / β1 = 4.703 / 0.821 = 5.726 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 5.726) / 5.726

= 0.1089

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 0.594 ㎠

Use As = D 13 @ 800 + D 13 @ 800 = 322.50 ㎠ (3 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00000	㎠	$A_{s,4/3req}$	=	0.8	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00000	㎠	$A_{s,min}$	=	0.8	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00151	㎠	$A_{s,min}$	=	322.5	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 4.703 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 25.740 kN·m > Mu = 0.048 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 322.50 / 1000 + 8 × 322.50 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 322.50)]

= 30.711 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.000 / [1000 × 30.711 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.000 / [322.500 x (213.5 - 30.711 / 3)] × 10^6

= 0.000 MPa

fst = fs x (H - d' - χ) / (d - χ) = 0 x (300 - 87 - 0) / (214 - 31) = 0.00 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 7.0E+08 ㎜

300 x (280 / fs) = 300 x (280 / 0.00) = 5.5E+08 ㎜

Sa = 5.51E+08 ㎜ Applying Minimum value

S = 1,000 / 3 Ea = 400.0 < Sa (5.5E+08 mm) Ā O.K

5) Wall - Middle(Out)

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	15.774	kN·m	V_u	=	0.000	kN	M_o	=		10.242	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Т ТФГТУКУФУZТ НПΣ ИЛ_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 172.320 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00094	㎟	$A_{s,4/3req}$	=	229.8	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00094	㎟	$A_{s,min}$	=	229.8	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 15.774 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 10.242 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 1.982 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 10.242 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 69.514 MPa

fst = fs x (H - d' - χ) / (d - χ) = 70 x (300 - 57 - 2) / (244 - 45) = 69.51 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 69.51) - 2.5 x 50.00 = 1405.62 ㅡ

300 x (280 / fs) = 300 x (280 / 69.51) = 1208.39 ㅡ

Sa = 1208.39 ㅡ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1208.39 mm) Ā O.K

6) Wall - Bottom

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	18.133	kN·m	V_u	=	33.898	kN	M_o	=		11.805	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 198.252 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00109	㎟	$A_{s,4/3req}$	=	264.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00109	㎟	$A_{s,min}$	=	264.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 18.133 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 33.898 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × {1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.805 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 2.285 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.805 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 80.125 MPa

fst = fs x (H - d' - χ) / (d - χ) = 80 x (300 - 57 - 2) / (244 - 45) = 80.13 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 80.13) - 2.5 x 50.00 = 1202.92 ㎜

300 x (280 / fs) = 300 x (280 / 80.13) = 1048.36 ㎜

Sa = 1048.36 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1048.36 mm) Ā O.K

7) Bottom Slab - At the end of the point

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	13.847	kN·m	V_u	=	74.227	kN	M_o	=		8.948	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 11.451) / 11.451

= 0.0608

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 151.165 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00083	㎟	$A_{s,4/3req}$	=	201.6	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00083	㎟	$A_{s,min}$	=	201.6	㎟
P_{use}	=	A_s / (B·d)	=	0.00265	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 58.221 kN·m > Mu = 13.847 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 74.227 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 645.00)]

= 45.234 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 8.948 / [1000 × 45.234 x (243.5 - 45.234 / 3)] × 10^6

= 1.732 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 8.948 / [645.000 x (243.5 - 45.234 / 3)] × 10^6

= 60.733 MPa

fst = fs x (H - d' - χ) / (d - χ) = 61 x (300 - 57 - 2) / (244 - 45) = 60.73 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 60.73) - 2.5 x 50.00 = 1626.93 ㎜

300 x (280 / fs) = 300 x (280 / 60.73) = 1383.11 ㎜

Sa = 1383.11 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (1383.11 mm) Ā O.K

8) Bottom Slab - Middle

(1) Section Design

ㅇ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm		
M_u	=	20.516	kN·m	V_u	=	0.000	kN	M_o	=		13.402	kN·m		

- Check of Strength reduction factor (Φ)

a = 9.406

Because T = C , *c* = 9.406 / β1 = 9.406 / 0.821 = 11.451 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 11.451) / 11.451

= 0.0529

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_T ТФГТУКУФУZТ НПΣ И□_T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 256.596 ㎟

Use As = D 13 @ 400 + D 13 @ 400 = 645.00 ㎟ (5 ea/m)

ㅇ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_c/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00160	㎟	$A_{s,4/3req}$	=	342.1	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00160	㎟	$A_{s,min}$	=	342.1	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	645.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅇ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 9.406 mm

ØMn = 0.9 × As × fy × (d - a/2) = 50.907 kN·m > Mu = 20.516 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 645.00 / 1000 + 8 × 645.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 645.00)]

= 42.062 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 13.402 / [1000 × 42.062 x (213.5 - 42.062 / 3)] × 10^6

= 3.195 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 13.402 / [645.000 x (213.5 - 42.062 / 3)] × 10^6

= 104.163 MPa

fst = fs x (H - d' - χ) / (d - χ) = 104 x (300 - 87 - 3) / (214 - 42) = 104.16 MPa

ⓐ Maximum center space of reinforcement

C_C = 86.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 104.16) - 2.5 x 80.00 = 821.47 ㎜

300 x (280 / fs) = 300 x (280 / 104.16) = 806.42 ㎜

Sa = 806.42 ㎜ Applying Minimum value

S = 1,000 / 5 Ea = 200.0 < Sa (806.42 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 1.800 m
- Thickness : H = 0.300 m

T_{min} = L / 28 × (0.43 + fy / 700) = 1.8 / 28 × (0.43 + 420 / 700)

= 0.066 m < H = 0.300 m Ā O.K

1.4.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Wall (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

3) Bottom Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

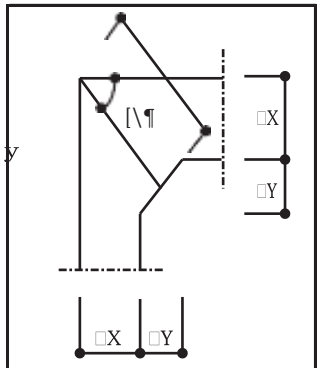
· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

1.4.9 Corner Design

1) Top slab Cheak



a1	=	300	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 13.313 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 565.7 \text{ mm}$$

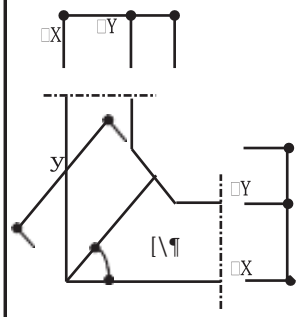
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 13.313 \times 10^6}{565.7^2 \times 1000} = 0.208 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.208 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	300	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm

M_o = 11.805 kN·m

R = □($a1^2$ + $a2^2$) = 424.3 mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 11.805 \times 10^6}{424.3^2 \times 1000} = 0.328 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.328 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2. CORRUGATED STEEL PIPE

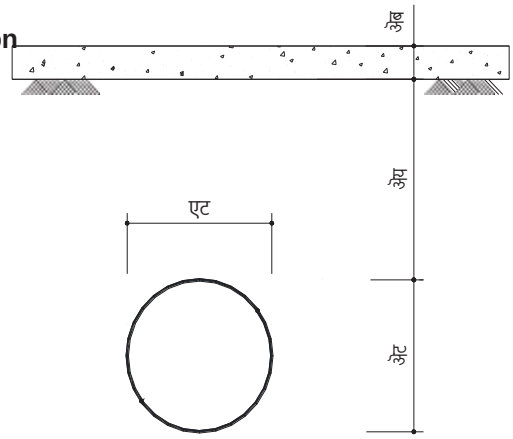
2.1 Corrugated Steel Pipe (D=1,200)

D=1.2M

H=1.23 m [SI UNIT]

2.1.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 0.911 m
- Pipe span : Sc = 1.200 m
- Corrugated steel pipe specifications
  - pitch x depth : **68x13** mm
  - thickness : **2.0** mm
  - yield strength : fy = 205 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

2.1.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			

152x51	17.326	17.375	17.425	17.475	17.523

2.1.3 Loads

1) Dead Load

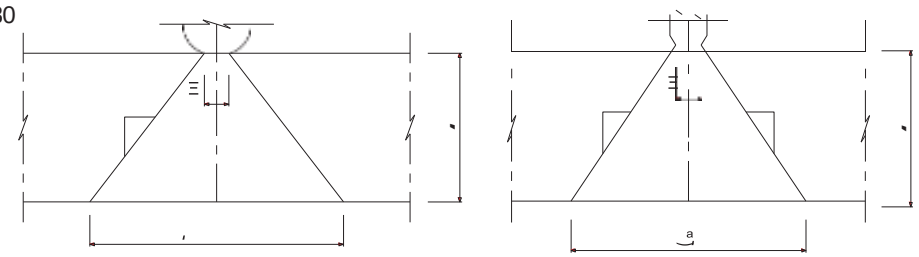
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 0.911 = 25.580 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pip
g = unit weight of the soil
H = height of cover over the pipe

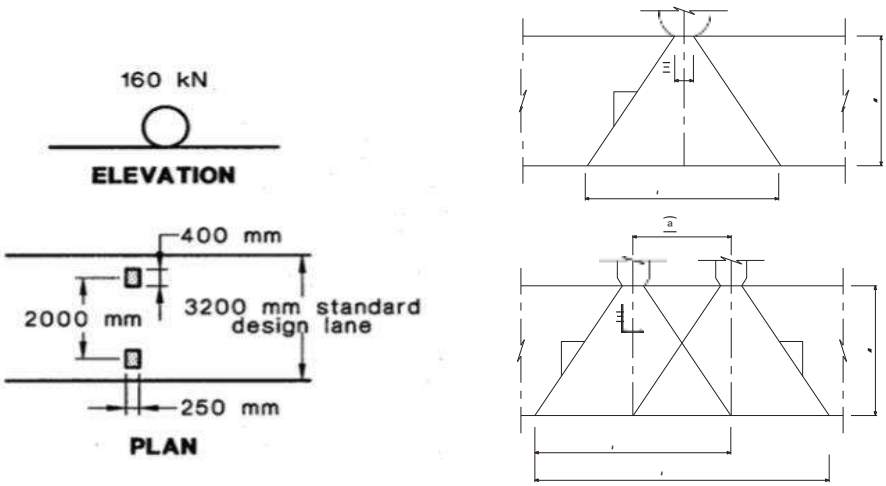
2) Live Load

(1) W80



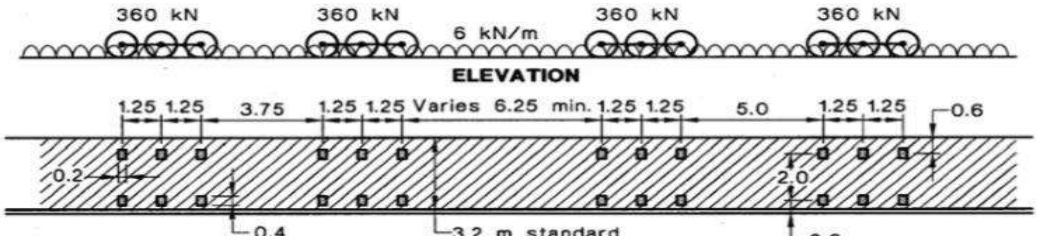
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 2.462) \times (0.4 + 2.462)} = 10.307 \text{ kN/m}^2$$

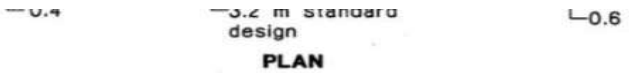
(2) A160



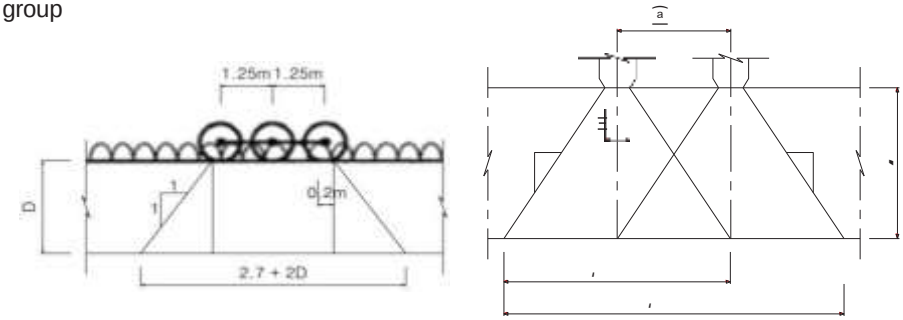
$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 2.462) \times (2.4 + 2.462)} = 12.134 \text{ kN/m}^2$$

(3) M1600





- Axle group

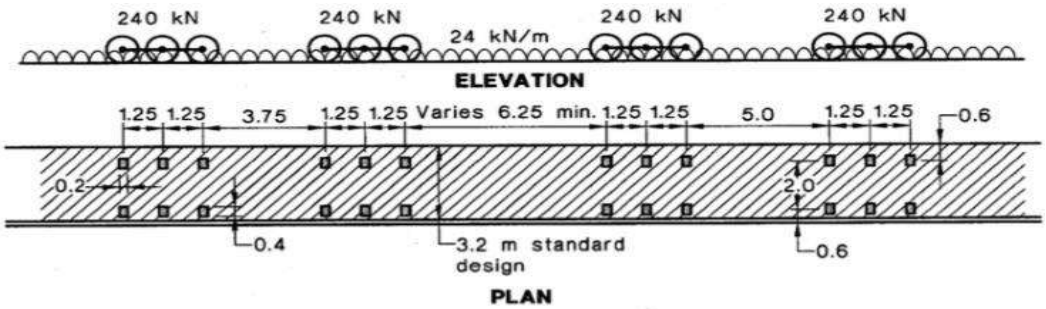


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 2.462) \times (2.4 + 2.462)} = 14.344 \text{ kN/m}^2$$

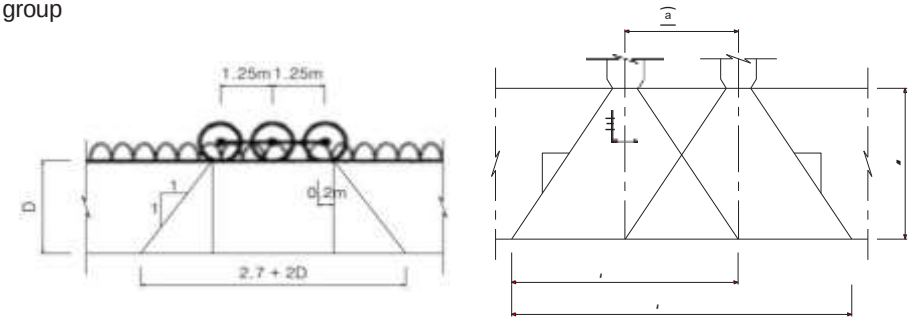
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 14.344 + 1.875 = 16.219 kN/m²

(4) S1600



- Axle group

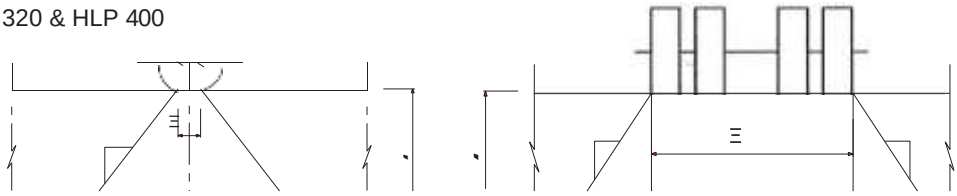


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 2.462) \times (2.4 + 2.462)} = 9.563 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 9.563 + 7.500 = 17.063 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2.462) \times (1.4 + 2.462)} = 12.159 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	10.307	0.22	12.527
A160	12.134	0.22	14.747
M1600	16.219	0.18	19.088
S1600	17.063	0.00	17.063
HLP	12.159	0.10	13.375

□ LL = 19.088 kN/m² = 19.088 kN/m²

2.1.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min}= 900 ¸ < H = 1231 ¸ ĀO.K

2.1.5 Backfill compaction

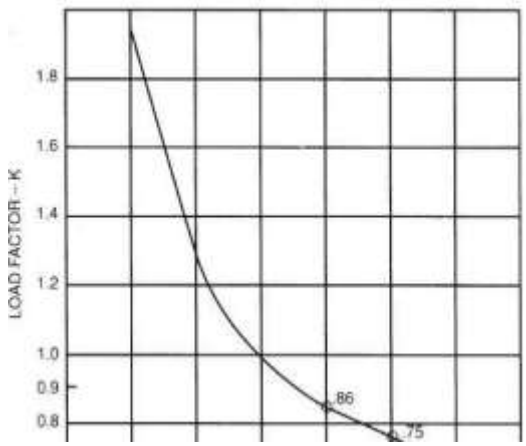
The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.
Therefore, the design assumes a backfill compaction densit 85%

2.1.6 Design Pressure

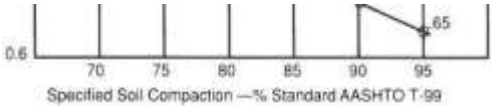
H = 1.2 m > S = 1.2 m

Pv = K (DL + LL), when H > S
= 0.86 x (25.58 + 19.088) = 38.414 kN/m²

where: Pv = design pressure, kPa
K = 0.86 , load factor
DL = dead load
LL = live load
H = height of cover



S = span or diameter



2.1.7 Ring compression

C = Pv x S / 2 = 38.414 x 1.200 / 2 = 23.049 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

2.1.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 276 < 294
fb = 205 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 102.50 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 1200 mm span or diameter
r = 4.345 ฟ , radius of gyration of the pipe wall

2.1.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

ฦ = C / fc = 23.049 / 102.500 = 0.225 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.966 ฯ/ฟ > 0.225 ฯ/ฟ ĀO.K

2.1.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

FF = D^2 / EI = 0.194 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span
I = 37.11 ¸¦,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installatio are as follows:

68x13	mm corrugation, FF	0.245 ฟ/N
125x25	ـمـ mm corrugation,	0.188 ฟ/N
76x25	FF ـمـ mm	0.188 ฟ/N
152x51	corrugation, FF ـمـ	0.114 ฟ/N
	mm corrugation, FF ـمـ	

2.1.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate sea strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for the types of pipe

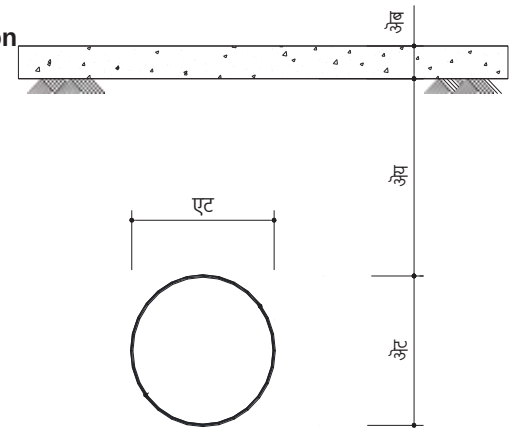
2.2 Corrugated Steel Pipe (D=900)

D=0.9M

H=1.34 m [SI UNIT]

2.2.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 1.016 m
- Pipe span : Sc = 0.900 m
- Corrugated steel pipe specifications
  - pitch x depth : **68x13** mm
  - thickness : **1.6** mm
  - yield strength : fy = 205 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

2.2.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523

2.2.3 Loads

1) Dead Load

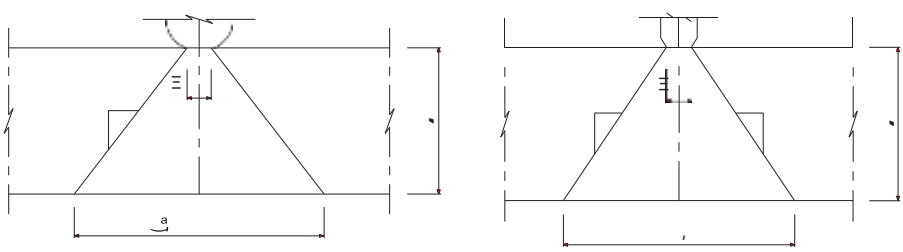
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 1.016 = 27.680 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pipe
g = unit weight of the soil
H = height of cover over the pipe

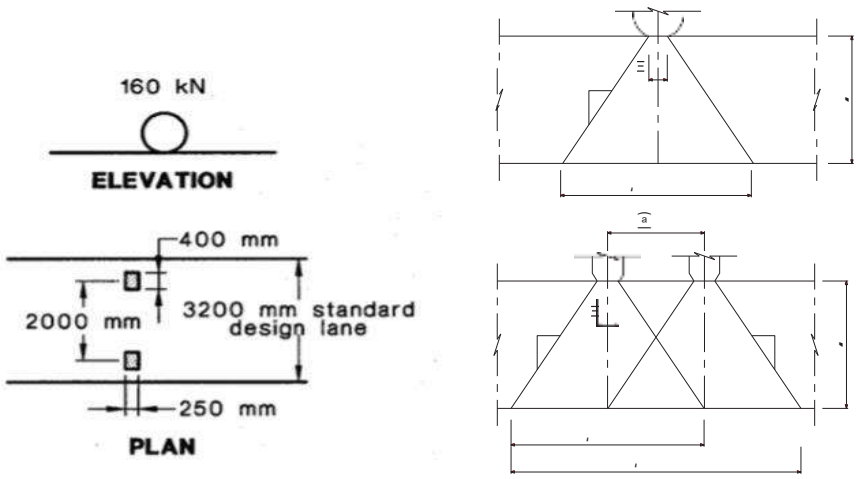
2) Live Load

(1) W80



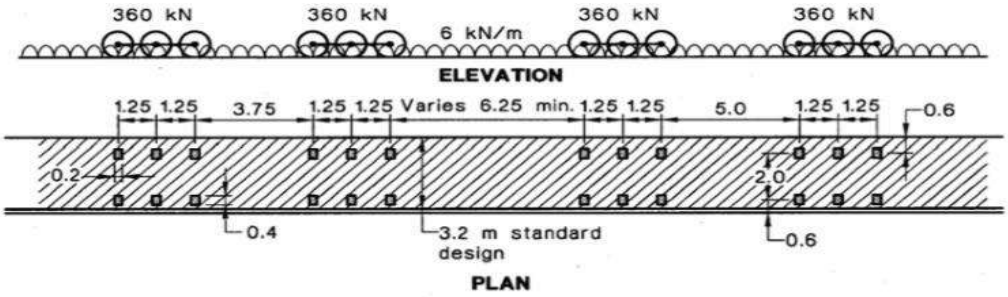
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 2.672) \times (0.4 + 2.672)} = 8.912 \text{ kN/m}^2$$

(2) A160

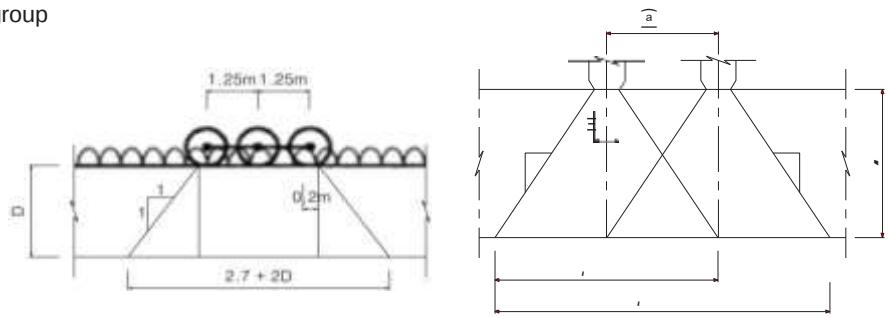


$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 2.672) \times (2.4 + 2.672)} = 10.796 \text{ kN/m}^2$$

(3) M1600



- Axle group

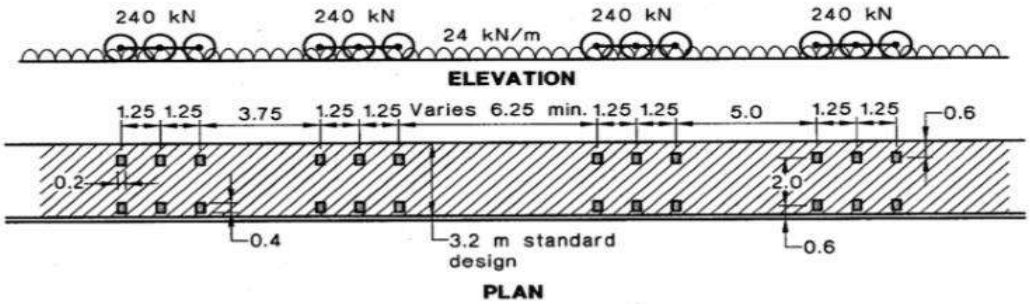


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 2.672) \times (2.4 + 2.672)} = 13.213 \text{ kN/m}^2$$

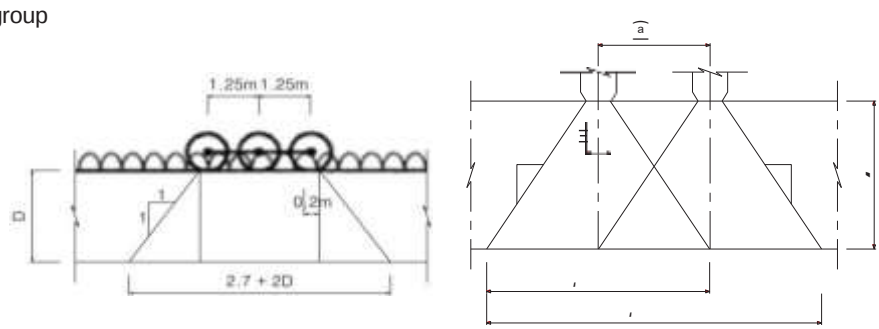
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 13.213 + 1.875 = 15.088 kN/m²

(4) S1600



- Axle group

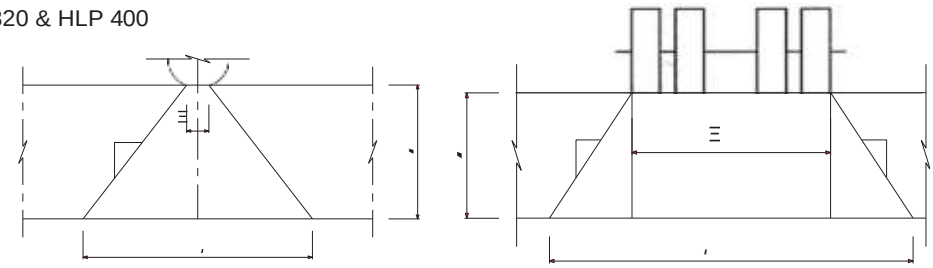


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 2.672) \times (2.4 + 2.672)} = 8.808 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 8.808 + 7.500 = 16.308 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2.672) \times (1.4 + 2.672)} = 10.689 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	8.912	0.20	10.691
A160	10.796	0.20	12.951
M1600	15.088	0.17	17.598
S1600	16.308	0.00	16.308
HLP	10.689	0.10	11.757

□ LL = 17.598 kN/m² = 17.598 kN/m²

2.2.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min}= 600 ㎝ < H = 1336 ㎝ ĀO.K

2.2.5 Backfill compaction

The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.

Therefore, the design assumes a backfill compaction density 85%

2.2.6 Design Pressure

H = 1.3 m > S = 0.9 m

Pv = K (DL + LL), when H > S

= 0.86 x (27.68 + 17.598) = 38.939 kN/m²

where: Pv = design pressure, kPa

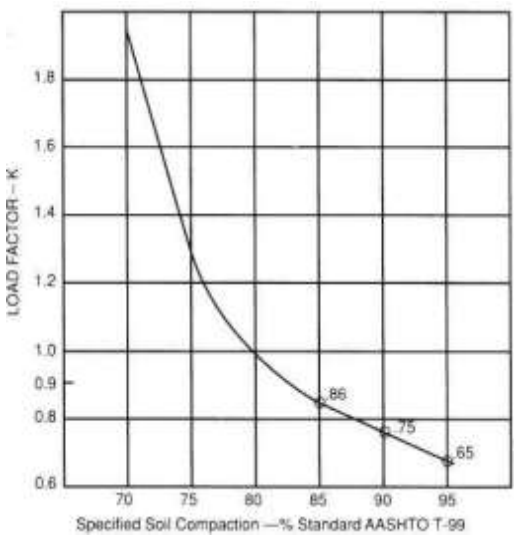
K = 0.86 , load factor

DL = dead load

LL = live load

H = height of cover

S = span or diameter



2.2.7 Ring compression

C = Pv x S / 2 = 38.939 x 0.900 / 2 = 17.523 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

2.2.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 208 < 294
fb = 205 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 102.50 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 900 mm span or diameter
r = 4.332 ฟ , radius of gyration of the pipe wall

2.2.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

Ī = C / fc = 17.523 / 102.500 = 0.171 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.512 ฯ/ฟ > 0.171 ฯ/ฟ ĀO.K

2.2.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

$FF = D^2 / EI$ = 0.143 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span
I = 28.37 ,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installation are as follows:

68x13	mm corrugation, FF	0.245 ฟ/N
125x25	mm corrugation,	0.188 ฟ/N
76x25	FF mm	0.188 ฟ/N
152x51	corrugation, FF	0.114 ฟ/N
	mm corrugation, FF	

2.2.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate seam strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for the types of pipe

2.3 Head/Wing Wall (H=1.5m)

2.3.1 Design Conditions (H=1.500m , N= 1 : 2.00 , Ho= 6.370)

1) General Items

(1)	Type of WingWall	:	Cantilever Type	
(2)	Height of WingWall	:	1.500	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	6.370	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

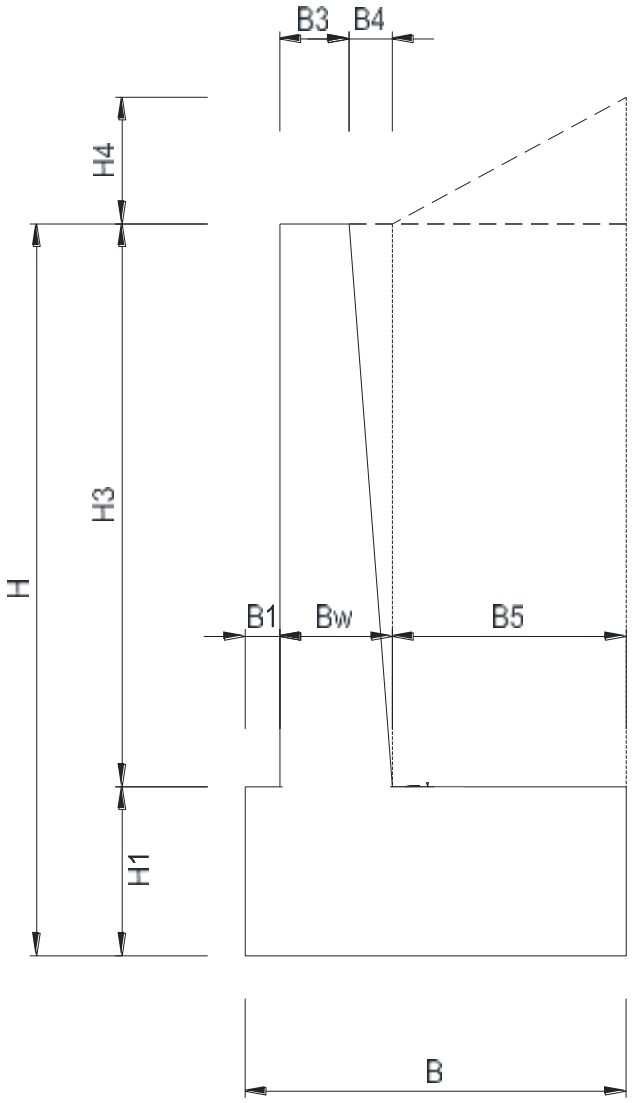
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

2.3.2 Section Assumption



※ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.100	0.000	0.200	0.125	0.675	1.100	0.325

- Height

H1	H2	H3	H4	H	Ho	
0.450	0.000	1.500	0.338	1.950	6.370	

2.3.3 Load Calculation

1) Self weight (D)

Automatic consideration in program

2) Earth pressure

▷ At Normal (H)

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	4.764

∵ (δ = Λ· x Φ)

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
32.2	9.333	1.500	165.183	12.280	0.575	0.557	0.140
32.3	9.333	1.500	161.576	12.294	0.575	0.558	0.140
32.4	**9.333**	**1.500**	**157.989**	**12.296**	**0.575**	**0.558**	**0.140**
32.5	9.333	1.500	154.422	12.288	0.575	0.558	0.140
32.6	9.333	1.500	150.875	12.268	0.574	0.557	0.140

Coefficient of earth pressure : Kah = 0.558

Horizontal earth pressure Pah = Kah x γt x H

Pah1 = 0.558 × 19 × 0.000 = 0.000 kN/m³

Pah2 = 0.558 × 19 × 1.500 = 15.903 kN/m³

◁ At Earthquake (E)

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	4.764	6.538

∵ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
28.2	0.000	1.500	328.536	38.670	1.8091	1.803	0.150
28.3	0.000	1.500	323.922	38.684	1.8098	1.804	0.150
28.4	**0.000**	**1.500**	**319.338**	**38.687**	**1.8099**	**1.804**	**0.150**
28.5	0.000	1.500	314.783	38.677	1.8094	1.803	0.150
28.6	0.000	1.500	310.258	38.655	1.8084	1.802	0.150

Coefficient of earthquake earth pressure : Kaeh' = Kaeh - Kah = 1.246

Earthquake earth pressure Paeh' = Kaeh' x γt x H

Paeh1' = 1.246 × 19 × 0.000 = 0.000 kN/m³

Paeh2' = 1.246 × 19 × 1.500 = 35.511 kN/m³

3) Inertia force at earthquake (E)

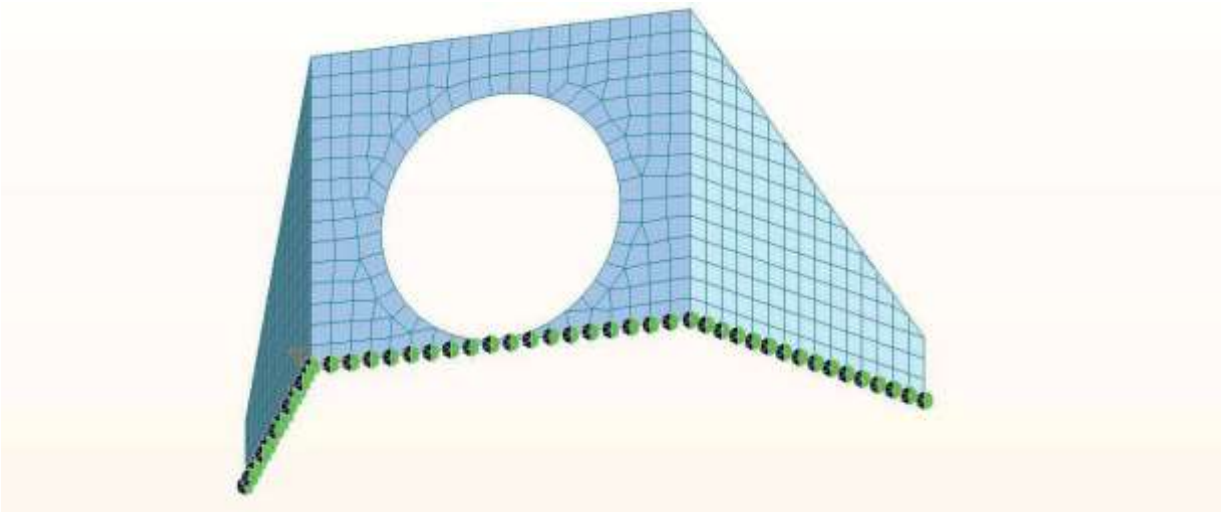
P = W x Kh

W : Weight of structure

Kh : 0.115 horizontal seismic coefficient

2.3.4 Modeling & Loading

1) Anaysis Model

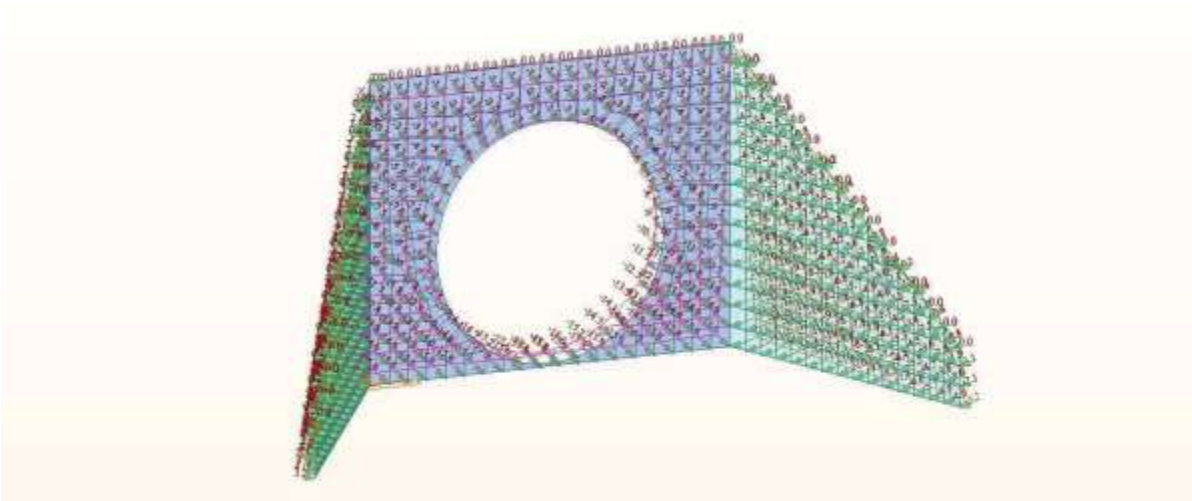


Boundary condition : Hinge

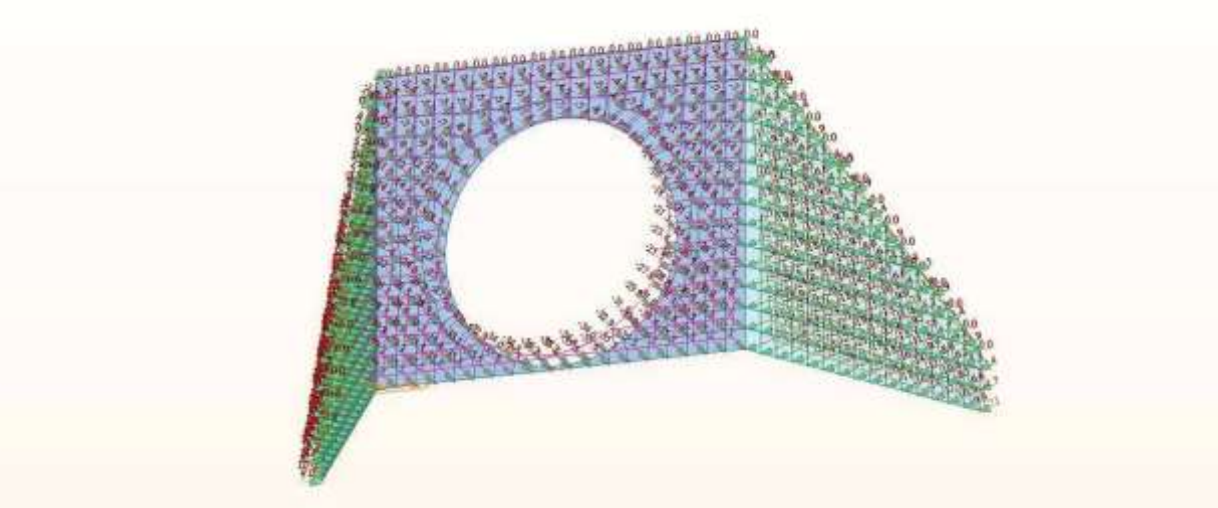
2) Loading

(1) Self weight - Automatic consideration in program (D)

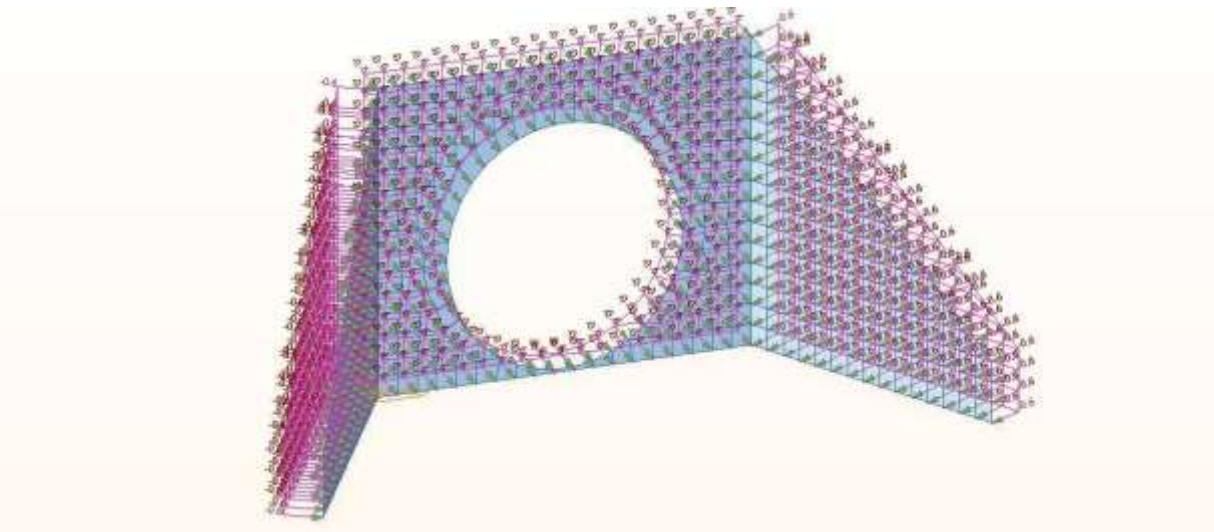
(2) Horizontal Earth Pressure at normal (H)



(3) Horizontal Earth Pressure at earthquake (E)



(4) Inertia force at earthquake (E)



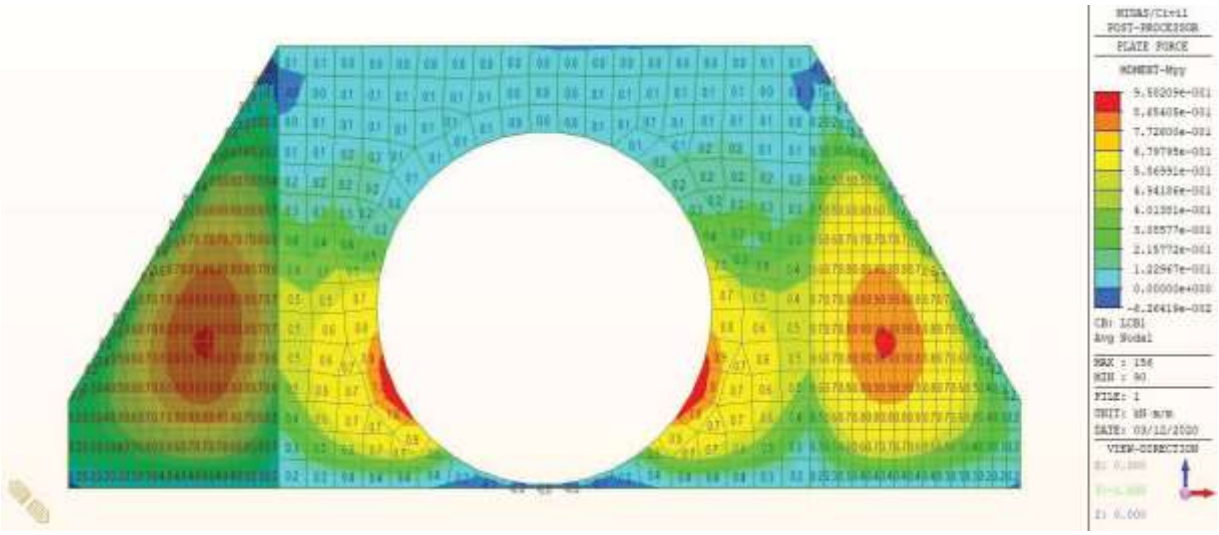
2.3.5 Load combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

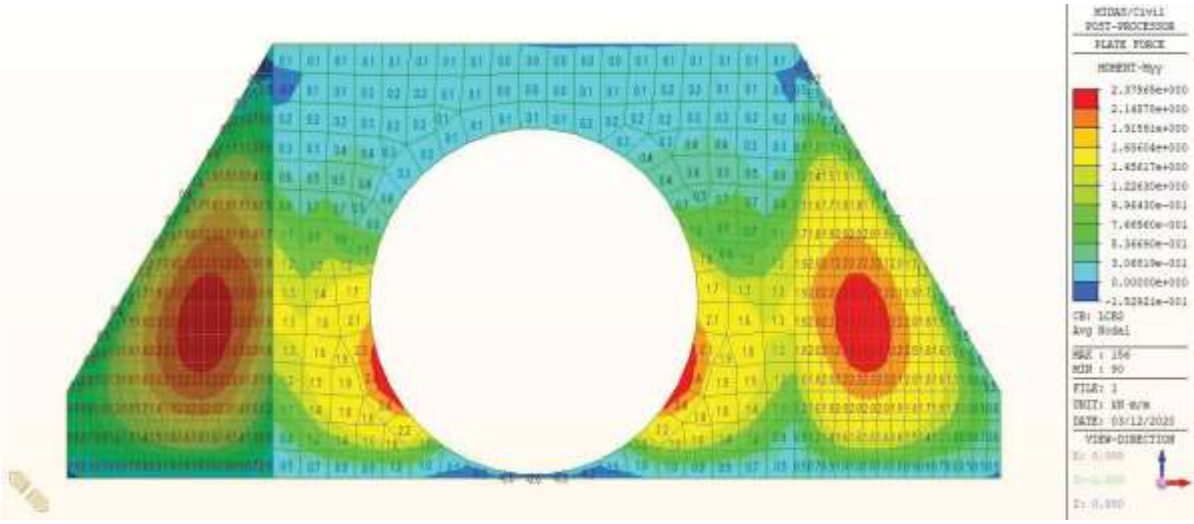
2.3.6 Summary of Analysis Results

(1) B.M.D (Ultimate Load) - Unit : kN.m

▷ LCB1

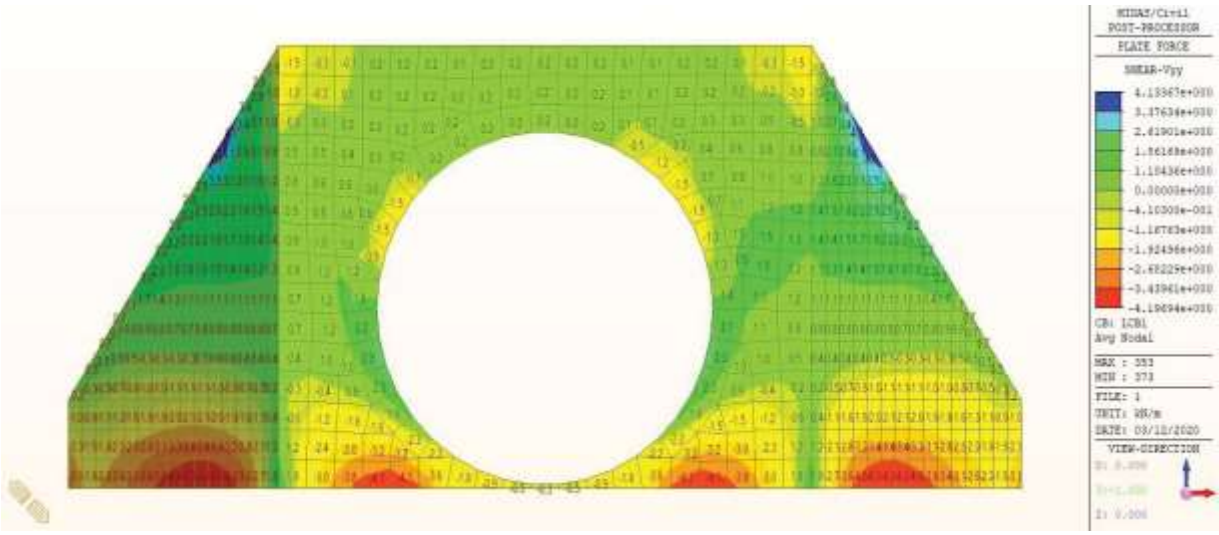


◁ LCB2

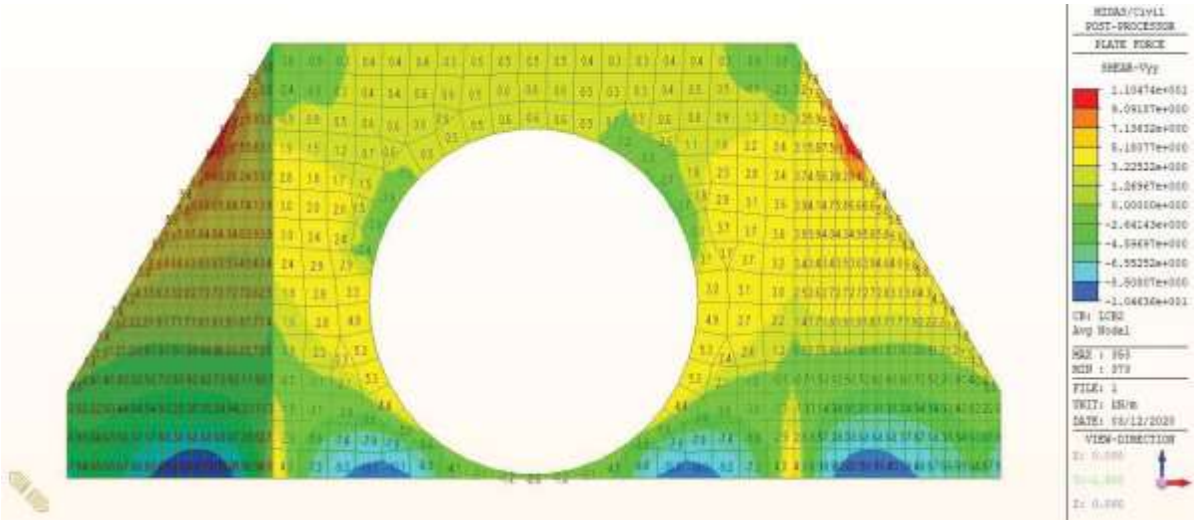


(2) S.F.D (Ultimate Load) - Unit : kN

▷ LCB1

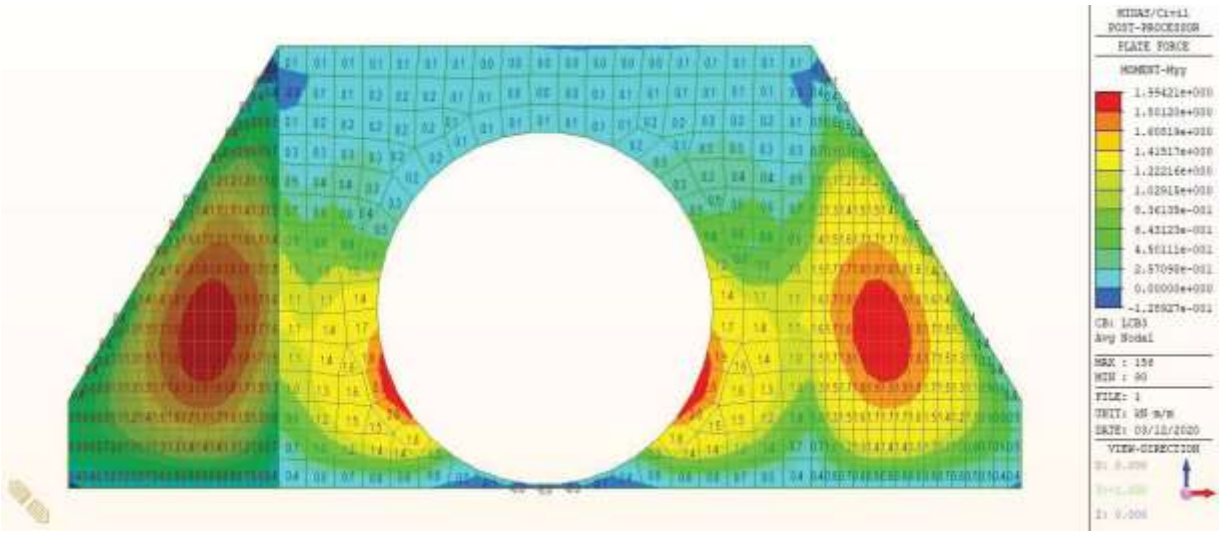


◁ LCB2



(3) B.M.D (Service Load) - Unit : kN.m

▷ LCB3



(4) Summary

Division	LCB1	LCB2	LCB3	Section
M(kN.m)	0.958	2.375	1.994	Middle of Wall
V(kN)	4.196	10.463		Bottom of Wall

□ Mu = 2.375 kN.m

Vu = 10.463 kN

Mo = 1.994 kN.m

2.3.7 Section Design

1) Middle of Wall

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	100.0	mm	
B	=	1000	mm	H	=	200	mm		d'	=	100.0	mm	
M_u	=	2.375	kN·m	V_u	=	10.463	kN	M_o	=		1.994	kN·m	

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (100.0 - 18.322) / 18.322

= 0.0134

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.ⁿ ТФГТУКУФУZТ НПΣ И□.ⁿ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 63.138 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	⊥	$A_{s,max}$	=	2485.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	⊥	$A_{s,min}$	=	336.7	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00084	⊥	$A_{s,4/3req}$	=	84.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00084	⊥	$A_{s,min}$	=	84.2	㎟
P_{use}	=	A_s / (B·d)	=	0.01032	⊥	$A_{s,min}$	=	1032.0	㎟

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 36.074 kN·m > Mu = 2.375 kN·m

Ā O.K

ᴖ Shear firction Check

Øv Vn = Øv × Avf × fy × μ 325.080 > Vu = 10.463 kN Ā O.K

(2) Crack Check

ᴖ Calculation of stress

n = 9

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 9 × 1,032.00 / 1000 + 9 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 100 / (9 × 1,032.00)]

= 34.801 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 1.994 / [1000 × 34.801 x (100.0 - 34.801 / 3)] × 10^6

= 1.296 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 1.994 / [1032.000 x (100.0 - 34.801 / 3)] × 10^6

= 21.857 MPa

fst = fs x (H - d' - χ) / (d - χ) = 22 x (200 - 100 - 1) / (100 - 35) = 21.86 MPa

ᴖ Maximum center space of reinforcement

C_C = 100.00 - 13.00 / 2 = 93.50 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 21.86) - 2.5 x 93.50 = 4634.21 ⌟

300 x (280 / fs) = 300 x (280 / 21.86) = 3843.12 ⌟

Sa = 3843.12 ⌟ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (3843.12 mm) Ā O.K

2.3.8 Distribution Reinforcement Check

1) Wall (H = 325 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 325 = 585.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : D 13@ 125 = 1032.0 ㎟

□ = 1032.0 ㎟ > 585.0 ㎟ Ā O.K

· Bar spacing : 125 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 450 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 450 = 810.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : Upper side : D 13@ 250 = 516.0 ㎟

Bottom side : D 13@ 250 = 516.0 ㎟

□ = 1032.0 ㎟ > 810.0 ㎟ Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

3. BOX CULVERT WING WALL

3.1 Wing Wall (H=2.2m)

3.1.1 Design Conditions (H=2.200m , N= 1 : 2.00 , Ho= 2.000)

1) General Items

(1)	Type of WingWall	:	Reverse T Type WingWall	
(2)	Height of WingWall	:	2.200	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	2.000	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(=0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

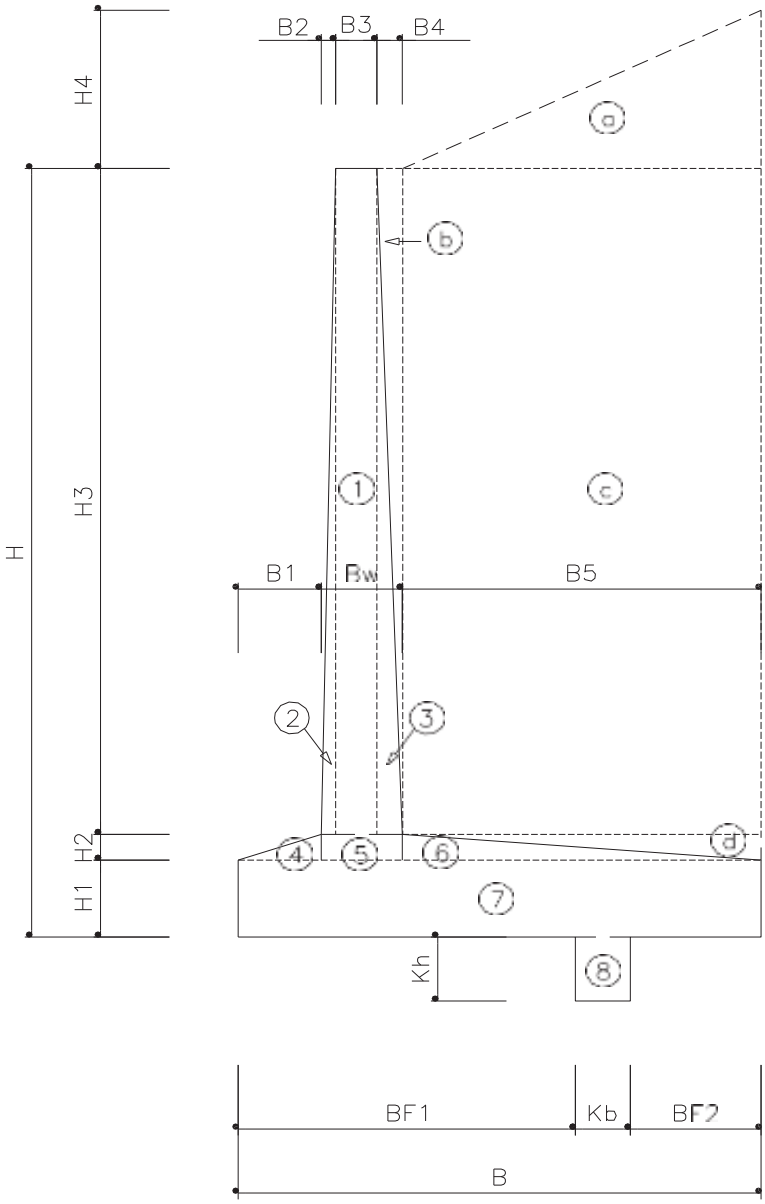
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

3.1.2 Section Assumption



Ŝ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.300	0.036	0.300	0.064	1.100	1.800	0.400

- Height

H1	H2	H3	H4	H	Ho	
0.400	0.000	1.800	0.550	2.200	2.000	

- Shear Key

BF1	BF2	Kb	Kh			
1.000	0.400	0.400	0.400			

3.1.3 Evaluation of serviceability

1) At Nomal

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta)} \times W$$

where,

Φ(rx)	β(rx)	δ(rx)	θ(rx)
28.0	26.565	23.420	2.036

∞ (Apply δ = β')

α(rx)	δ'(rx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
47.7	23.337	2.750	112.538	38.013	0.529	0.486	0.210
47.8	23.379	2.750	112.005	38.014	0.529	0.486	0.210
47.9	**23.420**	**2.750**	**111.473**	**38.015**	**0.529**	**0.486**	**0.210**
48.0	23.462	2.750	110.942	38.014	0.529	0.486	0.210
48.1	23.503	2.750	110.413	38.012	0.529	0.486	0.210

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 34.880 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 15.087 kN/m³

(2) Load

Division		Calculation							Unit Weight	Vertical Force(kN)
Concrete	▷	1.800	x	0.300			=	0.540	25.00	13.500
	◁	1.800	x	0.036	x	½	=	0.032	25.00	0.810
	V	1.800	x	0.064	x	½	=	0.058	25.00	1.440
	Å	0.000	x	0.300	x	½	=	0.000	25.00	0.000
	Λ	0.000	x	0.400			=	0.000	25.00	0.000
	Λ̇	0.000	x	1.100	x	½	=	0.000	25.00	0.000
	>	0.400	x	1.800			=	0.720	25.00	18.000
	>̇	0.400	x	0.400			=	0.160	25.00	4.000
Soil	▷·	0.550	x	1.100	x	½	=	0.303	19.00	5.748
	·▷̇	1.800	x	0.064	x	½	=	0.058	19.00	1.094
	▷̇·	1.800	x	1.100			=	1.980	19.00	37.620
	▷̇̇	0.000	x	1.100	x	½	=	0.000	19.00	0.000
Surface load		0.895					=	0.895	10.00	8.950

(3) Moment

Division		Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
		V (kN/m)	H (kN/m)	X	Y	V·X(Mr)	H·Y(Mo)
Concrete		13.500	-	0.486	-	6.561	-
		0.810	-	0.324	-	0.262	-
		1.440	-	0.657	-	0.947	-
		0.000	-	0.200	-	0.000	-
		0.000	-	0.500	-	0.000	-
		0.000	-	1.067	-	0.000	-
		18.000	-	0.900	-	16.200	-
		4.000	-	1.200	-	4.800	-
Sub Total		**37.750**				**28.770**	
Soil		5.748	-	1.433	-	8.238	-
		1.094	-	0.679	-	0.743	-
		37.620	-	1.250	-	47.025	-
		0.000	-	1.433	-	0.000	-
Sub Total		**44.462**				**56.006**	
Earth pressure		15.087	34.880	1.800	0.917	27.157	31.973
Surface load		1.879	4.345	1.800	0.917	3.383	3.983
Total		**99.179**	**39.225**			**115.316**	**35.957**

(4) Evaluation of serviceability

Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.800 - 2 x 0.100 = 1.600 m

C = 0.000 kN/m²

ΣV = 99.179 kN/m

ΣH = 39.225 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 49.589 kN/m

F.S = Hu / ΣH = 1.264 < 1.5 --- **N.G**

ÿ **Need Shear Key.**

Shear Key

V = 99.179 kN/m

tanΦB = 0.500

Kp = 3.688

$$Kp = \frac{\cos^2(\varphi + \theta)}{\cos^2\theta \cos(\theta - \delta)\left[1 - \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi + \beta)}{\cos(\theta - \delta)\cos(\theta - \beta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	0.00	9.33	0

Pp = 32.749 kN/m

Hk = V $\tan\Phi_B$ + Pp = 82.338 kN/m

F.S = Hk / ΣH = 2.099 > 1.5 --- **O.K**

ÿ **Stability of sliding is O.K.**

Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

= 1.800 / 2 - (115.316 - 35.957) / 99.179 = 0.100 m

B / 6 = 1.800 / 6 = 0.300 > e --- **O.K**

F.S = ΣMr / ΣMo = 3.207 > 2.0 --- **O.K**

ÿ **Stability of overturning is O.K.**

Bearing Capacity

e = 0.100 m < B/6 = 0.300 m

x = 3·[B/2 - e] = 3 x (1.800 / 2 - 0.100) = 2.400 m > 1.800 m

□ resultant in middle one-third of base

q(max,min) = (ΣV / B) x (1 ± 6·e / B)

= 99.179 / 1.800 x (1 ± 6 x 0.100 / 1.800)

qmax = 73.435 kN/m² (Toe)

qmin = 36.763 kN/m² (Heel)

qmax = 73.435 kN/m² < qa = 320.83 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

2) At Earthquake

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta')} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)	
28.0	26.565	28.0	2.036	6.538	∞(ω = $\tan^{-1}$Kh)
					È(δ = Φ)

if β' + ω ¯Φ, δ' = Φ

α(ƒx)	δ'(ƒx)	H (m)	We (kN/m)	Pae (kN/m)	Kae	Kaeh	Kaev
40.8	28.000	2.750	155.205	53.258	0.7413	0.655	0.348
40.9	28.000	2.750	154.517	53.260	0.7413	0.655	0.348
41.0	**28.000**	**2.750**	**153.832**	**53.261**	**0.7413**	**0.655**	**0.348**
41.1	28.000	2.750	153.149	53.261	0.7413	0.655	0.348
41.2	28.000	2.750	152.469	53.259	0.7413	0.655	0.348

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 47.058 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 25.002 kN/m³

(2) Load

Division	Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
	V (kN/m)	H (kN/m)	X	Y	V · X	H · Y
Concrete	37.750				28.770	
Soil	44.462				56.006	
Earth pressure	25.002	47.058	1.800	1.375	45.003	64.704
Total	**107.214**	**47.058**	1.210	1.375	**129.779**	**64.704**

(3) Moment

Load		Calculation	Horizontal seismic coefficient	Horizental Force	MOMENT (kN·m)	
				H (kN/m)	Y	H · Y
Concrete	⊳	13.500	0.115	1.547	1.300	2.011
	◁	0.810	0.115	0.093	1.000	0.093
	V	1.440	0.115	0.165	1.000	0.165
	Å	0.000	0.115	0.000	0.400	0.000
	Λ	0.000	0.115	0.000	0.400	0.000
	Λ̇	0.000	0.115	0.000	0.400	0.000
	>	18.000	0.115	2.063	0.200	0.413
Sub Total		**33.750**		**3.868**	-	**2.682**
Soil	⊳·	5.748	0.115	0.659	2.383	1.570
	·⊳	1.094	0.115	0.125	1.600	0.201
	⊳·	37.620	0.115	4.311	1.300	5.605
	⊳	0.000	0.115	0.000	0.400	0.000
Sub Total		**44.462**		**5.095**	-	**7.375**
Total		**78.212**		**8.963**	1.122	**10.057**

(4) Evaluation of serviceability

Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.800 - 2 x 0.387 = 1.026 m

C = 0.000 kN/m²

ΣV = 107.214 kN/m

ΣH = 56.021 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 53.607 kN/m --- **N.G**

F.S = Hu / ΣH = 0.957 < 1.1

ÿ **Need Shear Key.**

Shear Key

V = 107.214

$\tan\Phi_B$ = 0.500

Kpe = 2.138 kN/m

V =

tanΦB =

Kp =

$$Kp = \frac{\cos^2(\varphi - \omega + \theta)}{\cos\omega \cos^2\theta \cos(\delta - \theta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi - \delta)\sin(\varphi - \omega + \beta)}{\cos(\delta - \theta + \omega)\cos(\beta - \theta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.00	0.00	9.33	0	6.538

Ppe = 18.986 kN/m

Hk = V $\tan\Phi_B$ + Ppe = 72.593 kN/m

F.S = Hk / ΣH = 1.296 > 1.1 --- **O.K**

ÿ **Stability of sliding is O.K.**

Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

1.800 / 2 - (129.779 - (64.704 + 10.057)) / 107.214 = 0.387 m

B / 3 = 1.800 / 3 = 0.600 > e --- **O.K**

F.S = ΣMr / ΣMo = 1.736 > 1.5 --- **O.K**

ÿ **Stability of overturning is O.K.**

Bearing Capacity

e = 0.387 m < B/3 = 0.600 m

x = 3·[B/2 - e] = 3 x (1.800 / 2 -0.387) = 1.539 m < 1.800 m

Pressure for resultant outside middle one-third

qmax = 2·ΣV / X

= 107.214 x 2 / 1.539 = 139.329 kN/m²

qmax = 139.329 kN/m² (Toe)

qmin = 0.000 kN/m² (Heel)

qmax = 139.329 kN/m² < qa = 320.83 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

3.1.4 Load Calculation

1) Wall

(1) Earth pressure

At Nomal

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	2.036

∞ (δ = Λ· x Φ

α(ƒx)	δ'(ƒx)	)H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
42.1	9.333	1.800	76.914	18.759	0.609	0.597	0.120
42.2	9.333	1.800	76.383	18.760	0.610	0.598	0.120
42.3	**9.333**	**1.800**	**75.853**	**18.760**	**0.610**	**0.598**	**0.120**
42.4	9.333	1.800	75.325	18.759	0.610	0.598	0.120
42.5	9.333	1.800	74.800	18.756	0.609	0.597	0.120

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 18.406 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 3.694 kN/m³

At Earthquake

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	2.036	6.538

∞ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
37.3	0.000	1.800	104.756	28.821	0.9364	0.936	0.033
37.4	0.000	1.800	104.101	28.824	0.9364	0.936	0.033
37.5	**0.000**	**1.800**	**103.449**	**28.825**	**0.9365**	**0.936**	**0.033**
37.6	0.000	1.800	102.800	28.824	0.9365	0.936	0.033
37.7	0.000	1.800	102.154	28.823	0.9364	0.936	0.033

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 28.810 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 1.016 kN/m³

Earthquake earth pressure : Paeh' = 1/2 x kaeh' x γt x H^2 = 10.404 kN/m³

Division		Load	Horizental Force	length (m)	Mr (kN·m)
		W (kN)	H (kN/m)	Y	H x Y
Concrete	▷	13.500	1.547	0.900	1.392
	◁	0.810	0.093	0.600	0.056
	V	1.440	0.165	0.600	0.099
Bottom of Wall		**15.750**	**1.805**		**1.547**
Earthquake earth pressure			**10.404**	0.900	**9.363**
Concrete	▷	6.750	0.774	0.450	0.348
	◁	0.020	0.002	0.300	0.001
	V	0.360	0.041	0.300	0.012
Middle of Wall		**7.130**	**0.817**		**0.361**
Earthquake earth pressure			**2.601**	0.450	**1.170**

(2) Stress Resultant

At Nomal

Bottom of Wall	(H=	1.800 m	)				
V=	18.406				=	18.406	kN
M=	18.406	x	1.800	/ 3	=	11.044	kN/m
Middle of Wall	(H=	0.900 m	)				
V=	4.602				=	4.602	kN
M=	4.602	x	0.900	/ 3	=	1.380	kN/m

At Earthquake

Bottom of Wall	(H=	1.800 m	)				
Ve=	28.810				=	28.810	kN
Me=	28.810	x	1.800	/2	=	25.929	kN/m
Middle of Wall	(H=	0.900 m	)				
Ve=	7.203				=	7.203	kN
Me=	7.203	x	0.900	/ 2	=	3.241	kN/m

(3) Design Load for cross section

Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Summary

Division		Bottom of Wall		Middle of Wall	
		Horizontal earth pressure	Inertial force	Horizontal earth pressure	Inertial force
LCB1	Shear force	29.450	0.000	7.363	0.000
	Moment	17.670	0.000	2.209	0.000
LCB2	Shear force	29.450	12.209	7.363	3.418
	Moment	17.670	10.910	2.209	1.531
LCB3	Shear force	18.406	0.000	4.602	0.000
	Moment	11.044	0.000	1.380	0.000

V Design Load for cross section

-Bottom of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
29.450	17.670	41.659	28.580	18.406	11.044

-Middle of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
7.363	2.209	10.781	3.740	4.602	1.380

2) Foundation

(1) Stress resultant of Foundation

▷ Load

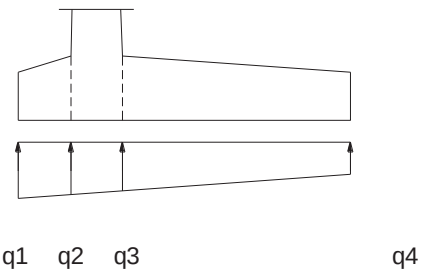
Division		V(kN)	H(kN)	Mr(kN.m)	Mo(kN.m)
At Nomal	Concrete	37.750	0.000	28.770	0.00
	Soil	44.462	0.000	56.006	0.00
	Earth pressure	15.087	34.880	27.157	31.973
	Surface load	1.879	4.345	3.383	3.983
	□	**99.179**	**39.225**	**115.316**	**35.96**
At Earthquake	Concrete	37.750	3.868	28.770	2.682
	Soil	44.462	5.095	56.006	7.375
	Earth pressure	15.087	34.880	27.157	31.973
	Earthquake earth pressure	9.914	12.178	17.846	32.731
	□	**107.214**	**56.021**	**129.779**	**74.76**

◁ Ultimate load Combination

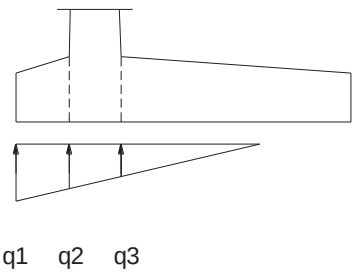
Division	□V	□Mr	□Mo	e	Load shape
LCB1	125.801	150.595	57.530	0.160	CASE1
LCB2	108.045	137.595	92.940	0.487	CASE2
LCB3	99.179	115.316	35.957	0.100	CASE1

V Stress resultant of Foundation

CASE1



CASE2



Division	q1	q2	q3	q4	e	CASE
LCB1	107.164	94.739	78.173	32.615	0.160	1
LCB2	174.406	132.177	75.872	-	0.487	2
LCB3	73.466	67.343	59.181	36.733	0.100	1

(2) Heel

▷ Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Λ'	0.000	0.367	0.000	
	>	11.000	0.550	6.050	
	>	4.000	0.500	2.000	
Sub Total		**15.000**	**0.537**	**8.050**	
Soil	▷	5.748	0.712	4.092	
	▷	37.620	0.550	20.691	
	▷	0.000	0.733	0.000	
Sub Total		**43.368**	**0.571**	**24.783**	
Total		**58.368**	**0.563**	**32.833**	

◁ Cross section force by Vertical Force

- At Nomal

V = 15.087 kN

M = 15.087 x 1.100 = 16.596 kN·m

- At Earthq ake

V = 15.087 + 9.914 = 25.002 kN

M1 = 15.087 x 1.100 = 16.596 kN·m

M2 = 9.914 x 1.100 = 10.906 kN·m

V Cross section force by Stress resultant of Foundation

Load	q3	q4	length(m)	V (kN)	M (kN·m)
LCB1	78.173	32.615	0.475	-60.933	-28.920
LCB2	75.872	0.000	0.180	-20.447	-3.674
LCB3	59.181	36.733	0.507	-52.752	-26.750

∞ V = (q4 + q3)/2 × B5

Λ Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	L	H	E	Stress resultant of Foundation	Total
LCB1	Vu	70.041	-	24.140	-	-60.933	33.247
	Mu	39.400	-	26.553	-	-28.920	37.034
LCB2	Vu	52.531	-	24.140	9.914	-20.447	66.137
	Mu	29.550	-	26.553	10.906	-3.674	63.336
LCB3	Vo	58.368	-	15.087	-	-52.752	20.702
	Mo	32.833	-	16.596	-	-26.750	22.679

(3) Toe

Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Å'	0.000	0.100	0.000	
	> '	3.000	0.150	0.450	
Sub Total		**3.000**	**0.150**	**0.450**	

Cross section force by Stress resultant of Foundation

Load	q1	q2	length(m)	V (kN)	M (kN·m)
LCB1	107.164	94.739	0.153	30.285	4.636
LCB2	174.406	132.177	0.157	45.987	7.215
LCB3	73.466	67.343	0.152	21.121	3.214

∞ V = (q1 + q2)/2 × B1

Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	H	Stress resultant of Foundation	Total
LCB1	Vu	-3.600	-	30.285	26.685
	Mu	-0.540	-	4.636	4.096
LCB2	Vu	-2.700	-	45.987	43.287
	Mu	-0.405	-	7.215	6.810
LCB3	Vo	-3.000	-	21.121	18.121
	Mo	-0.450	-	3.214	2.764

(4) Shear Key

Passive earth pressure

At Nomal : Pp = 32.749 kN/m

At Earthq : Ppe = 18.986 kN/m Ā Apply Cross section force at Nomal

Design Load for cross section

Division	qk1	qk2	H(m)	V (kN)	M (kN·m)
At Nomal	68.226	95.517	0.400	32.749	6.914

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)
Design Load for cross section	11.062	52.398	6.914

3) Summary

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)	ØMn(kN·m)	Bar	S.F
Bottom of Wall	28.580	41.659	11.044	131.063	D13 @ 125	4.59
Middle of Wall	3.740	10.781	1.380	56.513	D13 @ 250	15.11
Heel	28.580	66.137	11.044	131.063	D13 @ 125	4.59
Toe	6.810	43.287	2.764	66.265	D13 @ 250	9.73
Shear Key	11.062	52.398	6.914	66.265	D13 @ 250	5.99

3.1.5 Section Design

1) Bottom of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm	
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm	
M_u	=	28.580	kN·m	V_u	=	41.659	kN	M_o	=		11.044	kN·m	

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ̈ ТФГТУКУФУZТ НПΣ И□ ̈

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 221.215 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	ㅿ	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	ㅿ	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00086	ㅿ	$A_{s,4/3req}$	=	295.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00086	ㅿ	$A_{s,min}$	=	295.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00300	ㅿ	$A_{s,min}$	=	1032.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 28.580 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 41.659 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,032.00 / 1000 + 8 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,032.00)]

= 67.507 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.044 / [1000 × 67.507 x (343.5 - 67.507 / 3)] × 10^6

= 1.019 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.044 / [1032.000 x (343.5 - 67.507 / 3)] × 10^6

= 33.338 MPa

fst = fs x (H - d' - χ) / (d - χ) = 33 x (400 - 57 - 1) / (344 - 68) = 33.34 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 33.34) - 2.5 x 50.00 = 3066.55 ㎜

300 x (280 / fs) = 300 x (280 / 33.34) = 2519.65 ㎜

Sa = 2519.65 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (2519.65 mm) Ā O.K

2) Middle of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	293.5	mm	
B	=	1000	mm	H	=	350	mm		d'	=	56.5	mm	
M_u	=	3.740	kN·m	V_u	=	10.781	kN	M_o	=		1.380	kN·m	

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (293.5 - 9.161) / 9.161

= 0.0931

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₮ ТФГТУКУФУZТ НПΣ И□.₮

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 33.743 ㎡

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎡ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	7293.5	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	988.3	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00015	㎡	$A_{s,4/3req}$	=	45.0	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00015	㎡	$A_{s,min}$	=	45.0	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00176	㎡	$A_{s,min}$	=	516.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 56.513 kN·m > Mu = 3.740 kN·m

Ā O.K

ㅿ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 207.536 kN > Vu = 10.781 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

$n = 8$

$\chi = -nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

$= -8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 293.5 / (8 \times 516.00)]}$

= 45.270 ㅡ

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 1.380 / [1000 \times 45.270 \times (293.5 - 45.270 / 3)] \times 10^6$

= 0.219 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 1.380 / [516.000 \times (293.5 - 45.270 / 3)] \times 10^6$

= 9.609 MPa

fst = fs x (H - d' - χ) / (d - χ) = 10 x (350 - 57 - 0) / (294 - 45) = 9.61 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 9.61) - 2.5 x 50.00 = 10947.48 ㅡ

300 x (280 / fs) = 300 x (280 / 9.61) = 8741.44 ㅡ

Sa = 8741.44 ㅡ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (8741.44 mm) Ā O.K

3) Heel

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	28.580	kN·m	V_u	=	66.137	kN	M_o	=		11.044	kN·m		

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₸ ТФГТУКУФУZТ НПΣ И□.₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 221.215 ㎡

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎡ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	8536.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00086	㎡	$A_{s,4/3req}$	=	295.0	㎡
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00086	㎡	$A_{s,min}$	=	295.0	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00300	㎡	$A_{s,min}$	=	1032.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 28.580 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 66.137 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,032.00 / 1000 + 8 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,032.00)]

= 67.507 ㅡ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.044 / [1000 × 67.507 x (343.5 - 67.507 / 3)] × 10^6

= 1.019 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.044 / [1032.000 x (343.5 - 67.507 / 3)] × 10^6

= 33.338 MPa

fst = fs x (H - d' - χ) / (d - χ) = 33 x (400 - 57 - 1) / (344 - 68) = 33.34 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 33.34) - 2.5 x 50.00 = 3066.55 ㅡ

300 x (280 / fs) = 300 x (280 / 33.34) = 2519.65 ㅡ

Sa = 2519.65 ㅡ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (2519.65 mm) Ā O.K

4) Toe

(1) Section Design

ㅎ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	6.810	kN·m	V_u	=	43.287	kN	M_o	=		2.764	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₸ ТФГТУКУФУZТ НПΣ И□.₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 52.509 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ㅎ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00020	㎟	$A_{s,4/3req}$	=	70.0	㎟
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00020	㎟	$A_{s,min}$	=	70.0	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00150	㎟	$A_{s,min}$	=	516.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅎ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 6.810 kN·m

Ā O.K

ㅿ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 43.287 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

$\chi = -nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

$= -8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 516.00)]}$

= 49.285 ㅡ

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 2.764 / [1000 \times 49.285 \times (343.5 - 49.285 / 3)] \times 10^6$

= 0.343 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 2.764 / [516.000 \times (343.5 - 49.285 / 3)] \times 10^6$

= 16.378 MPa

fst = fs x (H - d' - χ) / (d - χ) = 16 x (400 - 57 - 0) / (344 - 49) = 16.38 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 16.38) - 2.5 x 50.00 = 6.4E+03 ㅡ

300 x (280 / fs) = 300 x (280 / 16.38) = 5.1E+03 ㅡ

Sa = 5.13E+03 ㅡ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (5.1E+03 mm) Ā O.K

5) Shear Key

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	11.062	kN·m	V_u	=	52.398	kN	M_o	=		6.914	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.¶ ТФГТУКУФУZТ НПΣ И□.¶

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 85.359 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00033	㎟	$A_{s,4/3req}$	=	113.8	㎟
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00033	㎟	$A_{s,min}$	=	113.8	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00150	㎟	$A_{s,min}$	=	516.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 11.062 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 52.398 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

$\chi = -nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

$= -8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 516.00)]}$

= 49.285 ㎜

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 6.914 / [1000 \times 49.285 \times (343.5 - 49.285 / 3)] \times 10^6$

= 0.858 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 6.914 / [516.000 \times (343.5 - 49.285 / 3)] \times 10^6$

= 40.967 MPa

fst = fs x (H - d' - χ) / (d - χ) = 41 x (400 - 57 - 1) / (344 - 49) = 40.97 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 40.97) - 2.5 x 50.00 = 2472.20 ㎜

300 x (280 / fs) = 300 x (280 / 40.97) = 2050.42 ㎜

Sa = 2050.42 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (2050.42 mm) Ā O.K

3.1.6 Distribution Reinforcement Check

1) Wall (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	250	=	516.0	㎟			
Compression side	D	13@	250	=	516.0	㎟			
			□	=	1032.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

3.2 Wing Wall (H=1.5m)

3.2.1 Design Conditions (H=1.500m , N= 1 : 2.00 , Ho= 2.000)

1) General Items

(1)	Type of WingWall	:	Reverse T Type WingWall	
(2)	Height of WingWall	:	1.500	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	2.000	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(=0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

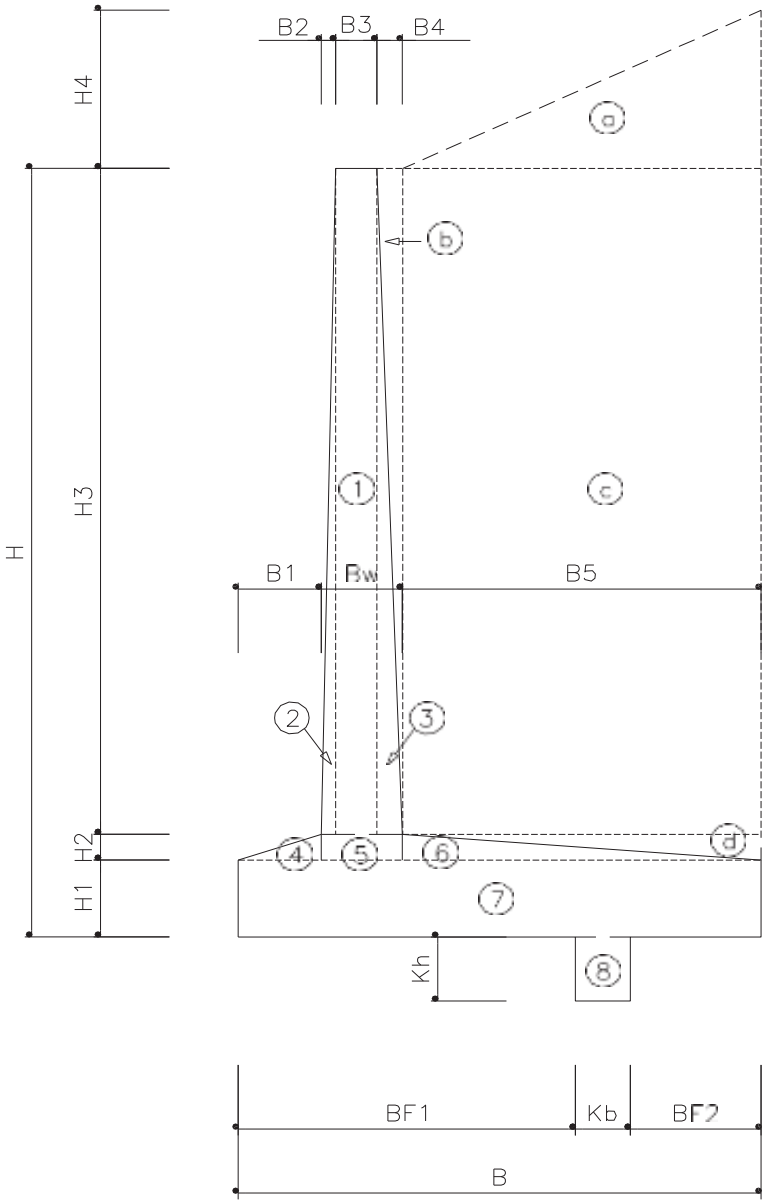
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

3.2.2 Section Assumption



※ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.300	0.022	0.300	0.078	0.800	1.500	0.400

- Height

H1	H2	H3	H4	H	Ho	
0.400	0.000	1.100	0.400	1.500	2.000	

- Shear Key

BF1	BF2	Kb	Kh			
0.700	0.400	0.400	0.400			

3.2.3 Evaluation of serviceability

1) At Nomal

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.0	26.565	25.245	4.056

∞ (Apply δ = β')

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
44.1	25.163	1.900	71.450	20.065	0.585	0.529	0.250
44.2	25.204	1.900	71.031	20.064	0.585	0.529	0.249
44.3	**25.245**	**1.900**	**70.614**	**20.063**	**0.585**	**0.529**	**0.249**
44.4	25.285	1.900	70.198	20.061	0.585	0.529	0.249
44.5	25.326	1.900	69.784	20.057	0.585	0.529	0.249

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 18.145 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 8.539 kN/m³

(2) Load

Division		Calculation							Unit Weight	Vertical Force(kN)
Concrete	▷	1.100	x	0.300			=	0.330	25.00	8.250
	◁	1.100	x	0.022	x	½	=	0.012	25.00	0.303
	V	1.100	x	0.078	x	½	=	0.043	25.00	1.073
	Å	0.000	x	0.300	x	½	=	0.000	25.00	0.000
	Λ	0.000	x	0.400			=	0.000	25.00	0.000
	Λ̇	0.000	x	0.800	x	½	=	0.000	25.00	0.000
	>	0.400	x	1.500			=	0.600	25.00	15.000
	>̇	0.400	x	0.400			=	0.160	25.00	4.000
Soil	▷·	0.400	x	0.800	x	½	=	0.160	19.00	3.040
	·▷̇	1.100	x	0.078	x	½	=	0.043	19.00	0.815
	▷̇·	1.100	x	0.800			=	0.880	19.00	16.720
	▷̈	0.000	x	0.800	x	½	=	0.000	19.00	0.000
Surface load		0.387					=	0.387	10.00	3.866

(3) Moment

Division		Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
		V (kN/m)	H (kN/m)	X	Y	V·X(Mr)	H·Y(Mo)
Concrete		8.250	-	0.472	-	3.894	-
		0.303	-	0.315	-	0.095	-
		1.073	-	0.648	-	0.695	-
		0.000	-	0.200	-	0.000	-
		0.000	-	0.500	-	0.000	-
		0.000	-	0.967	-	0.000	-
		15.000	-	0.750	-	11.250	-
		4.000	-	0.900	-	3.600	-
Sub Total		**28.625**				**19.534**	
Soil		3.040	-	1.233	-	3.749	-
		0.815	-	0.674	-	0.549	-
		16.720	-	1.100	-	18.392	-
		0.000	-	1.233	-	0.000	-
Sub Total		**20.575**				**22.691**	
Earth pressure		8.539	18.145	1.500	0.633	12.809	11.492
Surface load		0.963	2.045	1.500	0.633	1.444	1.295
Total		**58.702**	**20.191**			**56.478**	**12.788**

(4) Evaluation of serviceability

Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.500 - 2 x 0.006 = 1.489 m

C = 0.000 kN/m²

ΣV = 58.702 kN/m

ΣH = 20.191 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 29.351 kN/m

F.S = Hu / ΣH = 1.454 < 1.5 --- **N.G**

ÿ **Need Shear Key.**

Shear Key

V = 58.702 kN/m

tanΦB = 0.500

Kp = 3.688

$$Kp = \frac{\cos^2(\varphi + \theta)}{\cos^2\theta \cos(\theta - \delta)\left[1 - \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi + \beta)}{\cos(\theta - \delta)\cos(\theta - \beta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	0.00	9.33	0

Pp = 32.749 kN/m

Hk = V $\tan\Phi_B$ + Pp = 62.100 kN/m

F.S = Hk / ΣH = 3.076 > 1.5 --- **O.K**

ÿ **Stability of sliding is O.K.**

Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

= 1.500 / 2 - (56.478 - 12.788) / 58.702 = 0.006 m

B / 6 = 1.500 / 6 = 0.250 > e --- **O.K**

F.S = ΣMr / ΣMo = 4.417 > 2.0 --- **O.K**

ÿ **Stability of overturning is O.K.**

Bearing Capacity

e = 0.006 m < B/6 = 0.250 m

x = 3·[B/2 - e] = 3 x (1.500 / 2 - 0.006) = 2.233 m > 1.500 m

resultant in middle one-third of base

q(max,min) = (ΣV / B) x (1 ± 6·e / B)

= 58.702 / 1.500 x (1 ± 6 x 0.006 / 1.500)

qmax = 40.031 kN/m² (Toe)

qmin = 38.238 kN/m² (Heel)

qmax = 40.031 kN/m² < qa = 291.67 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

2) At Earthquake

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta')} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)	
28.0	26.565	28.0	4.056	6.538	∞(ω = $\tan^{-1}$Kh)
					È(δ = Φ)

if β' + ω ¯Φ, δ' = Φ

α(ƒx)	δ'(ƒx)	H (m)	We (kN/m)	Pae (kN/m)	Kae	Kaeh	Kaev
37.7	28.000	1.900	102.599	30.218	0.8811	0.778	0.414
37.8	28.000	1.900	102.053	30.220	0.8812	0.778	0.414
37.9	**28.000**	**1.900**	**101.510**	**30.221**	**0.8812**	**0.778**	**0.414**
38.0	28.000	1.900	100.970	30.220	0.8812	0.778	0.414
38.1	28.000	1.900	100.432	30.219	0.8811	0.778	0.414

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 26.682 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 14.198 kN/m³

(2) Load

Division	Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
	V (kN/m)	H (kN/m)	X	Y	V · X	H · Y
Concrete	28.625				19.534	
Soil	20.575				22.691	
Earth pressure	14.198	26.682	1.500	0.950	21.297	25.347
Total	**63.398**	**26.682**	1.002	0.950	**63.522**	**25.347**

(3) Moment

Load		Calculation	Horizontal seismic coefficient	Horizental Force	MOMENT (kN·m)	
				H (kN/m)	Y	H · Y
Concrete	▷	8.250	0.115	0.945	0.950	0.898
	◁	0.303	0.115	0.035	0.767	0.027
	V	1.073	0.115	0.123	0.767	0.094
	Å	0.000	0.115	0.000	0.400	0.000
	Λ	0.000	0.115	0.000	0.400	0.000
	Λ̇	0.000	0.115	0.000	0.400	0.000
	>	15.000	0.115	1.719	0.200	0.344
Sub Total		**24.625**		**2.822**	-	**1.363**
Soil	▷·	3.040	0.115	0.348	1.633	0.569
	·▷	0.815	0.115	0.093	1.133	0.106
	▷·	16.720	0.115	1.916	0.950	1.820
	▷	0.000	0.115	0.000	0.400	0.000
Sub Total		**20.575**		**2.358**	-	**2.495**
Total		**45.200**		**5.180**	0.745	**3.858**

(4) Evaluation of serviceability

▷ Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.500 - 2 x 0.209 = 1.082 m

C = 0.000 kN/m²

ΣV = 63.398 kN/m

ΣH = 31.861 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 31.699 kN/m --- **N.G**

F.S = Hu / ΣH = 0.995 < 1.1

ÿ **Need Shear Key.**

◁ Shear Key

V = V = 63.398 kN/m

tanΦB = $\tan\Phi_B$ = 0.500

Kp = Kpe = 2.138

$$Kp = \frac{\cos^2(\varphi - \omega + \theta)}{\cos\omega \cos^2\theta \cos(\delta - \theta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi - \delta)\sin(\varphi - \omega + \beta)}{\cos(\delta - \theta + \omega)\cos(\beta - \theta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.00	0.00	9.33	0	6.538

Ppe = 18.986 kN/m

Hk = V $\tan\Phi_B$ + Ppe = 50.686 kN/m

F.S = Hk / ΣH = 1.591 > 1.1 --- **O.K**

ÿ **Stability of sliding is O.K.**

¤ Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

1.500 / 2 - (63.522 - (25.347 + 3.858)) / 63.398 = 0.209 m

B / 3 = 1.500 / 3 = 0.500 > e --- **O.K**

F.S = ΣMr / ΣMo = 2.175 > 1.5 --- **O.K**

ÿ **Stability of overturning is O.K.**

¥ Bearing Capacity

e = 0.209 m < B/3 = 0.500 m

x = 3·[B/2 - e] = 3 x (1.500 / 2 -0.209) = 1.623 m > 1.500 m

□ resultant in middle one-third of base

q(max,mi = (ΣV / B) x (1 ± 6·e / B)

= 63.398 / 1.500 x (1 ± 6 x 0.209 / 1.500)

qmax = 77.599 kN/m² (Toe)

qmin = 6.932 kN/m² (Heel)

qmax = 77.599 kN/m² < qa = 291.67 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

3.2.4 Load Calculation

1) Wall

(1) Earth pressure

At Nomal

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ฑx)	β(ฑx)	δ(ฑx)	θ(ฑx)
28.00	26.565	9.33	4.056

∞ (δ = Λ· x Φ)

α(ฑx)	δ'(ฑx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
37.6	9.333	1.100	43.364	7.248	0.631	0.613	0.146
37.7	9.333	1.100	42.937	7.249	0.631	0.614	0.146
37.8	**9.333**	**1.100**	**42.512**	**7.250**	**0.631**	**0.614**	**0.146**
37.9	9.333	1.100	42.089	7.250	0.631	0.614	0.146
38.0	9.333	1.100	41.667	7.248	0.631	0.613	0.146

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 7.058 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 1.678 kN/m³

At Earthquake

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ฑx)	β(ฑx)	δ(ฑx)	θ(ฑx)	ω(ฑx)
28.000	26.565	0.000	4.056	6.538

∞ $\omega = \tan^{-1}Kh$

α(ฑx)	δ(ฑx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
33.1	0.000	1.100	64.465	13.007	1.1315	1.129	0.080
33.2	0.000	1.100	63.929	13.008	1.1316	1.129	0.080
33.3	**0.000**	**1.100**	**63.395**	**13.008**	**1.1316**	**1.129**	**0.080**
33.4	0.000	1.100	62.865	13.007	1.1316	1.129	0.080
33.5	0.000	1.100	62.337	13.005	1.1314	1.129	0.080

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 12.978 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 0.920 kN/m³

Earthquake earth pressure : Paeh' = 1/2 x kaeh' x γt x H^2 = 5.920 kN/m³

Division		Load	Horizental Force	length (m)	Mr (kN·m)
		W (kN)	H (kN/m)	Y	H x Y
Concrete	▷	8.250	0.945	0.550	0.520
	◁	0.303	0.035	0.367	0.013
	V	1.073	0.123	0.367	0.045
Bottom of Wall		**9.625**	**1.103**		**0.578**
Earthquake earth pressure			**5.920**	0.550	**3.256**
Concrete	▷	4.125	0.473	0.275	0.130
	◁	0.008	0.001	0.183	0.000
	V	0.268	0.031	0.183	0.006
Middle of Wall		**4.401**	**0.504**		**0.136**
Earthquake earth pressure			**1.480**	0.275	**0.407**

(2) Stress Resultant

At Nomal

Bottom of Wall	(H=	1.100 m	)				
V=	7.058				=	7.058	kN
M=	7.058	x	1.100	/ 3	=	2.588	kN/m
Middle of Wall	(H=	0.550 m	)				
V=	1.764				=	1.764	kN
M=	1.764	x	0.550	/ 3	=	0.323	kN/m

At Earthquake

Bottom of Wall	(H=	1.100 m	)				
Ve=	12.978				=	12.978	kN
Me=	12.978	x	1.100	/2	=	7.138	kN/m
Middle of Wall	(H=	0.550 m	)				
Ve=	3.244				=	3.244	kN
Me=	3.244	x	0.550	/ 2	=	0.892	kN/m

(3) Design Load for cross section

Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Summary

Division		Bottom of Wall		Middle of Wall	
		Horizontal earth pressure	Inertial force	Horizontal earth pressure	Inertial force
LCB1	Shear force	11.293	0.000	2.823	0.000
	Moment	4.141	0.000	0.518	0.000
LCB2	Shear force	11.293	7.023	2.823	1.984
	Moment	4.141	3.834	0.518	0.543
LCB3	Shear force	7.058	0.000	1.764	0.000
	Moment	2.588	0.000	0.323	0.000

V Design Load for cross section

-Bottom of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
11.293	4.141	18.316	7.975	7.058	2.588

-Middle of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
2.823	0.518	4.807	1.061	1.764	0.323

2) Foundation

(1) Stress resultant of Foundation

▷ Load

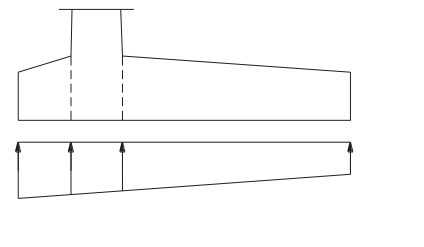
Division		V(kN)	H(kN)	Mr(kN.m)	Mo(kN.m)
At Nomal	Concrete	28.625	0.000	19.534	0.00
	Soil	20.575	0.000	22.691	0.00
	Earth pressure	8.539	18.145	12.809	11.492
	Surface load	0.963	2.045	1.444	1.295
	□	**58.702**	**20.191**	**56.478**	**12.79**
At Earthquake	Concrete	28.625	2.822	19.534	1.363
	Soil	20.575	2.358	22.691	2.495
	Earth pressure	8.539	18.145	12.809	11.492
	Earthquake earth pressure	5.659	8.536	8.488	13.855
	□	**63.398**	**31.861**	**63.522**	**29.21**

◁ Ultimate load Combination

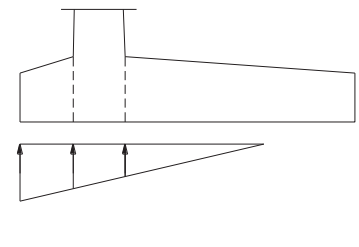
Division	□V	□Mr	□Mo	e	Load shape
LCB1	74.243	73.475	20.460	0.036	CASE1
LCB2	63.602	66.985	35.715	0.258	CASE2
LCB3	58.702	56.478	12.788	0.006	CASE1

V Stress resultant of Foundation

CASE1



CASE2



Division	q1	q2	q3	q4	e	CASE
LCB1	56.623	53.772	49.971	42.368	0.036	1
LCB2	86.181	68.665	45.309	-	0.258	2
LCB3	40.074	39.698	39.197	38.196	0.006	1

(2) Heel

Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete		0.000	0.267	0.000	
		8.000	0.400	3.200	
		4.000	0.200	0.800	
Sub Total		**12.000**	**0.333**	**4.000**	
Soil		3.040	0.507	1.542	
		16.720	0.400	6.688	
		0.000	0.533	0.000	
Sub Total		**19.760**	**0.417**	**8.230**	
Total		**31.760**	**0.385**	**12.230**	

Cross section force by Vertical Force

- At Nomal

V = 8.539 kN

M = 8.539 x 0.800 = 6.832 kN·m

- At Earthquake

V = 8.539 + 5.659 = 14.198 kN

M1 = 8.539 x 0.800 = 6.832 kN·m

M2 = 5.659 x 0.800 = 6.832 kN·m

V Cross section force by Stress resultant of Foundation

Load	q3	q4	length(m)	V (kN)	M (kN·m)
LCB1	49.971	42.368	0.389	-36.936	-14.369
LCB2	45.309	0.000	0.259	-17.580	-4.547
LCB3	39.197	38.196	0.398	-30.957	-12.329

∞ V = (q4 + q3)/2 × B5

Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	L	H	E	Stress resultant of Foundation	Total
LCB1	Vu	38.112	-	13.663	-	-36.936	14.840
	Mu	14.676	-	10.931	-	-14.369	11.238
LCB2	Vu	28.584	-	13.663	5.659	-17.580	30.326
	Mu	11.007	-	10.931	4.527	-4.547	21.917
LCB3	Vo	31.760	-	8.539	-	-30.957	9.342
	Mo	12.230	-	6.832	-	-12.329	6.732

(3) Toe

Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Å'	0.000	0.100	0.000	
	> '	3.000	0.150	0.450	
Sub Total		**3.000**	**0.150**	**0.450**	

Cross section force by Stress resultant of Foundation

Load	q1	q2	length(m)	V (kN)	M (kN·m)
LCB1	56.623	53.772	0.151	16.559	2.505
LCB2	86.181	68.665	0.156	23.227	3.615
LCB3	40.074	39.698	0.150	11.966	1.798

∞ V = (q1 + q2)/2 × B1

Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	H	Stress resultant of Foundation	Total
LCB1	Vu	-3.600	-	16.559	12.959
	Mu	-0.540	-	2.505	1.965
LCB2	Vu	-2.700	-	23.227	20.527
	Mu	-0.405	-	3.615	3.210
LCB3	Vo	-3.000	-	11.966	8.966
	Mo	-0.450	-	1.798	1.348

(4) Shear Key

Passive earth pressure

At Nomal : Pp = 32.749 kN/m

At Earthq : Ppe = 18.986 kN/m Ā Apply Cross section force at Nomal

Design Load for cross section

Division	qk1	qk2	H(m)	V (kN)	M (kN·m)
At Nomal	68.226	95.517	0.400	32.749	6.914

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)
Design Load for cross section	11.062	52.398	6.914

3) Summary

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)	ØMn(kN·m)	Bar	S.F
Bottom of Wall	7.975	18.316	2.588	131.063	D13 @ 125	16.43
Middle of Wall	1.061	4.807	0.323	56.513	D13 @ 250	53.28
Heel	7.975	30.326	2.588	131.063	D13 @ 125	16.43
Toe	3.210	20.527	1.348	66.265	D13 @ 250	20.64
Shear Key	11.062	52.398	6.914	66.265	D13 @ 250	5.99

3.2.5 Section Design

1) Bottom of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm	
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm	
M_u	=	7.975	kN·m	V_u	=	18.316	kN	M_o	=		2.588	kN·m	

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₸ ТФГТУКУФУZТ НПΣ И□.₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 61.502 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	ㅿ	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	ㅿ	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00024	ㅿ	$A_{s,4/3req}$	=	82.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00024	ㅿ	$A_{s,min}$	=	82.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00300	ㅿ	$A_{s,min}$	=	1032.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 7.975 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 18.316 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - $nA_s/b + nA_s/b \times$ □{1+2bd/nA_s}

= - 8 × 1,032.00 / 1000 + 8 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,032.00)]

= 67.507 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 2.588 / [1000 × 67.507 x (343.5 - 67.507 / 3)] × 10^6

= 0.239 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 2.588 / [1032.000 x (343.5 - 67.507 / 3)] × 10^6

= 7.812 MPa

fst = fs x (H - d' - χ) / (d - χ) = 8 x (400 - 57 - 0) / (344 - 68) = 7.81 MPa

ㅿ Maximum center space of reinforcement

Cc = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 7.81) - 2.5 x 50.00 = 13494.92 ㎜

300 x (280 / fs) = 300 x (280 / 7.81) = 10752.57 ㎜

Sa = 10752.57 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (10752.57 mm) Ā O.K

2) Middle of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	293.5	mm		
B	=	1000	mm	H	=	350	mm		d'	=	56.5	mm		
M_u	=	1.061	kN·m	V_u	=	4.807	kN	M_o	=		0.323	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (293.5 - 9.161) / 9.161

= 0.0931

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₮ ТФГТУКУФУZТ НПΣ И□.₮

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 9.562 ㎠

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎠ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	7293.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	988.3	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00004	㎠	$A_{s,4/3req}$	=	12.7	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00004	㎠	$A_{s,min}$	=	12.7	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00176	㎠	$A_{s,min}$	=	516.0	㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 56.513 kN·m > Mu = 1.061 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 207.536 kN > Vu = 4.807 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 516.00 / 1000 + 8 × 516.00 / 1000 × √[1 + 2 × 1000 × 293.5 / (8 × 516.00)]

= 45.270 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.323 / [1000 × 45.270 x (293.5 - 45.270 / 3)] × 10^6

= 0.051 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.323 / [516.000 x (293.5 - 45.270 / 3)] × 10^6

= 2.252 MPa

fst = fs x (H - d' - χ) / (d - χ) = 2 x (350 - 57 - 0) / (294 - 45) = 2.25 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 2.25) - 2.5 x 50.00 = 47126.69 ㎜

300 x (280 / fs) = 300 x (280 / 2.25) = 37303.97 ㎜

Sa = 37303.97 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (37303.97 mm) Ā O.K

3) Heel

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	7.975	kN·m	V_u	=	30.326	kN	M_o	=		2.588	kN·m		

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₸ ТФГТУКУФУZТ НПΣ И□.₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 61.502 ㎡

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎡ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	8536.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00024	㎡	$A_{s,4/3req}$	=	82.0	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00024	㎡	$A_{s,min}$	=	82.0	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00300	㎡	$A_{s,min}$	=	1032.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 7.975 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 30.326 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,032.00 / 1000 + 8 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,032.00)]

= 67.507 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 2.588 / [1000 × 67.507 x (343.5 - 67.507 / 3)] × 10^6

= 0.239 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 2.588 / [1032.000 x (343.5 - 67.507 / 3)] × 10^6

= 7.812 MPa

fst = fs x (H - d' - χ) / (d - χ) = 8 x (400 - 57 - 0) / (344 - 68) = 7.81 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 7.81) - 2.5 x 50.00 = 13494.92 ㎜

300 x (280 / fs) = 300 x (280 / 7.81) = 10752.57 ㎜

Sa = 10752.57 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (10752.57 mm) Ā O.K

4) Toe

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm			
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm			
M_u	=	3.210	kN·m	V_u	=	20.527	kN	M_o	=		1.348	kN·m			

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₸ ТФГТУКУФУZТ НПΣ И□.₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 24.739 ㎡

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎡ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	8536.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00010	㎡	$A_{s,4/3req}$	=	33.0	㎡
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00010	㎡	$A_{s,min}$	=	33.0	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00150	㎡	$A_{s,min}$	=	516.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 3.210 kN·m

Ā O.K

ㅿ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 20.527 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = $- nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

= $- 8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 516.00)]}$

= 49.285 ㎜

f_c = $2 \times M_o / [B \times \chi \times (d - \chi/3)]$

= $2.0 \times 1.348 / [1000 \times 49.285 \times (343.5 - 49.285 / 3)] \times 10^6$

= 0.167 MPa

f_s = $M_o / [A_s \times (d - \chi/3)]$

= $1.348 / [516.000 \times (343.5 - 49.285 / 3)] \times 10^6$

= 7.985 MPa

fst = fs x (H - d' - χ) / (d - χ) = 8 x (400 - 57 - 0) / (344 - 49) = 7.99 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 7.99) - 2.5 x 50.00 = 1.3E+04 ㎜

300 x (280 / fs) = 300 x (280 / 7.99) = 1.1E+04 ㎜

Sa = 1.05E+04 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (1.1E+04 mm) Ā O.K

5) Shear Key

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm	
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm	
M_u	=	11.062	kN·m	V_u	=	52.398	kN	M_o	=		6.914	kN·m	

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 85.359 ㎡

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎡ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	8536.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00033	㎡	$A_{s,4/3req}$	=	113.8	㎡
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00033	㎡	$A_{s,min}$	=	113.8	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.00150	㎡	$A_{s,min}$	=	516.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 11.062 kN·m

Ā O.K

ㅿ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 52.398 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = $- nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

= $- 8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 516.00)]}$

= 49.285 ㅡ

f_c = $2 \times M_o / [B \times \chi \times (d - \chi/3)]$

= $2.0 \times 6.914 / [1000 \times 49.285 \times (343.5 - 49.285 / 3)] \times 10^6$

= 0.858 MPa

f_s = $M_o / [A_s \times (d - \chi/3)]$

= $6.914 / [516.000 \times (343.5 - 49.285 / 3)] \times 10^6$

= 40.967 MPa

fst = fs x (H - d' - χ) / (d - χ) = 41 x (400 - 57 - 1) / (344 - 49) = 40.97 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 40.97) - 2.5 x 50.00 = 2472.20 ㅡ

300 x (280 / fs) = 300 x (280 / 40.97) = 2050.42 ㅡ

Sa = 2050.42 ㅡ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (2050.42 mm) Ā O.K

3.2.6 Distribution Reinforcement Check

1) Wall (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	250	=	516.0	㎟			
Compression side	D	13@	250	=	516.0	㎟			
			□	=	1032.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

4. RETAINING WALL

4.1 Retaining Wall (H=3.0m)

4.1.1 Design Conditions (H=3.000m , N= LEVEL , Ho= 0.000)

1) General Items

(1)	Type of Retaining Wall	:	L Type Retaining Wall	
(2)	Height of Retaining Wall	:	3.000	m
(3)	Slope of Backfill	:	LEVEL	
(4)	Height of Backfill	:	0.000	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(=0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

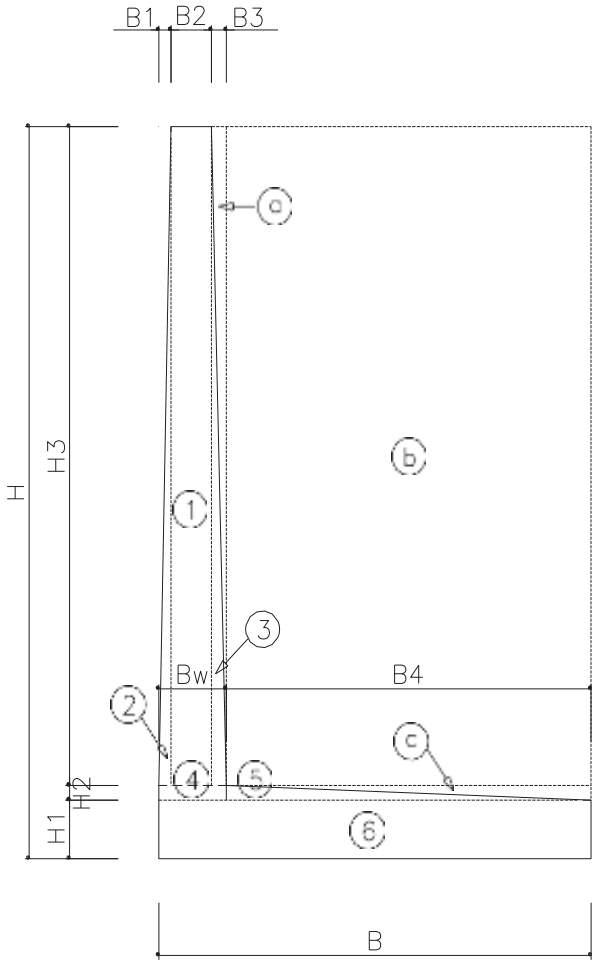
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

4.1.2 Section Assumption



※ Sectional specification

- Width

B1	B2	B3	B4	B	Bw	
0.052	0.300	0.048	1.800	2.200	0.400	

- Height

H1	H2	H3	H	Ho		
0.400	0.000	2.600	3.000	0.000		

4.1.3 Evaluation of serviceability

1) At Nomal

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta)} \times W$$

where,

Φ(ɾx)	β(ɾx)	δ(ɾx)	θ(ɾx)
28.0	0.000	0.000	1.058

α(ɾx)	δ(ɾx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
58.8	0.000	3.000	51.781	30.867	0.361	0.361	0.000
58.9	0.000	3.000	51.577	30.868	0.361	0.361	0.000
59.0	**0.000**	**3.000**	**51.374**	**30.868**	**0.361**	**0.361**	**0.000**
59.1	0.000	3.000	51.171	30.868	0.361	0.361	0.000
59.2	0.000	3.000	50.968	30.867	0.361	0.361	0.000

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 30.868 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 0.000 kN/m³

(2) Load

Division		Calculation							Unit Weight	Vertical Force(kN)
Concrete	▷	2.600	x	0.300			=	0.780	25.00	19.500
	◁	2.600	x	0.052	x	½	=	0.068	25.00	1.690
	V	2.600	x	0.048	x	½	=	0.062	25.00	1.560
	Λ	0.000	x	0.400			=	0.000	25.00	0.000
	Λ	0.000	x	1.800	x	½	=	0.000	25.00	0.000
	Λ	0.400	x	2.200			=	0.880	25.00	22.000
Soil	▷	2.600	x	0.048	x	½	=	0.062	19.00	1.186
	▷	2.600	x	1.800			=	4.680	19.00	88.920
	▷	0.000	x	1.800	x	½	=	0.000	19.00	0.000
Surface load				1.848			=	1.848	10.00	18.480

(3) Moment

Division		Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
		V (kN/m)	H (kN/m)	X	Y	V·X(Mr)	H·Y(Mo)
Concrete	▷	19.500	-	0.202	-	3.939	-
	◁	1.690	-	0.035	-	0.059	-
	V	1.560	-	0.368	-	0.574	-
	Λ	0.000	-	0.200	-	0.000	-
	Λ	0.000	-	1.000	-	0.000	-
	Λ	22.000	-	1.100	-	24.200	-
Sub Total		**44.750**				**28.772**	
Soil	▷	1.186	-	0.384	-	0.455	-
	▷	88.920	-	1.300	-	115.596	-
	▷	0.000	-	1.600	-	0.000	-
Sub Total		**90.106**				**116.051**	
Earth pressure		0.000	30.868	2.200	1.000	0.000	30.868
Surface load		**18.480**	**10.830**	1.276	1.500	**23.580**	**16.245**
Total		**153.336**	**41.698**			**168.403**	**47.113**

(4) Evaluation of serviceability

Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 2.200 - 2 x 0.309 = 1.582 m

C = 0.000 kN/m²

ΣV = 153.336 kN/m

ΣH = 41.698 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 76.668 kN/m

F.S = Hu / ΣH = 1.839 > 1.5 --- **O.K**

ÿ **Stability of sliding is O.K.**

Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

= 2.200 / 2 - (168.403 - 47.113) / 153.336 = 0.309 m

B / 6 = 2.200 / 6 = 0.367 > e --- **O.K**

F.S = ΣMr / ΣMo = 3.574 > 2.0 --- **O.K**

ÿ **Stability of overturning is O.K.**

V Bearing Capacity

e = 0.309 m < B/6 = 0.367 m

x = 3·[B/2 - e] = 3 x (2.200 / 2 - 0.309) = 2.373 m > 2.200 m

resultant in middle one-third of base

q(max,min) = (ΣV / B) x (1 ± 6·e / B)

= 153.336 / 2.200 x (1 ± 6 x 0.309 / 2.200)

qmax = 128.434 kN/m² (Toe)

qmin = 10.962 kN/m² (Heel)

qmax = 128.434 kN/m² < qa = 255.000 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

2) At Earthquake

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta')} \times \frac{We}{COS(\omega)}$$

where,

Φ(rx)	β(rx)	δ(rx)	θ(rx)	ω(rx)
28.0	0.000	0.000	1.058	6.538

∵ $\omega = \tan^{-1}Kh$

If β + ω ≦ Φ, δ' = Φ

α(rx)	δ'(rx)	H (m)	We (kN/m)	Pae (kN/m)	Kae	Kaeh	Kaev
53.0	0.000	3.000	64.851	37.428	0.438	0.438	0.000
53.1	0.000	3.000	64.615	37.428	0.438	0.438	0.000
53.2	**0.000**	**3.000**	**64.381**	**37.429**	**0.438**	**0.438**	**0.000**
53.3	0.000	3.000	64.147	37.429	0.438	0.438	0.000
53.4	0.000	3.000	63.914	37.428	0.438	0.438	0.000

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 37.429 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 0.000 kN/m³

(2) Load

Division	Vertical Force	Horizontal Force	length (m)		MOMENT (kN·m)	
	V (kN/m)	H (kN/m)	X	Y	V · X	H · Y
Concrete	44.750				28.772	
Soil	90.106				116.051	
Earth pressure	0.000	37.429	2.200	1.500	0.000	56.143
Total	**134.856**	**37.429**	1.074	1.500	**144.823**	**56.143**

(3) Moment

Load		Calculation	Horizontal seismic coefficient	Horizental Force	MOMENT (kN·m)	
				H (kN/m)	Y	H · Y
Concrete	▷	19.500	0.115	2.235	1.700	3.799
	◁	1.690	0.115	0.194	1.267	0.245
	V	1.560	0.115	0.179	1.267	0.226
	Å	0.000	0.115	0.000	0.400	0.000
	Λ	0.000	0.115	0.000	0.400	0.000
	Λ̇	22.000	0.115	2.521	0.200	0.504
Sub Total		**44.750**		**5.128**	-	**4.775**
Soil	▷·	1.186	0.115	0.136	2.133	0.290
	·▷	88.920	0.115	10.190	1.700	17.323
	▷·	0.000	0.115	0.000	0.400	0.000
Sub Total		**90.106**		**10.326**	-	**17.613**
Total		**134.856**		**15.454**	1.449	**22.388**

(4) Evaluation of serviceability

Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 2.200 - 2 x 0.608 = 0.984 m

C = 0.000 kN/m²

ΣV = 134.856 kN/m

ΣH = 52.883 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 67.428 kN/m

F.S = Hu / ΣH = 1.275 > 1.1 --- **O.K**

ÿ **Stability of sliding is O.K.**

Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

2.200 / 2 - (144.823 - (56.143 + 22.388)) / 134.856 = 0.608 m

B / 3 = 2.200 / 3 = 0.733 > e --- **O.K**

F.S = ΣMr / ΣMo = 1.844 > 1.5 --- **O.K**

ÿ **Stability of overturning is O.K.**

V Bearing Capacity

e = 0.608 m < B/3 = 0.733 m

x = 3·[B/2 - e] = 3 x (2.200 / 2 -0.608) = 1.476 m < 2.200 m

Pressure for resultant outside middle one-third

qmax = 2·ΣV / X

= 134.856 x 2 / 1.476 = 182.731 kN/m²

qmax = 182.731 kN/m² (Toe)

qmin = 0.000 kN/m² (Heel)

qmax = 182.731 kN/m² < qa = 255.000 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

4.1.4 Load Calculation

1) Wall

(1) Earth pressure

At Nomal

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	0.000	9.33	1.058

∞ (δ = Λ· x Φ)

α(ƒx)	δ(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
57.1	9.333	2.600	42.731	21.941	0.342	0.336	0.062
57.2	9.333	2.600	42.573	21.941	0.342	0.336	0.062
57.3	**9.333**	**2.600**	**42.414**	**21.941**	**0.342**	**0.336**	**0.062**
57.4	9.333	2.600	42.256	21.940	0.342	0.336	0.062
57.5	9.333	2.600	42.098	21.939	0.342	0.336	0.062

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 21.578 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 3.982 kN/m³

At Earthquake

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{We}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	0.000	0.000	1.058	6.538

∞ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
53.5	0.000	2.600	49.025	28.567	0.445	0.445	0.008
53.6	0.000	2.600	48.850	28.567	0.445	0.445	0.008
53.7	**0.000**	**2.600**	**48.676**	**28.567**	**0.445**	**0.445**	**0.008**
53.8	0.000	2.600	48.503	28.567	0.445	0.445	0.008
53.9	0.000	2.600	48.330	28.567	0.445	0.445	0.008

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 28.578 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 0.514 kN/m³

Earthquake earth pressure : Paeh' = 1/2 x kaeh' x γt x H^2 = 7.000 kN/m³

Division		Load	Horizental Force	length (m)	Mr (kN·m)
		W (kN)	H (kN/m)	Y	H x Y
Concrete		19.500	2.235	1.300	2.905
		1.690	0.194	0.867	0.168
	V	1.560	0.179	0.867	0.155
Bottom of Wall		**22.750**	**2.607**		**3.228**
Earthquake earth pressure			**7.000**	1.300	**9.100**
Concrete		9.750	1.117	0.650	0.726
		0.042	0.005	0.433	0.002
	V	0.390	0.045	0.433	0.019
Middle of Wall		**10.182**	**1.167**		**0.748**
Earthquake earth pressure			**1.750**	0.650	**1.137**

(2) Stress Resultant

▷ At Nomal

Bottom of Wall (H= 2.600 m)

V=	21.578				=	21.578	kN
M=	21.578	x	2.600	/ 3	=	18.701	kN/m

Middle of Wall (H= 1.300 m)

V=	5.394				=	5.394	kN
M=	5.394	x	1.300	/ 3	=	2.338	kN/m

◁ By surface load

Bottom of Wall (H= 2.600 m)

Vq=	kah x qL x H	=	8.736	kN/m
Mq=	Ph1 x y	=	11.357	kN.m

Middle of Wall (H= 1.300 m)

Vq=	kah x qL x H	=	4.368	kN/m
Mq=	Ph1 x y	=	2.839	kN.m

V At Earthquake

Bottom of Wall (H= 2.600 m)

Ve=	28.578				=	28.578	kN
Me=	28.578	x	2.600	/2	=	37.151	kN/m

Middle of Wall (H= 1.300 m)

Ve=	7.144				=	7.144	kN
Me=	7.144	x	1.300	/ 2	=	4.644	kN/m

(3) Design Load for cross section

▷ Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

◁ Summary

-Bottom of Wall

Division		Horizontal earth pressure	Surface load	Inertial force
LCB1	Shear force	34.525	13.978	0.000
	Moment	29.921	18.171	0.000
LCB2	Shear force	34.525	0.000	9.607
	Moment	29.921	0.000	12.328
LCB3	Shear force	21.578	8.736	0.000
	Moment	18.701	11.357	0.000

-Middle of Wall

Division		Horizontal earth pressure	Surface load	Inertial force
LCB1	Shear force	8.631	6.989	0.000
	Moment	3.740	4.542	0.000
LCB2	Shear force	8.631	0.000	2.917
	Moment	3.740	0.000	1.885
LCB3	Shear force	5.394	4.368	0.000
	Moment	2.338	2.839	0.000

V Design Load for cross section

-Bottom of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
48.502	48.093	44.132	42.249	30.314	30.058

-Middle of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
15.620	8.283	11.548	5.626	9.762	5.177

2) Foundation

(1) Stress resultant of Foundation

▷ Load

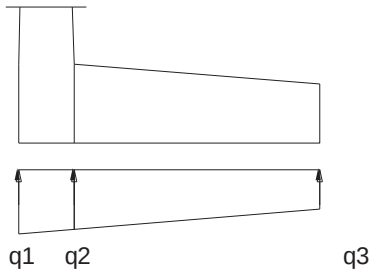
Division		V(kN)	H(kN)	Mr(kN.m)	Mo(kN.m)
At Nomal	Concrete	44.750	0.000	28.772	0.000
	Soil	90.106	0.000	116.051	0.000
	Earth pressure	0.000	30.868	0.000	30.868
	Surface load	18.480	10.830	23.580	16.245
	□	**153.336**	**41.698**	**168.403**	**47.113**
At Earthquake	Concrete	44.750	5.128	28.772	4.775
	Soil	90.106	10.326	116.051	17.613
	Earth pressure	0.000	30.868	0.000	30.868
	Earthquake earth pressure	0.000	6.560	0.000	25.275
	□	**134.856**	**52.883**	**144.823**	**78.53**

◁ Ultimate load Combination

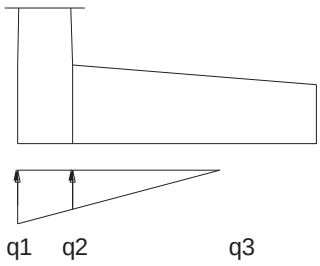
Division	□V	□Mr	□Mo	e	Load shape
LCB1	191.395	211.516	75.381	0.389	CASE2
LCB2	121.370	130.341	94.814	0.807	CASE2
LCB3	153.336	168.403	47.113	0.309	CASE1

V Stress resultant of Foundation

CASE1



CASE2



Division	q1	q2	q3	e	CASE
LCB1	179.461	145.806	-	0.389	2
LCB2	276.155	150.487	-	0.807	2
LCB3	128.434	107.076	10.962	0.309	1

(2) Heel

▶ Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Λ'	0.000	0.600	0.000	
	Λ̇'	18.000	0.900	16.200	
Sub Total		**18.000**	**0.900**	**16.200**	
Soil	·Ḋ	88.920	0.900	80.028	
	Ḋ·	0.000	1.200	0.000	
Sub Total		**88.920**	**0.900**	**80.028**	
Total		**106.920**	**0.900**	**96.228**	

◀ Cross section force by Stress resultant of Foundation

Load	q2	q3	length(m)	V (kN)	M (kN·m)
LCB1	145.806	0.000	0.578	-126.341	-72.983
LCB2	150.487	0.000	0.160	-36.042	-5.755
LCB3	107.076	10.962	0.656	-106.234	-69.659

∞ V = (q3 + q2)/2 × B5

V Design Load for cross section

-Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Division		D	L	H	E	Stress resultant of Foundation	Total
LCB1	Vu	128.304	28.8000	-	-	-126.341	30.763
	Mu	115.474	25.9200	-	-	-72.983	68.410
LCB2	Vu	96.228	-	-	-	-36.042	60.186
	Mu	86.605	-	-	-	-5.755	80.851
LCB3	Vo	106.920	18.0000	-	-	-106.234	18.686
	Mo	96.228	16.2000	-	-	-69.659	42.769

3) Summary

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)	ØMn(kN·m)	Bar	S.F
Bottom of Wall	48.093	48.502	30.058	131.063	D13 @ 125	2.73
Middle of Wall	8.283	15.620	5.177	56.513	D13 @ 250	6.82
Heel	48.093	60.186	30.058	131.063	D13 @ 125	2.73

4.1.5 Section Design

1) Bottom of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm	
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm	
M_u	=	48.093	kN·m	V_u	=	48.502	kN	M_o	=		30.058	kN·m	

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 373.526 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00145	㎟	$A_{s,4/3req}$	=	498.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00145	㎟	$A_{s,min}$	=	498.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00300	㎟	$A_{s,min}$	=	1032.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 48.093 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 48.502 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = $-nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

= $-8 \times 1{,}032.00 / 1000 + 8 \times 1{,}032.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 1{,}032.00)]}$

= 67.507 ㎜

f_c = $2 \times M_o / [B \times \chi \times (d - \chi/3)]$

= $2.0 \times 30.058 / [1000 \times 67.507 \times (343.5 - 67.507 / 3)] \times 10^6$

= 2.774 MPa

f_s = $M_o / [A_s \times (d - \chi/3)]$

= $30.058 / [1032.000 \times (343.5 - 67.507 / 3)] \times 10^6$

= 90.735 MPa

fst = fs x (H - d' - χ) / (d - χ) = 91 x (400 - 57 - 3) / (344 - 68) = 90.74 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 90.74) - 2.5 x 50.00 = 1047.64 ㎜

300 x (280 / fs) = 300 x (280 / 90.74) = 925.77 ㎜

Sa = 925.77 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (925.77 mm) Ā O.K

2) Middle of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	293.5	mm		
B	=	1000	mm	H	=	350	mm		d'	=	56.5	mm		
M_u	=	8.283	kN·m	V_u	=	15.620	kN	M_o	=		5.177	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (293.5 - 9.161) / 9.161

= 0.0931

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 74.803 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7293.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	988.3	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00034	㎟	$A_{s,4/3req}$	=	99.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00034	㎟	$A_{s,min}$	=	99.7	㎟
P_{use}	=	A_s / (B·d)	=	0.00176	㎟	$A_{s,min}$	=	516.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 56.513 kN·m > Mu = 8.283 kN·m

Ā O.K

ㅿ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 207.536 kN > Vu = 15.620 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = $- nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

= $- 8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 293.5 / (8 \times 516.00)]}$

= 45.270 ㎜

f_c = $2 \times M_o / [B \times \chi \times (d - \chi/3)]$

= $2.0 \times 5.177 / [1000 \times 45.270 \times (293.5 - 45.270 / 3)] \times 10^6$

= 0.821 MPa

f_s = $M_o / [A_s \times (d - \chi/3)]$

= $5.177 / [516.000 \times (293.5 - 45.270 / 3)] \times 10^6$

= 36.034 MPa

fst = fs x (H - d' - χ) / (d - χ) = 36 x (350 - 57 - 1) / (294 - 45) = 36.03 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 36.03) - 2.5 x 50.00 = 2827.78 ㎜

300 x (280 / fs) = 300 x (280 / 36.03) = 2331.14 ㎜

Sa = 2331.14 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (2331.14 mm) Ā O.K

3) Heel

(1) Section Design

ก. Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	48.093	kN·m	V_u	=	60.186	kN	M_o	=		30.058	kN·m		

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 18.322) / 18.322

= 0.0532

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 373.526 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ก. Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00145	㎟	$A_{s,4/3req}$	=	498.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00145	㎟	$A_{s,min}$	=	498.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00300	㎟	$A_{s,min}$	=	1032.0	㎟

ᴄ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ก. Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 131.063 kN·m > Mu = 48.093 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 60.186 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,032.00 / 1000 + 8 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,032.00)]

= 67.507 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 30.058 / [1000 × 67.507 x (343.5 - 67.507 / 3)] × 10^6

= 2.774 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 30.058 / [1032.000 x (343.5 - 67.507 / 3)] × 10^6

= 90.735 MPa

fst = fs x (H - d' - χ) / (d - χ) = 91 x (400 - 57 - 3) / (344 - 68) = 90.74 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 90.74) - 2.5 x 50.00 = 1047.64 ㎜

300 x (280 / fs) = 300 x (280 / 90.74) = 925.77 ㎜

Sa = 925.77 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (925.77 mm) Ā O.K

4.1.6 Distribution Reinforcement Check

1) Wall (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	250	=	516.0	㎟			
Compression side	D	13@	250	=	516.0	㎟			
			□	=	1032.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

5. GEOTECHNICAL CALCULATION

5.1 Bearing capacity & Settlement

▶ Bearing Capacity : BOX CULVERT 1 (STA. 0+012.000)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	16.60	(m)	■ width B	:	3.90	(m)
■ Penetration depth D_f	:	1.50	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 745 + 227 + 230 = 1{,}202 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}202 / 3.0 = 400.71 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}202 / 2.0 = 601.06 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 743 + 235 + 164 = 1{,}143 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}143 / 3.0 = 380.85 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}143 / 2.0 = 571.28 \quad (kN/m^2)$$

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	600 400 200	~ ~ ~	1,000 700 600	700 500 300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	400 200 100	~ ~ ~	600 400 300	400 300 150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	300 200 100	~ ~ ~	500 400 300	300 250 150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	300 100 50	~ ~ ~	600 300 100	400 200 50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard medium to stiff Soft	200 100 50	~ ~ ~	400 300 100	300 150 50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 1.00 (m)
- Additional Capacity : 41.67 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 41.67 = 291.67 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	400.71	380.85	291.67	291.67	119.02	OK
Seismic	601.06	571.28		291.67	-	

▶ Settlement : BOX CULVERT 1 (STA. 0+012.000)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	3.90	(m)	■ Length L	:	16.60	(m)
■ load q	:	119.021	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 119.02 \times 3.90 \times \frac{1 - 0.35^2}{16000} \times 1.481 = 37.70 \text{ (mm)}$$

There, q : load(kN/m^2)

B : width(m)

E : Young's modulus(Mpa)

v : Poisson's ratio

I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.98 \times 1.54 \times 114.77 \times 2.79E\text{-}04 = 48.40 \text{ (mm)}$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

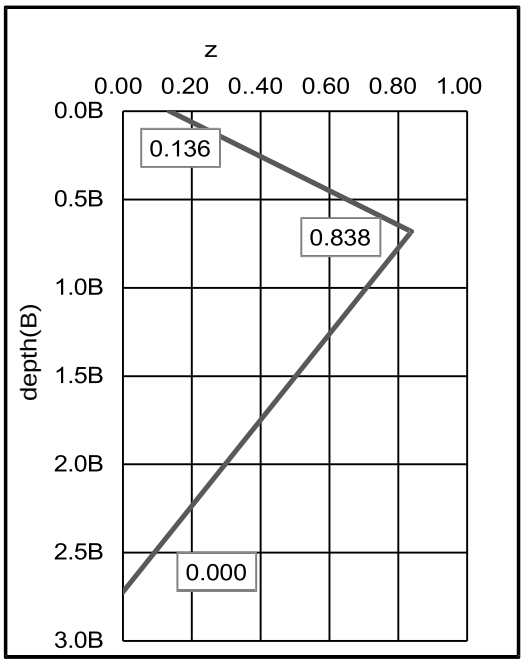
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ'_{vp})

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.136	0.00E+00
0.88	Silty sand	16,000	0.370	2.05E-05
1.77	Silty sand	16,000	0.604	3.34E-05
2.65	Silty sand	16,000	0.838	4.64E-05
3.79	Silty sand	16,000	0.718	5.11E-05
4.93	Silty sand	16,000	0.599	4.26E-05
6.07	Silty sand	16,000	0.479	3.41E-05
7.21	Silty sand	16,000	0.359	2.55E-05
8.34	Silty sand	16,000	0.239	1.70E-05
9.48	Silty sand	16,000	0.120	8.51E-06
10.62	Silty sand	16,000	0.000	0.00E+00
Sum				2.79E-04



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	37.70	
② Schmertmann's theory	48.40	
Maximum settlement	48.40	

▶ Bearing Capacity : BOX CULVERT 2 (STA. 1+400.00)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	13.00	(m)	■ width B	:	1.80	(m)
■ Penetration depth D_f	:	1.80	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 724 + 272 + 109 = 1{,}105 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}105 / 3.0 = 368.48 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}105 / 2.0 = 552.72 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 858 + 314 + 79 = 1{,}251 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}251 / 3.0 = 416.92 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}251 / 2.0 = 625.38 \quad (kN/m^2)$$

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	600 400 200	~ ~ ~	1,000 700 600	700 500 300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	400 200 100	~ ~ ~	600 400 300	400 300 150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	300 200 100	~ ~ ~	500 400 300	300 250 150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	300 100 50	~ ~ ~	600 300 100	400 200 50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard medium to stiff Soft	200 100 50	~ ~ ~	400 300 100	300 150 50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 1.30 (m)
- Additional Capacity : 54.17 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 54.17 = 304.17 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	368.48	416.92	304.17	304.17	148.37	OK
Seismic	552.72	625.38		304.17	-	

▶ Settlement : BOX CULVERT 2 (STA. 1+400.00)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	1.80	(m)	■ Length L	:	13.00	(m)
■ load q	:	148.371	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 148.37 \times 1.80 \times \frac{1 - 0.35^2}{16000} \times 1.778 = 26.04 \quad (mm)$$

There,
- q : load(kN/m^2)
- B : width(m)
- E : Young's modulus(Mpa)
- v : Poisson's ratio
- I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.96 \times 1.54 \times 138.17 \times 1.51E\text{-}04 = 30.85 \quad (mm)$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

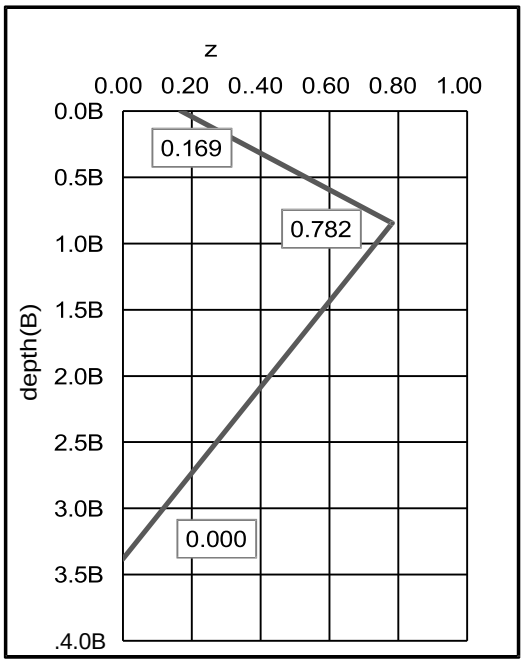
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ_{vp}')

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.169	0.00E+00
0.51	Silty sand	16,000	0.373	1.18E-05
1.01	Silty sand	16,000	0.578	1.83E-05
1.52	Silty sand	16,000	0.782	2.48E-05
2.17	Silty sand	16,000	0.670	2.73E-05
2.83	Silty sand	16,000	0.559	2.28E-05
3.48	Silty sand	16,000	0.447	1.82E-05
4.13	Silty sand	16,000	0.335	1.37E-05
4.78	Silty sand	16,000	0.223	9.11E-06
5.43	Silty sand	16,000	0.112	4.55E-06
6.09	Silty sand	16,000	0.000	0.00E+00
Sum				1.51E-04



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	26.04	
② Schmertmann's theory	30.85	
Maximum settlement	30.85	

▶ Bearing Capacity : BOX CULVERT 3 (STA. 1+400.00)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	10.50	(m)	■ width B	:	1.50	(m)
■ Penetration depth D_f	:	1.20	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 725 + 182 + 90 = 997 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 997 / 3.0 = 332.45 \quad (kN/m^2)$$

seismic condition

$$q_a = 997 / 2.0 = 498.68 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 810 + 200 + 66 = 1{,}076 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}076 / 3.0 = 358.80 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}076 / 2.0 = 538.20 \quad (kN/m^2)$$

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact	600	~	1,000	700
	Medium to compact	400	~	700	500
	Loose	200	~	600	300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	400	~	600	400
	Medium to compact	200	~	400	300
	Loose	100	~	300	150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	300	~	500	300
	Medium to compact	200	~	400	250
	Loose	100	~	300	150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	300	~	600	400
	Medium to stiff	100	~	300	200
	Soft	50	~	100	50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard	200	~	400	300
	medium to stiff	100	~	300	150
	Soft	50	~	100	50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 0.70 (m)
- Additional Capacity : 29.17 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 29.17 = 279.17 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	332.45	358.80	279.17	279.17	114.69	OK
Seismic	498.68	538.20		279.17	-	

▶ Settlement : BOX CULVERT 3 (STA. 1+400.00)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	1.50	(m)	■ Length L	:	10.50	(m)
■ load q	:	114.687	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 114.69 \times 1.50 \times \frac{1 - 0.35^2}{16000} \times 1.760 = 16.61 \quad (mm)$$

There,
- q : load(kN/m^2)
- B : width(m)
- E : Young's modulus(Mpa)
- v : Poisson's ratio
- I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.98 \times 1.54 \times 110.44 \times 1.28E\text{-}04 = 21.38 \quad (mm)$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

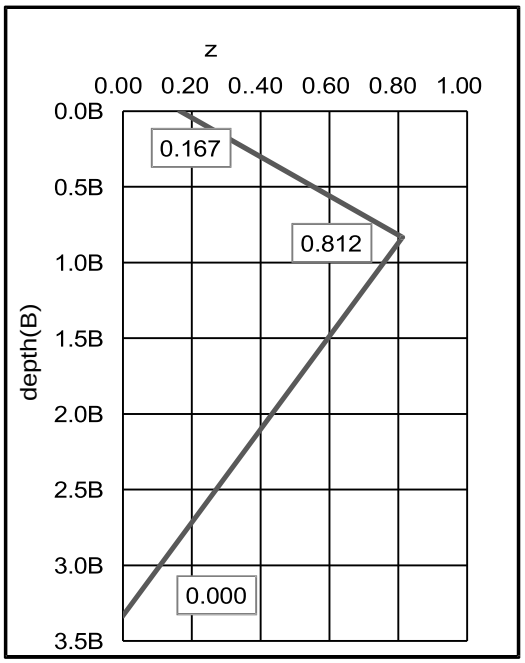
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ'_{vp})

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.167	0.00E+00
0.42	Silty sand	16,000	0.382	9.94E-06
0.83	Silty sand	16,000	0.597	1.55E-05
1.25	Silty sand	16,000	0.812	2.11E-05
1.79	Silty sand	16,000	0.696	2.33E-05
2.32	Silty sand	16,000	0.580	1.94E-05
2.86	Silty sand	16,000	0.464	1.55E-05
3.39	Silty sand	16,000	0.348	1.16E-05
3.93	Silty sand	16,000	0.232	7.77E-06
4.46	Silty sand	16,000	0.116	3.88E-06
5.00	Silty sand	16,000	0.000	0.00E+00
Sum				1.28E-04



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	16.61	
② Schmertmann's theory	21.38	
Maximum settlement	21.38	

▶ Bearing Capacity : BOX CULVERT 4 (STA. 3+300.000)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	9.50	(m)	■ width B	:	1.80	(m)
■ Penetration depth D_f	:	1.50	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 735 + 227 + 107 = 1{,}070 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}070 / 3.0 = 356.51 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}070 / 2.0 = 534.77 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 839 + 258 + 77 = 1{,}174 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$q_a = 1{,}174 / 3.0 = 391.40$ (kN/m^2)

seismic condition

$q_a = 1{,}174 / 2.0 = 587.10$ (kN/m^2)

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	600 400 200	~ ~ ~	1,000 700 600	700 500 300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	400 200 100	~ ~ ~	600 400 300	400 300 150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	300 200 100	~ ~ ~	500 400 300	300 250 150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	300 100 50	~ ~ ~	600 300 100	400 200 50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard medium to stiff Soft	200 100 50	~ ~ ~	400 300 100	300 150 50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 1.00 (m)
- Additional Capacity : 41.67 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 41.67 = 291.67 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	356.51	391.40	291.67	291.67	128.41	OK
Seismic	534.77	587.10		291.67	-	

▶ Settlement : BOX CULVERT 4 (STA. 3+300.000)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	1.80	(m)	■ Length L	:	9.50	(m)
■ load q	:	128.410	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 128.41 \times 1.80 \times \frac{1 - 0.35^2}{16000} \times 1.622 = 20.56 \quad (mm)$$

There,
- q : load(kN/m^2)
- B : width(m)
- E : Young's modulus(Mpa)
- v : Poisson's ratio
- I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.97 \times 1.54 \times 120.76 \times 1.33E\text{-}04 = 23.91 \quad (mm)$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

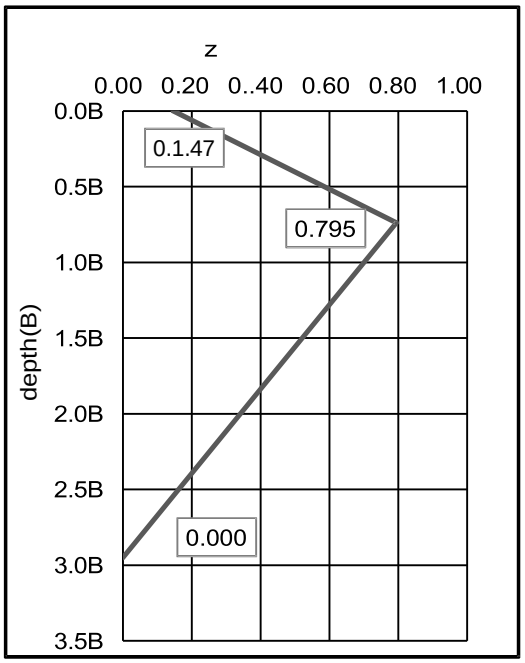
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ_{vp}')

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.147	0.00E+00
0.44	Silty sand	16,000	0.363	1.00E-05
0.88	Silty sand	16,000	0.579	1.60E-05
1.33	Silty sand	16,000	0.795	2.20E-05
1.90	Silty sand	16,000	0.681	2.42E-05
2.47	Silty sand	16,000	0.568	2.02E-05
3.03	Silty sand	16,000	0.454	1.61E-05
3.60	Silty sand	16,000	0.341	1.21E-05
4.17	Silty sand	16,000	0.227	8.07E-06
4.74	Silty sand	16,000	0.114	4.04E-06
5.31	Silty sand	16,000	0.000	0.00E+00
Sum				1.33E-04



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	20.56	
② Schmertmann's theory	23.91	
Maximum settlement	23.91	

▶ Bearing Capacity : Box Culvert Wing wall (Reverse T Type) (H=2.2m)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	1.80	(m)	■ width B	:	1.00	(m)
■ Penetration depth D_f	:	2.20	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 811 + 333 + 55 = 1{,}200 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}200 / 3.0 = 399.87 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}200 / 2.0 = 599.81 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 1{,}090 + 479 + 36 = 1{,}605 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$q_a = 1{,}605 / 3.0 = 534.87$ (kN/m^2)

seismic condition

$q_a = 1{,}605 / 2.0 = 802.30$ (kN/m^2)

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	600 400 200	~ ~ ~	1,000 700 600	700 500 300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	400 200 100	~ ~ ~	600 400 300	400 300 150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	300 200 100	~ ~ ~	500 400 300	300 250 150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	300 100 50	~ ~ ~	600 300 100	400 200 50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard medium to stiff Soft	200 100 50	~ ~ ~	400 300 100	300 150 50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 1.70 (m)
- Additional Capacity : 70.83 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 70.83 = 320.83 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	399.87	534.87	320.83	320.83	73.44	OK
Seismic	599.81	802.30		320.83	139.33	OK

▶ Settlement : Box Culvert Wing wall (Reverse T Type) (H=2.2m)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	1.00	(m)	■ Length L	:	1.80	(m)
■ load q	:	73.435	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 73.44 \times 1.00 \times \frac{1 - 0.35^2}{16000} \times 1.088 = 4.38 \text{ (mm)}$$

There, q : load(kN/m^2)

B : width(m)

E : Young's modulus(Mpa)

v : Poisson's ratio

I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.96 \times 1.54 \times 67.49 \times 5.12E\text{-}05 = 5.09 \text{ (mm)}$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

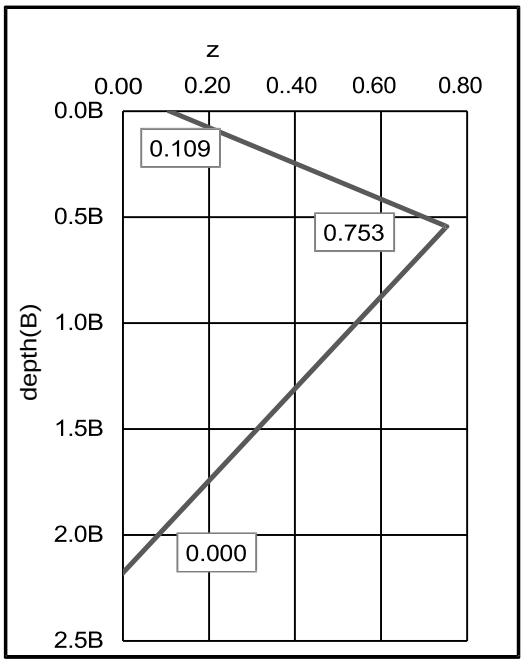
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ_{vp}')

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.109	0.00E+00
0.18	Silty sand	16,000	0.323	3.67E-06
0.36	Silty sand	16,000	0.538	6.10E-06
0.54	Silty sand	16,000	0.753	8.54E-06
0.78	Silty sand	16,000	0.645	9.41E-06
1.01	Silty sand	16,000	0.538	7.84E-06
1.24	Silty sand	16,000	0.430	6.27E-06
1.48	Silty sand	16,000	0.323	4.70E-06
1.71	Silty sand	16,000	0.215	3.14E-06
1.94	Silty sand	16,000	0.108	1.57E-06
2.18	Silty sand	16,000	0.000	0.00E+00
Sum				5.12E-05



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	4.38	
② Schmertmann's theory	5.09	
Maximum settlement	5.09	

▶ Bearing Capacity : Box Culvert Wing wall (Reverse T Type) (H=1.5m)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	1.50	(m)	■ width B	:	1.00	(m)
■ Penetration depth D_f	:	1.50	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 835 + 227 + 54 = 1{,}115 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}115 / 3.0 = 371.82 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}115 / 2.0 = 557.73 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 1{,}092 + 329 + 34 = 1{,}455 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 1{,}455 / 3.0 = 484.90 \quad (kN/m^2)$$

seismic condition

$$q_a = 1{,}455 / 2.0 = 727.35 \quad (kN/m^2)$$

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	600 400 200	~ ~ ~	1,000 700 600	700 500 300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	400 200 100	~ ~ ~	600 400 300	400 300 150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	300 200 100	~ ~ ~	500 400 300	300 250 150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	300 100 50	~ ~ ~	600 300 100	400 200 50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard medium to stiff Soft	200 100 50	~ ~ ~	400 300 100	300 150 50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 1.00 (m)
- Additional Capacity : 41.67 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 41.67 = 291.67 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	371.82	484.90	291.67	291.67	40.12	OK
Seismic	557.73	727.35		291.67	77.60	OK

▶ Settlement : Box Culvert Wing wall (Reverse T Type) (H=1.5m)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	1.00	(m)	■ Length L	:	1.50	(m)
■ load q	:	40.115	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 40.12 \times 1.00 \times \frac{1 - 0.35^2}{16000} \times 1.040 = 2.29 \text{ (mm)}$$

There, q : load(kN/m^2)

B : width(m)

E : Young's modulus(Mpa)

v : Poisson's ratio

I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.91 \times 1.54 \times 34.17 \times 4.50E\text{-}05 = 2.16 \text{ (mm)}$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

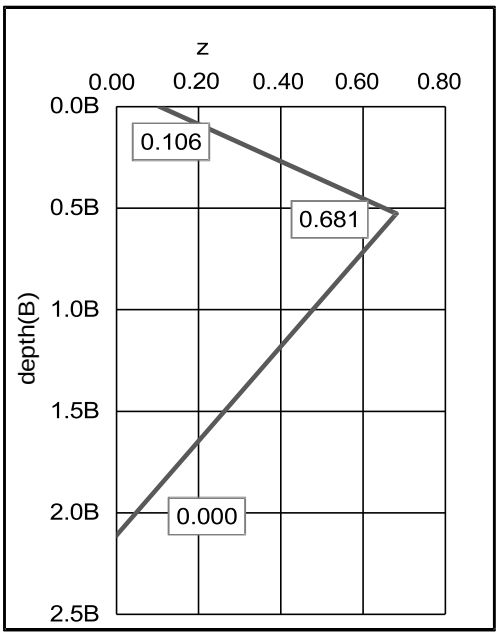
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ_{vp}')

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.106	0.00E+00
0.18	Silty sand	16,000	0.297	3.27E-06
0.35	Silty sand	16,000	0.489	5.38E-06
0.53	Silty sand	16,000	0.681	7.49E-06
0.75	Silty sand	16,000	0.584	8.25E-06
0.98	Silty sand	16,000	0.486	6.88E-06
1.21	Silty sand	16,000	0.389	5.50E-06
1.43	Silty sand	16,000	0.292	4.13E-06
1.66	Silty sand	16,000	0.195	2.75E-06
1.88	Silty sand	16,000	0.097	1.38E-06
2.11	Silty sand	16,000	0.000	0.00E+00
Sum				4.50E-05



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	2.29	
② Schmertmann's theory	2.16	
Maximum settlement	2.29	

▶ Bearing Capacity : L-Type Retaining Wall (H=3.0m)

1. Ground Condition (RBH-5)

■ Bottom layer	:	Silty sand		■ Groundwater GL.	:	0.0	(m)
■ Friction angle	:	28	(°)	■ Cohesion	:	22.0	(kN/m^2)
■ Bottom unit weight	:	18.5	(kN/m^3)	■ top unit weight	:	18.5	(kN/m^3)
■ N-value	:	11					

2. Foundation

■ Length L	:	20.00	(m)	■ width B	:	2.20	(m)
■ Penetration depth D_f	:	0.62	(m)				

3. Calculation Bearing capacity through theory

1) Terzaghi's Bearing Capacity Theory

$$q_u = \alpha \cdot c \cdot N_c + q \cdot N_q + \beta \cdot \gamma_2 \cdot B' \cdot N_\gamma$$

there, γ_1 : Unit weight of top layer

γ_2 : Unit weight of bottom layer

q : $\gamma_1 D_f$

α, β : shape factor

☞ Ultimate bearing capacity

$$q_u = \alpha \cdot c \cdot Nc + q \cdot Nq + \beta \cdot \gamma \cdot B' \cdot N\gamma$$

$$= 718 + 94 + 133 = 946 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 946 / 3.0 = 315.26 \quad (kN/m^2)$$

seismic condition

$$q_a = 946 / 2.0 = 472.88 \quad (kN/m^2)$$

2) Hansen's Bearing Capacity Theory

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

there, s_c, s_r, s_q : shape factor

d_c, d_r, d_q : Penetration depth factor

i_c, i_r, i_q : load slope factor

g_c, g_r, g_q : ground slope factor

b_c, b_r, b_q : foundation slope factor

$N_c = (N_q - 1) \cdot \cot\varphi = 25.80$

$N_q = \tan^2(45+\varphi/2) \cdot e^{\pi \cdot \tan\varphi} = 14.72$

$N_r = 1.5 \cdot (N_q - 1) \cdot \tan\varphi = 10.94$

☞ Ultimate bearing capacity

$$q_u = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot g_c \cdot b_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot g_q \cdot b_q + 0.5 \cdot \gamma \cdot B' \cdot N_r \cdot s_r \cdot d_r \cdot i_r \cdot g_r \cdot b_r$$

$$= 671 + 89 + 98 = 858 \quad (kN/m^2)$$

☞ Allowable bearing capacity

ordinary condition

$$q_a = 858 / 3.0 = 286.05 \quad (kN/m^2)$$

seismic condition

$$q_a = 858 / 2.0 = 429.08 \quad (kN/m^2)$$

4. Empirical Allowable Bearing Capacity (U. S. Navy, 1982)

Type of Bearing Material	Consistency In Place	Allowable Bearing Capacity (kN/m^2)			
		Range			Recommend Value for use
Well graded mixture of fine and coarse-grained soil : glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	800	~	1,200	1,000
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact	600	~	1,000	700
	Medium to compact	400	~	700	500
	Loose	200	~	600	300
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	400	~	600	400
	Medium to compact	200	~	400	300
	Loose	100	~	300	150
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	300	~	500	300
	Medium to compact	200	~	400	250
	Loose	100	~	300	150
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	300	~	600	400
	Medium to stiff	100	~	300	200
	Soft	50	~	100	50
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard	200	~	400	300
	medium to stiff	100	~	300	150
	Soft	50	~	100	50

1. Extend footings on soft rock or on any soil to a minimum depth of 0.5m below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.
2. For footings on soft rock or on coarse-grained soil, increase allowable bearing capacity by 5 percent of the nominal values for each 0.3m below the minimum depth

- Recommended Value : 250 (kN/m^2)
- Depth to consider additional capacity : 0.12 (m)
- Additional Capacity : 5 (kN/m^2)

☞ Allowable Bearing Capacity q_a = 250.00 + 5.00 = 255.00 (kN/m^2)

5. Conclusion

Condition	Calculation capacity through theory (kN/m^2)		Empirical capacity (kN/m^2)	Conclusion Bearing Capacity (kN/m^2)	Foundation load (kN/m^2)	Judgment
	Terzaghi Theory	Hansen Theory				
Ordinary	315.26	286.05	255.00	255.00	128.43	OK
Seismic	472.88	429.08		255.00	182.73	OK

▶ Settlement : L-Type Retaining Wall (H=3.0m)

1. Ground Condition of foudation layer (RBH-5)

■ Top layer	:	Silty sand		■ Friction angle	:	28	()
■ Poisson's ratio	:	0.35		■ Cohesion	:	22.0	(kN/m^2)
■ Young's modulus	:	16	(MPa)	■ Unit weight	:	18.5	(kN/m^3)

2. Foundation

■ width B	:	2.20	(m)	■ Length L	:	20.00	(m)
■ load q	:	128.434	(kN/m^2)				

3. Calculation settlement

1) Theory of elastic settlement

$$S = q \cdot B \cdot \frac{1-v^2}{E} \cdot I_s$$

$$= 128.43 \times 2.20 \times \frac{1 - 0.35^2}{16000} \times 1.927 = 29.87 \quad (mm)$$

There,
- q : load(kN/m^2)
- B : width(m)
- E : Young's modulus(Mpa)
- v : Poisson's ratio
- I_s : Elastic settlement coefficient

2) Schmertmann's theory

$$S = C_1 \cdot C_2 \cdot (q_b - \sigma_{vo}') \cdot \Sigma (I_{zi} / E_i) \cdot \Delta z_i$$

$$= 0.98 \times 1.54 \times 122.48 \times 2.11E\text{-}04 = 38.82 \quad (mm)$$

There,

C_1	:	Penetration depth factor	I_{zi}	:	Strain factor at layer i
C_2	:	Creep factor	E_i	:	Young's modulus at layer i
σ_{vo}'	:	Effective stress at penetraion depth	Δz_i	:	Thickness of layer i
q_b	:	Pressure at foundation			

■ maximum I_{zp} , depth z_{fp} calculation

z_{f0} = 2B (L/B = 1) z_{f0} = 4B (L/B = 10)

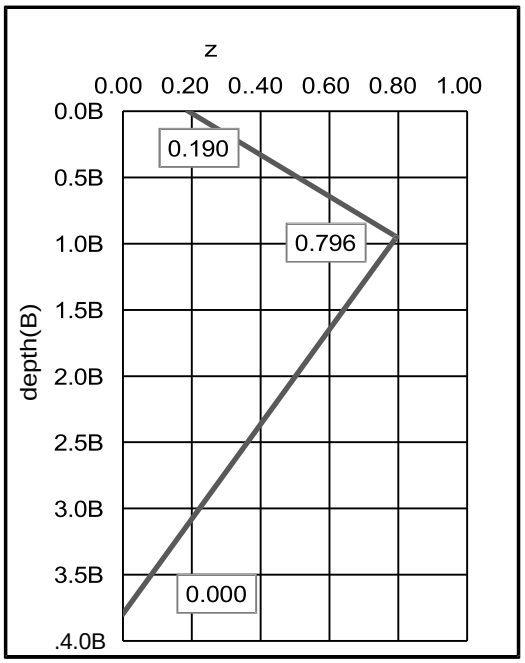
z_{f0} = {2 + 0.222(L/B - 1)}· B , (1<L/B<10)

- Strain factor for depth, I_{z0} = 0.1 + 0.0111(L/B - 1)

- maximum I_{zp} depth, z_{fp} = 0.5 + 0.0555(L/B - 1)

- maximum I_{zp} = 0.5 + 0.1F(q'/σ_{vp}')

depth z (m)	Layer	Young's modulus E_s (kPa)	Strain factor I_z	Iz/Es·Δz
0.00	Silty sand	16,000	0.190	0.00E+00
0.70	Silty sand	16,000	0.392	1.70E-05
1.39	Silty sand	16,000	0.594	2.58E-05
2.09	Silty sand	16,000	0.796	3.46E-05
2.98	Silty sand	16,000	0.682	3.81E-05
3.88	Silty sand	16,000	0.568	3.18E-05
4.77	Silty sand	16,000	0.455	2.54E-05
5.67	Silty sand	16,000	0.341	1.91E-05
6.56	Silty sand	16,000	0.227	1.27E-05
7.46	Silty sand	16,000	0.114	6.36E-06
8.35	Silty sand	16,000	0.000	0.00E+00
Sum				2.11E-04



4. Conclusion

Theory	Ground settlement (mm)	Remark
① Theory of elastic settlement	29.87	
② Schmertmann's theory	38.82	
Maximum settlement	38.82	

5.2 Anchored Retaining wall

1. The internal stability of anchor(Type-1)

1-1 Anchor structure Calculation

(1) Anchor Design Tensile Force (ton)

Td = 310.00 KN

(2) Specificcations of the permanent anchor

- PS Strand : Ø 12.7 mm x 4 EA
- Tensile stress of PS Strand (Fu) = 1,895 Mpa
- Yield stress of PS Strand(Fy) = 1,611 Mpa
- Cross section area of Single PS Strand(As) = 98.71 mm^2
- Cross Section Area of whole PS Strand(As) = 394.8 mm^2
- Ultimate Tensile Load : Tus = Fu x As = 748.2 KN (ASTM A416, Grade 270)
- Yield load : Tys = Fy x As = 636.09 KN
  0.75Tys = **477.07**
  0.60Tus = **448.93**
- Allowable tensile force : Tas = 0.6Tus = 448.9 KN > **310.0** KN -- (OK)

(3) Bond Length Calculations of Permanent Anchor

- Bond length(Lb) is applied to the largest of the following values.

① Minimum bond length = 4.0 m

② Adhesion length(Lsa) - Calculation between anchor and grout

$$Lsa = \frac{Td}{d_s \cdot \tau_b} = \frac{310.0 \times 1000}{159.6 \times 0.80} = 2428.1 \text{ mm} = 2.50 \text{ m}$$

There, ds = O.D of adhesion body = 3.14 x 4 x 1.27 = 16.0 cm
N = Number of PS Strand = 4 EA
τb = allowable bond stress = 0.80 Mpa (Fck= 25 Mpa)

③ Friction length(La) - Calculation between soil and grout

$$La = \frac{Fs \cdot Td}{\pi \cdot d_a \cdot \tau} = \frac{3.0 \times 310.0 \times 1000}{\pi \times 127.0 \times 0.25} = 9{,}323.7 \text{ mm} = 9.40 \text{ m}$$

There, Fs : safety factor = 3.0
da : Diameter of Boring hole = 127 mm
τ : Ultimate friction resistance = 0.25 Mpa (Sand N=30)

∴ **Bond Length =** 9.50 m

<Table> Ultimate friction resistance due to soil

Soil Type			Ultimate friction resistance(MPa)
Rock		Hard rock	1.5 ~ 2.5
		Soft rock	1.0 ~ 1.5
		Weathered rock	0.4 ~ 1.0
		Mud Stone	0.4 ~ 1.2
Gravel	N-value	10	0.1 ~ 0.2
		20	0.17 ~ 0.25
		30	0.25 ~ 0.35
		40	0.35 ~ 0.45
		50	0.45 ~ 0.70
Sand	N-value	10	0.1 ~ 0.14
		20	0.18 ~ 0.22
		30	0.23 ~ 0.27
		40	0.29 ~ 0.35
		50	0.30 ~ 0.40
Clay			1.0C

* Ref : Structural foundation design criteria commentary
(Japanese Society of Soil Mechanics and Foundation Engineering)

(3) Anchor of free length calculation

- Free length is applied to the largest of the following values.

① Minimum Free length = 4.0 m

② The ditance from ground surface to expected slip surface or bearing stratum = 14.00 m

∴ Free Length = TYPE-1 = 14.00 m

(4) Anchor Jacking Force (KN)

Ts1 = Td + ΔPr + ΔPs1 = 372.4 KN

① The loss for relaxation test (Δpr)

- Loss to 90% of Yield load = **5%**

ΔPr = 0.9 x Tys x 0.05 = 0.9 x 636.1 x 0.05 = 28.6 KN

② The loss for setting(Δps)

$$\Delta Ps1 = \frac{As \times Es \times \Delta L}{Lf} = \frac{394.84 \times 200000 \times 6.0}{14{,}000.0} = 33.84 \text{ KN}$$

ΔL : slip length = 6.0 mm

Es : Elastic coefficient of PS strand = 2.00E+05 Mpa

(5) Stability check of anchor Jacking Force

① Maximum initial tension force is select the smaller both 0.75Tus and 0.85Tys

0.75	x	748.2	=	636.0	kN	>	372.4	kN	--	(OK)
0.85	x	636.1	=	540.7	kN	>	372.4	kN	--	(OK)

② Maximum jacking force is select the smaller both 0.70Tus and 0.80Tys

0.7	x	748.2	=	523.8	kN	>	372.4	kN	--	(OK)
0.8	x	636.1	=	508.9	kN	>	372.4	kN	--	(OK)

③ Maximum efficient tension force is 0.9 multiple of maximum jacking force

0.9	x	508.9	=	458.0	kN	>	372.4	kN	--	(OK)

(6) Elastic enlongation

$$\delta = \frac{Ts \times Ls}{Es \times As}$$

There,

Ts : Jacking Force

Ls : Total length - bond length - marginal tension length = 24.5 - (14.00 + 0.58)

Es : Elastic modulus of Strand = 2.0 x 10^5 (MPa)

As : Cross Section Area of Strand = (3.948 cm^2)

(7) Specific of permanent Anchor

DIVISION	Relaxation Length(m)	Bond Length	Free Length	Total Length	Jacking Force	Elastic enlongation (cm)
	(m)	(m)	(m)	(m)	(KN)	
TYPE-1	1.0	9.5	14.00	24.5	372.4	6.80

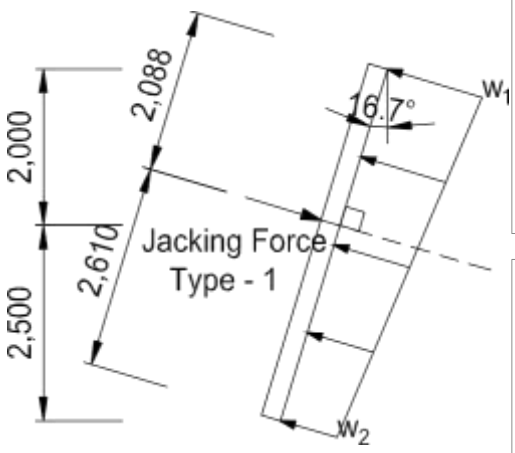
※ **Stick out Length is Head Top to Length on structure**

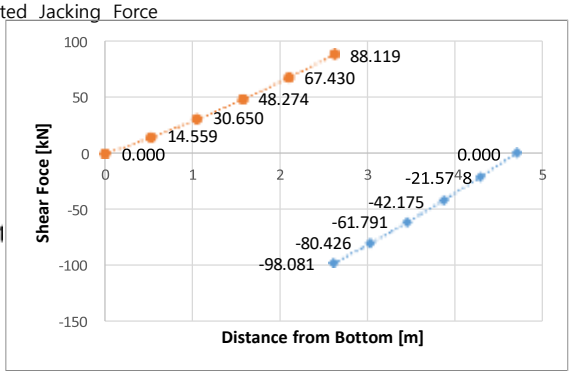
= Head Length for jacking+Plate thickness+Nut thickness

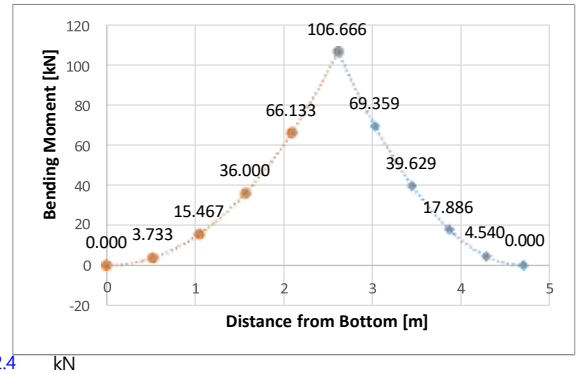
2. Section Design

(1) Member Force

- Calculate with simple beam considering concentrated Jacking Force







① Load

- Jacking Force Type - A = 372.4 kN

② Reaction

- M_o = 106.666 kN·m - V_o = 98.081 kN
- M_u = 1.2 × M_o = 1.2 × 106.666 = 127.999 kN·m (ACI 5.3.12)
- V_u = 1.2 × V_u = 1.2 × 98.081 = 117.697 kN (ACI 5.3.12)

(1) Section Design

◈ Section specification and design condition

$f_{c'}$	=	25	MPa	f_y	=	420	MPa		k_1	=	0.85			
Øf	=	0.90		Øv	=	0.75			d	=	241	mm		
B	=	1000	mm	H	=	300	mm		d'	=	60	mm		
M_u	=	127.999	kN·m	V_u	=	117.697	kN	M_o	=		106.666	kN·m		

- Check of Strength reduction factor (Φ)

a = 35.342

Because T = C , c = 35.342 / β1 = 35.342 / 0.850 = 41.579 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (240.5 - 41.579) / 41.579

= 0.0144

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

Eq. (B) substitutes for Eq. (A)

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

∴ Req. As = 1500.507 ㎟

Use As = D 19 @ 300 + D 19 @ 300 = 1893.33 ㎟ (7 ea/m)

◈ Evaluation of reinforcement

P_b	= $k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.02679					
P_{max}	= 0.75 - Pb	=	0.02009	⇒	$A_{s,max}$	=	4831.6	㎟
P_{min}	= max (1.4 / fy , 0.25 √ fc' / fy	=	0.00333	⇒	$A_{s,min}$	=	801.7	㎟
$P_{4/3req}$	= 4/3 - $A_{s.req}$ / (B·d)	=	0.00832	⇒	$A_{s,4/3req}$	=	2000.7	㎟
P_{min}	= min (P_{min}, $P_{4/3req}$)	=	0.00333	⇒	$A_{s,min}$	=	801.7	㎟
P_{use}	= A_s / (B·d)	=	0.00787	⇒	$A_{s,min}$	=	1893.3	㎟

▶ Pmin ≤ Puse ≤ Pmax ∴ O.K

◈ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 35.342 mm

ØMn = 0.9 × As × fy × (d - a/2) = 159.474 kN·m > Mu = 127.999 kN·m

∴ O.K

◈ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × = 150.313 kN > Vu = 117.697 kN

∴ Need not shear reinforcement

(2) Crack Check

◈ Calculation of stress

n = 9

$X = - nA_s/b + nA_s/b \times \sqrt{1+2bd/nA_s}$

$= - 9 \times 1,893.33 / 1000 + 9 \times 1,893.33 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 240.5 / (9 \times 1,893.33)]}$

= 75.083 mm

$f_c = 2 \times M_o / [B \times X \times (d - X/3)]$

$= 2.0 \times 106.666 / [1000 \times 75.083 \times (240.5 - 75.083 / 3)] \times 10^6$

= 13.186 MPa

$f_s = M_o / [A_s \times (d - X/3)]$

$= 106.666 / [1893.333 \times (240.5 - 75.083 / 3)] \times 10^6$

= 261.461 MPa

fst = fs x (H - d' - X) / (d - X) = 2 / 3 x 420 = 280.00 MPa

◈ Maximum center space of reinforcement

C_C = 59.50 - 19.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 261.46) - 2.5 x 50.00 = 281.94 ㎜

300 x (280 / fs) = 300 x (280 / 261.46) = 321.27 ㎜

Sa = 281.94 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (281.94 mm) ∴ O.K

(3) Distribution Reinforcement Check

- $A_{s,\ min}$ = (0.0018 × 420 / fy) × B × H = 0.002 × 127.999 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

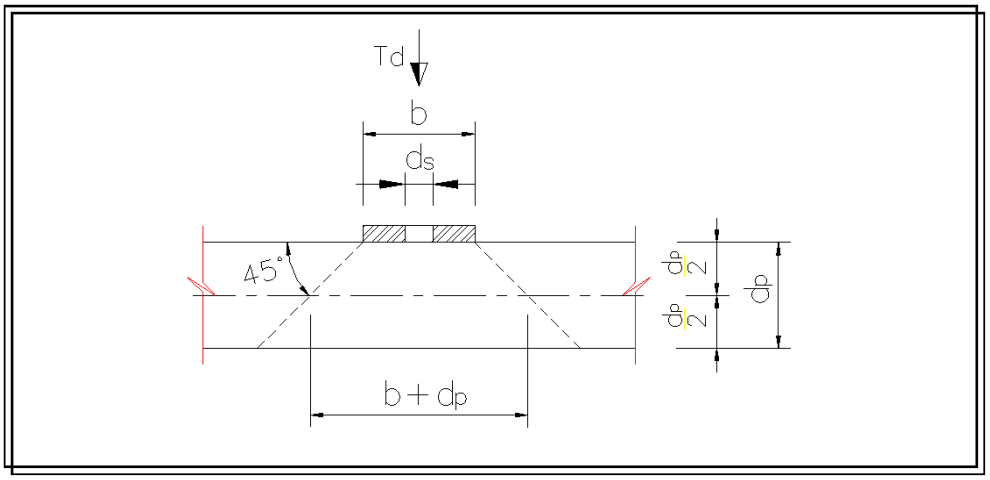
- Used As :	Tension side	D	13@	300	=	430.0	㎟			
	Compression side	D	13@	300	=	430.0	㎟			
				∑	=	860.0	㎟	>	540.0 ㎟	∴ O.K

- Bar spacing : 300 ㎜ < 450 ㎜ ∴ O.K

2. Punching failure and bearing strength Review

1) Design Conditions

- Design Strength of Concrete : f'c = 25 MPa
- Yield Strength of Steel : fy = 420 MPa
- Shear Strength Reduction Factor : φv = 0.75
- Bearing Strength Reduction Factor : φb = 0.85



2) Punching failure Review

1) Width of anchor plate (b1) = 250.00 mm
(b2) = 250.00 mm
2) Effective height of beam (d) = 240.50 mm
3) Tensile force of anchor (Td) = 372.4 kN

- Bp = 2 X (491 + 491) = 1962 mm

- βc = 1.0 (Aspect ratio of long ans short sides)

▶ Concrete Shear Strength Review

- Su ≤ Φv Sc ≤ Φv X 0.33 X √f'c X Bp X d

Φv Sc = 0.75 X 0.17 X (1 + 2 /βc) X √f'c X Bp X d

= 0.75 X 0.17 X (1 + 2 / 1.0) X √ 25 X 1962 X 241 = 902 kN

Φv Sc ≤ 0.75 X 0.33 X √f'c X Bp X d

= 0.75 X 0.33 X √ 25 X 1962 X 241 = 584 kN

∴ Shear strength of ConcretΦv Sc = 584 kN

∴ Su = 372.4 kN < Sc = 584 kN **Shear reinforcement is not necessary!!**

3) Bearing strength Rewivw

▶ Bearing strength of concrete on anchor plate

- $\varphi P_{nb} = \varphi(0.85 \cdot f_{ck} \cdot A_1) = 0.85 \times (0.85 \times 25 \times 41118)$

$= 742.685$ kN > Td = 372.4 kN ∴ **O.K**

From above, $A_1 = b_1 \times b_2 - \pi \times 165.0^2 / 4 = 41118$ ㎟

1. The internal stability of anchor(Type-2)

1-1 Anchor structure Calculation

(1) Anchor Design Tensile Force (ton)

Td = 320.00 KN

(2) Specificcations of the permanent anchor

- PS Strand : Ø 12.7 mm x 4 EA
- Tensile stress of PS Strand (Fu) = 1,895 Mpa
- Yield stress of PS Strand(Fy) = 1,611 Mpa
- Cross section area of Single PS Strand(As) = 98.71 mm^2
- Cross Section Area of whole PS Strand(As) = 394.8 mm^2
- Ultimate Tensile Load : Tus = Fu x As = 748.2 KN (ASTM A416, Grade 270)
- Yield load : Tys = Fy x As = 636.09 KN
  - 0.75Tys = **477.07**
  - 0.60Tus = **448.93**
- Allowable tensile force : Tas = 0.6Tus = 448.9 KN > **320.0** KN -- (OK)

(3) Bond Length Calculations of Permanent Anchor

- Bond length(Lb) is applied to the largest of the following values.

① Minimum bond length = 4.0 m

② Adhesion length(Lsa) - Calculation between anchor and grout

$$Lsa = \frac{Td}{d_s \cdot \tau_b} = \frac{320.0 \times 1000}{159.6 \times 0.80} = 2506.4 \text{ mm} = 2.60 \text{ m}$$

There, ds = O.D of adhesion body = 3.14 x 4 x 1.27 = 16.0 cm

N = Number of PS Strand = 4 EA

τb = allowable bond stress = 0.80 Mpa (Fck= 25 Mpa)

③ Friction length(La) - Calculation between soil and grout

$$La = \frac{Fs \cdot Td}{\pi \cdot d_a \cdot \tau} = \frac{3.0 \times 320.0 \times 1000}{\pi \times 127.0 \times 0.25} = 9{,}624.5 \text{ mm} = 9.70 \text{ m}$$

There, Fs : safety factor = 3.0

da : Diameter of Boring hole = 127 mm

τ : Ultimate friction resistance = 0.25 Mpa (Sand N=30)

∴ **Bond Length =** 10.00 m

<Table> Ultimate friction resistance due to soil

Soil Type			Ultimate friction resistance(MPa)
Rock		Hard rock	1.5 ~ 2.5
		Soft rock	1.0 ~ 1.5
		Weathered rock	0.4 ~ 1.0
		Mud Stone	0.4 ~ 1.2
Gravel	N-value	10	0.1 ~ 0.2
		20	0.17 ~ 0.25
		30	0.25 ~ 0.35
		40	0.35 ~ 0.45
		50	0.45 ~ 0.70
Sand	N-value	10	0.1 ~ 0.14
		20	0.18 ~ 0.22
		30	0.23 ~ 0.27
		40	0.29 ~ 0.35
		50	0.30 ~ 0.40
Clay			1.0C

* Ref : Structural foundation design criteria commentary

(Japanese Society of Soil Mechanics and Foundation Engineering)

(3) Anchor of free length calculation

- Free length is applied to the largest of the following values.

① Minimum Free length = 4.0 m

② The ditance from ground surface to expected slip surface or bearing stratum = 14.00 m

∴ Free Length = TYPE-1 = 12.00 m

(4) Anchor Jacking Force (KN)

Ts1 = Td + ΔPr + ΔPs1 = 388.1 KN

① The loss for relaxation test (Δpr)

- Loss to 90% of Yield load = 5%

ΔPr = 0.9 x Tys x 0.05 = 0.9 x 636.1 x 0.05 = 28.6 KN

② The loss for setting(Δps)

$$\Delta Ps1 = \frac{As \times Es \times \Delta L}{Lf} = \frac{394.84 \times 200000 \times 6.0}{12{,}000.0} = 39.48 \text{ KN}$$

ΔL : slip length = 6.0 mm

Es : Elastic coefficient of PS strand = 2.00E+05 Mpa

(5) Stability check of anchor Jacking Force

① Maximum initial tension force is select the smaller both 0.75Tus and 0.85Tys

0.75	x	0.0	=	0.0	kN	>	388.1	kN	--	(OK)
0.85	x	636.1	=	540.7	kN	>	388.1	kN	--	(OK)

② Maximum jacking force is select the smaller both 0.70Tus and 0.80Tys

0.7	x	0.0	=	0.0	kN	>	388.1	kN	--	(OK)
0.8	x	636.1	=	508.9	kN	>	388.1	kN	--	(OK)

③ Maximum efficient tension force is 0.9 multiple of maximum jacking force

0.9	x	0.0	=	0.0	kN	>	388.1	kN	--	(OK)

(6) Elastic enlongation

$$\delta = \frac{Ts \times Ls}{Es \times As}$$

There,

Ts : Jacking Force

Ls : Total length - bond length - marginal tension length = 23.0 - (12.00 + 0.58)

Es : Elastic modulus of Strand = 2.0 x 10^5 (MPa)

As : Cross Section Area of Strand = (3.948 cm^2)

(7) Specific of permanent Anchor

DIVISION	Relaxation Length(m)	Bond Length	Free Length	Total Length	Jacking Force	Elastic enlongation (cm)
	(m)	(m)	(m)	(m)	(KN)	
TYPE-1	1.0	10.0	12.00	23.0	388.1	6.10

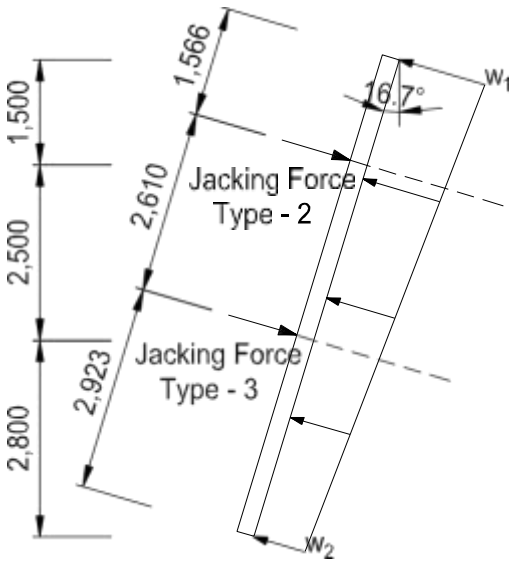
※ Stick out Length is Head Top to Length on structure

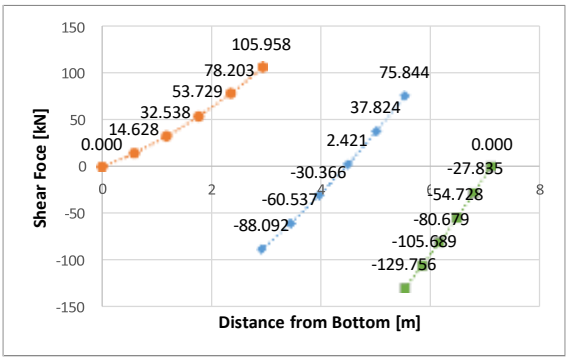
= Head Length for jacking+Plate thickness+Nut thickness

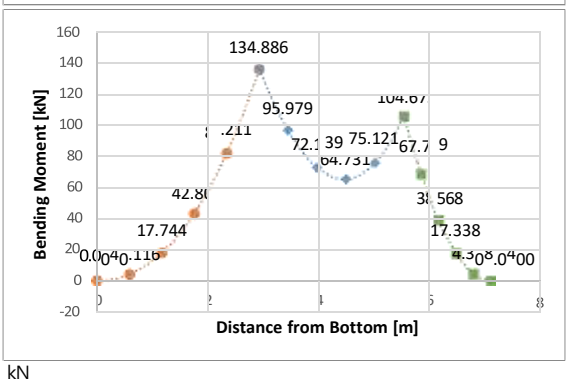
2. Section Design

(1) Member Force

- Calculate with simple beam considering concentrated Jacking Force







① Load

- Jacking Force Type - A = 388.1 kN

② Reaction

- W_1 = 38.653 kN/m - W_2 = 19.327 kN/m

③ Member Force

- M_o = 104.675 kN·m - V_o = 129.756 kN
- M_u = 1.2 × M_o = 1.2 × 104.675 = 125.610 kN·m (ACI 5.3.12)
- V_u = 1.2 × V_u = 1.2 × 129.756 = 155.707 kN (ACI 5.3.12)

(1) Section Design

◈ Section specification and design condition

$f_{c'}$	=	25	MPa	f_y	=	420	MPa		k_1	=	0.85				
Øf	=	0.90		Øv	=	0.75			d	=	341	mm			
B	=	1000	mm	H	=	400	mm		d'	=	60	mm			
M_u	=	125.610	kN·m	V_u	=	155.707	kN	M_o	=	104.675	kN·m				

- Check of Strength reduction factor (Φ)

a = 35.342

Because T = C , c = 35.342 / β1 = 35.342 / 0.850 = 41.579 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (340.5 - 41.579) / 41.579

= 0.0216

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

Eq. (B) substitutes for Eq. (A)

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

∴ Req. As = 1005.251 ㎟

Use As = D 19 @ 300 + D 19 @ 300 = 1893.33 ㎟ (7 ea/m)

◈ Evaluation of reinforcement

P_b	= $k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.02679					
P_{max}	= 0.75 - Pb	=	0.02009	⇒	$A_{s,max}$	=	6840.6	㎟
P_{min}	= max (1.4 / fy , 0.25 √ fc' / fy	=	0.00333	⇒	$A_{s,min}$	=	1135.0	㎟
$P_{4/3req}$	= 4/3 - $A_{s.req}$ / (B·d)	=	0.00394	⇒	$A_{s,4/3req}$	=	1340.3	㎟
P_{min}	= min (P_{min}, $P_{4/3req}$)	=	0.00333	⇒	$A_{s,min}$	=	1135.0	㎟
P_{use}	= A_s / (B·d)	=	0.00556	⇒	$A_{s,min}$	=	1893.3	㎟

▶ Pmin ≤ Puse ≤ Pmax ∴ O.K

◈ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 35.342 mm

ØMn = 0.9 × As × fy × (d - a/2) = 231.042 kN·m > Mu = 125.610 kN·m

∴ O.K

◈ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × = 212.813 kN > Vu = 155.707 kN

∴ Need not shear reinforcement

(2) Crack Check

◈ Calculation of stress

n = 9

$X = - nA_s/b + nA_s/b \times \sqrt{1+2bd/nA_s}$

$= - 9 \times 1,893.33 / 1000 + 9 \times 1,893.33 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 340.5 / (9 \times 1,893.33)]}$

= 92.022 mm

$f_c = 2 \times M_o / [B \times X \times (d - X/3)]$

$= 2.0 \times 104.675 / [1000 \times 92.022 \times (340.5 - 92.022 / 3)] \times 10^6$

= 7.343 MPa

$f_s = M_o / [A_s \times (d - X/3)]$

$= 104.675 / [1893.333 \times (340.5 - 92.022 / 3)] \times 10^6$

= 178.442 MPa

fst = fs x (H - d' - X) / (d - X) = 2 / 3 x 420 = 280.00 MPa

◈ Maximum center space of reinforcement

C_C = 59.50 - 19.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 178.44) - 2.5 x 50.00 = 471.27 ㎜

300 x (280 / fs) = 300 x (280 / 178.44) = 470.74 ㎜

Sa = 470.74 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (470.74 mm) ∴ O.K

(3) Distribution Reinforcement Check

- $A_{s,\ min}$ = (0.0018 × 420 / fy) × B × H = 0.002 × 125.61 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

- Used As : Tension side D 13@ 300 = 430.0 ㎟

Compression side D 13@ 300 = 430.0 ㎟

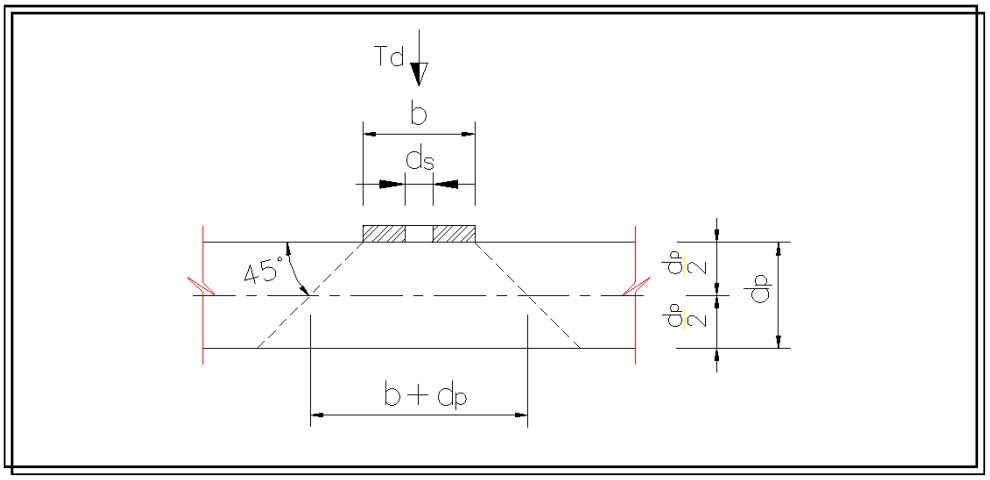
∑ = 860.0 ㎟ > 720.0 ㎟ ∴ O.K

- Bar spacing : 300 ㎜ < 450 ㎜ ∴ O.K

2. Punching failure and bearing strength Review

1) Design Conditions

- Design Strength of Concrete	: f'c	=	25	MPa
- Yield Strength of Steel	: fy	=	420	MPa
- Shear Strength Reduction Factor	: φv	=	0.75	
- Bearing Strength Reduction Factor	: φb	=	0.85	



2) Punching failure Review

1) Width of anchor plate	(b1)	=	250.00	mm
	(b2)	=	250.00	mm
2) Effective height of beam	(d)	=	340.50	mm
3) Tensile force of anchor	(Td)	=	388.1	kN

- Bp = 2 X (591 + 591) = 2362 mm

- βc = 1.0 (Aspect ratio of long ans short sides)

▶ Concrete Shear Strength Review

- Su ≤ Φv Sc ≤ Φv X 0.33 X √f'c X Bp X d

Φv Sc = 0.75 X 0.17 X (1 + 2 /βc) X √f'c X Bp X d

= 0.75 X 0.17 X (1 + 2 / 1.0) X √ 25 X 2362 X 341 = 1538 kN

Φv Sc ≤ 0.75 X 0.33 X √f'c X Bp X d

= 0.75 X 0.33 X √ 25 X 2362 X 341 = 995 kN

∴ Shear strength of ConcretΦv Sc = 995 kN

∴ Su = 388.1 kN < Sc = 995 kN **Shear reinforcement is not necessary!!**

3) Bearing strength Rewivw

▶ Bearing strength of concrete on anchor plate

- $\varphi P_{nb} = \varphi(0.85 \cdot f_{ck} \cdot A_1) = 0.85 \times (0.85 \times 25 \times 41118)$

$= 742.685$ kN > Td = 388.1 kN ∴ **O.K**

From above, $A_1 = b_1 \times b_2 - \pi \times 165.0^2 / 4 = 41118$ ㎟

1. The internal stability of anchor(Type-3)

1-1 Anchor structure Calculation

(1) Anchor Design Tensile Force (ton)

Td = 330.00 KN

(2) Specificcations of the permanent anchor

- PS Strand : Ø 12.7 mm x 4 EA
- Tensile stress of PS Strand (Fu) = 1,895 Mpa
- Yield stress of PS Strand(Fy) = 1,611 Mpa
- Cross section area of Single PS Strand(As) = 98.71 mm^2
- Cross Section Area of whole PS Strand(As) = 394.8 mm^2
- Ultimate Tensile Load : Tus = Fu x As = 748.2 KN (ASTM A416, Grade 270)
- Yield load : Tys = Fy x As = 636.09 KN
  - 0.75Tys = **477.07**
  - 0.60Tus = **448.93**
- Allowable tensile force : Tas = 0.6Tus = 448.9 KN > **330.0** KN -- (OK)

(3) Bond Length Calculations of Permanent Anchor

- Bond length(Lb) is applied to the largest of the following values.

① Minimum bond length = 4.0 m

② Adhesion length(Lsa) - Calculation between anchor and grout

$$Lsa = \frac{Td}{d_s \cdot \tau_b} = \frac{330.0 \times 1000}{159.6 \times 0.80} = 2584.7 \text{ mm} = 2.60 \text{ m}$$

There, ds = O.D of adhesion body = 3.14 x 4 x 1.27 = 16.0 cm

N = Number of PS Strand = 4 EA

τb = allowable bond stress = 0.80 Mpa (Fck= 25 Mpa)

③ Friction length(La) - Calculation between soil and grout

$$La = \frac{Fs \cdot Td}{\pi \cdot d_a \cdot \tau} = \frac{3.0 \times 330.0 \times 1000}{\pi \times 127.0 \times 0.25} = 9{,}925.3 \text{ mm} = 10.00 \text{ m}$$

There, Fs : safety factor = 3.0

da : Diameter of Boring hole = 127 mm

τ : Ultimate friction resistance = 0.25 Mpa (Sand N=30)

∴ **Bond Length =** 10.00 m

<Table> Ultimate friction resistance due to soil

Soil Type			Ultimate friction resistance(MPa)
Rock		Hard rock	1.5 ~ 2.5
		Soft rock	1.0 ~ 1.5
		Weathered rock	0.4 ~ 1.0
		Mud Stone	0.4 ~ 1.2
Gravel	N-value	10	0.1 ~ 0.2
		20	0.17 ~ 0.25
		30	0.25 ~ 0.35
		40	0.35 ~ 0.45
		50	0.45 ~ 0.70
Sand	N-value	10	0.1 ~ 0.14
		20	0.18 ~ 0.22
		30	0.23 ~ 0.27
		40	0.29 ~ 0.35
		50	0.30 ~ 0.40
Clay			1.0C

* Ref : Structural foundation design criteria commentary

(Japanese Society of Soil Mechanics and Foundation Engineering)

(3) Anchor of free length calculation

- Free length is applied to the largest of the following values.

① Minimum Free length = 4.0 m

② The ditance from ground surface to expected slip surface or bearing stratum = 14.00 m

∴ Free Length = TYPE-1 = 9.00 m

(4) Anchor Jacking Force (KN)

Ts1 = Td + ΔPr + ΔPs1 = 411.2 KN

① The loss for relaxation test (Δpr)

- Loss to 90% of Yield load = **5%**

ΔPr = 0.9 x Tys x 0.05 = 0.9 x 636.1 x 0.05 = 28.6 KN

② The loss for setting(Δps)

$$\Delta Ps1 = \frac{As \times Es \times \Delta L}{Lf} = \frac{394.84 \times 200000 \times 6.0}{9{,}000.0} = 52.65 \text{ KN}$$

ΔL : slip length = 6.0 mm

Es : Elastic coefficient of PS strand = 2.00E+05 Mpa

(5) Stability check of anchor Jacking Force

① Maximum initial tension force is select the smaller both 0.75Tus and 0.85Tys

0.75	x	0.0	=	0.0	kN	>	411.2	kN	--	(OK)
0.85	x	636.1	=	540.7	kN	>	411.2	kN	--	(OK)

② Maximum jacking force is select the smaller both 0.70Tus and 0.80Tys

0.7	x	0.0	=	0.0	kN	>	411.2	kN	--	(OK)
0.8	x	636.1	=	508.9	kN	>	411.2	kN	--	(OK)

③ Maximum efficient tension force is 0.9 multiple of maximum jacking force

0.9	x	0.0	=	0.0	kN	>	411.2	kN	--	(OK)

(6) Elastic enlongation

$$\delta = \frac{Ts \times Ls}{Es \times As}$$

There,

Ts : Jacking Force

Ls : Total length - bond length - marginal tension length = 20.0 - (9.00 + 0.58)

Es : Elastic modulus of Strand = 2.0 x 10^5 (MPa)

As : Cross Section Area of Strand = (3.948 cm^2)

(7) Specific of permanent Anchor

DIVISION	Relaxation Length(m)	Bond Length	Free Length	Total Length	Jacking Force	Elastic enlongation (cm)
	(m)	(m)	(m)	(m)	(KN)	
TYPE-1	1.0	10.0	9.00	20.0	411.2	4.91

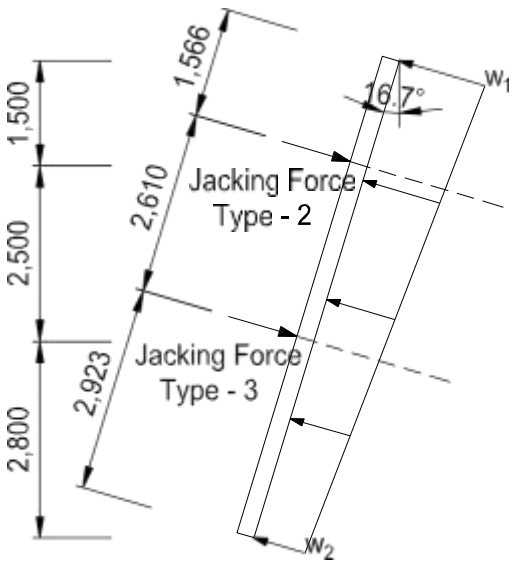
※ **Stick out Length is Head Top to Length on structure**

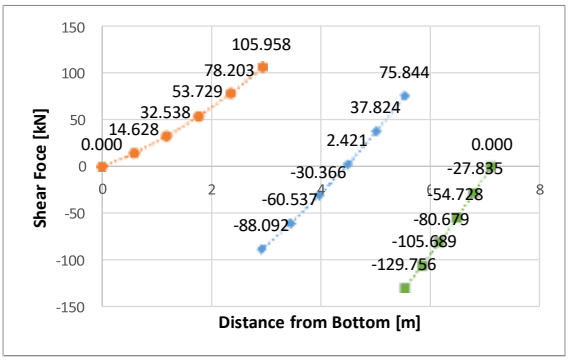
= Head Length for jacking+Plate thickness+Nut thickness

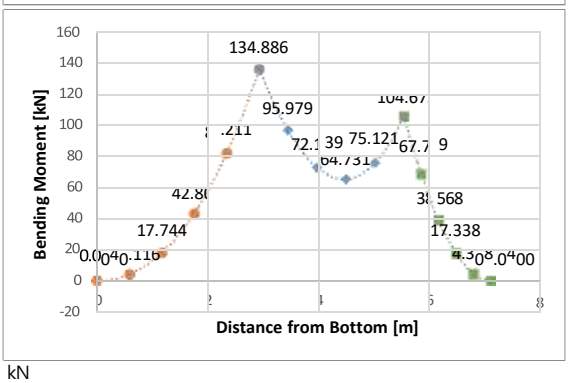
2. Section Design

(1) Member Force

- Calculate with simple beam considering concentrated Jacking Force







① Load

- Jacking Force Type - A = 411.2 kN

② Reaction

- W_1 = 38.653 kN/m - W_2 = 19.327 kN/m

③ Member Force

- M_o = 134.886 kN·m - V_o = 105.958 kN
- M_u = 1.2 × M_o = 1.2 × 134.886 = 161.863 kN·m (ACI 5.3.12)
- V_u = 1.2 × V_u = 1.2 × 105.958 = 127.150 kN (ACI 5.3.12)

(1) Section Design

◈ Section specification and design condition

$f_{c'}$	=	25	MPa	f_y	=	420	MPa		k_1	=	0.85			
Øf	=	0.90		Øv	=	0.75			d	=	341	mm		
B	=	1000	mm	H	=	400	mm		d'	=	60	mm		
M_u	=	161.863	kN·m	V_u	=	127.150	kN	M_o	=		134.886	kN·m		

- Check of Strength reduction factor (Φ)

a = 35.342

Because T = C , c = 35.342 / β1 = 35.342 / 0.850 = 41.579 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (340.5 - 41.579) / 41.579

= 0.0216

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

Eq. (B) substitutes for Eq. (A)

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

∴ Req. As = 1307.181 ㎟

Use As = D 19 @ 300 + D 19 @ 300 = 1893.33 ㎟ (7 ea/m)

◈ Evaluation of reinforcement

P_b	= $k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.02679					
P_{max}	= 0.75 - Pb	=	0.02009	⇒	$A_{s,max}$	=	6840.6	㎟
P_{min}	= max (1.4 / fy , 0.25 √ fc' / fy	=	0.00333	⇒	$A_{s,min}$	=	1135.0	㎟
$P_{4/3req}$	= 4/3 - $A_{s.req}$ / (B·d)	=	0.00512	⇒	$A_{s,4/3req}$	=	1742.9	㎟
P_{min}	= min (P_{min}, $P_{4/3req}$)	=	0.00333	⇒	$A_{s,min}$	=	1135.0	㎟
P_{use}	= A_s / (B·d)	=	0.00556	⇒	$A_{s,min}$	=	1893.3	㎟

▶ Pmin ≤ Puse ≤ Pmax ∴ O.K

◈ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 35.342 mm

ØMn = 0.9 × As × fy × (d - a/2) = 231.042 kN·m > Mu = 161.863 kN·m

∴ O.K

◈ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × = 212.813 kN > Vu = 127.150 kN

∴ Need not shear reinforcement

(2) Crack Check

◈ Calculation of stress

n = 9

$X = - nA_s/b + nA_s/b \times \sqrt{1+2bd/nA_s}$

$= - 9 \times 1,893.33 / 1000 + 9 \times 1,893.33 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 340.5 / (9 \times 1,893.33)]}$

= 92.022 mm

$f_c = 2 \times M_o / [B \times X \times (d - X/3)]$

$= 2.0 \times 134.886 / [1000 \times 92.022 \times (340.5 - 92.022 / 3)] \times 10^6$

= 9.462 MPa

$f_s = M_o / [A_s \times (d - X/3)]$

$= 134.886 / [1893.333 \times (340.5 - 92.022 / 3)] \times 10^6$

= 229.944 MPa

fst = fs x (H - d' - X) / (d - X) = 2 / 3 x 420 = 280.00 MPa

◈ Maximum center space of reinforcement

C_C = 59.50 - 19.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 229.94) - 2.5 x 50.00 = 337.72 ㎜

300 x (280 / fs) = 300 x (280 / 229.94) = 365.31 ㎜

Sa = 337.72 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (337.72 mm) ∴ O.K

(3) Distribution Reinforcement Check

- $A_{s,\ min}$ = (0.0018 × 420 / fy) × B × H = 0.002 × 161.863 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

- Used As : Tension side D 13@ 300 = 430.0 ㎟

Compression side D 13@ 300 = 430.0 ㎟

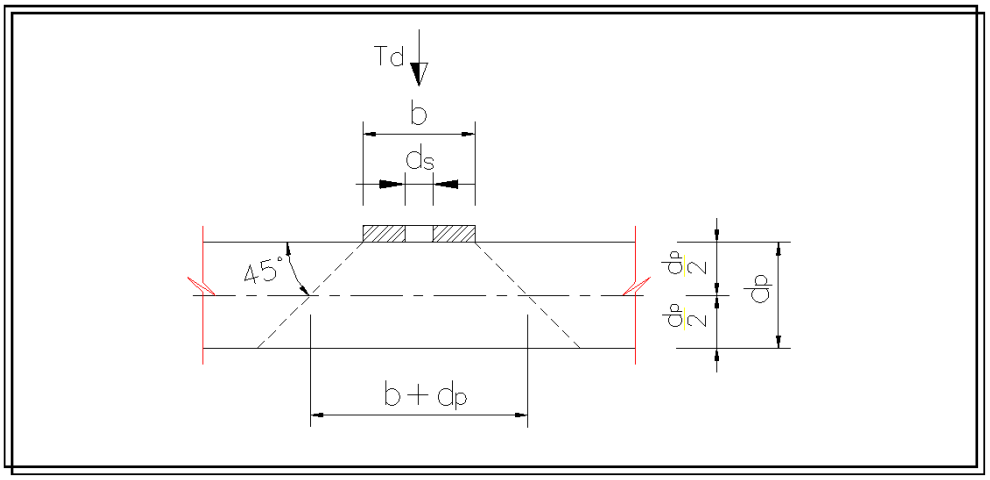
∑ = 860.0 ㎟ > 720.0 ㎟ ∴ O.K

- Bar spacing : 300 ㎜ < 450 ㎜ ∴ O.K

2. Punching failure and bearing strength Review

1) Design Conditions

- Design Strength of Concrete : f'c = 25 MPa
- Yield Strength of Steel : fy = 420 MPa
- Shear Strength Reduction Factor : φv = 0.75
- Bearing Strength Reduction Factor : φb = 0.85



2) Punching failure Review

1) Width of anchor plate (b1) = 250.00 mm
(b2) = 250.00 mm
2) Effective height of beam (d) = 340.50 mm
3) Tensile force of anchor (Td) = 411.2 kN

- Bp = 2 X (591 + 591) = 2362 mm

- βc = 1.0 (Aspect ratio of long ans short sides)

▶ Concrete Shear Strength Review

- Su ≤ Φv Sc ≤ Φv X 0.33 X √f'c X Bp X d

Φv Sc = 0.75 X 0.17 X (1 + 2 /βc) X √f'c X Bp X d

= 0.75 X 0.17 X (1 + 2 / 1.0) X √ 25 X 2362 X 341 = 1538 kN

Φv Sc ≤ 0.75 X 0.33 X √f'c X Bp X d

= 0.75 X 0.33 X √ 25 X 2362 X 341 = 995 kN

∴ Shear strength of ConcretΦv Sc = 995 kN

∴ Su = 411.2 kN < Sc = 995 kN **Shear reinforcement is not necessary!!**

3) Bearing strength Rewivw

▶ Bearing strength of concrete on anchor plate

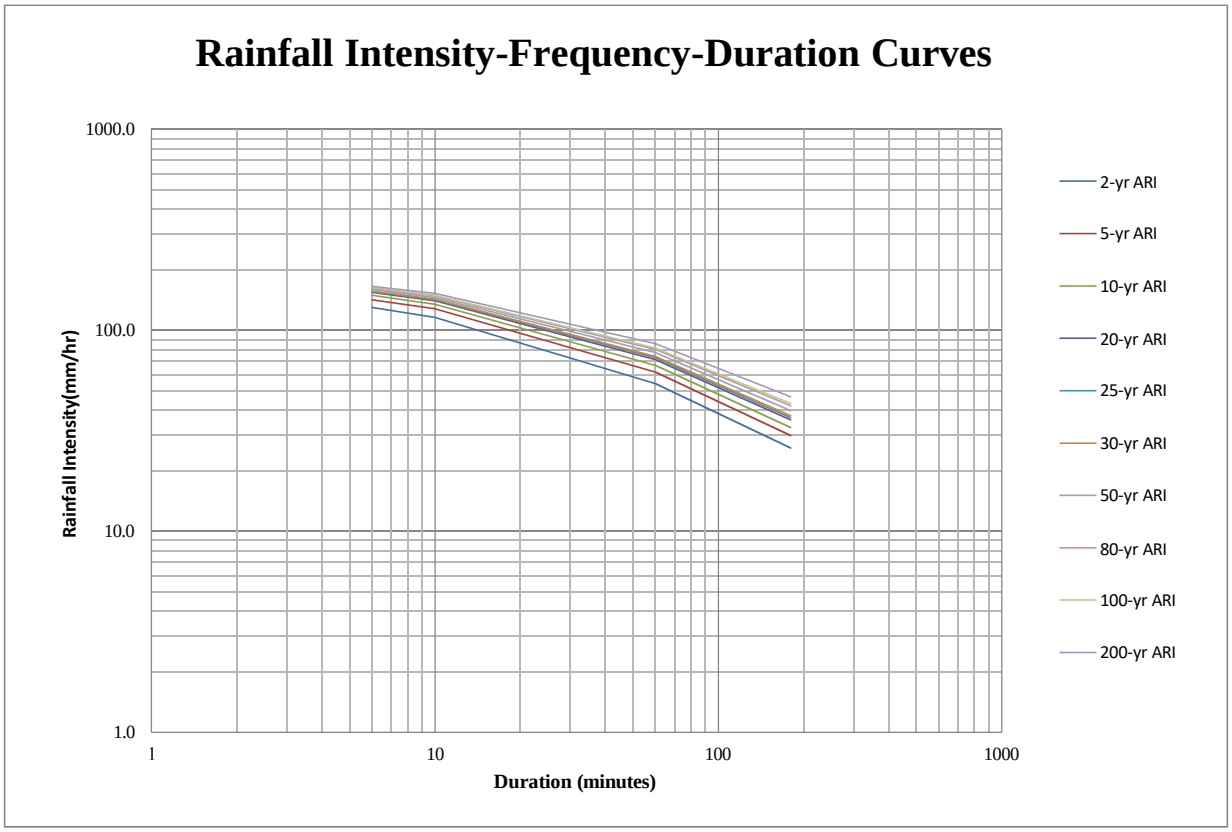
- $\varphi Pnb = \varphi(0.85 \cdot fck \cdot A1) = 0.85 \times (0.85 \times 25 \times 41118)$

$= 742.685$ kN > Td = 411.2 kN ∴ **O.K**

From above, $A1 = b1 \times b2 - \pi \times 165.0^2 / 4 = 41118$ ㎟

6. HYDRAULIC CALCULATION

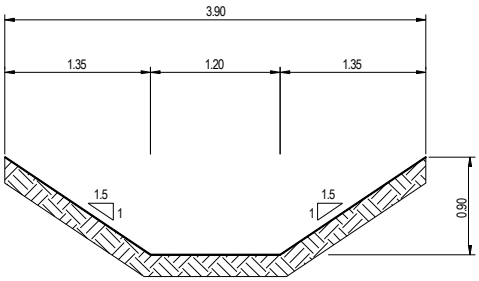
6.1 Earth Drain Ditch Calculation



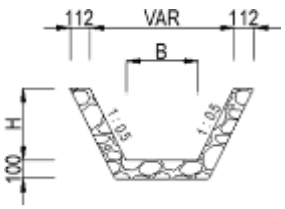
Duration (mins) \ Rainfall Intensity (mm/hr)	2-yr ARI	5-yr ARI	10-yr ARI	20-yr ARI	25-yr ARI	30-yr ARI	50-yr ARI	80-yr ARI	100-yr ARI	200-yr ARI
6	129.7	**142.1**	149.2	154.2	**155.8**	156.8	159.3	161.9	162.9	165.6
10	115.9	127.9	134.9	140.1	141.8	142.9	145.7	148.4	149.5	152.6
60	54.3	62.0	66.9	71.5	73.0	74.2	77.4	80.3	81.7	85.9
180	25.9	30.0	32.9	35.8	36.8	37.6	39.9	42.1	43.2	46.6
360	14.5	16.9	18.7	20.7	21.4	22.0	23.8	25.7	26.7	30.7
540	10.5	12.7	14.7	17.0	17.8	18.4	20.6	22.8	23.9	28.1
720	8.2	10.4	12.3	14.6	15.4	16.2	18.4	20.7	21.9	26.3
900	6.8	8.9	10.7	13.0	13.8	14.5	16.8	19.1	20.4	24.8
1080	5.8	7.8	9.6	11.8	12.6	13.3	15.5	17.9	19.2	23.6
1440	4.6	6.3	8.0	10.0	10.8	11.5	13.7	16.0	17.3	21.7

A. Calculation of Earth Drain & U-type Ditch

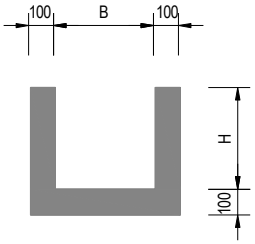
1. EARTH DRAIN



U-type ditch(Stone pitching)



U-type ditch(Concret)



1) Flow rate in Earth drain

① Earth Drain Q= A X V = A × (1/n) × $R^{2/3}$ × $I^{1/2}$ (n=0.020)

TYPE	Depth of Flow (0.8H)	Width(D) (m)	Area of the flow (m^2)	Wetted perimeter (m)	Hydraulic Radius (m)	$R^{2/3}$	Q m^3/sec
TYPE-1(H=0.9M)	0.72	3.36	1.6416	3.796	0.4325	0.5719	46.9416 × $I^{1/2}$

① Utype- ditch Q= A X V = A × (1/n) × $R^{2/3}$ × $I^{1/2}$ ·Stonenpitching 0.025

·Concrete 0.012

TYPE	Depth of Flow (0.8H)	Width(D) (m)	Area of the flow (m^2)	Wetted perimeter (m)	Hydraulic Radius (m)	$R^{2/3}$	Q m^3/sec
TYPE-4(H=0.40M)Stone	0.32	0.72	0.1792	1.116	0.1606	0.2955	2.1181 × $I^{1/2}$
TYPE-6(H=0.60M) Con	0.48	0.600	0.2880	1.560	0.1846	0.3242	7.7808 × $I^{1/2}$

2) The Rational Method formula for flow estimation Q(m^3/sec)

① Plain farming area

$Qd_1 = 0.278 \times C_1 \times I_1 \times A_1$

where, C_1, C_2 : Run-off Coefficient

I_1, I_2 : Rainfall intensity(mm/hr)

A_1, A_2 : Catchment Area(km^2)

② Paved surface

$Qd_2 = 0.278 \times C_2 \times I_2 \times A_2$

C_1 : 0.5 C_2 : 0.9

I_1(5yr) : 142.1 I_2(5yr) : 142.1

③ Peak Catchment Discharge $Qd = Qd_1 + Qd_2$

3) Determination of Cross section

Compare Flow rate with the Peak Catchment Discharge according to Runoff Coefficient and Catchment Area

※ **Q > Qd (OK)**

Result of Side Earth Drain Analyses

No.	Station		Direction	Catchment Area (km²) (A1,A2)	Run-off Coefficient (C_1,C_2)	Slope (%)	Flow rate according to slope Q	Peak Catchment Discharge (m³/sec) (Qd_1,Qd_2)	Qd	Determination Check Capacity	Remarks
	BP	EP									
1	0+010.0	0+525.0	L	0.0874	0.5	0.33	2.697	1.726	**1.903**	**OK**	
				0.0050	0.9			0.177			
2	0+525.0	0+925.0	L	0.0497	0.5	0.16	1.878	0.981	**1.080**	**OK**	
				0.0028	0.9			0.099			
3	0+925.0	1+212.5	L	0.0293	0.5	0.25	2.347	0.578	**0.627**	**OK**	
				0.0014	0.9			0.049			
4	0+020.0	1+710.0	R	0.0827	0.5	0.24	2.300	1.634	**2.053**	**OK**	
				0.0118	0.9			0.419			
5	1+710.0	1+882.5	R	0.0637	0.5	0.25	2.347	1.259	**1.470**	**OK**	
				0.0060	0.9			0.212			
6	1+882.5	2+480.0	R	0.0522	0.5	0.38	2.894	1.030	**1.148**	**OK**	
				0.0033	0.9			0.118			
7	2+480.0	2+560.0	R	0.0157	0.5	0.45	3.149	0.311	**0.322**	**OK**	
				0.0003	0.9			0.011			
8	2+560.0	2+734.0	R	0.0056	0.5	0.15	1.818	0.112	**0.133**	**OK**	
				0.0006	0.9			0.022			
9	2+734.0	3+300.0	R	0.0699	0.5	0.34	2.737	1.381	**1.500**	**OK**	
				0.0033	0.9			0.119			

U-Ditch Hydrological Caculation

NO.	Station(Lot 1)		Direction	Catchment Area(㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope		Peak Catchment Discharge (㎥/sec)		Determination		Remark
	BP	EP					TYPE-4	TYPE-6	(Qd_1,Qd_2)	Qd	Check Capacity	Apply	
1	12+120	12+139	R	0.0071	0.8	1.32	0.243	0.894	0.224	0.256	U6	U6	
				0.0009	0.9				0.032				
2	12+139	12+407	R	0.0064	0.8	4.82	0.465	1.708	0.202	0.241	U4	U4	
				0.0011	0.9				0.039				
3	12+407	12+670	R	0.0038	0.8	9.94	0.668	2.453	0.120	0.163	U4	U4	
				0.0012	0.9				0.043				

6.2 Culvert Design Calculation

Culvert Design Calculations

STATION : (Access Road LOT 1) 0+012.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

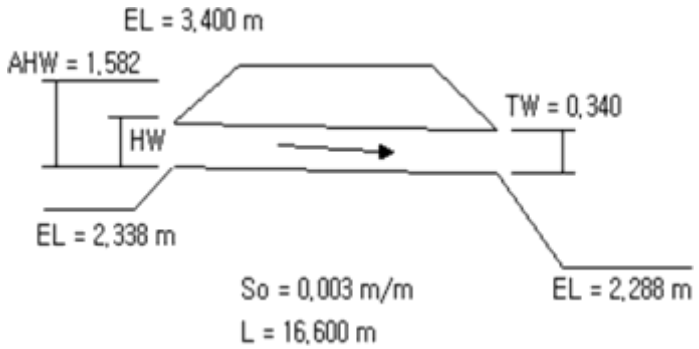
Catchment Area(A) = 0.0913 ㎢
Qd = 1.977 ㎥/sec
Arrival Distance(L) = 2620.000 m

Recurrence Interrval for Culv. 25yr
Elev. Difference: 15.50 m - 2.34 m
= 13.16 m

Rational Formula Qd = 0.278 C I A	STD Run-off Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.80 mm/h C = 0.500 A = 0.091 ㎢ Qd = 1.977 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.266

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
BOX	0.989	2@ 1.5 x 0.9	0.57	0.51	0.2	0.1191	0.223	0.562	0.26	0.72	0.05	0.79	0.79	1.22	Outlet	O.K

Summary and Recommndations :

1. Outlet velocity is 1.24 m/sec, will require a rock apron Protection Outlet.
2. Qd = Avo, Qd = 1.977 < AVo=2 × (B × H) × 0.8 × 1.22 = 2.635 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 0+012.0000

⊙ **Input Sectional Shape (BOX 2@ 1.5 × 0.9)**

·Catchment Area(A) = 0.0913 ㎢	**·Rainfall Intensity(I) = 166.4mm/hr**
·Peak Discharge(Qd) = 1.977 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 13.160 m**

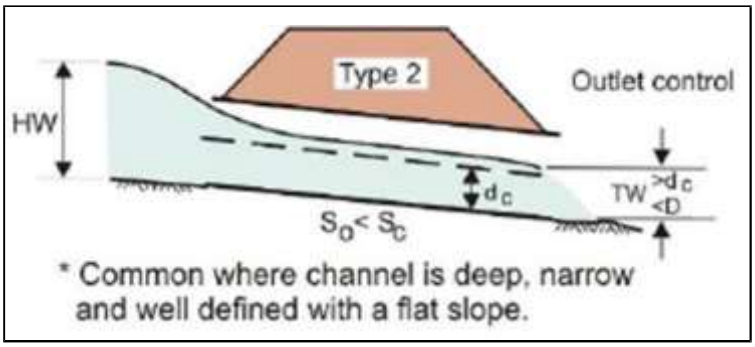
1. Hydrological Analysis

HW (0.79) ≤ 1.2 D (1.08) , So (0.0030) < Sc (0.0052)

dc (0.233) ≤ TW(0.26) < D(0.9) , dn (0.266) > dc (0.233) therefore,

The shape proposed is 2nd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = h_e + h_f + d_{tw} + \frac{V_{tw}^2}{2g} - S_0L$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in average of the outlet

and Vtw = 1.240 m / sec.

2. Analysis of the discharge area

Qd =1.977 ㎥/sec of the drainage area at the current STA., and the box size is 2@1.5 × 0.9

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (1.977) < Q = AVo = 2 × (B × H) × 0.8 × 2 × 1.220 = 2.635 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 1+710.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

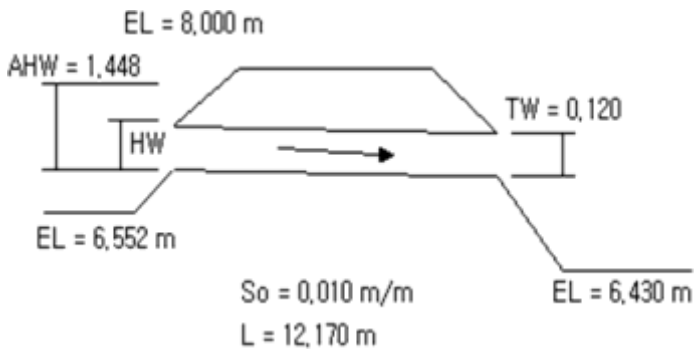
Catchment Area(A) = 0.0125 km²
Qd = 0.271 m³/sec
Arrival Distance(L) = 205.600 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 9.50 m - 6.55 m
= 2.95 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.500 A = 0.013 km² Qd = 0.271 m³/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m³/sec Qd = 0.000 m³/sec	Chupu Karma gauging station

Water level in Culvert = 0.226

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m³/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.271	Φ900	0.42	0.37	0.5	0.009	0.299	0.599	0.11	0.67	0.12	0.56	0.37	2.17	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 2.21 m/sec, will require a rock apron protection Outlet.
2. 2. Qd = Avo, Qd = 0.271 < AVo= (πd²/4) × 0.75 × 2.17 = 1.035 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 1+710.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0125 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.289 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 2.950 m**

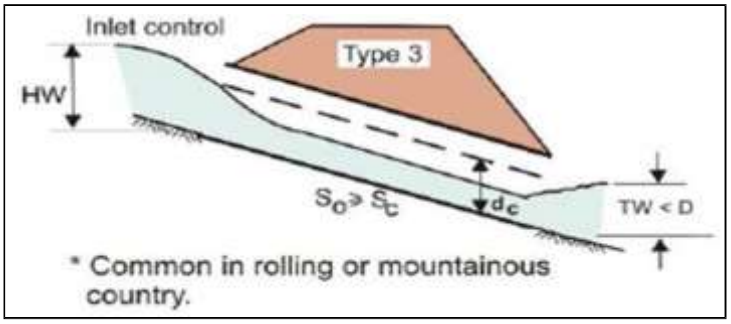
1. Hydrological Analysis

HW (0.37) ≤ 1.2 D (1.08) , So (0.0100) ≥ Sc (0.0033)

TW (0.11) ≤ dc (0.299) < D (0.9) , dn (0.226) < dc (0.299) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 2.170 m / sec.

2. Analysis of the discharge area

Qd =0.271㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.271) < Q = AVo = (πd²/4) × 0.75 × 2.170 = 1.035 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 1+882.50
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-3

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

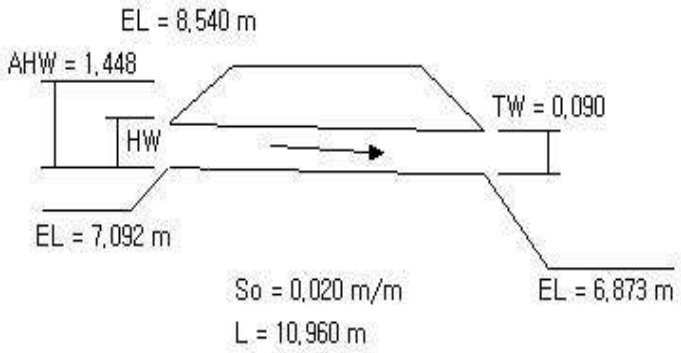
Catchment Area(A) = 0.0118 ㎢
Qd = 0.256 ㎥/sec
Arrival Distance(L) = 590.700 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 10.50 m - 7.09 m
= 3.41 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.500 A = 0.012 ㎢ Qd = 0.256 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.190

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.256	Φ900	0.4	0.36	0.5	0.0048	0.291	0.60	0.09	0.67	0.22	0.46	0.36	2.72	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 2.72 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.256 < AVo= (πd²/4) × 0.75 × 2.72 = 1.298 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 1+882.5000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0118 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.256 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 3.410 m**

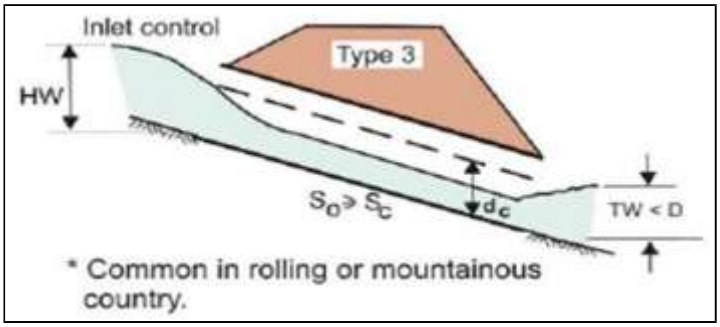
1. Hydrological Analysis

HW (0.36) ≤ 1.2 D (1.08) , So (0.0200) ≥ Sc (0.0033)

TW (0.09) ≤ dc (0.291) < D (0.9) , dn (0.185) < dc (0.291) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 2.720 m / sec.

2. Analysis of the discharge area

Qd =0.256㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.256) < Q = AVo = (πd²/4) × 0.75 × 2.720 = 1.298 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 2+465.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-4

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

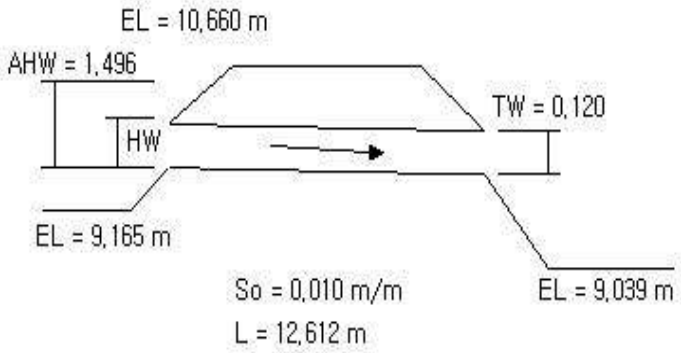
Catchment Area(A) = 0.0147 km²
Qd = 0.318 m³/sec
Arrival Distance(L) = 294.6 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 11.50 m - 9.16 m
= 2.34 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.500 A = 0.015 km² Qd = 0.318 m³/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m³/sec Qd = 0.000 m³/sec	Chupu Karma gauging station

Water level in Culvert = 0.245

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m³/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
BOX	0.318	Φ900	0.46	0.41	0.5	0.0068	0.325	0.612	0.12	0.67	0.13	0.56	0.41	2.27	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 2.50 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.318 < AVo= (πd²/4) × 0.75 × 2.27 = 1.083 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 2+465.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0147 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.340 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 4.340 m**

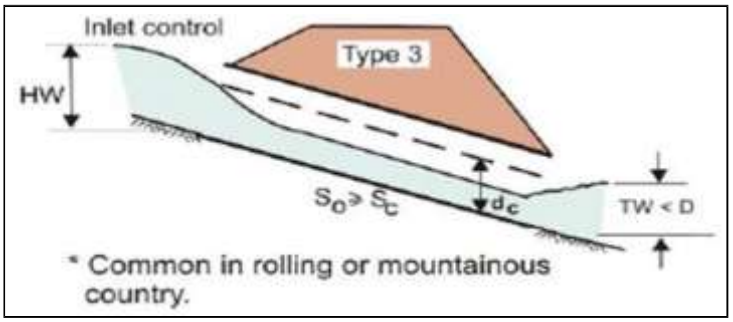
1. Hydrological Analysis

HW (0.41) ≤ 1.2 D (1.08) , So (0.0100) ≥ Sc (0.0034)

TW (0.12) ≤ dc (0.325) < D (0.9) , dn (0.245) < dc (0.325) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_\phi) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 2.270 m / sec.

2. Analysis of the discharge area

Qd =0.180㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.318) < Q = AVo = (πd²/4) × 0.75 × 2.270 = 1.083 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 2+734.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-5

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

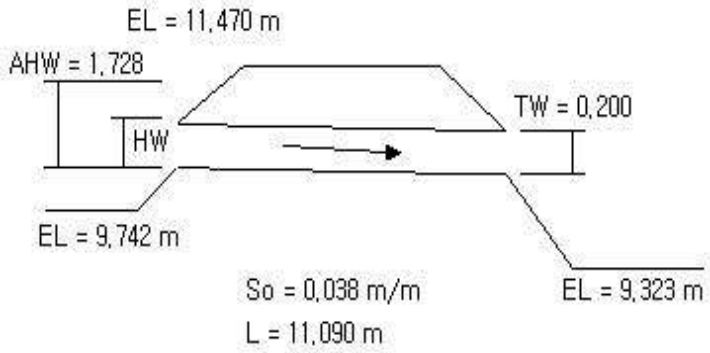
Catchment Area(A) = 0.0670 ㎢
Qd = 1.451 ㎥/sec
Arrival Distance(L) = 376.500 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 12.50 m - 9.74 m
= 2.76 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.500 A = 0.067 ㎢ Qd = 1.451 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.341

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	1.451	Φ900	3.15	2.84	0.5	0.7837	0.712	0.806	0.20	0.67	0.42	1.04	2.84	5.58	INLET	N.G
PIPE	1.451	Φ1200	0.77	0.93	0.5	0.2091	0.658	0.929	0.20	0.90	0.42	0.69	0.93	5.47	INLET	O.K

Summary and Recommndations :

1. Outlet velocity is 5.47 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 1.451 < AVo= (πd²/4) × 0.75 × 5.47 = 4.640 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 2+734.0000

⊙ **Input Sectional Shape (Φ1200)**

·**Catchment Area(A) = 0.0670 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 1.451 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 2.760 m**

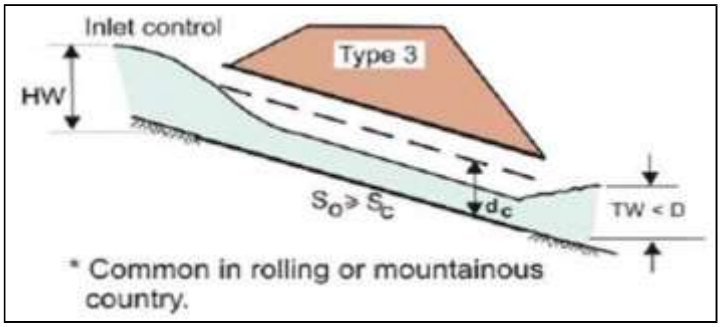
1. Hydrological Analysis

HW (0.93) ≤ 1.2 D (1.44) , So (0.0378) ≥ Sc (0.0035)

TW (0.20) ≤ dc (0.658) < D (1.2) , dn (0.341) < dc (0.658) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 5.470 m / sec.

2. Analysis of the discharge area

Qd =1.451㎥/sec of the drainage area at the current STA., and the Pipe size is φ1200

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (1.451) < Q = AVo = (πd²/4) × 0.75 × 5.470 = 4.640 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 3+300.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-6

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

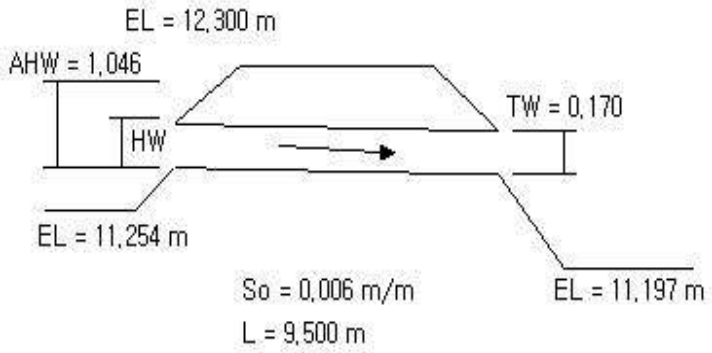
Catchment Area(A) = 0.0198 km²
Qd = 0.429 m³/sec
Arrival Distance(L) = 161.677 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 14.50 m - 11.25 m
= 3.25 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.500 A = 0.020 km² Qd = 0.429 m³/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m³/sec Qd = 0.000 m³/sec	Chupu Karma gauging station

Water level in Culvert = 0.197

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m³/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
BOX	0.429	1.2 x 0.9	0.36	0.32	0.2	0.0169	0.235	0.568	0.17	0.72	0.06	0.68	0.32	1.81	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 1.85 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.429 < AVo= (B × H) × 0.8 × 1.81 = 1.564 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 3+300.0000

⊙ **Input Sectional Shape (BOX 1.2 × 0.9)**

·**Catchment Area(A) = 0.0198 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.429 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 3.250 m**

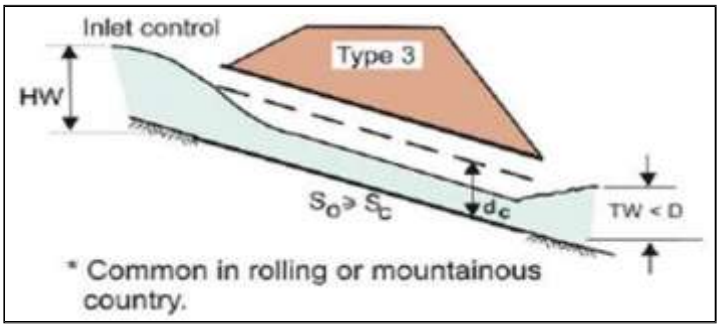
1. Hydrological Analysis

HW (0.32) ≤ 1.2 D (1.08) , So (0.0060) ≥ Sc (0.0036)

TW (0.17) ≤ dc (0.235) < D (0.9) , dn (0.197) < dc (0.235) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = (0.701 + 0.234 \cdot C_s) \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}}$$

HW can be found from the equation.

The flow velocity at this time is the uniform flow velocity of the outlet

and Vn = 1.810 m / sec.

2. Analysis of the discharge area

Qd =0.429㎥/sec of the drainage area at the current STA., and the Box culvert size is 1.2 × 0.9

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.429) < Q = AVo = (B × H) × 0.8 × 1.810 = 1.564 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 7+463.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-7

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

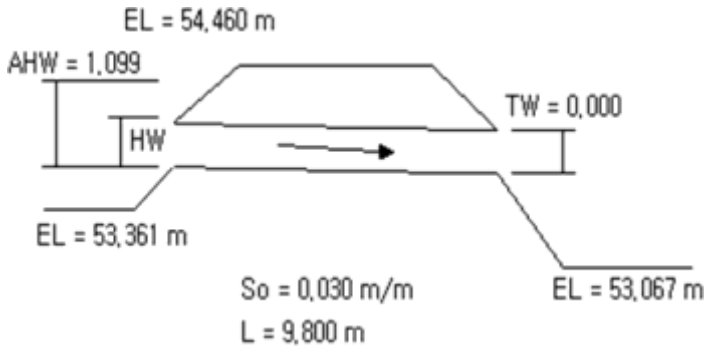
Catchment Area(A) = 0.1044 ㎢
Qd = 3.165 ㎥/sec
Arrival Distance(L) = 689.000 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 100.50 m - 53.36 m
= 47.14 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.700 A = 0.104 ㎢ Qd = 3.165 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.296

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	3.165	Φ1200	2.98	3.58	0.5	0.9489	0.976	1.088	0.20	0.90	0.29	1.55	3.58	6.24	Inlet	N.G
PIPE	1.583	2@ Φ900	1.12	1.01	0.5	0.531	0.524	0.712	0.08	0.67	0.29	0.91	1.05	4.36	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 4.36 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 3.165 < AVo= 2 x (πd²/4) × 0.75 × 4.36 = 4.160 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 7+463.0000

⊙ **Input Sectional Shape (2@ Φ900)**

·**Catchment Area(A) = 0.1044 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 3.165 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 47.14 m**

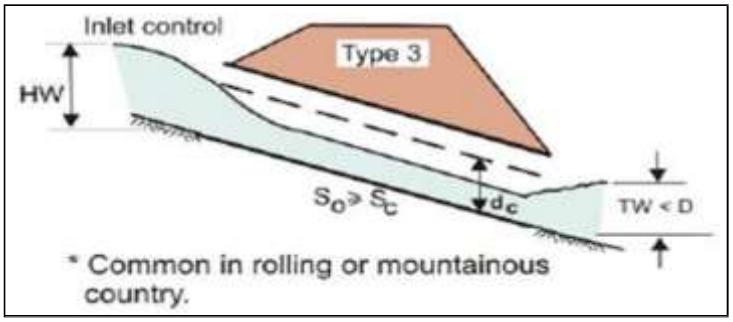
1. Hydrological Analysis

HW (1.01) ≤ 1.2 D (1.08) , So (0.0300) ≥ Sc (0.0040)

TW (0.08) ≤ dc (0.542) < D (0.9) , dn (0.296) < dc (0.524) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_\phi) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 4.360 m / sec.

2. Analysis of the discharge area

Qd =3.165㎥/sec of the drainage area at the current STA., and the Pipe size is 2@ φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (3.165) < Q = AVo = (πd²/4) × 0.75 × 2 × 4.360 = 4.161 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 7+664.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-8

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

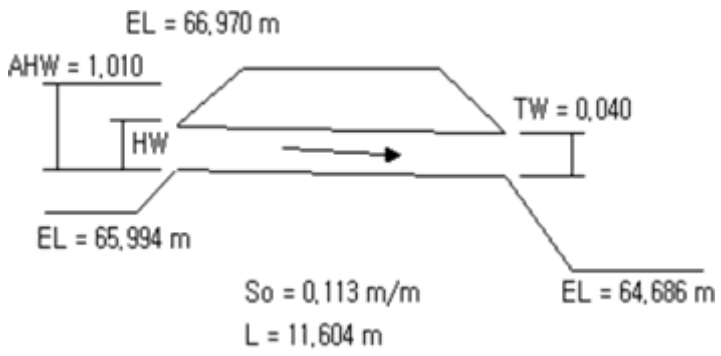
Catchment Area(A) = 0.0064 ㎢
Qd = 0.222 ㎥/sec
Arrival Distance(L) = 164.500 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 91.50 m - 65.99 m
= 25.51 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.800 A = 0.006 ㎢ Qd = 0.222 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.113

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.222	Φ900	0.37	0.33	0.5	0.0188	0.27	0.585	0.04	0.67	1.31	-0.61	0.33	4.79	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 4.79 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.222 < AVo= (πd²/4) × 0.75 × 4.79 = 2.285 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 7+664.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0064 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.222 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 25.510 m**

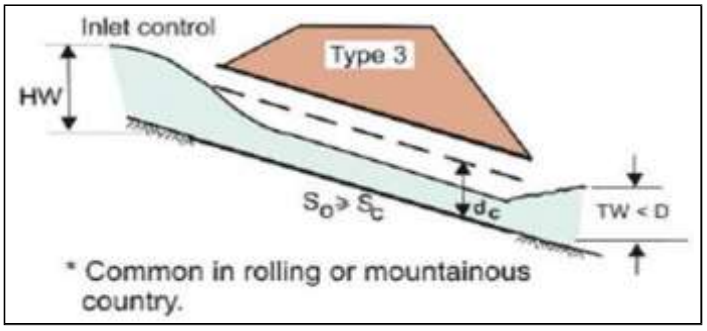
1. Hydrological Analysis

HW (0.33) ≤ 1.2 D (1.08) , So (0.1127) ≥ Sc (0.0033)

TW (0.04) ≤ dc (0.279) < D (0.9) , dn (0.113) < dc (0.270) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 4.790 m / sec.

2. Analysis of the discharge area

Qd =0.222㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.222) < Q = AVo = (πd²/4) × 0.75 × 4.790 = 2.285 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 7+846.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-9

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

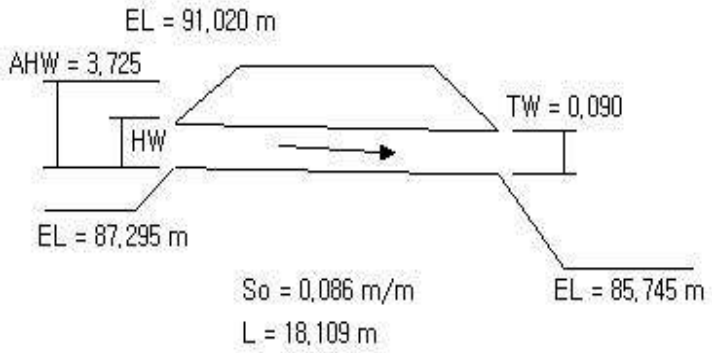
Catchment Area(A) = 0.0197 ㎢
Qd = 0.683 ㎥/sec
Arrival Distance(L) = 225.600 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 100.50 m - 87.30 m
= 13.20 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.800 A = 0.0197 ㎢ Qd = 0.683 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.210

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.683	Φ900	0.76	0.68	0.5	0.2277	0.485	0.692	0.09	0.67	1.55	-0.62	0.68	6.07	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 6.07 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.683 < AVo= (πd²/4) × 0.75 × 6.07 = 2.896 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 7+846.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0197 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.683 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 13.29 m**

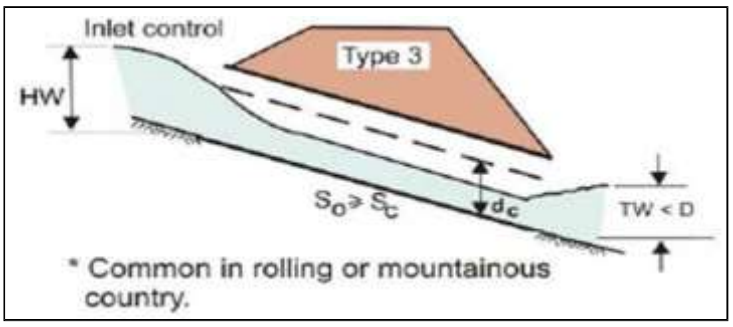
1. Hydrological Analysis

HW (0.68) ≤ 1.2 D (1.08) , So (0.0856) ≥ Sc (0.0038)

TW (0.09) ≤ dc (0.485) < D (0.9) , dn (0.210) < dc (0.485) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_\phi) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 6,070 m / sec.

2. Analysis of the discharge area

Qd =0.683㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.683) < Q = AVo = (πd²/4) × 0.75 × 6.070 = 2.896 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 7+984.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-10

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

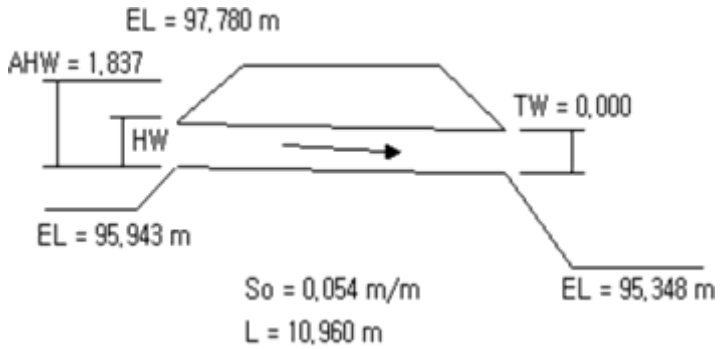
Catchment Area(A) = 0.0084 ㎢
Qd = 0.291 ㎥/sec
Arrival Distance(L) = 190.400 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 102.53 m - 95.94 m
= 6.56 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.800 A = 0.008 ㎢ Qd = 0.291 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.154

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.291	Φ900	0.43	0.39	0.5	0.0313	0.311	0.605	0.07	0.67	0.6	0.11	0.39	4.02	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 4.02 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.291 < AVo= (πd²/4) × 0.75 × 4.02 = 1.918 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 7+984.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0084 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.291 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 6.65 m**

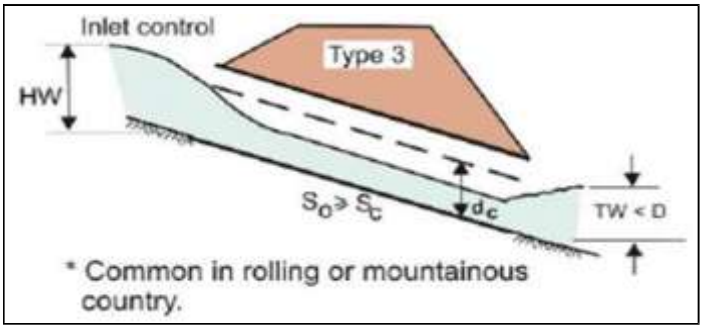
1. Hydrological Analysis

HW (0.39) ≤ 1.2 D (1.08) , So (0.0543) ≥ Sc (0.0034)

TW (0.07) ≤ dc (0.311) < D (0.9) , dn (0.154) < dc (0.311) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 4.020 m / sec.

2. Analysis of the discharge area

Qd =0.291㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.291) < Q = AVo = (πd²/4) × 0.75 × 4.020 = 1.918 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 9+395.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-11

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

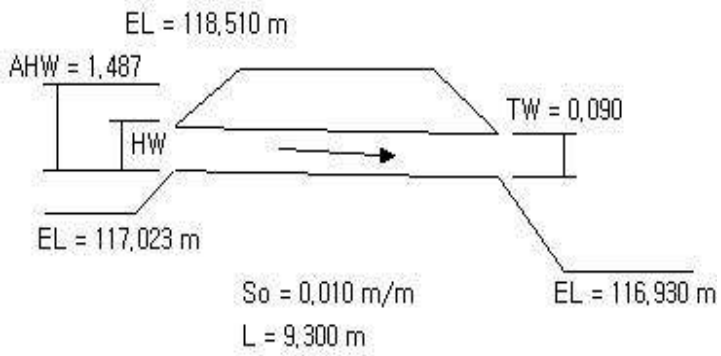
Catchment Area(A) = 0.0058 ㎢
Qd = 0.215 ㎥/sec
Arrival Distance(L) = 88.500 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 134.08 m - 117.02 m
= 17.06 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.800 A = 0.006 ㎢ Qd = 0.201 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.195

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.201	Φ900	0.34	0.31	0.5	0.0136	0.257	0.579	0.09	0.67	0.09	0.60	0.31	1.99	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 1.99 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.201 < AVo= (πd²/4) × 0.75 × 1.99 = 0.949 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 9+395.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0058 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.201 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 17.06 m**

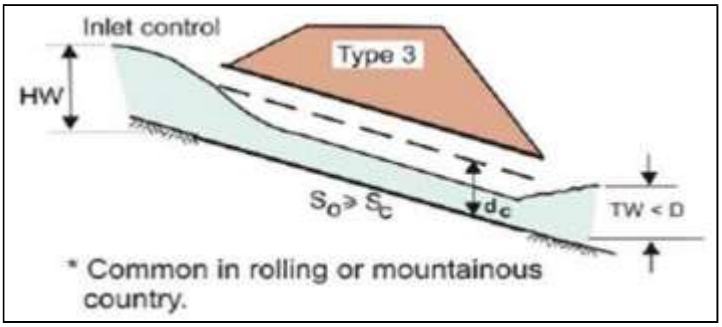
1. Hydrological Analysis

HW (0.31) ≤ 1.2 D (1.08) , So (0.0100) ≥ Sc (0.0033)

TW (0.09) ≤ dc (0.257) < D (0.9) , dn (0.195) < dc (0.257) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 1.990 m / sec.

2. Analysis of the discharge area

Qd =0.201㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.201) < Q = AVo = (πd²/4) × 0.75 × 1.990 = 0.949 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 1) 12+139.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 1 C-12

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

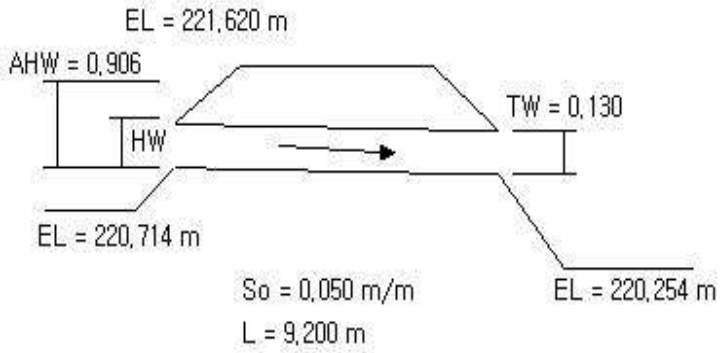
Catchment Area(A) = 0.0157 km²
Qd = 0.544 m³/sec
Arrival Distance(L) = 90.800 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 243.19 m - 220.71 m
= 22.48 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.8 mm/h C = 0.800 A = 0.0157 km² Qd = 0.544 m³/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m³/sec Qd = 0.000 m³/sec	Chupu Karma gauging station

Water level in Culvert = 0.214

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m³/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.544	Φ900	0.65	0.58	0.5	0.1009	0.43	0.665	0.13	0.67	0.46	0.32	0.58	4.69	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 4.69 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.544 < AVo= (πd²/4) × 0.75 × 4.69 = 2.238 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 12+139.0000

⊙ **Input Sectional Shape (Φ900)**

·**Catchment Area(A) = 0.0157 ㎢**	·**Rainfall Intensity(I) = 155.8mm/hr**
·**Peak Discharge(Qd) = 0.544 ㎥/sec**	·**Rain fall Area = Chupu Karma**
·**Recurrence Interval = 25 years**	·**Elevation Diffence = 22.480 m**

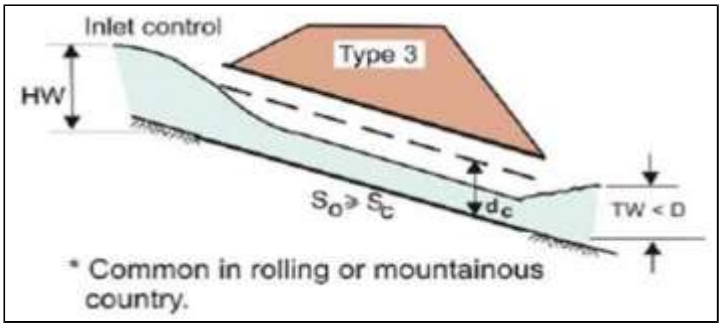
1. Hydrological Analysis

HW (0.58) ≤ 1.2 D (1.08) , So (0.0500) ≥ Sc (0.0036)

TW (0.13) ≤ dc (0.430) < D (0.9) , dn (0.214) < dc (0.430) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 4.690 m / sec.

2. Analysis of the discharge area

Qd =0.544㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.544) < Q = AVo = (πd²/4) × 0.75 × 4.690 = 2.238 : O.K

C. Calculation of Longitudinal Pipes

1) Design Discharge of Flow

$Qd = 1/360 \times C \times r \times W \times L \times 10^{-4}$

Qd : Design Discharge (m^3/sec)

C : Run-off Coefficient(0.9)

r : Rainfall Intensity(5yr) :142.1mm/hr

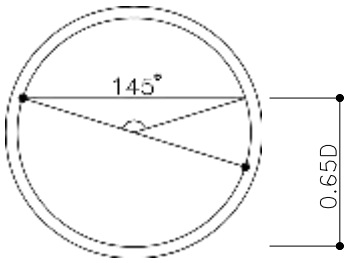
W : Width of Catchment

L : Length of longitudinal pipe(m)

2) Flow rate

$Q = A \times V = A \times (1/n) \times R^{2/3} \times I^{1/2}$

Calculation of flow rate by diameter



$A = \pi D^2/4 \times 215/360 + D/2 \times \sin 72.5 \times D/2 \times \cos 72.5 = 0.5407D^2$

$P = \pi \times D \times 215/360 = 1.87623D$

$R = A / P = 0.2882D$

n = 0.012 Fully pave C.S.P

Depth :0.65 D

DIA (mm)	Depth of Flow (0.65D)m	Area of the flow (A)m	Wetted perimeter (P)m	Hydraulic Radius (R)m	$R^{2/3}$	1/n 0.012	Q (m^3/sec)	Remarks
300	0.1950	0.0487	0.5629	0.0865	0.1955	83.333	$0.7929 \times I^{1/2}$	
450	0.2925	0.1095	0.8443	0.1297	0.2562	83.333	$2.3377 \times I^{1/2}$	
600	0.3900	0.1947	1.1257	0.1729	0.3104	83.333	$5.0345 \times I^{1/2}$	
900	0.5850	0.4380	1.6886	0.2594	0.4067	83.333	$14.8435 \times I^{1/2}$	
1200	0.7800	0.7786	2.2515	0.3458	0.4927	83.333	$31.9672 \times I^{1/2}$	

Longitudinal Pipe List

NO.	Beginning Point	End Point	Direction	Length (m)	Spec	Slope	Flow rate (m3/sec)	Catchment Area(Km²)	Run-off Coefficient	Peak Catchment Discharge (m³/sec)		Determination	Remarks
										Qd1,Qd2	Qd		
1	0+108	0+120	LHS	12	900	0.0020	0.663	0.0061	0.9	0.218	0.41	**O.K**	
								0.0098	0.5	0.192			
2	0+214	0+230	LHS	16	900	0.0020	0.663	0.0056	0.9	0.2	0.38	**O.K**	
								0.0092	0.5	0.18			
3	0+319	0+331	LHS	12	900	0.0040	0.938	0.0052	0.9	0.185	0.354	**O.K**	
								0.0086	0.5	0.169			
4	0+491	0+503	LHS	12	900	0.0040	0.938	0.0045	0.9	0.161	0.311	**O.K**	
								0.0076	0.5	0.15			
5	0+591	0+603	LHS	12	900	0.0020	0.663	0.0041	0.9	0.146	0.281	**O.K**	
								0.0068	0.5	0.135			
6	0+685	0+697	LHS	12	900	0.0015	0.574	0.0037	0.9	0.132	0.256	**O.K**	
								0.0063	0.5	0.124			
7	0+942	0+958	LHS	16	900	0.0020	0.663	0.0028	0.9	0.097	0.188	**O.K**	
								0.0046	0.5	0.091			
8	1+934	1+942	RHS	8	900	0.0035	0.878	0.0032	0.9	0.114	0.822	**O.K**	
								0.0358	0.5	0.708			
9	2+468	2+478	RHS	10	600	0.0075	0.436	0.0004	0.9	0.012	0.322	**O.K**	
								0.0157	0.5	0.31			
10	2+991	3+003	RHS	12	900	0.0025	0.742	0.0040	0.9	0.142	0.54	**O.K**	
								0.0202	0.5	0.398			

PROJECT : TINA RIVER HYDROPOWER DEVELOPMENT PROJECT

CALCULATION
(ACCESS ROAD LOT 2-1,2-2,2-3,3-1,3-2)

DOCUMENT No.: E-PR-CVGD-C2-00010

EMPLOYER : TINA HYDROPOWER LIMITED (THL)

EPC CONTRACTOR : HYUNDAI ENGINEERING CO., LTD. (HEC)

ISSUE STATUS

0	24-JUN-2020	ISSUED FOR CONSTRUCTION	H.K CHO		H.K YU		T.H. KIM		J.K. LEE	
REV. No.	DATE	DESCRIPTION	PREPARED		CHECKED		REVIEWED		APPROVED	

Tina River Hydropower Development P oject (TRHDP)

CALCULATION
(ACCESS ROAD LOT 2)

24 - JUN -2020

Tina Hydropower Limited

HYUNDAI ENGINEERING CO., LTD

Content

1 .Box Culvert

(LOT 2-2)

1.1 Box Culvert 1 (STA.2+423.00)

2@2.0×2.00

FH=2.57 m [SI UNIT]

1.1.1 Design Conditions

1) General Items

(1) Type of Culvert : 2 Box

(2) Width (w) : 2 @ 2.0 m

(3) Height (h) : 2.00 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength	:	fc'	=	32	MPa
¤ Modulus of Elasticity	:	Ec	=	26587	MPa

(2) Reinforcement bar

⊳ Yield Strength	:	fy	=	420	MPa
⊲ Modulus of Elasticity	:	Es	=	200000	MPa

3) Material weight

(1) Reinforced Concrete	:	ωc	=	25.00	kN/m³
(2) plain concrete	:	γcn	=	23.50	kN/m³
(3) Pavement	:	γasp	=	23.00	kN/m³
(4) Subterranean	:	γw	=	10.00	kN/m³

4) Soil

(1) Wet Unit Weight	:	γt	=	19.00	kN/m³
(2) Submerged Unit Weight	:	γsub	=	10.00	kN/m³
(3) angle of internal friction	:	φ	=	28.00	°
(4) coefficient of earth pressure atrest	:	Ko	=	1-sinΦ	= 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

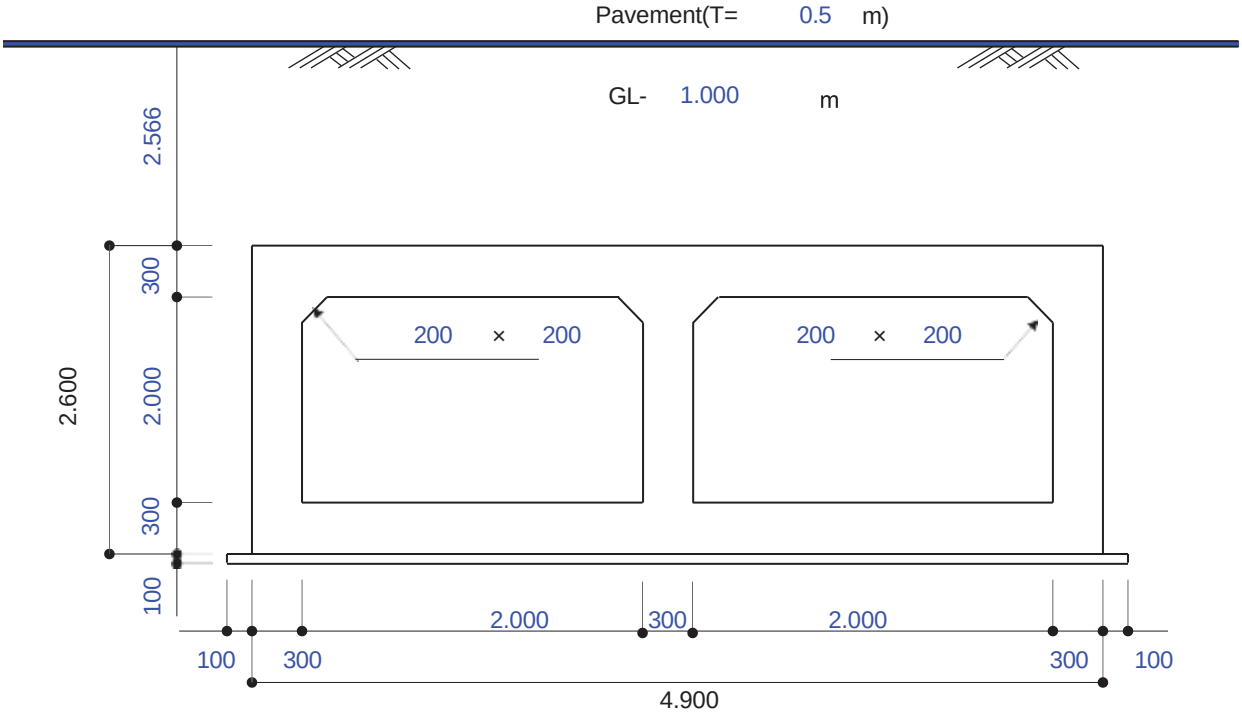
· SAP2000 (Structure Analysis Program)

8) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.1.2 Section Assumption



1.1.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.500 × 4.900 × 23.0	56.350
Vertical earth pressure (EV)	No exist ground water	2.066 × 4.900 × 19.0	192.345
	Exist ground water	(0.500 × 19.0 + 1.566 × 10.0) × 4.900	123.284
Ground Water(WA')		1.566 × 4.900 × 10.0	76.734
Sub Total		Surcharge Load for Bouyancy Check	256.368
Slab(DC)	Top	0.300 × 4.900 × 25.0	36.750
	Bottom	0.300 × 4.900 × 25.0	36.750
Wall(DC)	Left	0.300 × 2.000 × 25.0	15.000
	Right	0.300 × 2.000 × 25.0	15.000
	Inner	0.300 × 2.000 × 25.0	15.000
Hunch(DC)		0.200 × 0.200 / 2 × 25.0 × 4 EA	2.000
Sub Total		Surcharge Load for Bouyancy Check	120.500

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.00 m)

- Total Load for Bouyancy Check : 376.868 kN
- Uplife force : 4.900 ×(5.166 - 1.000)× 10.0 kN/□ = 204.134 kN
- Safety factor = 1.25

□ F.S = 376.868 / 204.134 = 1.846 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL- 3.000 m)

- Total Load for Bouyancy Check :

120.500 +(0.000 × 4.900 × 10.000 kN/□) = 120.500 kN

- Uplife force : 4.900 ×(5.166 - 3.000)× 10.0 kN/□ = 106.134 kN
- Safety factor = 1.1

□ F.S = 120.500 / 106.134 = 1.135 > 1.1 - O.K

Ā Ground water level should not exceed GL- 3.000m

3) Allowable vertical bearing capacity check

(1) Load

- Dead load
  - Self weight of Structure = 120.500 / 4.900 = 24.592 kN/m²
  - Vertical earth pressure = 248.695 / 4.900 = 50.754 kN/m² (No exist ground water)
- Live load = 11.568 kN/m² (Refer to 1.1.4.2)
- Water load in Culvrt = 2.000 ɜ 10.000 = 20.000 kN/m²

(2) Allowable vertical bearing capacity

- Q_{max} = 106.914 kN/m²
- Q_a = 337.500 kN/m² (Refer to Geotechnic Report)

□ Q_a = 337.500 kN/□ > Q_{max} = 106.914 kN/□ - O.K

1.1.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

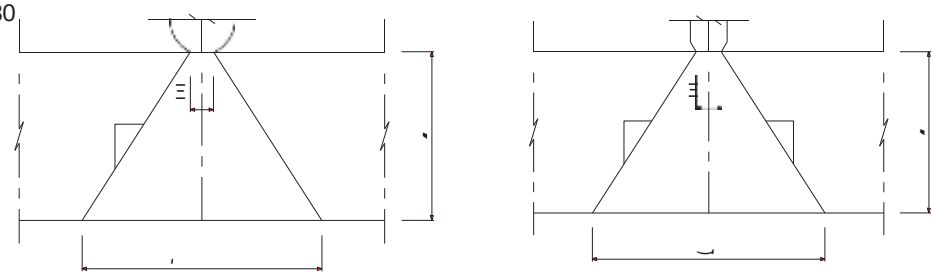
TYPE	Depth (m)	Unit weight (kN/ب)	Load (kN/m²)	
Pavement	0.500	23.000	1.000 × 0.500 × 23.000 =	11.500
Vertical earth pressure	2.066	19.000	1.000 × 2.066 × 19.000 =	39.254
□	2.566		Psv =	50.754 kN/m²

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/ب)	Load (kN/m²)	
Pavement	0.500	23.000	1.000 × 0.500 × 23.000 =	11.500
Vertical earth pressure	0.500	19.000	1.000 × 0.500 × 19.000 =	9.500
	1.566	10.000	1.000 × 1.566 × 10.000 =	15.660
□	2.566		Psvh =	36.660 kN/m²

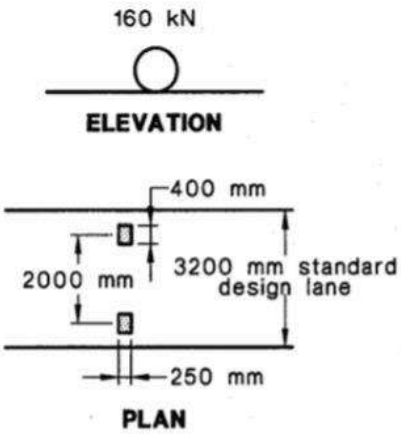
2) Live Load

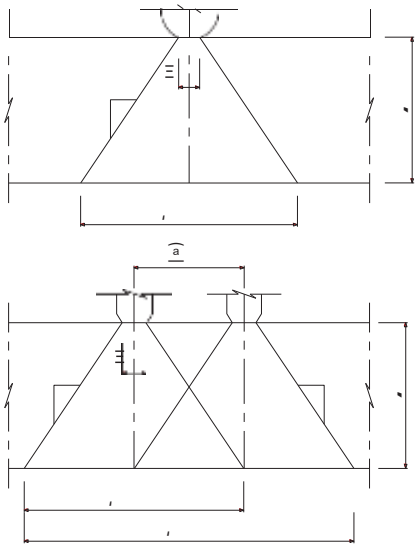
(1) W80



$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 5.132) \times (0.4 + 5.132)} = 2.687 \text{ kN/m}^2$$

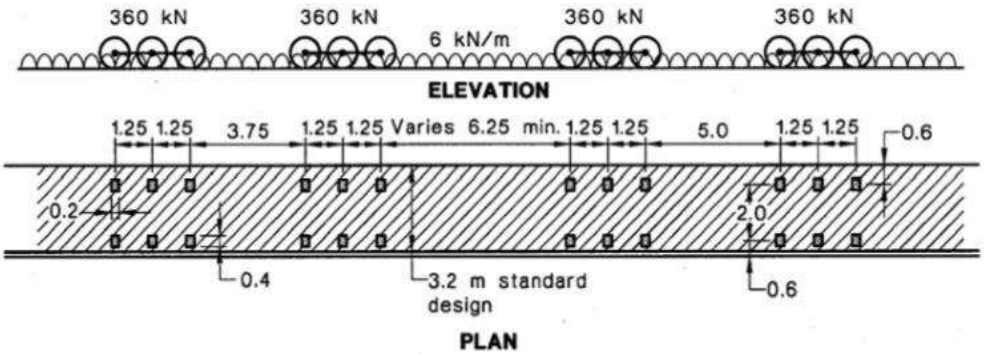
(2) A160



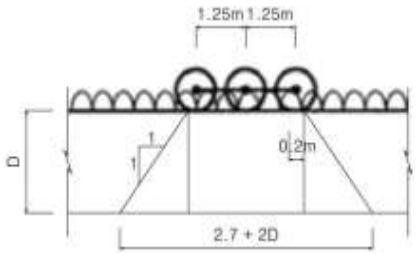


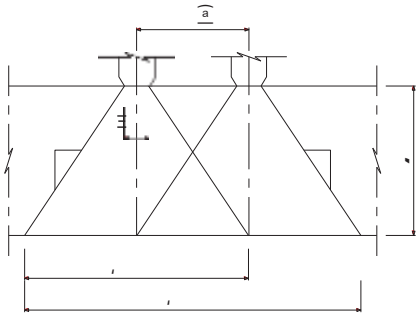
$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 2\times2.566) \times (2.4 + 2\times2.566)} = 3.947 \text{ kN/m}^2$$

(3) M1600



- Axle group



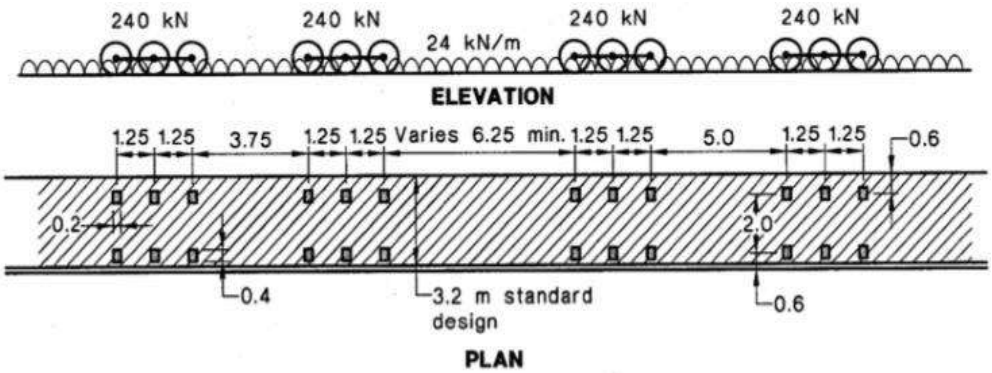


$$P_{v1} = \frac{6 \times 60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 5.132) \times (2.4 + 5.132)} = 6.103 \text{ kN/m}^2$$

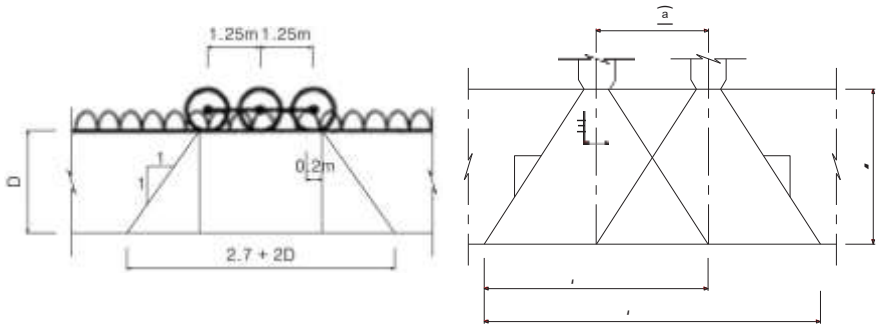
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 6.103 + 1.875 = 7.978 kN/m²

(4) S1600



- Axle group

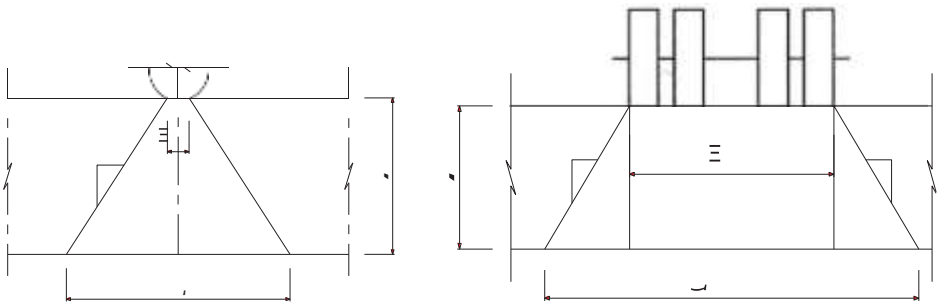


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 5.132) \times (2.4 + 5.132)} = 4.068 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 4.068 + 7.500 = 11.568 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2\times2.566) \times (1.4 + 2\times2.566)} = 3.589 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	2.687	0.10	2.956
A160	3.947	0.10	4.342
M1600	7.978	0.10	8.775
S1600	11.568	0.00	11.568
HLP	3.589	0.10	3.948

□ P_{vl} = 11.568 kN/m² = 11.568 kN/m²

(7) Live Load Surcharge

□ P_{vlh} = 11.568 kN/m² × 0.500 = 5.784 kN/m²

3) Lateral Earth Pressure

↳ coefficient of earth pressure at rest : Ko = 1 - sin30= 0.500

- No exist ground water

P_{sh} = k_o × γ_t × H

P_{sh1} = 0.500 x (23 x 0.500 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 2.066) = 25.377 kN/m²

P_{sh2} = 25.377 + 0.500 × 19.0 × 0.150 = 26.802 kN/m²

P_{sh3} = 26.802 + 0.500 × 19.0 × 0.350 = 30.127 kN/m²

P_{sh4} = 30.127 + 0.500 × 19.0 × 0.900 = 38.677 kN/m²

P_{sh5} = 38.677 + 0.500 × 19.0 × 0.900 = 47.227 kN/m²

P_{sh6} = 47.227 + 0.500 × 19.0 × 0.150 = 48.652 kN/m²

- Exist ground water

$P_{sh'}$ = k_o × (γ_t × H_1 + γ_{sub} × H_2)

$P_{sh1'}$ = 0.500 x (23 x 0.500 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 0.500 + 10 x 1.566) = 18.330 kN/m²

$P_{sh2'}$ = 18.330 + 0.500 × 10.0 × 0.150 = 19.080 kN/m²

$P_{sh3'}$ = 19.080 + 0.500 × 10.0 × 0.350 = 20.830 kN/m²

$P_{sh4'}$ = 20.830 + 0.500 × 10.0 × 0.900 = 25.330 kN/m²

$P_{sh5'}$ = 25.330 + 0.500 × 10.0 × 0.900 = 29.830 kN/m²

$P_{sh6'}$ = 29.830 + 0.500 × 10.0 × 0.150 = 30.580 kN/m²

4) Ground Water Load

1) Horizontal ground Water Pressure

$P_{wh} = \gamma_w \times H_2$

$P_{wh1} = 10.0 \times 1.566 = 15.660$ kN/m²

$P_{wh2} = 15.660 + 10.0 \times 0.150 = 17.160$ kN/m²

$P_{wh3} = 17.160 + 10.0 \times 0.350 = 20.660$ kN/m²

$P_{wh4} = 20.660 + 10.0 \times 0.900 = 29.660$ kN/m²

$P_{wh5} = 29.660 + 10.0 \times 0.900 = 38.660$ kN/m²

$P_{wh6} = 38.660 + 10.0 \times 0.150 = 40.160$ kN/m²

2) Vertical ground Water Pressure

- Top Slab : $P_{wv1} = 10.0 \times 1.566 = 15.660$ kN/m²

-Bottom Slab : $P_{wv2} = 10.0 \times 4.166 = 41.660$ kN/m²

5) Load Combination

(1) Ultimate Load

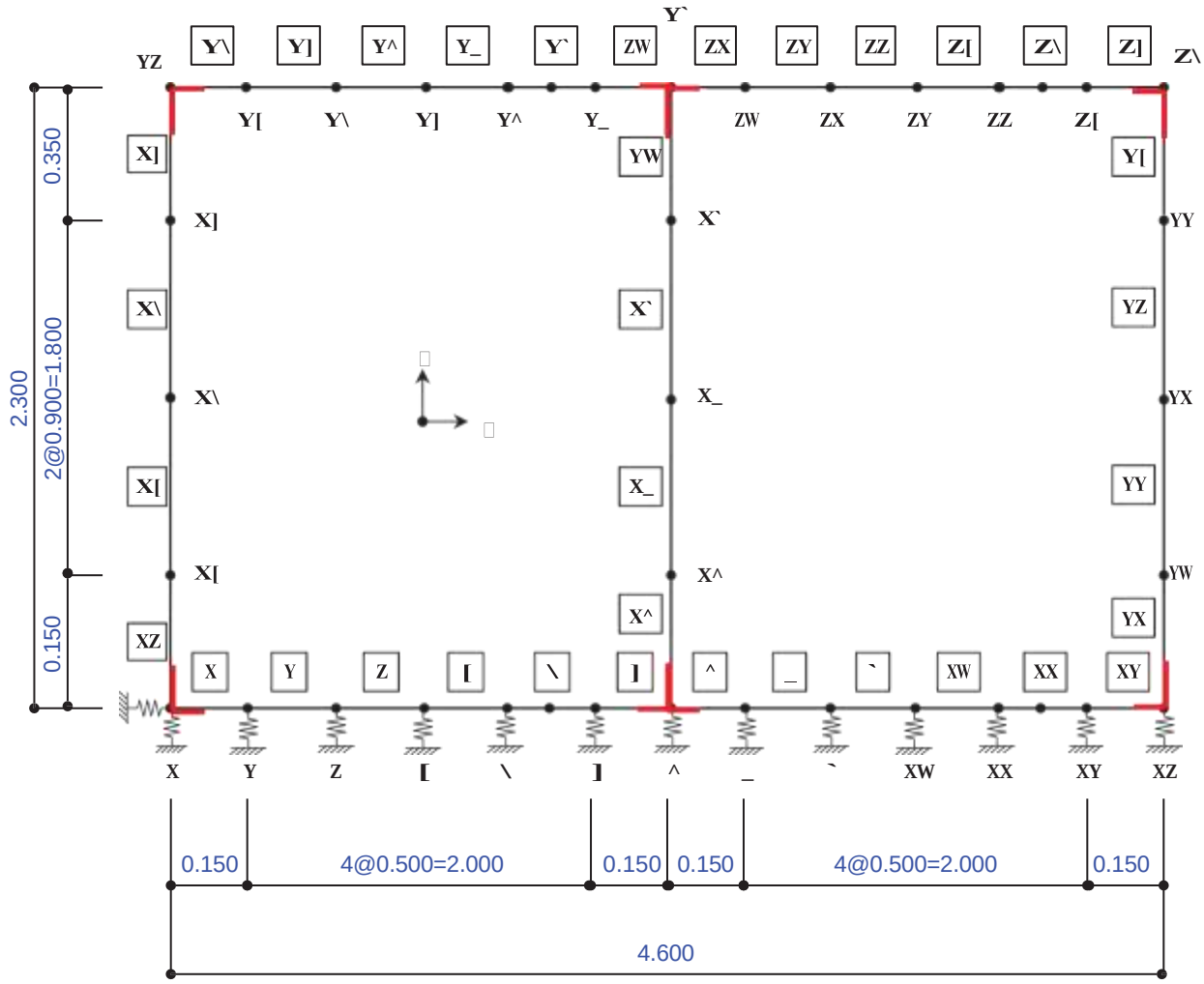
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.58		1.60	1.60	1.60			
COMB 3	1.20	1.58		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.58	1.60	1.60		1.60	1.60	1.58
COMB 7	1.20		1.58	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	0.986		1.00	1.00	1.00			
SCOMB 2	1.00	0.986		1.00	1.00	0.56			
SCOMB 3	1.00	0.986				0.00			
SCOMB 4	1.00		0.986	1.00	1.00		1.00	1.00	0.986
SCOMB 5	1.00		0.986	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		0.986				0.00	0.00	0.00

1.1.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	Z	Section	Node	X	Z	Section
1	0.150	0.150	Bottom Slab	19	2.450	2.100	Middle Wall
2	0.300	0.150		20	4.750	0.300	Right Wall
3	0.800	0.150		21	4.750	1.200	
4	1.300	0.150		22	4.750	2.100	
5	1.800	0.150		23	0.150	2.450	Top Slab
6	2.300	0.150		24	0.500	2.450	
7	2.450	0.150		25	0.900	2.450	
8	2.600	0.150		26	1.300	2.450	
9	3.100	0.150		27	1.700	2.450	
10	3.600	0.150		28	2.100	2.450	
11	4.100	0.150		29	2.450	2.450	
12	4.600	0.150		30	2.800	2.450	
13	4.750	0.150		31	3.200	2.450	
14	0.150	0.300	Left Wall	32	3.600	2.450	
15	0.150	1.200		33	4.000	2.450	
16	0.150	2.100		34	4.400	2.450	
17	2.450	0.300	Middle Wall	35	4.750	2.450	
18	2.450	1.200					

(2) Section

NO.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~5 , 8~11	Bottom Slab
2	0.300	1.000	0.300	0.002250	14~15	Left Wall
3	0.300	1.000	0.300	0.002250	18~19	Middle Wall
4	0.300	1.000	0.300	0.002250	22~23	Right Wall
5	0.300	1.000	0.300	0.002250	26~29 , 32~35	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$$Kv = Kvo\ (Bv / 0.3)^{-3/4}$$

$$kvo = 1/0.3 \times \alpha \times Eo$$

Eo : the modulus of subgrade elasticity (kN/m²)

α : correction factor for calculating Eo

$Eo = 7000$ kN/m² (Refer to Geotechnic Report)

$\alpha = 4$

$Kvo = 1/0.3 \times \alpha \times Eo = 1/0.3 \times 4 \times 7000 = 93333.333$ kN/m

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{4.90 \times 4.90} = 4.900$ m²

$Kv = Kvo\,(Bv / 0.3)^{-3/4}$

$= 93333.333 \times (4.900 / 0.3)^{-3/4} = 11487.6$ kN/m

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 13	11487.600	0.2250	1.0000	0.2250	2584.7
2, 12	11487.600	0.3250	1.0000	0.3250	3733.5
3~5, 9~11	11487.600	0.5000	1.0000	0.5000	5743.8
6, 8	11487.600	0.3250	1.0000	0.3250	3733.5
7	11487.600	0.1500	1.0000	0.1500	1723.1

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

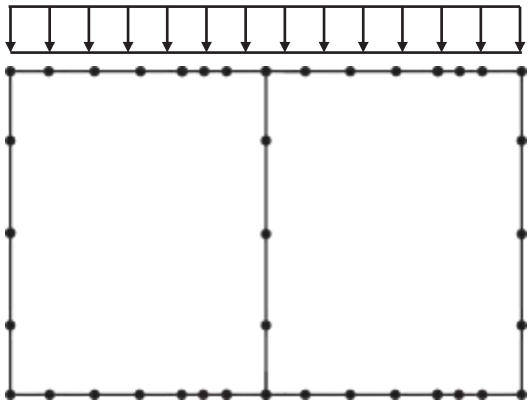
3) Loading

(1) LOAD-1 : Self weight - Automatic consideration in program

(2) LOAD-2,3 : Vertical earth pressure

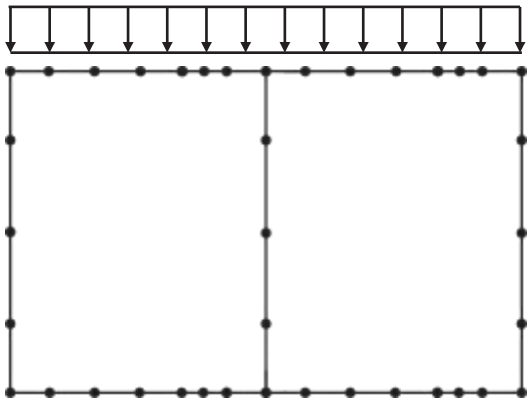
Psvh= 36.660 kN/m² (Exist ground water)

Psv= 50.754 kN/m² (No exist ground water)



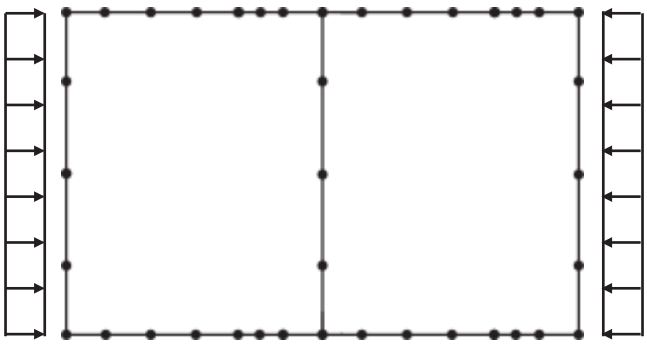
(3) LOAD-4 : Live Load

Pvl= 11.568 kN/m²



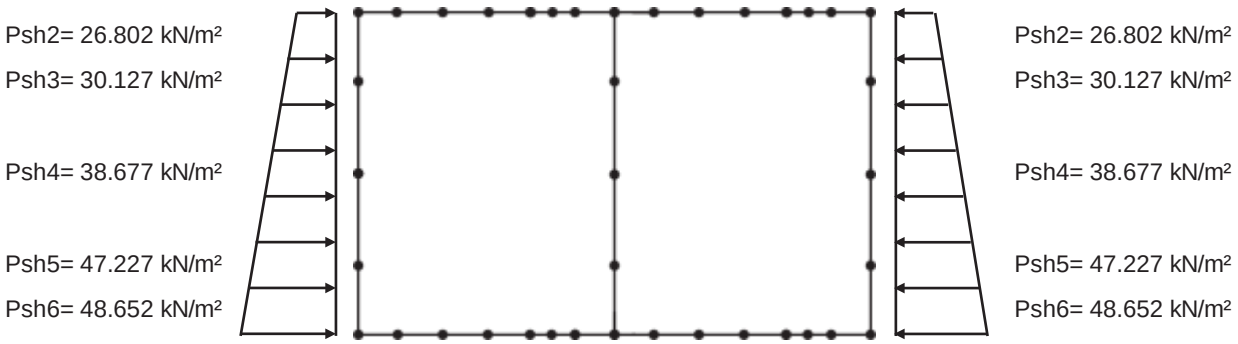
(4) LOAD-5 : Live Load Surcharge

Pvlh= 5.784 kN/m²

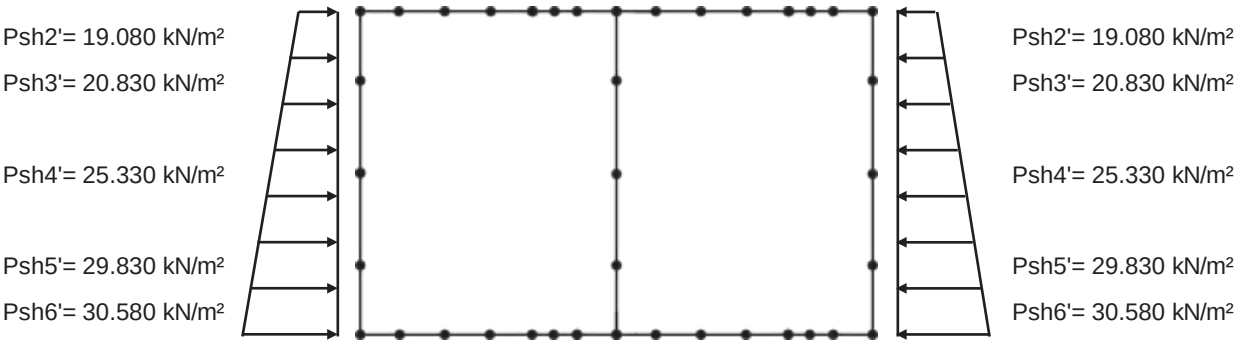


Pvlh= 5.784 kN/m²

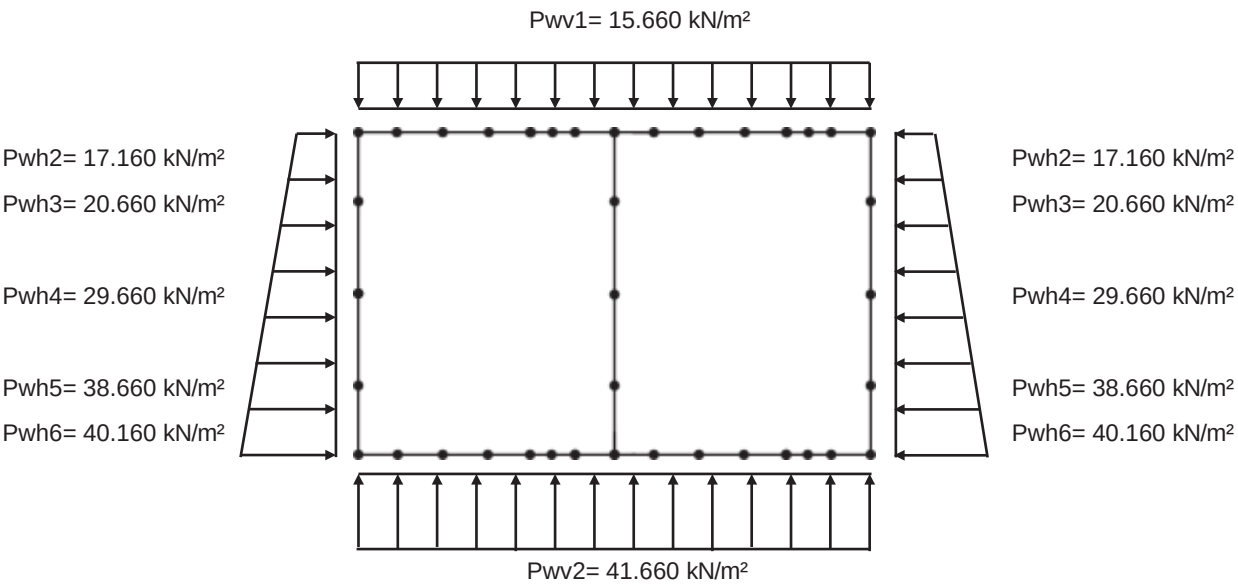
(5) LOAD-6 : Horizontal Earth Pressure (No Ground Water)



(6) LOAD-7 : Horizontal Earth Pressure (Ground Water)

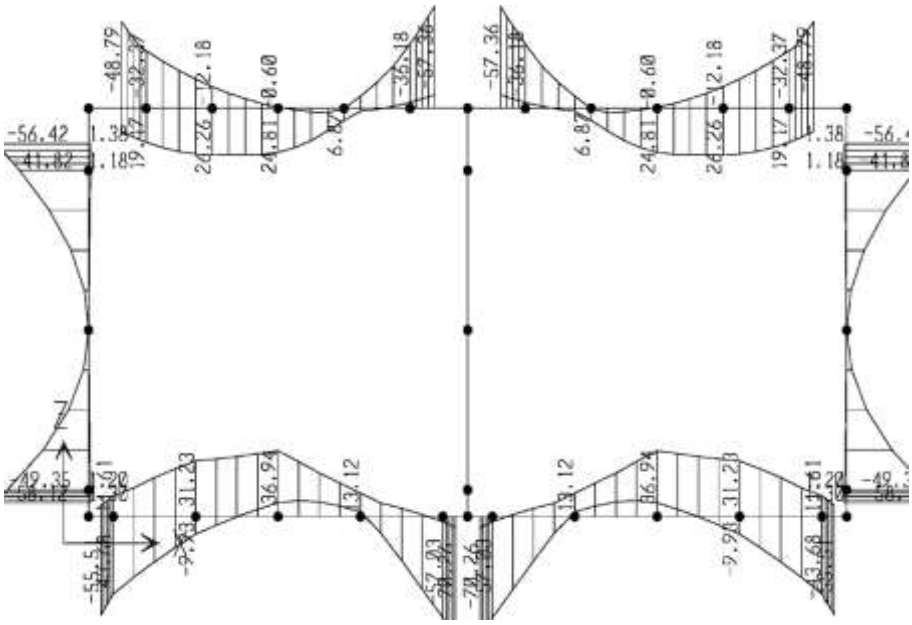


(7) LOAD-8 : Ground Water Pressure

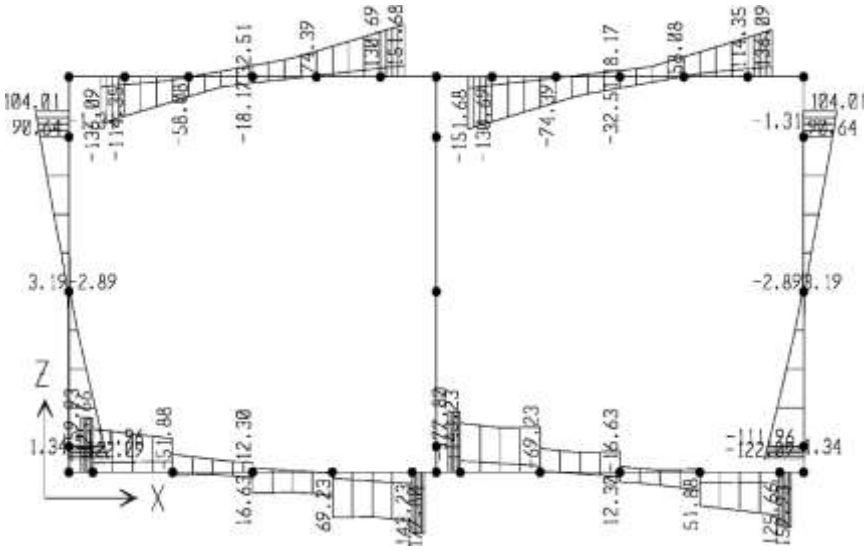


1.1.6 Summary of Analysis Results

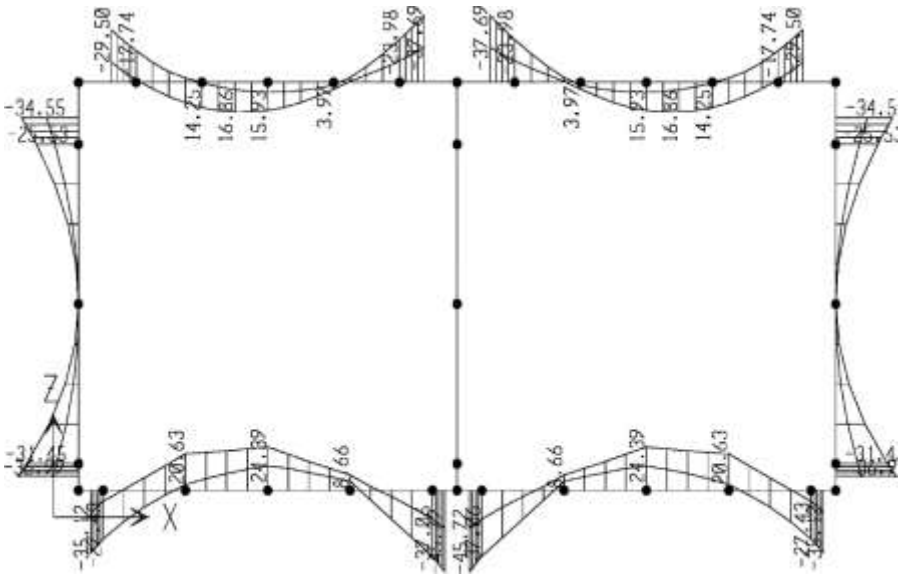
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	ØMn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	42.167	108.764	25.045	367	310.5	98.899	D13 @ 150	2.35
	Middle(+)	32.352	0.000	15.630	300	213.5	67.366	D13 @ 150	2.08
	End of Middle Wall(-)	48.708	120.801	31.125	367	310.5	98.899	D13 @ 150	2.03
Wall	Top(-)	46.940	86.935	28.625	367	310.5	98.899	D13 @ 150	2.11
	Middle(+)	1.206	0.000	0.000	300	213.5	34.193	D13 @ 300	28.34
	Middle(-)	11.081	0.000	7.231	300	243.5	39.069	D13 @ 300	3.53
	Bottom(-)	51.813	99.170	33.045	300	243.5	77.118	D13 @ 150	1.49
Middle Wall	Top & Bottom(-)	0.000	0.000	0.000	300	213.5	34.193	D13 @ 300	-
Bottom Slab	End of the point(-)	50.129	108.166	31.759	300	243.5	77.118	D13 @ 150	1.54
	Middle(+)	43.177	0.000	25.328	300	213.5	67.366	D13 @ 150	1.56
	End of Middle Wall(-)	57.005	117.926	37.147	300	243.5	77.118	D13 @ 150	1.35

1.1.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm			
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm			
M_u	=	42.167	kN·m	V_u	=	108.764	kN	M_o	=	25.045	kN·m				

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 15.268) / 15.268

= 0.0580

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□_ТΦΓΤΥΚΥΦΥΖΤ ΗΠΣ И□_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 362.535 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00156	㎟	$A_{s,4/3req}$	=	483.4	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00156	㎟	$A_{s,min}$	=	483.4	㎟
P_{use}	=	A_s / (B·d)	=	0.00277	㎟	$A_{s,min}$	=	860.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

ØMn = 0.9 × As × fy × (d - a/2) = 98.899 kN·m > Mu = 42.167 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 108.764 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 860.00)]

= 58.845 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 25.045 / [1000 × 58.845 x (310.5 - 58.845 / 3)] × 10^6

= 2.926 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 25.045 / [860.000 x (310.5 - 58.845 / 3)] × 10^6

= 100.114 MPa

fst = fs x (H - d' - χ) / (d - χ) = 100 x (367 - 57 - 3) / (311 - 59) = 100.11 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 100.11) - 2.5 x 50.00 = 937.79 ㎜

300 x (280 / fs) = 300 x (280 / 100.11) = 839.05 ㎜

Sa = 839.05 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (839.05 mm) Ā O.K

2) Top Slab - Middle

(1) Section Design

ⁿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm		
M_u	=	32.352	kN·m	V_u	=	0.000	kN	M_o	=		15.630	kN·m		

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 15.268) / 15.268

= 0.0390

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 406.865 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⁿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00254	㎟	$A_{s,4/3req}$	=	542.5	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00254	㎟	$A_{s,min}$	=	542.5	㎟
P_{use}	=	A_s / (B·d)	=	0.00403	㎟	$A_{s,min}$	=	860.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⁿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 67.366 kN·m > Mu = 32.352 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 860.00)]

= 47.756 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 15.630 / [1000 × 47.756 x (213.5 - 47.756 / 3)] × 10^6

= 3.313 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 15.630 / [860.000 x (213.5 - 47.756 / 3)] × 10^6

= 91.986 MPa

fst = fs x (H - d' - χ) / (d - χ) = 92 x (300 - 87 - 3) / (214 - 48) = 91.99 MPa

ㅿ Maximum center space of reinforcement

Cc = 56.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 91.99) - 2.5 x 80.00 = 956.70 ㎜

300 x (280 / fs) = 300 x (280 / 91.99) = 913.18 ㎜

Sa = 913.18 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (913.18 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 4.900 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 4.9 / 28 × (0.43 + 420 / 700)

= 0.180 m < H = 0.300 m Ā O.K

3. Top Slab - At the end of middle Wall

(1) Section Design

ⅾ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm	
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm	
M_u	=	48.708	kN·m	V_u	=	120.801	kN	M_o	=		31.125	kN·m	

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 15.268) / 15.268

= 0.0580

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 419.370 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⅾ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00180	㎟	$A_{s,4/3req}$	=	559.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00180	㎟	$A_{s,min}$	=	559.2	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00277	㎟	$A_{s,min}$	=	860.0	㎟

4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⅾ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 98.899 kN·m > Mu = 48.708 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 120.801 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 860.00)]

= 58.845 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 31.125 / [1000 × 58.845 x (310.5 - 58.845 / 3)] × 10^6

= 3.637 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 31.125 / [860.000 x (310.5 - 58.845 / 3)] × 10^6

= 124.418 MPa

fst = fs x (H - d' - χ) / (d - χ) = 124 x (367 - 57 - 4) / (311 - 59) = 124.42 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 124.42) - 2.5 x 50.00 = 730.18 ㎜

300 x (280 / fs) = 300 x (280 / 124.42) = 675.14 ㎜

Sa = 675.14 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (675.14 mm) Ā O.K

4. Wall - Top

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	310.5	mm		
B	=	1000	mm	H	=	367	mm		d'	=	56.5	mm		
M_u	=	46.940	kN·m	V_u	=	86.935	kN	M_o	=		28.625	kN·m		

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (310.5 - 15.268) / 15.268

= 0.0580

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ₸ ТФГТУКУФУZТ НПΣ И□ ₸

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 403.997 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7715.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1045.5	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00173	㎟	$A_{s,4/3req}$	=	538.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00173	㎟	$A_{s,min}$	=	538.7	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00277	㎟	$A_{s,min}$	=	860.0	㎟

ᴖ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ᴖ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 98.899 kN·m > Mu = 46.940 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.416 kN > Vu = 86.935 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 310.5 / (8 × 860.00)]

= 58.845 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 28.625 / [1000 × 58.845 x (310.5 - 58.845 / 3)] × 10^6

= 3.345 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 28.625 / [860.000 x (310.5 - 58.845 / 3)] × 10^6

= 114.426 MPa

fst = fs x (H - d' - χ) / (d - χ) = 114 x (367 - 57 - 3) / (311 - 59) = 114.43 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 114.43) - 2.5 x 50.00 = 804.86 ㎜

300 x (280 / fs) = 300 x (280 / 114.43) = 734.10 ㎜

Sa = 734.10 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (734.10 mm) Ā O.K

5. Wall - Middle(In)

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	1.206	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (Φ)

a = 6.271

Because T = C , *c* = 6.271 / β1 = 6.271 / 0.821 = 7.634 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 7.634) / 7.634

= 0.0809

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ̈ ТФГТУКУФУZТ НПΣ И□ ̈

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 14.956 ㎠

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ㎠ (3 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00009	㎠	$A_{s,4/3req}$	=	19.9	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00009	㎠	$A_{s,min}$	=	19.9	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00201	㎠	$A_{s,min}$	=	430.0	㎠

ᴖ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ᴖ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 6.271 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 34.193 kN·m > Mu = 1.206 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 430.00 / 1000 + 8 × 430.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 430.00)]

= 35.040 ㎝

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.000 / [1000 × 35.040 x (213.5 - 35.040 / 3)] × 10^6

= 0.000 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.000 / [430.000 x (213.5 - 35.040 / 3)] × 10^6

= 0.000 MPa

fst = fs x (H - d' - χ) / (d - χ) = 0 x (300 - 87 - 0) / (214 - 35) = 0.00 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 80.00 ㎝

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 4.0E+17 ㎝

300 x (280 / fs) = 300 x (280 / 0.00) = 3.2E+17 ㎝

Sa = 3.18E+17 ㎝ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (3.2E+17 mm) Ā O.K

6. Wall - Middle(Out)

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	11.081	kN·m	V_u	=	0.000	kN	M_o	=		7.231	kN·m	

- Check of Strength reduction factor (Φ)

a = 6.271

Because T = C , *c* = 6.271 / β1 = 6.271 / 0.821 = 7.634 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 7.634) / 7.634

= 0.0927

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ̈ ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ И□ ̈

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 120.848 ㎟

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ㎟ (3 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00066	㎟	$A_{s,4/3req}$	=	161.1	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00066	㎟	$A_{s,min}$	=	161.1	㎟
P_{use}	=	A_s / (B·d)	=	0.00177	㎟	$A_{s,min}$	=	430.0	㎟

ᴖ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ᴖ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 6.271 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 39.069 kN·m > Mu = 11.081 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 430.00 / 1000 + 8 × 430.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 430.00)]

= 37.634 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 7.231 / [1000 × 37.634 x (243.5 - 37.634 / 3)] × 10^6

= 1.664 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 7.231 / [430.000 x (243.5 - 37.634 / 3)] × 10^6

= 72.811 MPa

fst = fs x (H - d' - χ) / (d - χ) = 73 x (300 - 57 - 2) / (244 - 38) = 72.81 MPa

ⓐ Maximum center space of reinforcement

Cc = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 72.81) - 2.5 x 50.00 = 1336.31 ㎜

300 x (280 / fs) = 300 x (280 / 72.81) = 1153.67 ㎜

Sa = 1153.67 ㎜ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (1153.67 mm) Ā O.K

7. Wall - Bottom

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm	
M_u	=	51.813	kN·m	V_u	=	99.170	kN	M_o	=		33.045	kN·m	

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 15.268) / 15.268

= 0.0448

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ̈ ΤΦΓΤΥΚΥΦΥΖΤ ΗΠΣ И□ ̈

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 573.341 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00314	㎟	$A_{s,4/3req}$	=	764.5	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00314	㎟	$A_{s,min}$	=	764.5	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00353	㎟	$A_{s,min}$	=	860.0	㎟

ᴖ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ᴖ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 77.118 kN·m > Mu = 51.813 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 99.170 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 860.00)]

= 51.411 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 33.045 / [1000 × 51.411 x (243.5 - 51.411 / 3)] × 10^6

= 5.679 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 33.045 / [860.000 x (243.5 - 51.411 / 3)] × 10^6

= 169.747 MPa

fst = fs x (H - d' - χ) / (d - χ) = 170 x (300 - 57 - 6) / (244 - 51) = 169.75 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 169.75) - 2.5 x 50.00 = 501.82 ㎜

300 x (280 / fs) = 300 x (280 / 169.75) = 494.86 ㎜

Sa = 494.86 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (494.86 mm) Ā O.K

8. Bottom Slab - At the end of the point

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm		
M_u	=	50.129	kN·m	V_u	=	108.166	kN	M_o	=		31.759	kN·m		

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 15.268) / 15.268

= 0.0448

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 554.375 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00304	㎟	$A_{s,4/3req}$	=	739.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00304	㎟	$A_{s,min}$	=	739.2	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00353	㎟	$A_{s,min}$	=	860.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 77.118 kN·m > Mu = 50.129 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 108.166 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 860.00)]

= 51.411 ㎝

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 31.759 / [1000 × 51.411 x (243.5 - 51.411 / 3)] × 10^6

= 5.458 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 31.759 / [860.000 x (243.5 - 51.411 / 3)] × 10^6

= 163.140 MPa

fst = fs x (H - d' - χ) / (d - χ) = 163 x (300 - 57 - 5) / (244 - 51) = 163.14 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎝

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 163.14) - 2.5 x 50.00 = 527.20 ㎝

300 x (280 / fs) = 300 x (280 / 163.14) = 514.90 ㎝

Sa = 514.90 ㎝ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (514.90 mm) Ā O.K

9. Bottom Slab - Middle

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm		
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm		
M_u	=	43.177	kN·m	V_u	=	0.000	kN	M_o	=		25.328	kN·m		

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 15.268) / 15.268

= 0.0390

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 545.787 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5305.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00341	㎟	$A_{s,4/3req}$	=	727.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00337	㎟	$A_{s,min}$	=	718.9	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00403	㎟	$A_{s,min}$	=	860.0	㎟

ㄴ Pmin ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 67.366 kN·m > Mu = 43.177 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 150.967 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 860.00)]

= 47.756 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 25.328 / [1000 × 47.756 x (213.5 - 47.756 / 3)] × 10^6

= 5.369 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 25.328 / [860.000 x (213.5 - 47.756 / 3)] × 10^6

= 149.058 MPa

fst = fs x (H - d' - χ) / (d - χ) = 149 x (300 - 87 - 5) / (214 - 48) = 149.06 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 80.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 149.06) - 2.5 x 80.00 = 513.82 ㎜

300 x (280 / fs) = 300 x (280 / 149.06) = 563.54 ㎜

Sa = 513.82 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (513.82 mm) Ā O.K

(3) Deflection Check

- Boundary condition : One-way Slab, Both ends continuous
- Span : L = 4.900 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 × (0.43 + fy / 700) = 4.9 / 28 × (0.43 + 420 / 700)

= 0.180 m < H = 0.300 m Ā O.K

10. Bottom Slab - At the end of middle Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	243.5	mm			
B	=	1000	mm	H	=	300	mm		d'	=	56.5	mm			
M_u	=	57.005	kN·m	V_u	=	117.926	kN	M_o	=	37.147	kN·m				

- Check of Strength reduction factor (Φ)

a = 12.542

Because T = C , *c* = 12.542 / β1 = 12.542 / 0.821 = 15.268 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (243.5 - 15.268) / 15.268

= 0.0448

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.₮ ТФГТУКУФУZТ НПΣ И□.₮

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 631.992 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	6051.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00346	㎟	$A_{s,4/3req}$	=	842.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00337	㎟	$A_{s,min}$	=	819.9	㎟
P_{use}	=	A_s / (B·d)	=	0.00353	㎟	$A_{s,min}$	=	860.0	㎟

ㄴ Pmin ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 12.542 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 77.118 kN·m > Mu = 57.005 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 172.181 kN > Vu = 117.926 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 860.00 / 1000 + 8 × 860.00 / 1000 × √[1 + 2 × 1000 × 243.5 / (8 × 860.00)]

= 51.411 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 37.147 / [1000 × 51.411 x (243.5 - 51.411 / 3)] × 10^6

= 6.384 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 37.147 / [860.000 x (243.5 - 51.411 / 3)] × 10^6

= 190.820 MPa

fst = fs x (H - d' - χ) / (d - χ) = 191 x (300 - 57 - 6) / (244 - 51) = 190.82 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 190.82) - 2.5 x 50.00 = 432.59 ㎜

300 x (280 / fs) = 300 x (280 / 190.82) = 440.21 ㎜

Sa = 432.59 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (432.59 mm) Ā O.K

11. Middle Wall - Top & Bottom

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	213.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	86.5	mm	
M_u	=	0.000	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (Φ)

a = 6.271

Because T = C , *c* = 6.271 / β1 = 6.271 / 0.821 = 7.634 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (213.5 - 7.634) / 7.634

= 0.0809

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ̈ ТФГТYKYФYZT НПΣ И□ ̈

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 0.000 ㎠

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ㎠ (3 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	5305.5	㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	718.9	㎠
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00000	㎠	$A_{s,4/3req}$	=	0.0	㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00000	㎠	$A_{s,min}$	=	0.0	㎠
P_{use}	=	$A_s / (B \cdot d)$	=	0.00201	㎠	$A_{s,min}$	=	430.0	㎠

ᴖ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ᴖ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 6.271 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 34.193 kN·m > Mu = 0.000 kN·m

Ā O.K

ㅿ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 103.827 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 430.00 / 1000 + 8 × 430.00 / 1000 × √[1 + 2 × 1000 × 213.5 / (8 × 430.00)]

= 35.040 ㄱ

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.000 / [1000 × 35.040 x (213.5 - 35.040 / 3)] × 10^6

= 0.000 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.000 / [430.000 x (213.5 - 35.040 / 3)] × 10^6

= 0.000 MPa

fst = fs x (H - d' - χ) / (d - χ) = 0 x (300 - 87 - 0) / (214 - 35) = 0.00 MPa

ㅿ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 80.00 ㄱ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 8.18E+14 ㄱ

300 x (280 / fs) = 300 x (280 / 0.00) = 6.46E+14 ㄱ

Sa = 6.46125E+14 ㄱ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (6.46125E+14 mm) Ā O.K

1.1.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Wall (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

3) Bottom Slab (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

4) Middle Wall (H = 300 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

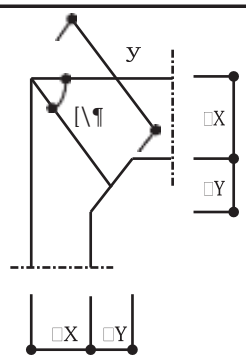
· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	540.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

1.1.9 Corner Design

1) Top slab Cheak



a1	=	300	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 28.625 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 565.7 \text{ mm}$$

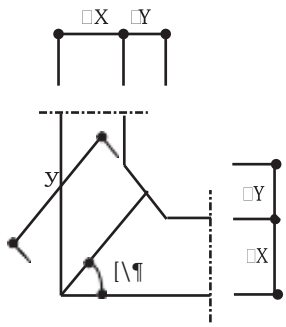
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 28.625 \times 10^6}{565.7^2 \times 1000} = 0.447 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.447 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	300	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm
f_{sa}	=	180	MPa

M_o = 33.045 kN·m

R = □$(a1^2 + a2^2)$ = 424.3 mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R^2 \cdot w} = \frac{5 \times 33.045 \times 10^6}{424.3^2 \times 1000} = 0.918 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.918 > 0.13 √fc' = 0.735 Ā Need reinforcements

$$As = \frac{2 \cdot Mo}{R \cdot fsa} = \frac{2 \times 33.045 \times 10^6}{424.3 \times 180} = 865.3 \text{ ㎡}$$

Use As :	D	13	-	1	ea	c.t.c	150	=	860.0 ㎡
(Main Reinforcment)	D	13	-	1	ea	c.t.c	300	=	430.0 ㎡

As = 1290 ㎡ < As,req = 865.3 ㎡ Ā O.K

1.2 Box Culvert 1 (STA.2+423.00) skew

2@2.7x2.00

FH=1.25 m [SI UNIT]

1.2.1 Design Conditions

This calculation covers the skewed end part of culvert.

1) General Items

(1) Type of Culvert : 2 Box

(2) Width (w) : 2 @ 2.7 m

(3) Height (h) : 2.00 m

(4) Underground Water Level : GL -1.000 m

2) Design Material

(1) Concrete

£ Compressive Strength	:	fc'	=	32	MPa
¤ Modulus of Elasticity	:	Ec	=	26587	MPa

(2) Reinforcement bar

⊳ Yield Strength	:	fy	=	420	MPa
⊲ Modulus of Elasticity	:	Es	=	200000	MPa

3) Material weight

(1) Reinforced Concrete	:	wc	=	25.00	kN/m3
(2) plain concrete	:	ycn	=	23.50	kN/m3
(3) Pavement	:	yasp	=	23.00	kN/m3
(4) Subterranean	:	yw	=	10.00	kN/m3

4) Soil

(1) Wet Unit Weight	:	yt	=	19.00	kN/m3
(2) Submerged Unit Weight	:	ysub	=	10.00	kN/m3
(3) angle of internal friction	:	cp	=	28.00	0
(4) coefficient of earth pressure atrest	:	Ko	=	1-sincD	= 0.500

5) Live Load

Structure is to be designed by SM1600 traffic design loads in accordance with AS 5100.2

6) Method of Design

(1) Evaluation of stability : Allowable Strength Method

(2) Design of Cross Section : Ultimate Strength Design

7) Program (S/W)

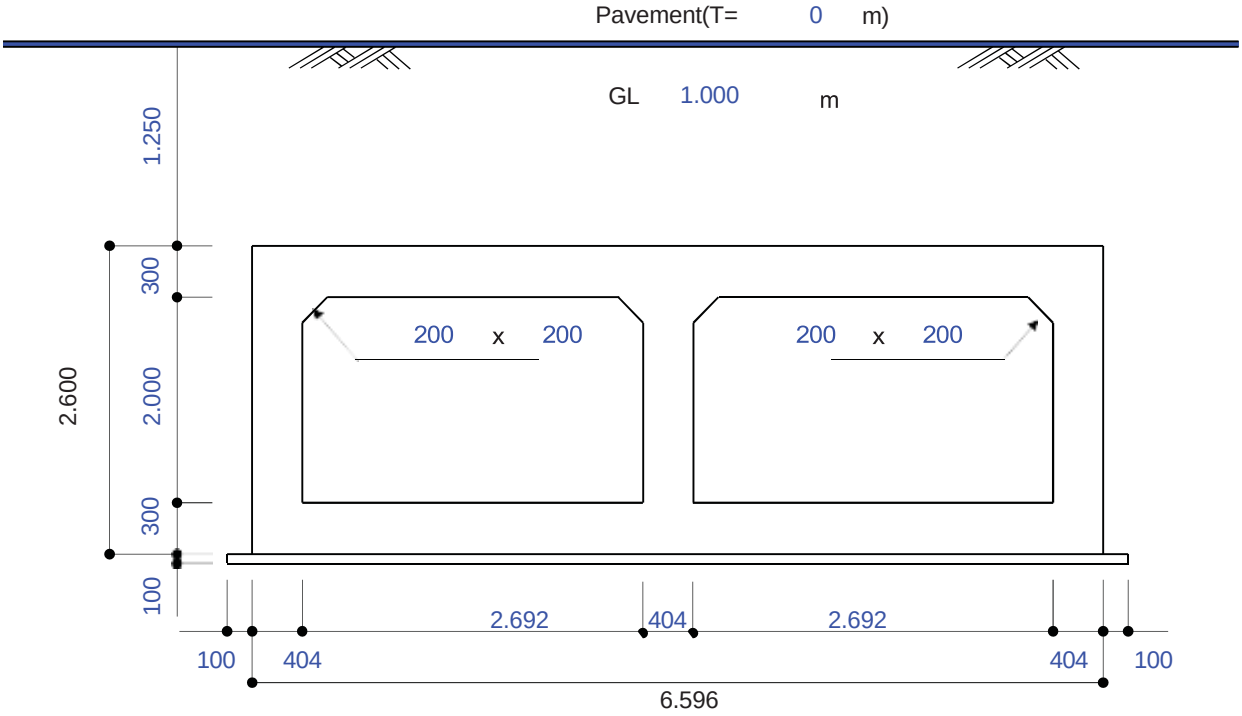
· SAP2000 (Structure Analysis Program)

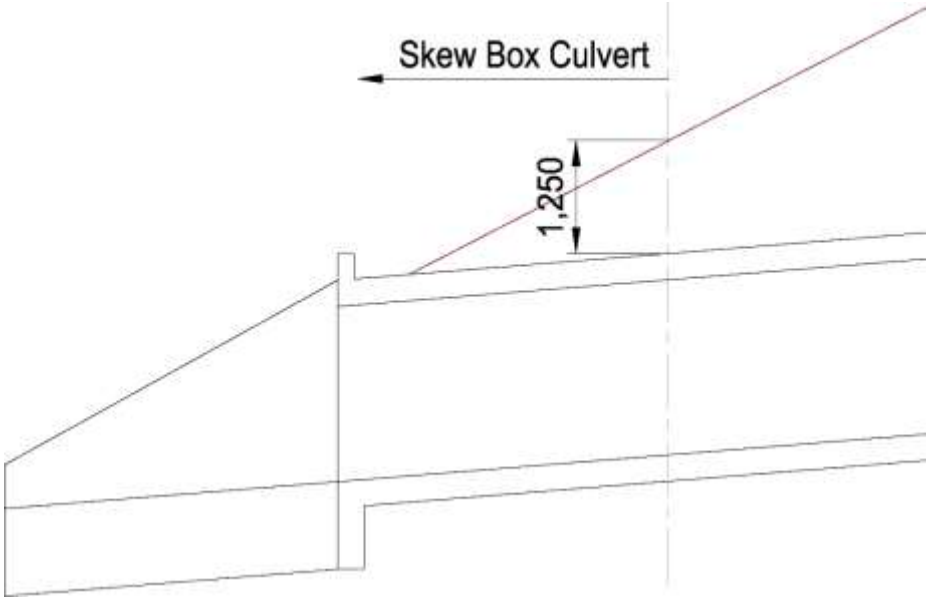
8) Reference

(1) American Concrete Institute - Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.2.2 Section Assumption





1.2.3 Stability Check

1) Load Summary and combinations

(1) Load Summary

Type		Calculation	Load(kN)
Paverment(DC)		0.000 x 6.596 x 23.0	0.000
Vertical earth pressure (EV)	No exist ground water	1.250 x 6.596 x 19.0	156.655
	Exist ground water	(1.000 x 19.0 + 0.250 x 10.0) x 6.596	141.814
Ground Water(WA')		0.250 x 6.596 x 10.0	16.490
Sub Total		Surcharge Load for Bouyancy Check	158.304
Slab(DC)	Top	0.300 x 6.596 x 25.0	49.470
	Bottom	0.300 x 6.596 x 25.0	49.470
Wall(DC)	Left	0.404 x 2.000 x 25.0	20.200
	Right	0.404 x 2.000 x 25.0	20.200
	Inner	0.404 x 2.000 x 25.0	20.200
Hunch(DC)		0.200 x 0.200 / 2 x 25.0 x 4 EA	2.000
Sub Total		Surcharge Load for Bouyancy Check	161.540

2) Bouyancy Check

(1) After construction (Ground water Level :GL- 1.00 m)

- Total Load for Bouyancy Check : 319.844 kN
- Uplife force : 6.596 x(3.850 - 1.000)x 10.0 kN/□ = 187.986 kN
- Safety factor = 1.25

□ F.S = 319.844 / 187.986 = 1.701 > 1.25 - O.K

(2) Under construction (Assumed Ground water Level :GL- 3.000 m)

- Total Load for Bouyancy Check :

161.540 +(0.000 x 6.596 x 10.000 kN/□) = 161.540 kN

- Uplife force : 6.596 x(3.850 - 3.000)x 10.0 kN/□ = 56.066 kN
- Safety factor = 1.1

□ F.S = 161.540 / 56.066 = 2.881 > 1.1 - O.K

Ā Ground water level should not exceed GL- 3.000m

3) Allowable vertical bearing capacity check

(1) Load

- Dead load
  - Self weight of Structure = 161.540 / 6.596 = 24.491 kN/m2
  - Vertical earth pressure = 156.655 / 6.596 = 23.750 kN/m2 (No exist ground water)
- Live load = 0.000 kN/m2 (Refer to 1.1.4.2)
- Water load in Culvrt = 2.000 ɜ 10.000 = 20.000 kN/m2

(2) Allowable vertical bearing capacity

- Q_{max} = 68.241 kN/m2
- Q_a = 337.500 kN/m2 (Refer to Geotechnic Report)

□ Q_a = 337.500 kN/□ > Q_{max} = 68.241 kN/□ - 0.K

1.2.4 Load and Combination

1) Dead Load

(1) Self weight : Automatic consideration in program

(2) Vertical earth pressure

- No exist ground water

TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m2)	
Pavement	0.000	23.000	1.000 x 0.000 x 23.000 =	0.000
Vertical earth pressure	1.250	19.000	1.000 x 1.250 x 19.000 =	23.750
□	1.250		Psv =	23.750 kN/m2

- Exist ground water

TYPE	Depth (m)	Unit weight (kN/㎥)	Load (kN/m2)	
Pavement	0.000	23.000	1.000 x 0.000 x 23.000 =	0.000
Vertical earth pressure	1.000	19.000	1.000 x 1.000 x 19.000 =	19.000
	0.250	10.000	1.000 x 0.250 x 10.000 =	2.500
□	1.250		Psvh =	21.500 kN/m2

2) Lateral Earth Pressure

↳ coefficient of earth pressure at rest : Ko = 1 - sin30 = 0.500

- No exist ground water

$P_{sh} = k_o \times y_t \times H$

P_{sh1} = 0.500 x (23 x 0.000 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 1.250) = 11.875 kN/m2

P_{sh2} = 11.875 + 0.500 x 19.0 x 0.150 = 13.300 kN/m2

P_{sh3} = 13.300 + 0.500 x 19.0 x 0.350 = 16.625 kN/m2

P_{sh4} = 16.625 + 0.500 x 19.0 x 0.900 = 25.175 kN/m2

P_{sh5} = 25.175 + 0.500 x 19.0 x 0.900 = 33.725 kN/m2

P_{sh6} = 33.725 + 0.500 x 19.0 x 0.150 = 35.150 kN/m2

- Exist ground water

$P_{sh'} = k_o \times (y_t \times H_1 + y_{sub} \times H_2)$

$P_{sh1'}$ = 0.500 x (23 x 0.000 + 23 x 0.000 + 20 x 0.000 + 20 x 0.000 + 19 x 1.000 + 10 x 0.250) = 10.750 kN/m2

$P_{sh2'}$ = 10.750 + 0.500 x 10.0 x 0.150 = 11.500 kN/m2

$P_{sh3'}$ = 11.500 + 0.500 x 10.0 x 0.350 = 13.250 kN/m2

$P_{sh4'}$ = 13.250 + 0.500 x 10.0 x 0.900 = 17.750 kN/m2

$P_{sh5'}$ = 17.750 + 0.500 x 10.0 x 0.900 = 22.250 kN/m2

$P_{sh6'}$ = 22.250 + 0.500 x 10.0 x 0.150 = 23.000 kN/m2

3) Ground Water Load

1) Horizontal ground Water Pressure

P_{wh} = y_w x H_2

P_{wh1}	=	10.0	x	0.250			=	2.500 kN/m2
P_{wh2}	=	2.500	+	10.0	x	0.150	=	4.000 kN/m2
P_{wh3}	=	4.000	+	10.0	x	0.350	=	7.500 kN/m2
P_{wh4}	=	7.500	+	10.0	x	0.900	=	16.500 kN/m2
P_{wh5}	=	16.500	+	10.0	x	0.900	=	25.500 kN/m2
P_{wh6}	=	25.500	+	10.0	x	0.150	=	27.000 kN/m2

2) Vertical ground Water Pressure

- Top Slab :	P_{wv1}	=	10.0	x	0.250		=	2.500 kN/m2
-Bottom Slab :	P_{wv2}	=	10.0	x	2.850		=	28.500 kN/m2

4) Load Combination

(1) Ultimate Load

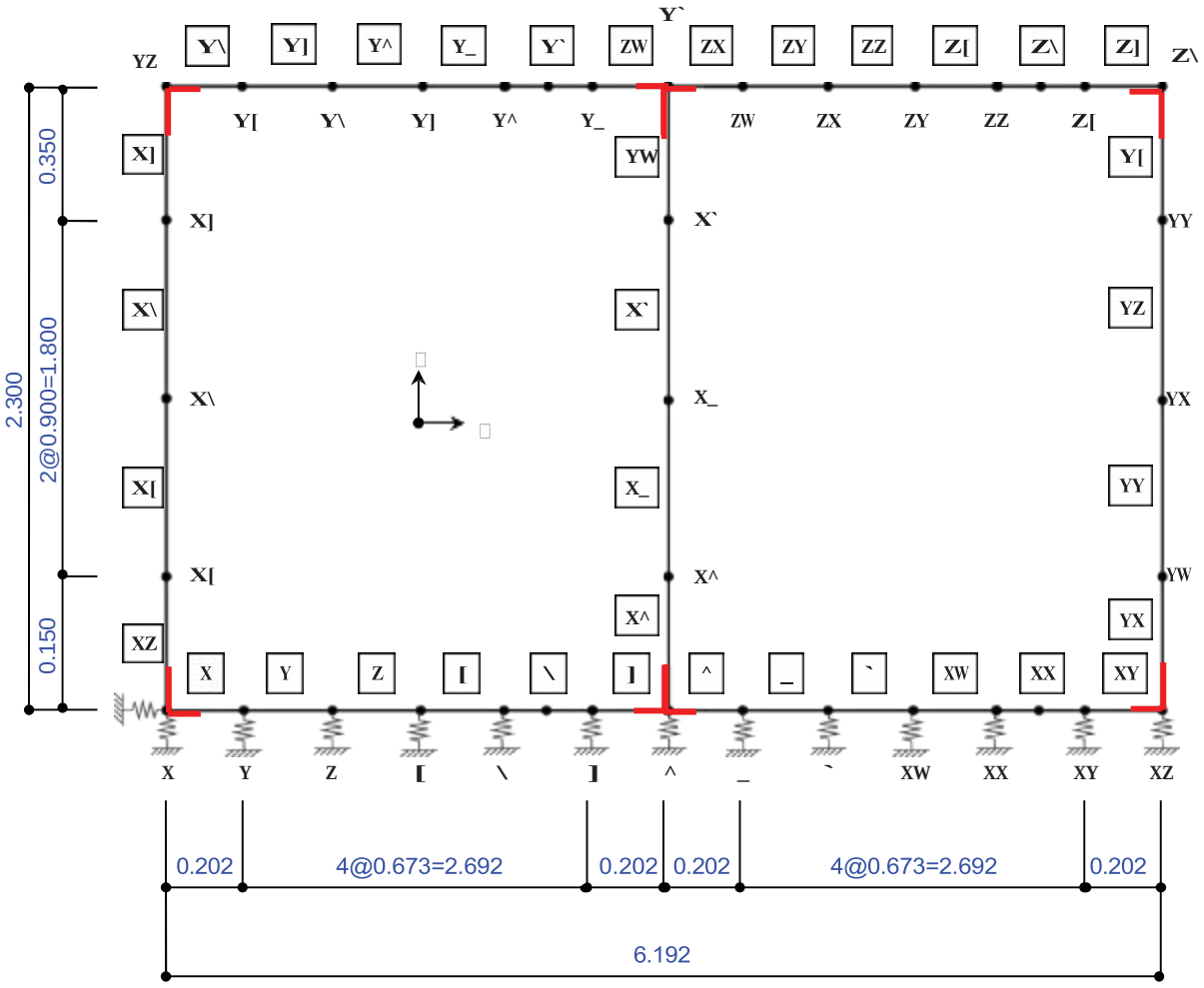
	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
COMB 1	1.40	1.40							
COMB 2	1.20	1.60		1.60	1.60	1.60			
COMB 3	1.20	1.60		1.60	1.60	0.90			
COMB 4	0.90	0.90				0.00			
COMB 5	1.40		1.40						1.40
COMB 6	1.20		1.60	1.60	1.60		1.60	1.60	1.60
COMB 7	1.20		1.60	1.60	1.60		0.90	0.90	1.60
COMB 8	0.90		0.90				0.00	0.00	0.00
COMB 9	1.20			1.00	1.00				
COMB 10	0.90	0.90				0.80			
COMB 11	0.90		0.90				0.00	0.00	0.00

(2) Service Load

	DEAD	USAT DEAD	SAT DEAD	LIVE	LIVE SOIL	USAT SOIL	SAT SOIL	WATER	UP WATER
SCOMB 1	1.00	1.000		1.00	1.00	1.00			
SCOMB 2	1.00	1.000		1.00	1.00	0.56			
SCOMB 3	1.00	1.000				0.00			
SCOMB 4	1.00		1.000	1.00	1.00		1.00	1.00	1.000
SCOMB 5	1.00		1.000	1.00	1.00		0.56	0.56	1.000
SCOMB 6	1.00		1.000				0.00	0.00	0.00

1.2.5 Modeling & Loading

1) Analysis Model



(1) Node

(Unit : m)

Node	X	z	Section	Node	X	z	Section
1	0.202	0.150	Bottom Slab	19	3.298	2.100	Middle Wall
2	0.404	0.150		20	6.394	0.300	Right Wall
3	1.077	0.150		21	6.394	1.200	
4	1.750	0.150		22	6.394	2.100	
5	2.423	0.150		23	0.202	2.450	Top Slab
6	3.096	0.150		24	0.604	2.450	
7	3.298	0.150		25	1.177	2.450	
8	3.500	0.150		26	1.750	2.450	
9	4.173	0.150		27	2.323	2.450	
10	4.846	0.150		28	2.896	2.450	
11	5.519	0.150		29	3.298	2.450	
12	6.192	0.150		30	3.700	2.450	
13	6.394	0.150		31	4.273	2.450	
14	0.202	0.300	Left Wall	32	4.846	2.450	
15	0.202	1.200		33	5.419	2.450	
16	0.202	2.100		34	5.992	2.450	
17	3.298	0.300	Middle Wall	35	6.394	2.450	
18	3.298	1.200					

(2) Section

N0.	H(m)	B(m)	A(m^2)	I(m^4)	Node	Section
1	0.300	1.000	0.300	0.002250	2~5 , 8~11	Bottom Slab
2	0.404	1.000	0.404	0.005495	14~15	Left Wall
3	0.404	1.000	0.404	0.005495	18~19	Middle Wall
4	0.404	1.000	0.404	0.005495	22~23	Right Wall
5	0.300	1.000	0.300	0.002250	26~29 , 32~35	Top Slab

2) Coefficient of subgrade reaction

(1) Vertical coefficient of subgrade reaction (Kv)

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

kvo = 1/0.3 xaxEo

Eo : the modulus of subgrade elasticity (kN/m2)

a : correction factor for calculating Eo

Eo = 7000 kN/m2 (Refer to Geotechnic Report)

a = 4

Kvo = 1/0.3 x a x Eo = 1/0.3 x 4 x 7000 = 93333.333 kN/m

$Bv = \sqrt{Av} = \sqrt{B \times B} = \sqrt{6.60 \times 6.60} = 6.596$ m2

$Kv = Kvo\ (Bv / 0.3)^{-3/4}$

$= 93333.333 \times (6.596 / 0.3)^{-3/4} = 9192.2$ kN/m

Joint No.	Kv	Lateral Length (m)	Longitudinal Length (m)	Area (m^2)	Coefficient of subgrade reaction (kN/m)
1, 13	9192.200	0.3030	1.0000	0.3030	2785.2
2, 12	9192.200	0.4375	1.0000	0.4375	4021.6
3~5, 9~11	9192.200	0.6730	1.0000	0.6730	6186.4
6, 8	9192.200	0.4375	1.0000	0.4375	4021.6
7	9192.200	0.2020	1.0000	0.2020	1856.8

(2) Horizontal coefficient of subgrade reaction (Kh)

kh = Infinite rigidity = 1.0E+10 kN/m

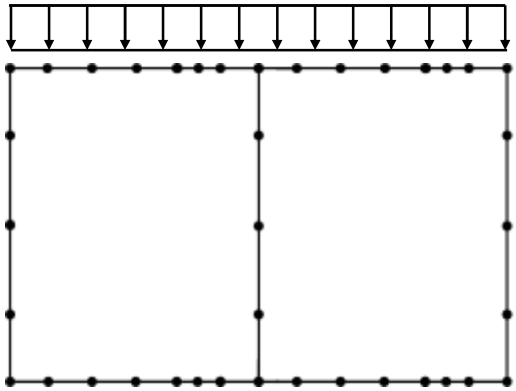
3) Loading

(1) L0AD-1 : Self weight - Automatic consideration in program

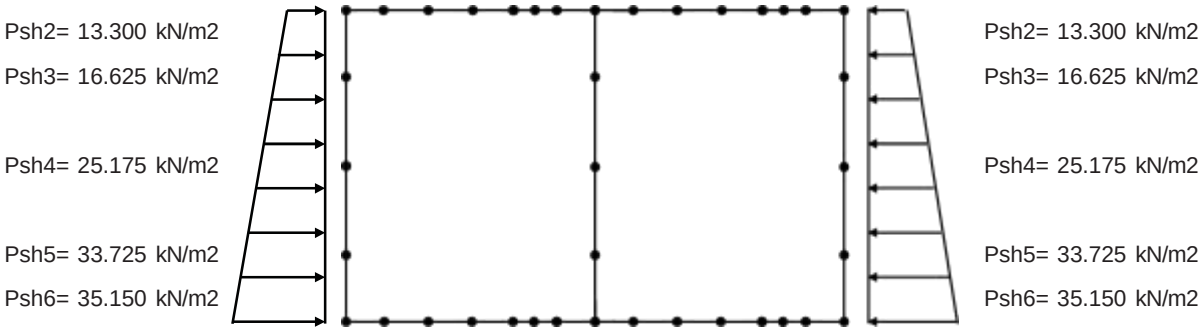
(2) L0AD-2,3 : Vertical earth pressure

Psvh= 21.500 kN/m2 (Exist ground water)

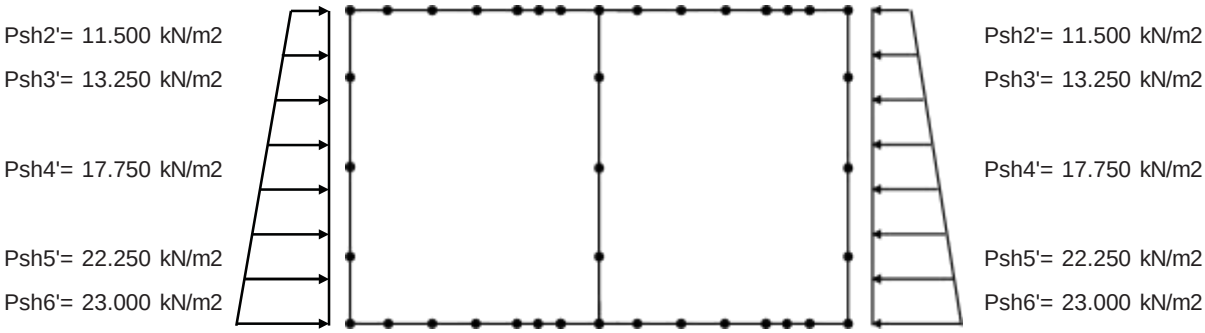
Psv= 23.750 kN/m2 (No exist ground water)



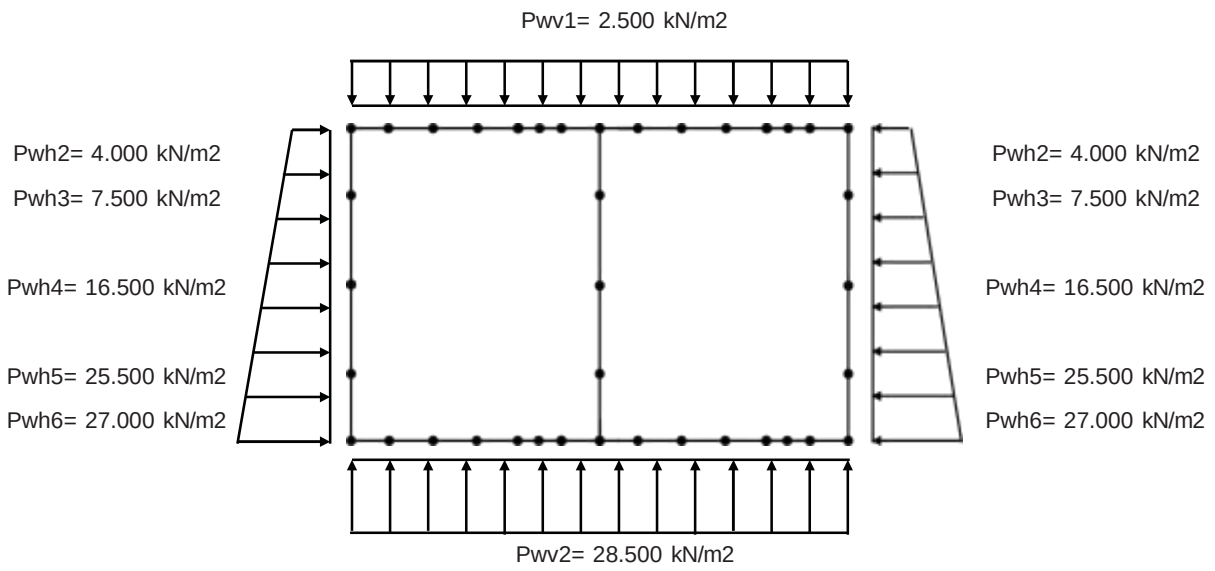
(3) L0AD-6 : Horizontal Earth Pressure (No Ground Water)



(4) L0AD-7 : Horizontal Earth Pressure (Ground Water)

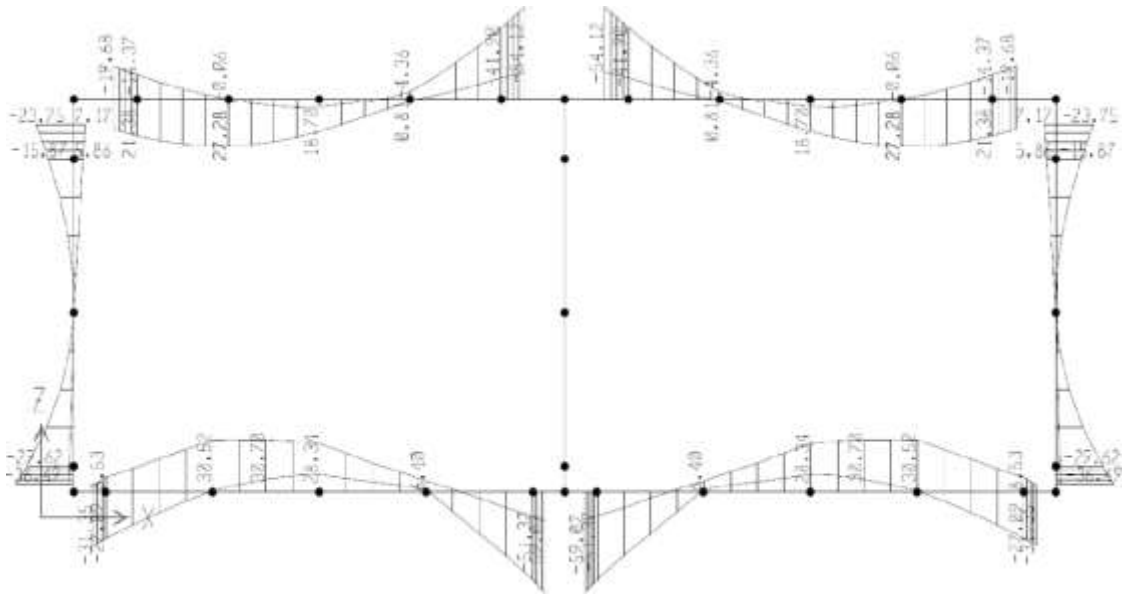


(5) L0AD-8 : Ground Water Pressure

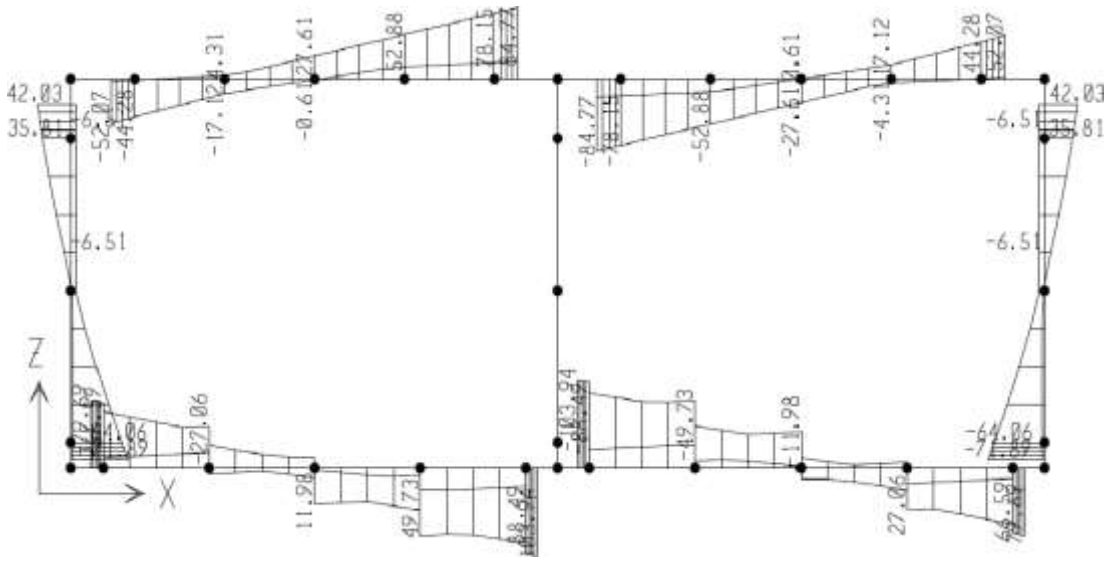


1.2.6 Summary of Analysis Results

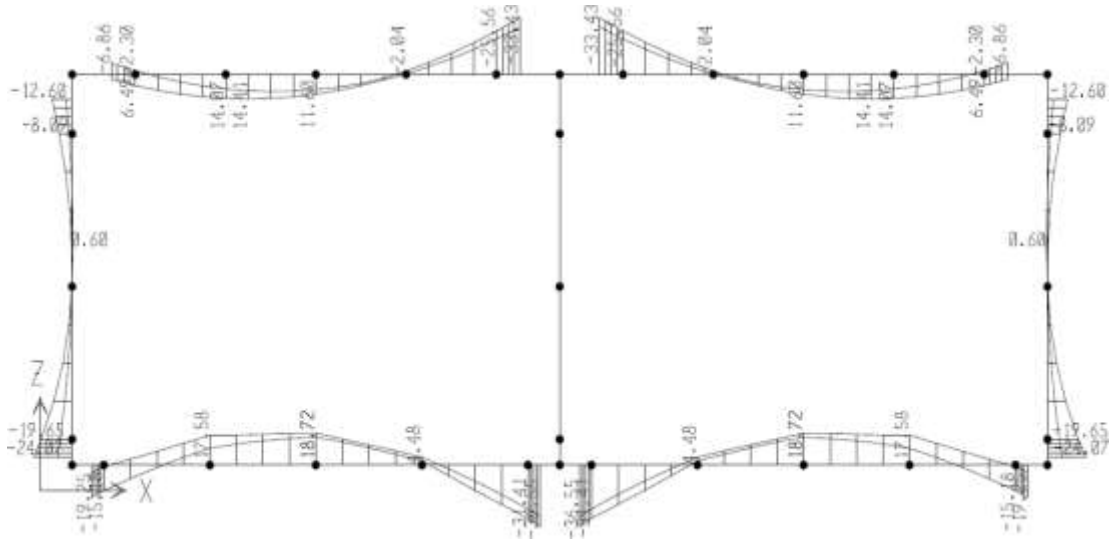
1) B.M.D (Ultimate Load) - Unit : kN.m



2) S.F.D (Ultimate Load) - Unit : kN



3) B.M.D (Service Load) - Unit : kN.m



4) Summary

Division		Mu(kN·m)	Vu(kN)	Mo(kN·m)	H(mm)	d(mm)	0Mn(kN·m)	Bar	S.F
Top Slab	End of the point(-)	19.682	52.070	6.863	367	284.5	90.447	D13 @ 150	4.60
	Middle(+)	27.281	0.000	14.410	300	187.5	58.914	D13 @ 150	2.16
	End of Middle Wall(-)	54.120	84.773	33.431	367	284.5	90.447	D13 @ 150	1.67
Wall	Top(-)	23.748	42.030	12.601	471	414.5	132.707	D13 @ 150	5.59
	Middle(+)	5.858	0.000	0.599	404	317.5	51.097	D13 @ 300	8.72
	Middle(-)	8.170	0.000	5.676	404	347.5	55.973	D13 @ 300	6.85
	Bottom(-)	36.487	64.055	24.072	404	347.5	110.927	D13 @ 150	3.04
Middle Wall	Top y Bottom(-)	0.000	0.000	0.000	404	317.5	51.097	D13 @ 300	-
Bottom Slab	End of the point(-)	31.353	66.591	19.245	300	217.5	68.666	D13 @ 150	2.19
	Middle(+)	30.704	0.000	18.719	300	187.5	58.914	D13 @ 150	1.92
	End of Middle Wall(-)	59.071	88.493	36.547	300	217.5	68.666	D13 @ 150	1.16

1.2.7 Section Design

1) Top Slab - At the end of the point

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	284.5	mm	
B	=	1000	mm	H	=	367	mm		d'	=	82.5	mm	
M_u	=	19.682	kN·m	V_u	=	52.070	kN	M_o	=		6.863	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ⓪1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (284.5 - 15.268) / 15.268

= 0.0529

Et > 0.0050 Tension-controlled sections <If = 0.900

a = A_s x f_y / (0 x $f_{c'}$ x b) ---------- (1)

M_u / 0 = A_s x f_y x (d - a / 2) ---------- (2)

И□_ТΦГТУКУΦУZТ НПΣ И□_Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 183.940 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	k_1 x 0 x ($f_{c'}$/f_y) x {600/(600 + f_y)}	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7069.8	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	958.0	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00086	㎟	$A_{s,4/3req}$	=	245.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00086	㎟	$A_{s,min}$	=	245.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00302	㎟	$A_{s,min}$	=	860.0	㎟

ⓐ 4/3 x Preq s Puse s Pmax Ā 0.K

ⓐ Bending Check

a = A_s x f_y / (0 x $f_{c'}$ x b) = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 90.447 kN·m > Mu = 19.682 kN·m

Ā 0.K

ⓐ Shear Check

0Vc = 0.75 x 1/6 x ✓ fc' x B x d = 154.031 kN > Vu = 52.070 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x ✓[1 + 2 x 1000 x 284.5 / (8 x 860.00)]

= 56.065 ㎜

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 6.863 / [1000 x 56.065 x (284.5 - 56.065 / 3)] x 10^6

= 0.921 MPa

f_s = M_o / [A_s x (d - x/3)]

= 6.863 / [860.000 x (284.5 - 56.065 / 3)] x 10^6

= 30.024 MPa

fst = fs x (H - d' - x) / (d - x) = 30 x (367 - 83 - 1) / (285 - 56) = 30.02 MPa

ⓐ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 76.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 30.02) - 2.5 x 76.00 = 3353.87 ㎜

300 x (280 / fs) = 300 x (280 / 30.02) = 2797.79 ㎜

Sa = 2797.79 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (2797.79 mm) Ā 0.K

2) Top Slab - Middle

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	187.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	112.5	mm	
M_u	=	27.281	kN·m	V_u	=	0.000	kN	M_o	=		14.410	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (187.5 - 15.268) / 15.268

= 0.0338

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ ТФГТУКУФУZT НПΣ ИЛ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 391.220 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	4659.4	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	631.3	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00278	㎟	$A_{s,4/3req}$	=	521.6	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00278	㎟	$A_{s,min}$	=	521.6	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00459	㎟	$A_{s,min}$	=	860.0	㎟

ᴖ 4/3 x Preq s Puse s Pmax Ā 0.K

ᴖ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 58.914 kN·m > Mu = 27.281 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x ✓ fc' x B x d = 132.583 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x ✓[1 + 2 x 1000 x 187.5 / (8 x 860.00)]

= 44.378 ㄴ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 14.410 / [1000 x 44.378 x (187.5 - 44.378 / 3)] x 10^6

= 3.760 MPa

f_s = M_o / [A_s x (d - x/3)]

= 14.410 / [860.000 x (187.5 - 44.378 / 3)] x 10^6

= 97.021 MPa

fst = fs x (H - d' - x) / (d - x) = 97 x (300 - 113 - 4) / (188 - 44) = 97.02 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 106.00 ㄴ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 97.02) - 2.5 x 106.00 = 831.67 ㄴ

300 x (280 / fs) = 300 x (280 / 97.02) = 865.79 ㄴ

Sa = 831.67 ㄴ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (831.67 mm) Ā 0.K

(3) Deflection Check

- Boundary condition : 0ne-way Slab, Both ends continuous
- Span : L = 6.596 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 x (0.43 + fy / 700) = 6.596 / 28 x (0.43 + 420 / 700)

= 0.243 m < H = 0.300 m Ā 0.K

3. Top Slab - At the end of middle Wall

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	284.5	mm	
B	=	1000	mm	H	=	367	mm		d'	=	82.5	mm	
M_u	=	54.120	kN·m	V_u	=	84.773	kN	M_o	=		33.431	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (284.5 - 15.268) / 15.268

= 0.0529

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 510.321

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7069.8	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	958.0	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00239	㎟	$A_{s,4/3req}$	=	680.4	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00239	㎟	$A_{s,min}$	=	680.4	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00302	㎟	$A_{s,min}$	=	860.0	㎟

4/3 x Preq s Puse s Pmax Ā 0.K

ᴖ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 90.447 kN·m > Mu = 54.120 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 154.031 kN > Vu = 84.773 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 284.5 / (8 x 860.00)]

= 56.065 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 33.431 / [1000 x 56.065 x (284.5 - 56.065 / 3)] x 10^6

= 4.487 MPa

f_s = M_o / [A_s x (d - x/3)]

= 33.431 / [860.000 x (284.5 - 56.065 / 3)] x 10^6

= 146.243 MPa

fst = fs x (H - d' - x) / (d - x) = 146 x (367 - 83 - 4) / (285 - 56) = 146.24 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 76.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 146.24) - 2.5 x 76.00 = 537.56 ㅡ

300 x (280 / fs) = 300 x (280 / 146.24) = 574.39 ㅡ

Sa = 537.56 ㅡ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (537.56 mm) Ā 0.K

4. Wall - Top

(1) Section Design

ᴖ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	414.5	mm	
B	=	1000	mm	H	=	471	mm		d'	=	56.5	mm	
M_u	=	23.748	kN·m	V_u	=	42.030	kN	M_o	=		12.601	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (414.5 - 15.268) / 15.268

= 0.0784

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 152.002

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ᴖ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	10300.3	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1395.7	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00049	㎟	$A_{s,4/3req}$	=	202.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00049	㎟	$A_{s,min}$	=	202.7	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00207	㎟	$A_{s,min}$	=	860.0	㎟

ᴖ 4/3 x Preq s Puse s Pmax Ā 0.K

ᴖ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 132.707 kN·m > Mu = 23.748 kN·m

Ā 0.K

ⓐ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 245.955 kN > Vu = 42.030 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 414.5 / (8 x 860.00)]

= 68.954 ㎜

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 12.601 / [1000 x 68.954 x (414.5 - 68.954 / 3)] x 10^6

= 0.934 MPa

f_s = M_o / [A_s x (d - x/3)]

= 12.601 / [860.000 x (414.5 - 68.954 / 3)] x 10^6

= 37.425 MPa

fst = fs x (H - d' - x) / (d - x) = 37 x (471 - 57 - 1) / (415 - 69) = 37.43 MPa

ⓐ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 37.43) - 2.5 x 50.00 = 2717.98 ㎜

300 x (280 / fs) = 300 x (280 / 37.43) = 2244.46 ㎜

Sa = 2244.46 ㎜ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (2244.46 mm) Ā 0.K

5. Wall - Middle(In)

(1) Section Design

ⱨ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	317.5	mm	
B	=	1000	mm	H	=	404	mm		d'	=	86.5	mm	
M_u	=	5.858	kN·m	V_u	=	0.000	kN	M_o	=		0.599	kN·m	

- Check of Strength reduction factor (cD)

a = 6.271

Because T = C , *c* = 6.271 / ◆1 = 6.271 / 0.821 = 7.634 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (317.5 - 7.634) / 7.634

= 0.1218

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 48.867

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ⊢ (3 ea/m)

ⱨ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	⊢	$A_{s,max}$	=	7889.9	⊢
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	⊢	$A_{s,min}$	=	1069.1	⊢
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00021	⊢	$A_{s,4/3req}$	=	65.2	⊢
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00021	⊢	$A_{s,min}$	=	65.2	⊢
P_{use}	=	$A_s / (B \cdot d)$	=	0.00135	⊢	$A_{s,min}$	=	430.0	⊢

ⱨ 4/3 x Preq s Puse s Pmax Ā 0.K

ⱨ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 6.271 mm

0Mn = 0.9 x As x fy x (d - a/2) = 51.097 kN·m > Mu = 5.858 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 224.506 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 430.00 / 1000 + 8 x 430.00 / 1000 x √[1 + 2 x 1000 x 317.5 / (8 x 430.00)]

= 43.424 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 0.599 / [1000 x 43.424 x (317.5 - 43.424 / 3)] x 10^6

= 0.091 MPa

f_s = M_o / [A_s x (d - x/3)]

= 0.599 / [430.000 x (317.5 - 43.424 / 3)] x 10^6

= 4.596 MPa

fst = fs x (H - d' - x) / (d - x) = 5 x (404 - 87 - 0) / (318 - 43) = 4.60 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 4.60) - 2.5 x 80.00 = 2.3E+04 ㅡ

300 x (280 / fs) = 300 x (280 / 4.60) = 1.8E+04 ㅡ

Sa = 1.83E+04 ㅡ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (1.8E+04 mm) Ā 0.K

6. Wall - Middle(Out)

(1) Section Design

ⱨ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	347.5	mm	
B	=	1000	mm	H	=	404	mm		d'	=	56.5	mm	
M_u	=	8.170	kN·m	V_u	=	0.000	kN	M_o	=		5.676	kN·m	

- Check of Strength reduction factor (cD)

a = 6.271

Because T = C , *c* = 6.271 / ◆1 = 6.271 / 0.821 = 7.634 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (347.5 - 7.634) / 7.634

= 0.1336

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Ţ ТФГТУКУФУZТ НПΣ ИЛ_Ţ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 62.283 ㎟

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ㎟ (3 ea/m)

ⱨ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8635.4	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1170.1	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00024	㎟	$A_{s,4/3req}$	=	83.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00024	㎟	$A_{s,min}$	=	83.0	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00124	㎟	$A_{s,min}$	=	430.0	㎟

ⱨ 4/3 x Preq s Puse s Pmax Ā 0.K

ⱨ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 6.271 mm

0Mn = 0.9 x As x fy x (d - a/2) = 55.973 kN·m > Mu = 8.170 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 245.720 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 430.00 / 1000 + 8 x 430.00 / 1000 x √[1 + 2 x 1000 x 347.5 / (8 x 430.00)]

= 45.577 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 5.676 / [1000 x 45.577 x (347.5 - 45.577 / 3)] x 10^6

= 0.750 MPa

f_s = M_o / [A_s x (d - x/3)]

= 5.676 / [430.000 x (347.5 - 45.577 / 3)] x 10^6

= 39.722 MPa

fst = fs x (H - d' - x) / (d - x) = 40 x (404 - 57 - 1) / (348 - 46) = 39.72 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 39.72) - 2.5 x 50.00 = 2553.60 ㅡ

300 x (280 / fs) = 300 x (280 / 39.72) = 2114.69 ㅡ

Sa = 2114.69 ㅡ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (2114.69 mm) Ā 0.K

7. Wall - Bottom

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	347.5	mm	
B	=	1000	mm	H	=	404	mm		d'	=	56.5	mm	
M_u	=	36.487	kN·m	V_u	=	64.055	kN	M_o	=		24.072	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (347.5 - 15.268) / 15.268

= 0.0653

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ⅊ ТФГТУКУФУZТ НПΣ И□ ⅊

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 279.511 ㎟

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8635.4	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1170.1	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00107	㎟	$A_{s,4/3req}$	=	372.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00107	㎟	$A_{s,min}$	=	372.7	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00247	㎟	$A_{s,min}$	=	860.0	㎟

ⓐ 4/3 x Preq s Puse s Pmax Ā 0.K

ⓐ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 110.927 kN·m > Mu = 36.487 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 245.720 kN > Vu = 64.055 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 347.5 / (8 x 860.00)]

= 62.611 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 24.072 / [1000 x 62.611 x (347.5 - 62.611 / 3)] x 10^6

= 2.354 MPa

f_s = M_o / [A_s x (d - x/3)]

= 24.072 / [860.000 x (347.5 - 62.611 / 3)] x 10^6

= 85.696 MPa

fst = fs x (H - d' - x) / (d - x) = 86 x (404 - 57 - 2) / (348 - 63) = 85.70 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 50.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 85.70) - 2.5 x 50.00 = 1116.61 ㅡ

300 x (280 / fs) = 300 x (280 / 85.70) = 980.21 ㅡ

Sa = 980.21 ㅡ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (980.21 mm) Ā 0.K

8. Bottom Slab - At the end of the point

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	217.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	82.5	mm	
M_u	=	31.353	kN·m	V_u	=	66.591	kN	M_o	=		19.245	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (217.5 - 15.268) / 15.268

= 0.0397

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$ □ Req. As = 386.665

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5404.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	732.4	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00237	㎟	$A_{s,4/3req}$	=	515.6	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00237	㎟	$A_{s,min}$	=	515.6	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00395	㎟	$A_{s,min}$	=	860.0	㎟

ⓐ 4/3 x Preq s Puse s Pmax Ā 0.K

ⓐ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 12.542 mm

0Mn = 0.9 x As x fy x (d - a/2) = 68.666 kN·m > Mu = 31.353 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 153.796 kN > Vu = 66.591 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 217.5 / (8 x 860.00)]

= 48.257 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 19.245 / [1000 x 48.257 x (217.5 - 48.257 / 3)] x 10^6

= 3.960 MPa

f_s = M_o / [A_s x (d - x/3)]

= 19.245 / [860.000 x (217.5 - 48.257 / 3)] x 10^6

= 111.106 MPa

fst = fs x (H - d' - x) / (d - x) = 111 x (300 - 83 - 4) / (218 - 48) = 111.11 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 76.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 111.11) - 2.5 x 76.00 = 767.64 ㅡ

300 x (280 / fs) = 300 x (280 / 111.11) = 756.03 ㅡ

Sa = 756.03 ㅡ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (756.03 mm) Ā 0.K

9. Bottom Slab - Middle

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	187.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	112.5	mm	
M_u	=	30.704	kN·m	V_u	=	0.000	kN	M_o	=		18.719	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (187.5 - 15.268) / 15.268

= 0.0338

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 441.227

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	4659.4	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	631.3	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00314	㎟	$A_{s,4/3req}$	=	588.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00314	㎟	$A_{s,min}$	=	588.3	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00459	㎟	$A_{s,min}$	=	860.0	㎟

ⓐ 4/3 x Preq s Puse s Pmax Ā 0.K

ⓐ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$	=	12.542	mm				
0Mn = 0.9 x As x fy x (d - a/2)	=	58.914	kN·m	>	Mu =	30.704	kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 132.583 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 187.5 / (8 x 860.00)]

= 44.378 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 18.719 / [1000 x 44.378 x (187.5 - 44.378 / 3)] x 10^6

= 4.885 MPa

f_s = M_o / [A_s x (d - x/3)]

= 18.719 / [860.000 x (187.5 - 44.378 / 3)] x 10^6

= 126.032 MPa

fst = fs x (H - d' - x) / (d - x) = 126 x (300 - 113 - 5) / (188 - 44) = 126.03 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 106.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 126.03) - 2.5 x 106.00 = 579.23 ㅡ

300 x (280 / fs) = 300 x (280 / 126.03) = 666.50 ㅡ

Sa = 579.23 ㅡ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (579.23 mm) Ā 0.K

(3) Deflection Check

- Boundary condition : 0ne-way Slab, Both ends continuous
- Span : L = 6.596 m
- Thickness : H = 0.300 m

ㄱ T_{min} = L / 28 x (0.43 + fy / 700) = 6.596 / 28 x (0.43 + 420 / 700)

= 0.243 m < H = 0.300 m Ā 0.K

10. Bottom Slab - At the end of middle Wall

(1) Section Design

Ᾰ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	217.5	mm	
B	=	1000	mm	H	=	300	mm		d'	=	82.5	mm	
M_u	=	59.071	kN·m	V_u	=	88.493	kN	M_o	=		36.547	kN·m	

- Check of Strength reduction factor (cD)

a = 12.542

Because T = C , *c* = 12.542 / ◆1 = 12.542 / 0.821 = 15.268 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (217.5 - 15.268) / 15.268

= 0.0397

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 737.824

Use As = D 13 @ 300 + D 13 @ 300 = 860.00 ㎟ (7 ea/m)

Ᾰ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	5404.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	732.4	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00452	㎟	$A_{s,4/3req}$	=	983.8	㎟
P_{min}	=	$\min (P_{min}, P_{4/3req})$	=	0.00337	㎟	$A_{s,min}$	=	732.4	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00395	㎟	$A_{s,min}$	=	860.0	㎟

Ᾰ Pmin s Puse s Pmax Ā 0.K

Ᾰ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$	=	12.542	mm				
0Mn = 0.9 x As x fy x (d - a/2)	=	68.666	kN·m	>	Mu =	59.071	kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 153.796 kN > Vu = 88.493 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 860.00 / 1000 + 8 x 860.00 / 1000 x √[1 + 2 x 1000 x 217.5 / (8 x 860.00)]

= 48.257 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 36.547 / [1000 x 48.257 x (217.5 - 48.257 / 3)] x 10^6

= 7.520 MPa

f_s = M_o / [A_s x (d - x/3)]

= 36.547 / [860.000 x (217.5 - 48.257 / 3)] x 10^6

= 210.989 MPa

fst = fs x (H - d' - x) / (d - x) = 211 x (300 - 83 - 8) / (218 - 48) = 210.99 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 76.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 210.99) - 2.5 x 76.00 = 314.29 ㅡ

300 x (280 / fs) = 300 x (280 / 210.99) = 398.13 ㅡ

Sa = 314.29 ㅡ Applying Minimum value

S = 1,000 / 7 Ea = 150.0 < Sa (314.29 mm) Ā 0.K

11. Middle Wall - Top & Bottom

(1) Section Design

ⱨ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Of	=	0.90		Ov	=	0.75			d	=	317.5	mm	
B	=	1000	mm	H	=	404	mm		d'	=	86.5	mm	
M_u	=	0.000	kN·m	V_u	=	0.000	kN	M_o	=		0.000	kN·m	

- Check of Strength reduction factor (cD)

a = 6.271

Because T = C , *c* = 6.271 / ◆1 = 6.271 / 0.821 = 7.634 mm

Ey = fy / Es = 420.000 / 200000 = 0.0021

Et = 0.0030 x (dt - c) / c = 0.003 x (317.5 - 7.634) / 7.634

= 0.1218

Et > 0.0050 Tension-controlled sections <If = 0.900

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ ---------- (1)

$M_u / 0 = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЛ_Ţ ТФГТУКУФУZТ НПΣ ИЛ_Ţ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{0} = 0$$

□ Req. As = 0.000 ㎟

Use As = D 13 @ 600 + D 13 @ 600 = 430.00 ㎟ (3 ea/m)

ⱨ Evaluation of reinforcement

P_b	=	$k_1 \times 0 \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7889.9	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1069.1	㎟
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00000	㎟	$A_{s,4/3req}$	=	0.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00000	㎟	$A_{s,min}$	=	0.0	㎟
P_{use}	=	$A_s / (B \cdot d)$	=	0.00135	㎟	$A_{s,min}$	=	430.0	㎟

ⱨ 4/3 x Preq s Puse s Pmax Ā 0.K

ⱨ Bending Check

$a = A_s \times f_y / (0 \times f_{c'} \times b)$ = 6.271 mm

0Mn = 0.9 x As x fy x (d - a/2) = 51.097 kN·m > Mu = 0.000 kN·m

Ā 0.K

ㅿ Shear Check

0Vc = 0.75 x 1/6 x √ fc' x B x d = 177.366 kN > Vu = 0.000 kN

Ā No shear reinforcement is required

(2) Crack Check

ㅿ Calculation of stress

n = 8

x = - nA_s/b + nA_s/b x □{1+2bd/nA_s}

= - 8 x 430.00 / 1000 + 8 x 430.00 / 1000 x √[1 + 2 x 1000 x 317.5 / (8 x 430.00)]

= 43.424 ㅡ

f_c = 2 x M_o / [B x x x (d - x/3)]

= 2.0 x 0.000 / [1000 x 43.424 x (317.5 - 43.424 / 3)] x 10^6

= 0.000 MPa

f_s = M_o / [A_s x (d - x/3)]

= 0.000 / [430.000 x (317.5 - 43.424 / 3)] x 10^6

= 0.000 MPa

fst = fs x (H - d' - x) / (d - x) = 0 x (404 - 87 - 0) / (318 - 43) = 0.00 MPa

ㅿ Maximum center space of reinforcement

C_C = 82.50 - 13.00 / 2 = 80.00 ㅡ

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 0.00) - 2.5 x 80.00 = 6.66E+14 ㅡ

300 x (280 / fs) = 300 x (280 / 0.00) = 5.25E+14 ㅡ

Sa = 5.25432E+14 ㅡ Applying Minimum value

S = 1,000 / 3 Ea = 300.0 < Sa (5.25432E+14 mm) Ā 0.K

1.2.8 Distribution Reinforcement Check

1) Top Slab (H = 300 mm)

· $A_{s,min}$ = (0.0018 x 420 / fy) x B x H = (0.0018 x 420 / 420) x 1000 x 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā 0.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā 0.K

2) Wall (H = 404 mm)

· $A_{s,min}$ = (0.0018 x 420 / fy) x B x H = (0.0018 x 420 / 420) x 1000 x 404 = 727.2 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	727.2 ㎟	Ā 0.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā 0.K

3) Bottom Slab (H = 300 mm)

· $A_{s,min}$ = (0.0018 x 420 / fy) x B x H = (0.0018 x 420 / 420) x 1000 x 300 = 540.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	540.0 ㎟	Ā 0.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā 0.K

4) Middle Wall (H = 404 mm)

· $A_{s,min}$ = (0.0018 x 420 / fy) x B x H = (0.0018 x 420 / 420) x 1000 x 404 = 727.2 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

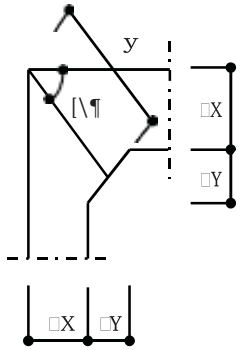
· Used As :

Tension side	D	13@	200	=	645.0	㎟				
Compression side	D	13@	200	=	645.0	㎟				
				□	=	1290.0	㎟	>	727.2 ㎟	Ā 0.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā 0.K

1.2.9 Corner Design

1) Top slab Cheak



a1	=	404	mm
a2	=	200	mm
b1	=	300	mm
b2	=	200	mm

M_o = 12.601 kN·m

$$R = \frac{a2 \cdot b2 + b2 \cdot a1 + a2 \cdot b1}{a2 + b2} \times \square 2 = 639.2 \text{ mm}$$

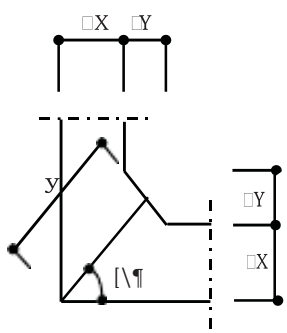
W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R2 \cdot w} = \frac{5 \times 12.601 \times 10A6}{639.22 \times 1000} = 0.154 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.154 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2) Bottom slab Cheak



a1	=	404	mm
a2	=	0	mm
b1	=	300	mm
b2	=	0	mm
f_{sa}	=	180	MPa

M_o = 24.072 kN·m

R = □(a12 + a22) = 503.2 mm

W = 1000 mm

$$f_{t.max} = \frac{5 \cdot M_o}{R2 \cdot w} = \frac{5 \times 24.072 \times 10A6}{503.22 \times 1000} = 0.475 \text{ MPa}$$

0.13 □fc' = 0.735 MPa

$f_{t.max}$ = 0.475 < 0.13 √fc' = 0.735 Ā No reinforcement is requireds

2 .Corrugated Steel Pipe

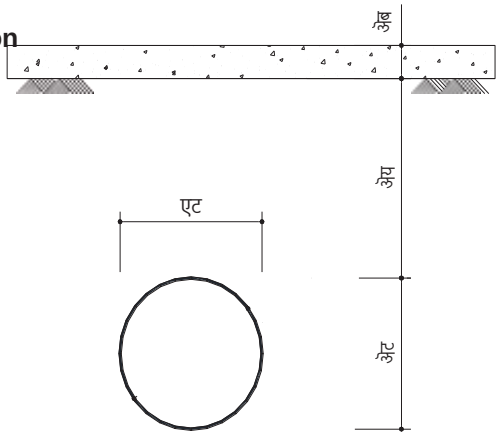
2.1 Corrugated Steel Pipe (D=1,200)

D=1.2M

H=4.85 m [SI UNIT]

2.1.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 4.850 m
- Pipe span : Sc = 1.200 m
- Corrugated steel pipe specifications
  - pitch x depth : **68x13** mm
  - thickness : **2.0** mm
  - yield strength : fy = 230 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

2.1.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523

2.1.3 Loads

1) Dead Load

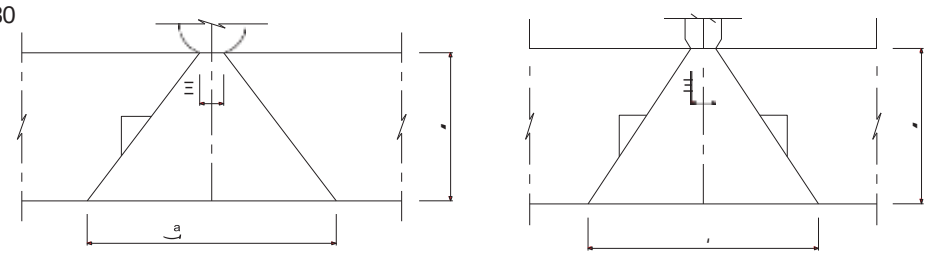
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 4.850 = 104.360 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pi
g = unit weight of the soil
H = height of cover over the pipe

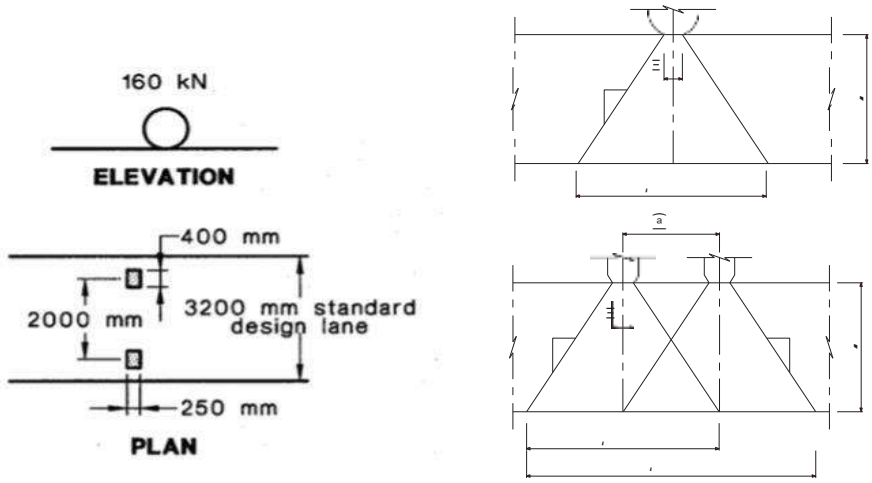
2) Live Load

(1) W80



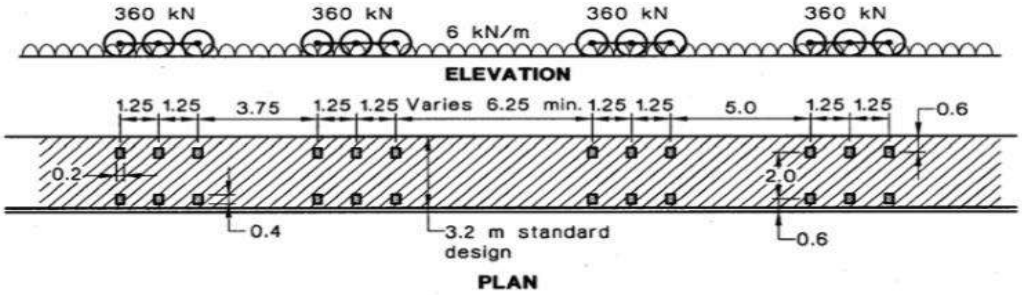
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 10.34) \times (0.4 + 10.34)} = 0.703 \text{ kN/m}^2$$

(2) A160

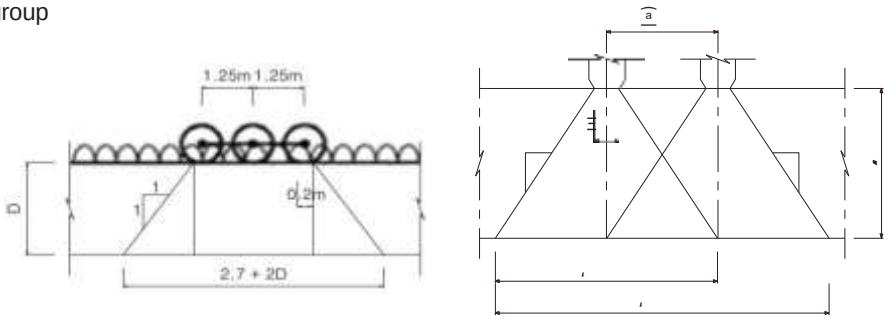


$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 10.34) \times (2.4 + 10.34)} = 1.186 \text{ kN/m}^2$$

(3) M1600



- Axle group

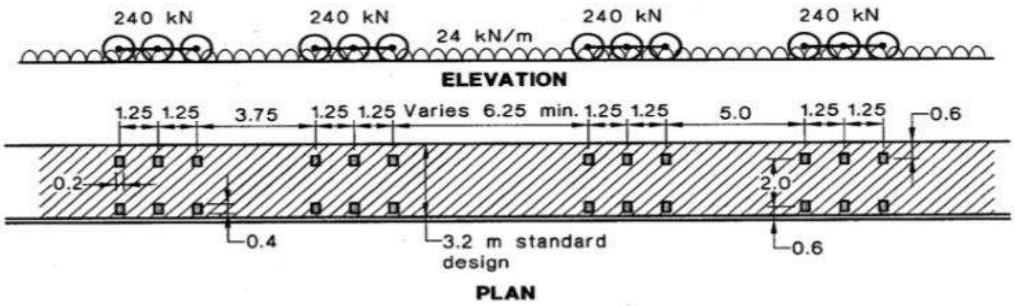


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 10.34) \times (2.4 + 10.34)} = 2.167 \text{ kN/m}^2$$

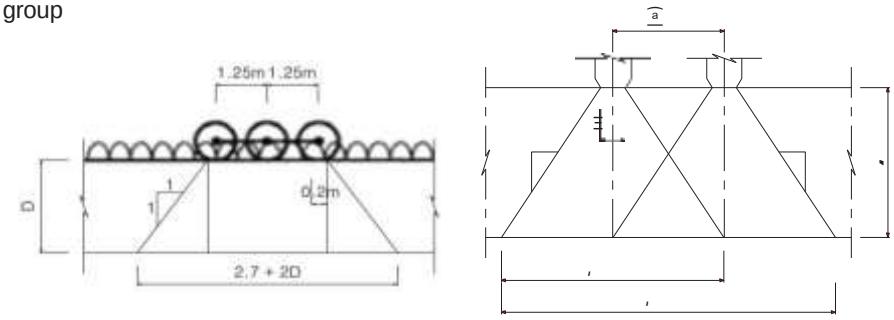
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 2.167 + 1.875 = 4.042 kN/m²

(4) S1600



- Axle group

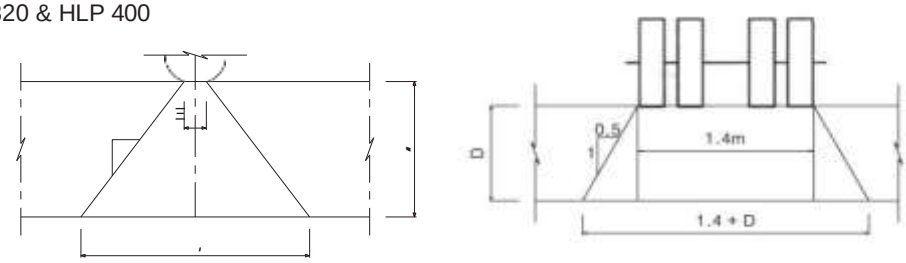


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 10.34) \times (2.4 + 10.34)} = 1.445 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 1.445 + 7.500 = 8.945 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 10.34) \times (1.4 + 10.34)} = 1.010 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	0.703	0.10	0.774
A160	1.186	0.10	1.305
M1600	4.042	0.10	4.446
S1600	8.945	0.00	8.945
HLP	1.010	0.10	1.111

□ LL = 8.945 kN/m² = 8.945 kN/m²

2.1.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min} = 900 ¸ < H = 5170 ¸ ĀO.K

2.1.5 Backfill compaction

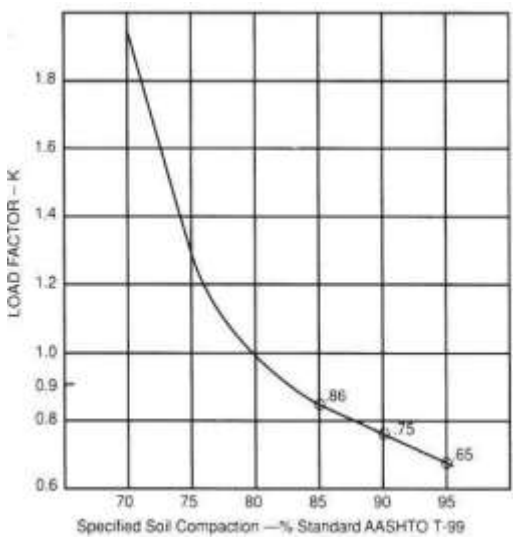
The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.
Therefore, the design assumes a backfill compaction densi 85%

2.1.6 Design Pressure

H = 5.2 m > S = 1.2 m

Pv = K (DL + LL), when H > S
= 0.86 x (104.36 + 8.945) = 97.442 kN/m²

where:
Pv = design pressure, kPa
K = 0.86 , load factor
DL = dead load
LL = live load
H = height of cover
S = span or diameter



2.1.7 Ring compression

C = Pv x S / 2 = 97.442 x 1.200 / 2 = 58.465 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

2.1.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 276 < 294
fb = 230 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 115.00 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 1200 mm span or diameter
r = 4.345 ฟ , radius of gyration of the pipe wall

2.1.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

ฦ = C / fc = 58.465 / 115.000 = 0.508 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.966 ฯ/ฟ > 0.508 ฯ/ฟ ĀO.K

2.1.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

FF = D^2 / EI = 0.194 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span
I = 37.11 ,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installati are as follows:

68x13	mm corrugation, FF	0.245 ฟ/N
125x25	mm corrugation,	0.188 ฟ/N
76x25	FF mm	0.188 ฟ/N
152x51	corrugation, FF	0.114 ฟ/N
	mm corrugation, FF	

2.1.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate se strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for t types of pipe

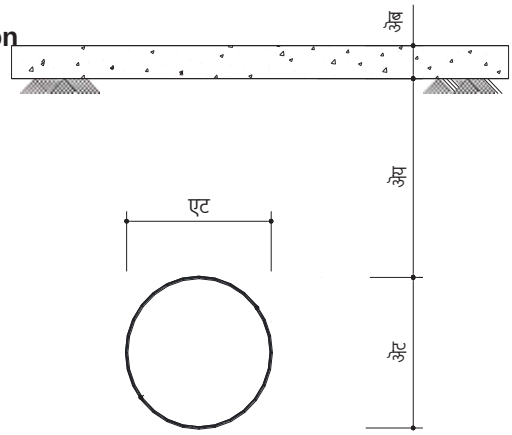
2.2 Corrugated Steel Pipe (D=1000)

D=1M

H=8.17 m [SI UNIT]

2.2.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 8.170 m
- Pipe span : Sc = 1.000 m
- Corrugated steel pipe specifications
  - pitch x depth : **68x13** mm
  - thickness : **1.6** mm
  - yield strength : fy = 230 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

2.2.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523

2.2.3 Loads

1) Dead Load

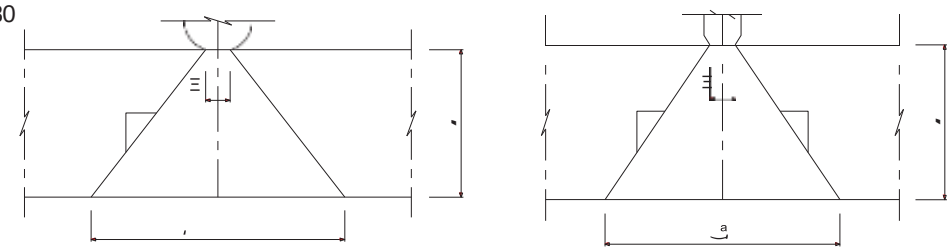
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 8.170 = 170.760 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pipe
g = unit weight of the soil
H = height of cover over the pipe

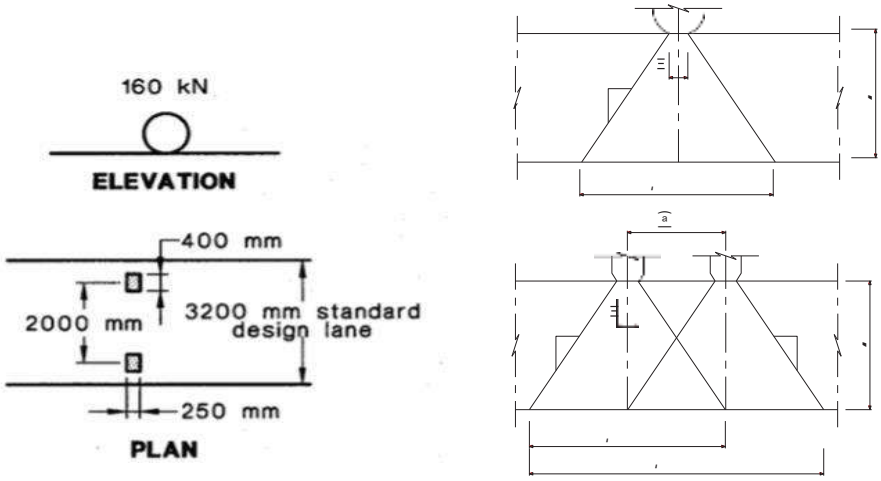
2) Live Load

(1) W80



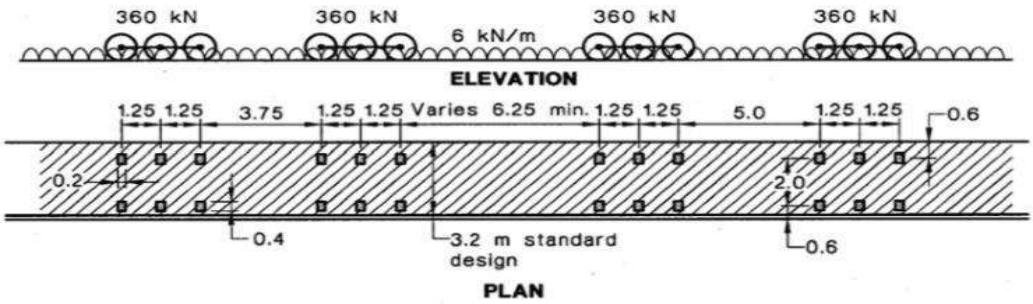
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 16.98) \times (0.4 + 16.98)} = 0.267 \text{ kN/m}^2$$

(2) A160

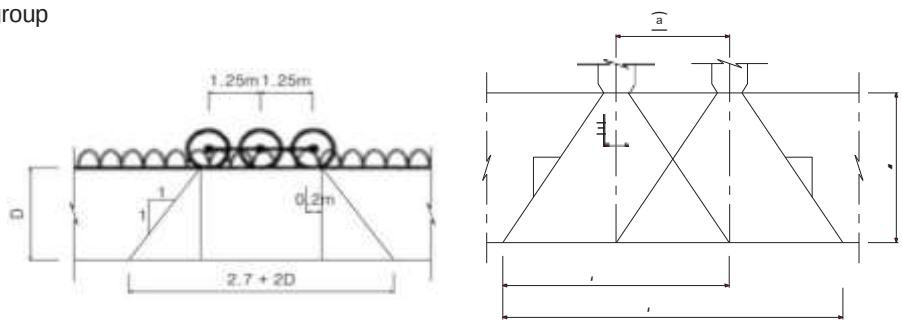


$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 16.98) \times (2.4 + 16.98)} = 0.479 \text{ kN/m}^2$$

(3) M1600



- Axle group

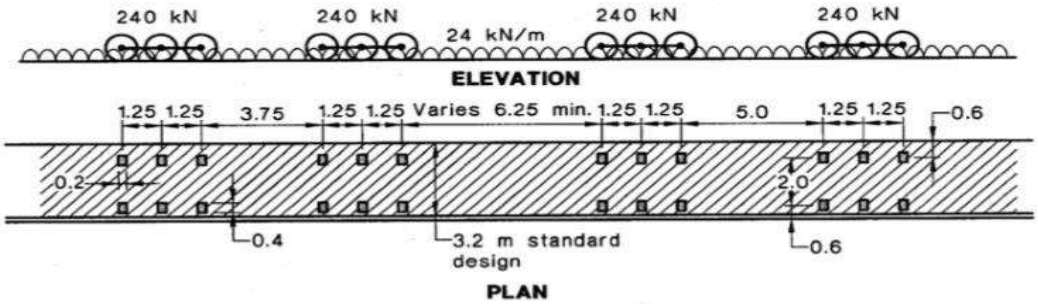


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 16.98) \times (2.4 + 16.98)} = 0.944 \text{ kN/m}^2$$

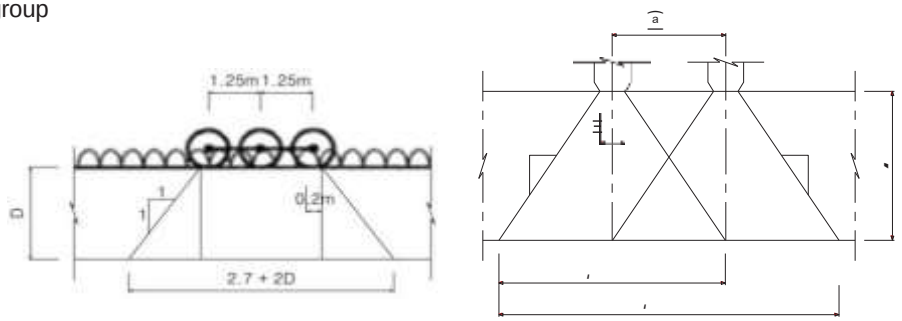
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 0.944 + 1.875 = 2.819 kN/m²

(4) S1600



- Axle group

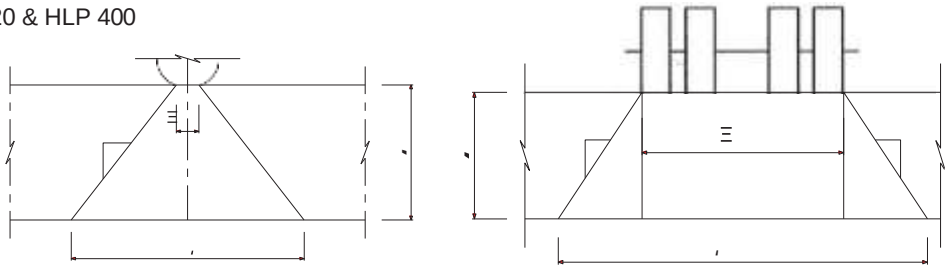


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 16.98) \times (2.4 + 16.98)} = 0.629 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 0.629 + 7.500 = 8.129 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 16.98) \times (1.4 + 16.98)} = 0.396 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	0.267	0.10	0.294
A160	0.479	0.10	0.527
M1600	2.819	0.10	3.101
S1600	8.129	0.00	8.129
HLP	0.396	0.10	0.435

□ LL = 8.129 kN/m² = 8.129 kN/m²

2.2.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min}= 600 ㎜ < H = 8490 ㎜ ĀO.K

2.2.5 Backfill compaction

The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.

Therefore, the design assumes a backfill compaction density 85%

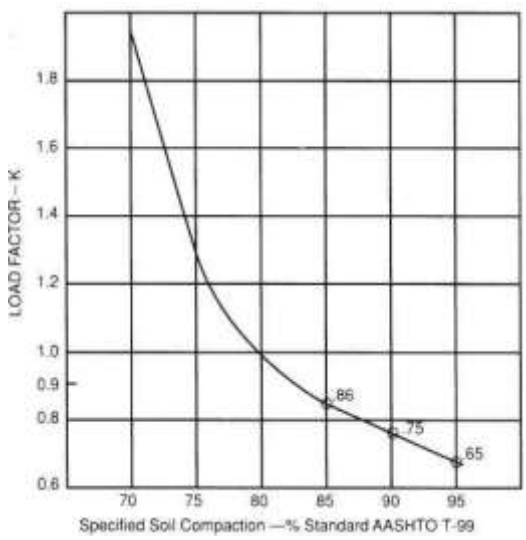
2.2.6 Design Pressure

H = 8.5 m > S = 1.0 m

Pv = K (DL + LL), when H > S

= 0.86 x (170.76 + 8.129) = 153.845 kN/m²

where:
- Pv = design pressure, kPa
- K = 0.86 , load factor
- DL = dead load
- LL = live load
- H = height of cover
- S = span or diameter



2.2.7 Ring compression

C = Pv x S / 2 = 153.845 x 1.000 / 2 = 76.922 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

2.2.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 231 < 294
fb = 230 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 115.00 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 1000 mm span or diameter
r = 4.332 ฟ , radius of gyration of the pipe wall

2.2.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

ฑ = C / fc = 76.922 / 115.000 = 0.669 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.512 ฯ/ฟ > 0.669 ฯ/ฟ ĀO.K

2.2.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

FF = D^2 / EI = 0.176 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span
I = 28.37 ,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installations are as follows:

68x13	mm corrugation, FF	0.245 พ/N
125x25	mm corrugation,	0.188 พ/N
76x25	FF mm	0.188 พ/N
152x51	corrugation, FF	0.114 พ/N
	mm corrugation, FF	

2.2.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate seam strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for the types of pipe

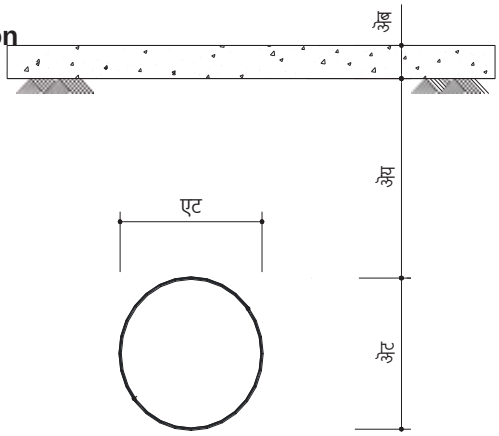
2.3 Corrugated Steel Pipe (D=900)

D=0.9M

H=1.34 m [SI UNIT]

2.3.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 1.016 m
- Pipe span : Sc = 0.900 m
- Corrugated steel pipe specifications
  - pitch x depth : **68x13** mm
  - thickness : **1.6** mm
  - yield strength : fy = 230 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

2.3.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523

2.3.3 Loads

1) Dead Load

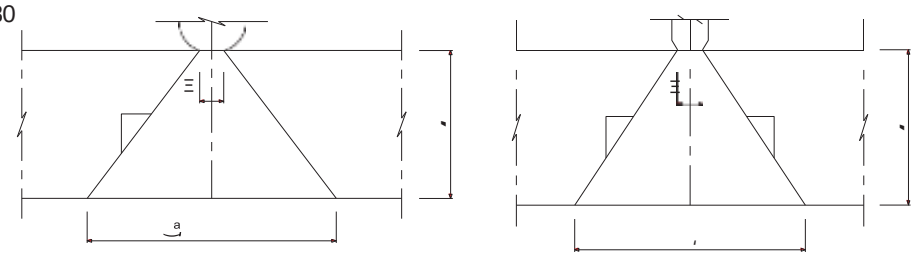
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 1.016 = 27.680 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pi
g = unit weight of the soil
H = height of cover over the pipe

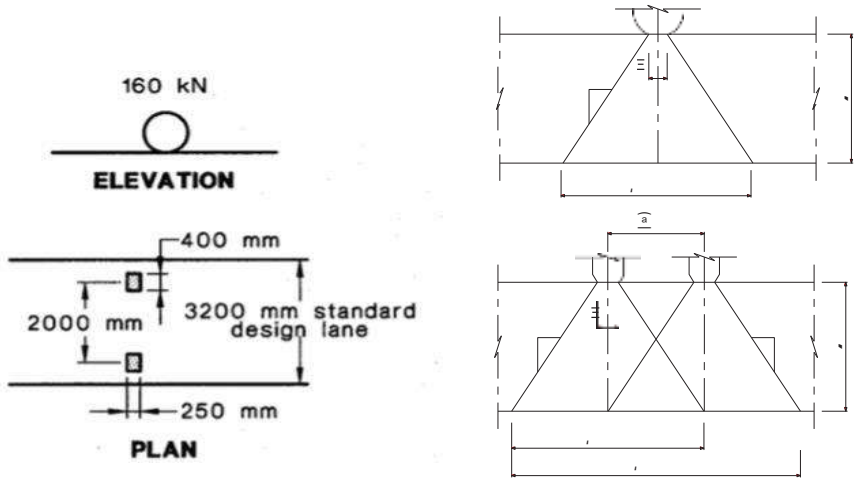
2) Live Load

(1) W80



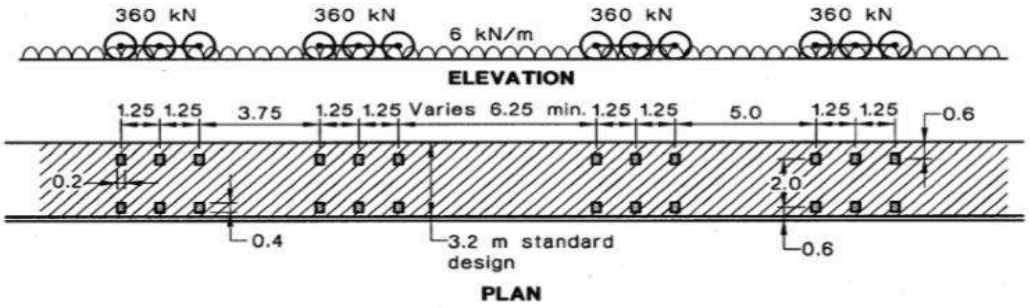
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 2.672) \times (0.4 + 2.672)} = 8.912 \text{ kN/m}^2$$

(2) A160

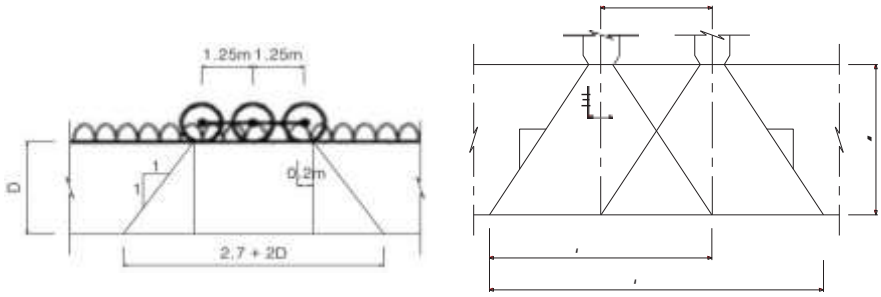


$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 2.672) \times (2.4 + 2.672)} = 10.796 \text{ kN/m}^2$$

(3) M1600



- Axle group

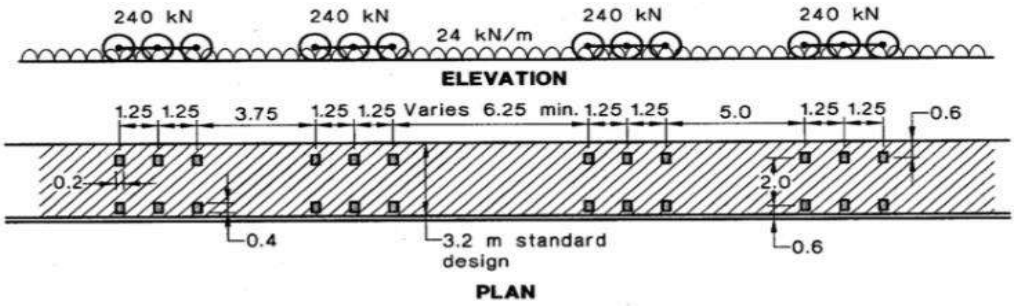


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 2.672) \times (2.4 + 2.672)} = 13.213 \text{ kN/m}^2$$

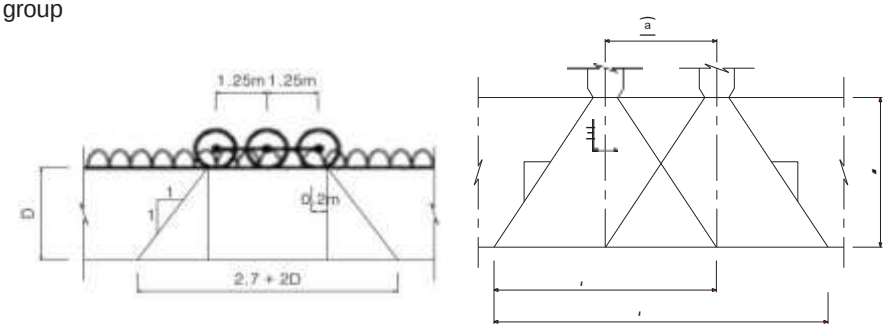
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 13.213 + 1.875 = 15.088 kN/m²

(4) S1600



- Axle group

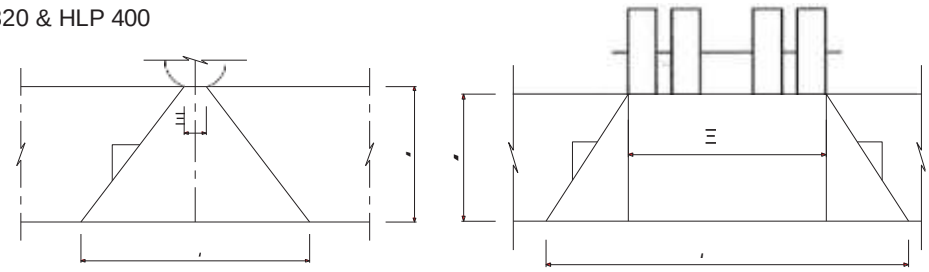


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 2.672) \times (2.4 + 2.672)} = 8.808 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 8.808 + 7.500 = 16.308 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2.672) \times (1.4 + 2.672)} = 10.689 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	8.912	0.20	10.691
A160	10.796	0.20	12.951
M1600	15.088	0.17	17.598
S1600	16.308	0.00	16.308
HLP	10.689	0.10	11.757

□ LL = 17.598 kN/m² = 17.598 kN/m²

2.3.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min} = 600 ¸ < H = 1336 ¸ ĀO.K

2.3.5 Backfill compaction

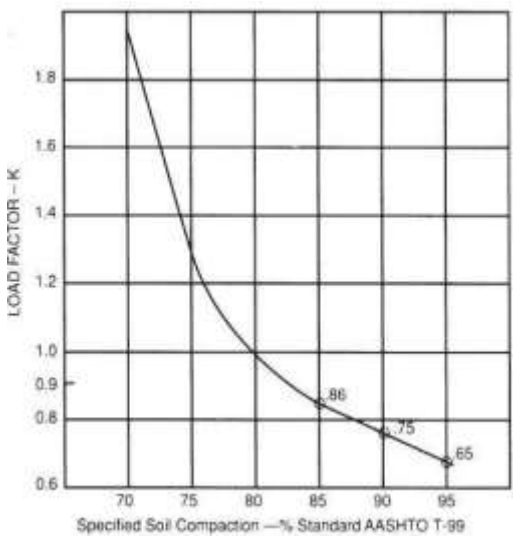
The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.
Therefore, the design assumes a backfill compaction densi 85%

2.3.6 Design Pressure

H = 1.3 m > S = 0.9 m

Pv = K (DL + LL), when H > S
= 0.86 x (27.68 + 17.598) = 38.939 kN/m²

where: Pv = design pressure, kPa
K = 0.86 , load factor
DL = dead load
LL = live load
H = height of cover
S = span or diameter



2.3.7 Ring compression

C = Pv x S / 2 = 38.939 x 0.900 / 2 = 17.523 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

2.3.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 208 < 294
fb = 230 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 115.00 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 900 mm span or diameter
r = 4.332 ฟ , radius of gyration of the pipe wall

2.3.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

ฦ = C / fc = 17.523 / 115.000 = 0.152 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.512 ฯ/ฟ > 0.152 ฯ/ฟ ĀO.K

2.3.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

FF = D^2 / EI = 0.143 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span
I = 28.37 ,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installati are as follows:

68x13	mm corrugation, FF	0.245 ห/N
125x25	mm corrugation,	0.188 ห/N
76x25	FF mm	0.188 ห/N
152x51	corrugation, FF	0.114 ห/N
	mm corrugation, FF	

2.3.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate se strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for t types of pipe

2.4 Head/Wing Wall (H=1.5m)

2.4.1 Design Conditions (H=1.500m , N= 1 : 2.00 , Ho= 6.370)

1) General Items

(1)	Type of WingWall	:	Cantilever Type	
(2)	Height of WingWall	:	1.500	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	6.370	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

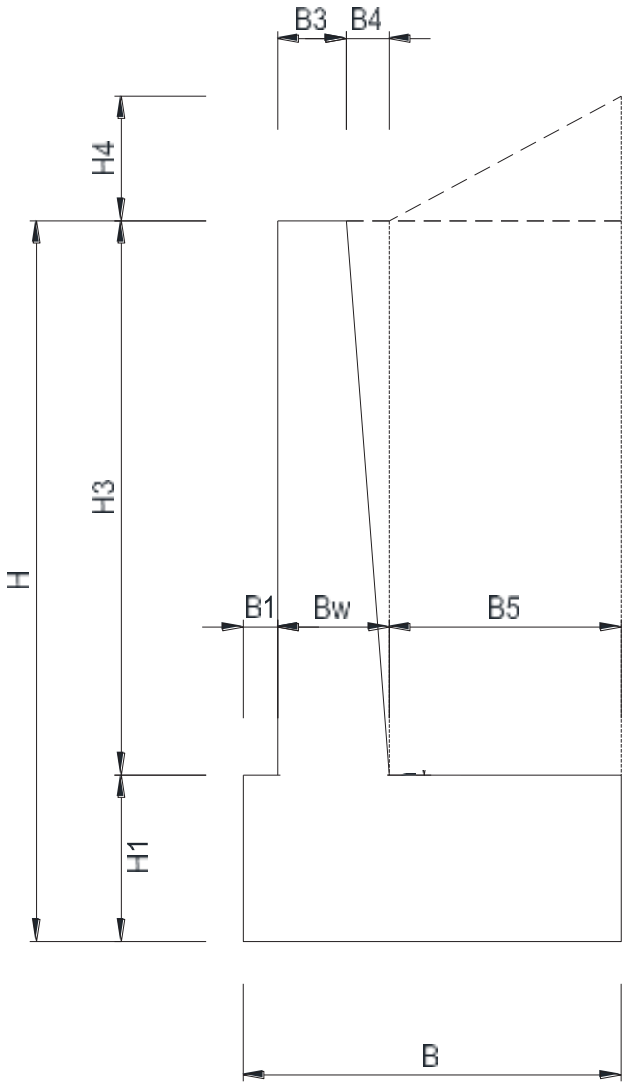
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

2.4.2 Section Assumption



Ŝ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.100	0.000	0.200	0.125	0.675	1.100	0.325

- Height

H1	H2	H3	H4	H	Ho	
0.450	0.000	1.500	0.338	1.950	6.370	

2.4.3 Load Calculation

1) Self weight (D)

Automatic consideration in program

2) Earth pressure

▷ At Normal (H)

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	4.764

∞ (δ = Λ· x Φ)

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
32.2	9.333	1.500	165.183	12.280	0.575	0.557	0.140
32.3	9.333	1.500	161.576	12.294	0.575	0.558	0.140
32.4	**9.333**	**1.500**	**157.989**	**12.296**	**0.575**	**0.558**	**0.140**
32.5	9.333	1.500	154.422	12.288	0.575	0.558	0.140
32.6	9.333	1.500	150.875	12.268	0.574	0.557	0.140

Coefficient of earth pressure : Kah = 0.558

Horizontal earth pressure Pah = Kah x γt x H

Pah1 = 0.558 × 19 × 0.000 = 0.000 kN/m³

Pah2 = 0.558 × 19 × 1.500 = 15.903 kN/m³

◁ At Earthquake (E)

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	4.764	6.538

∞ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
28.2	0.000	1.500	328.536	38.670	1.8091	1.803	0.150
28.3	0.000	1.500	323.922	38.684	1.8098	1.804	0.150
28.4	**0.000**	**1.500**	**319.338**	**38.687**	**1.8099**	**1.804**	**0.150**
28.5	0.000	1.500	314.783	38.677	1.8094	1.803	0.150
28.6	0.000	1.500	310.258	38.655	1.8084	1.802	0.150

Coefficient of earthquake earth pressure : Kaeh' = Kaeh - Kah = 1.246

Earthquake earth pressure Paeh' = Kaeh' x γt x H

Paeh1' = 1.246 × 19 × 0.000 = 0.000 kN/m³

Paeh2' = 1.246 × 19 × 1.500 = 35.511 kN/m³

3) Inertia force at earthquake (E)

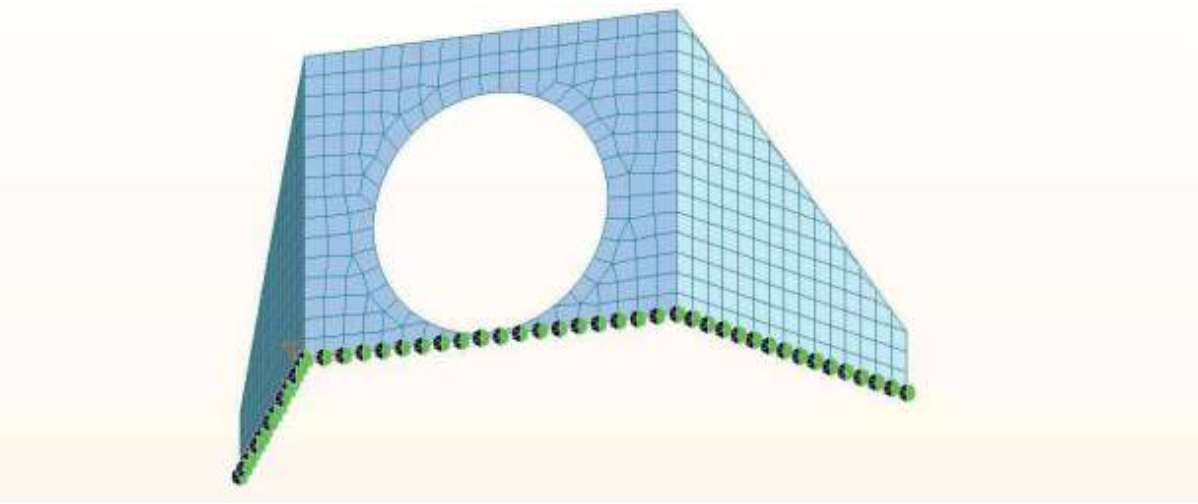
P = W x Kh

W : Weight of structure

Kh : 0.115 horizontal seismic coefficient

2.4.4 Modeling & Loading

1) Anaysis Model

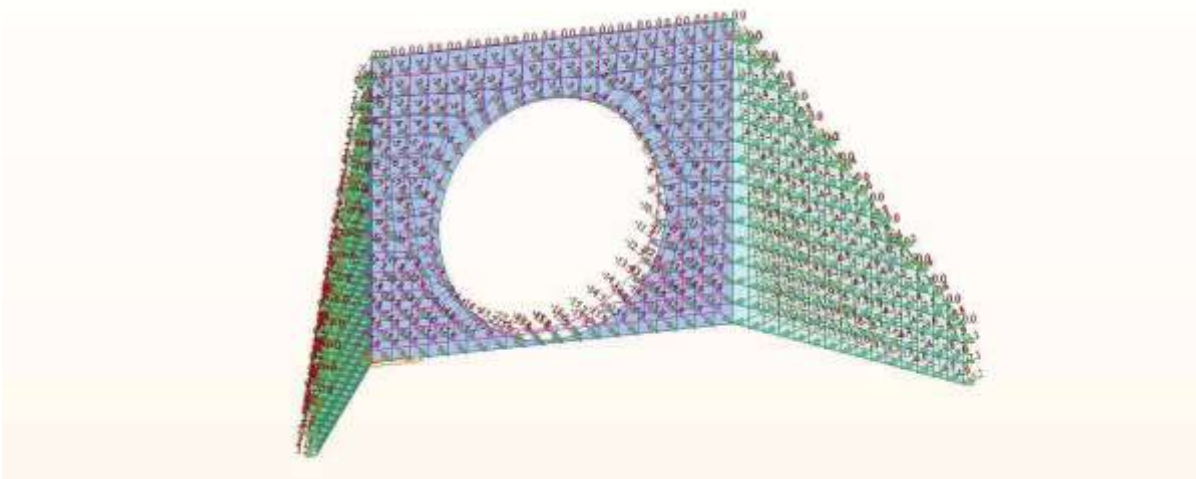


Boundary condition : Hinge

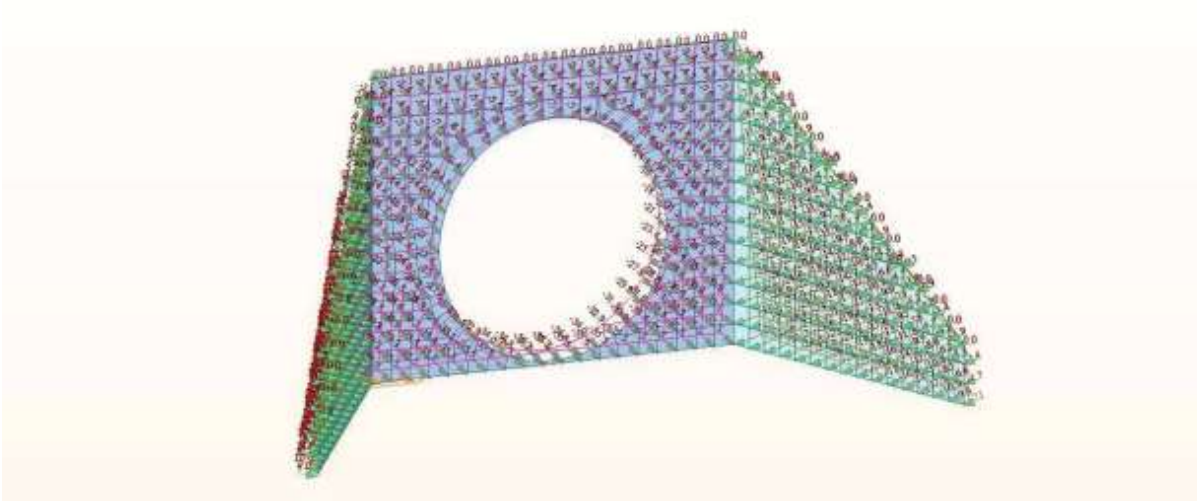
2) Loading

(1) Self weight - Automatic consideration in program (D)

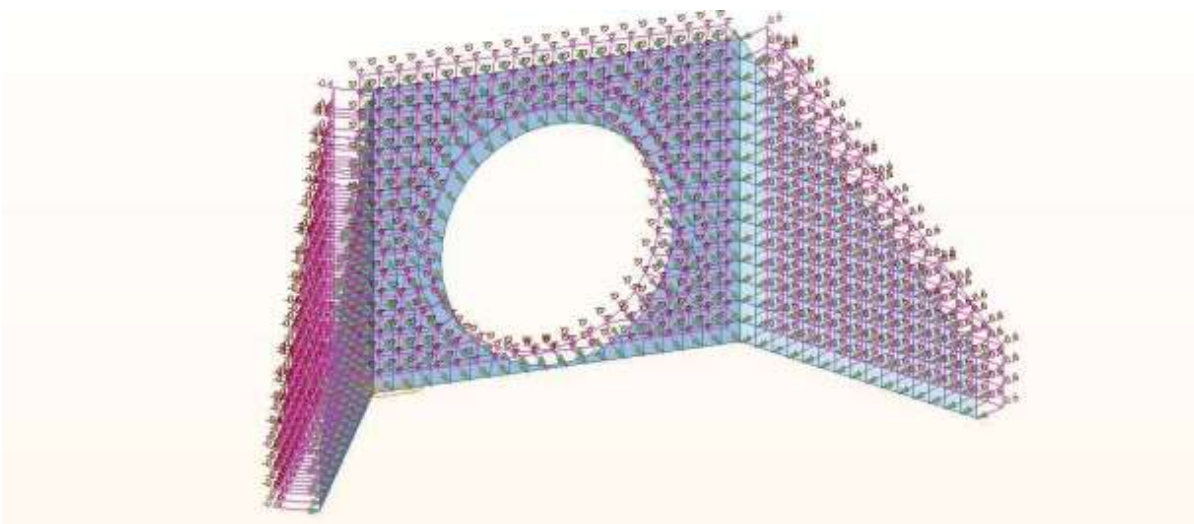
(2) Horizontal Earth Pressure at normal (H)



(3) Horizontal Earth Pressure at earthquake (E)



(4) Inertia force at earthquake (E)



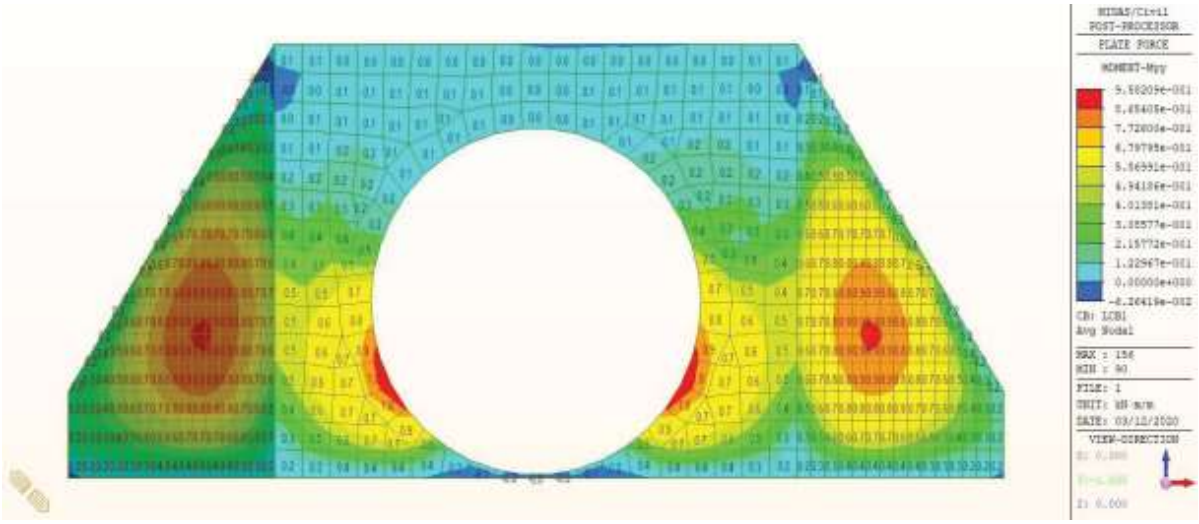
2.4.5 Load combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

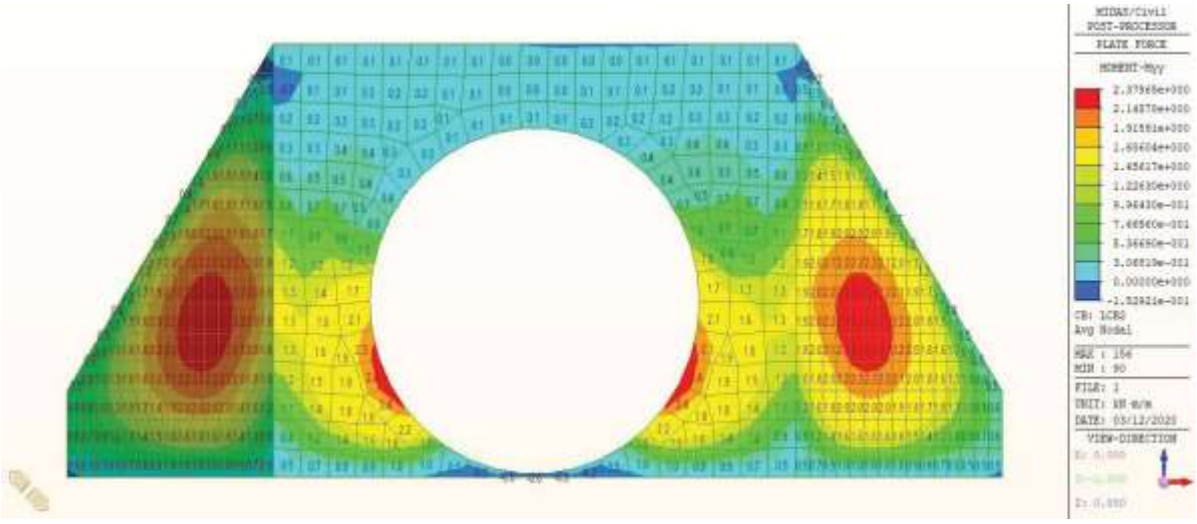
2.4.6 Summary of Analysis Results

(1) B.M.D (Ultimate Load) - Unit : kN.m

▷ LCB1

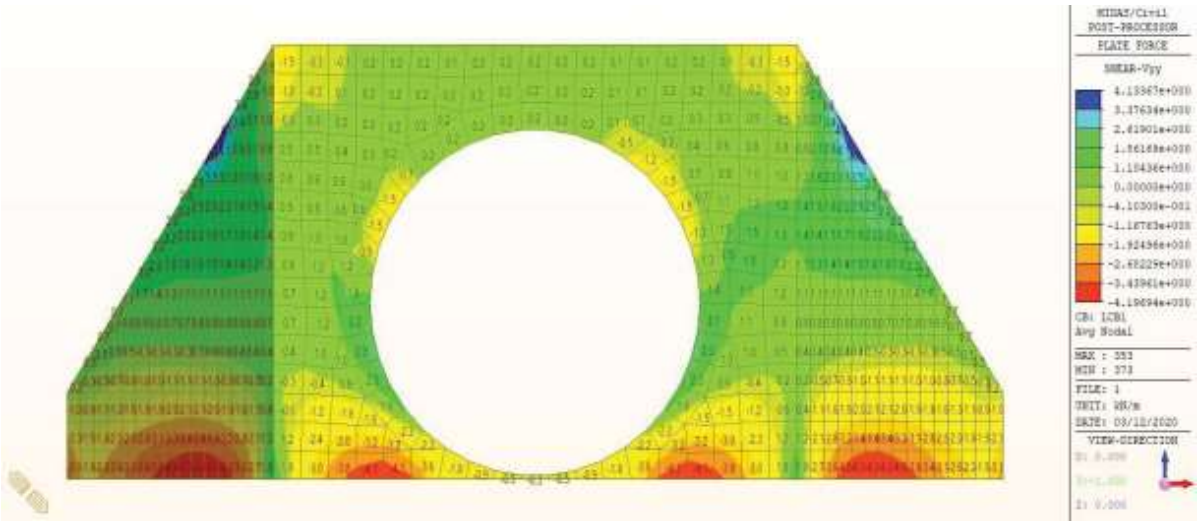


◁ LCB2

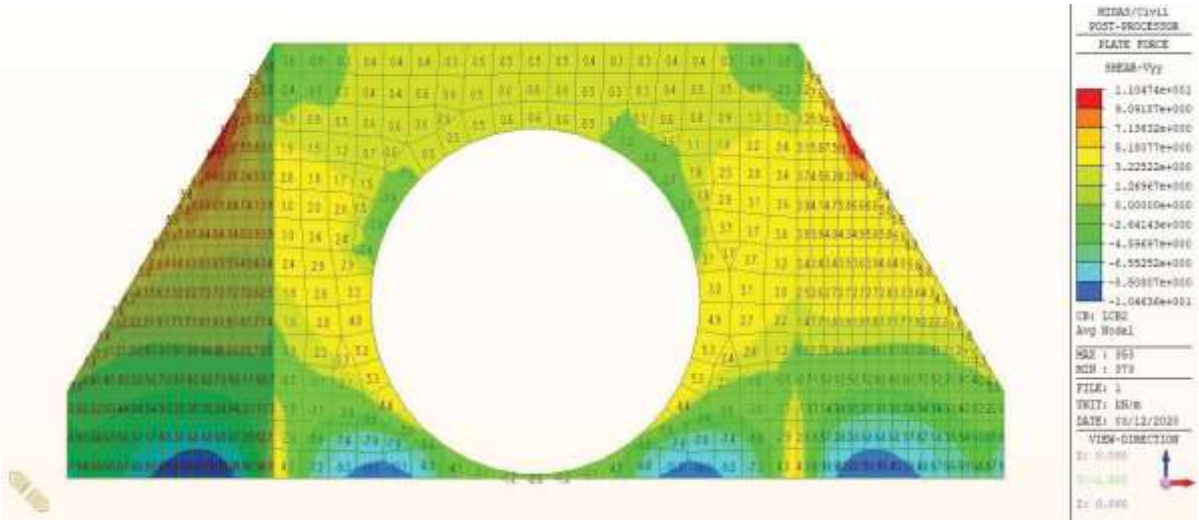


(2) S.F.D (Ultimate Load) - Unit : kN

▷ LCB1

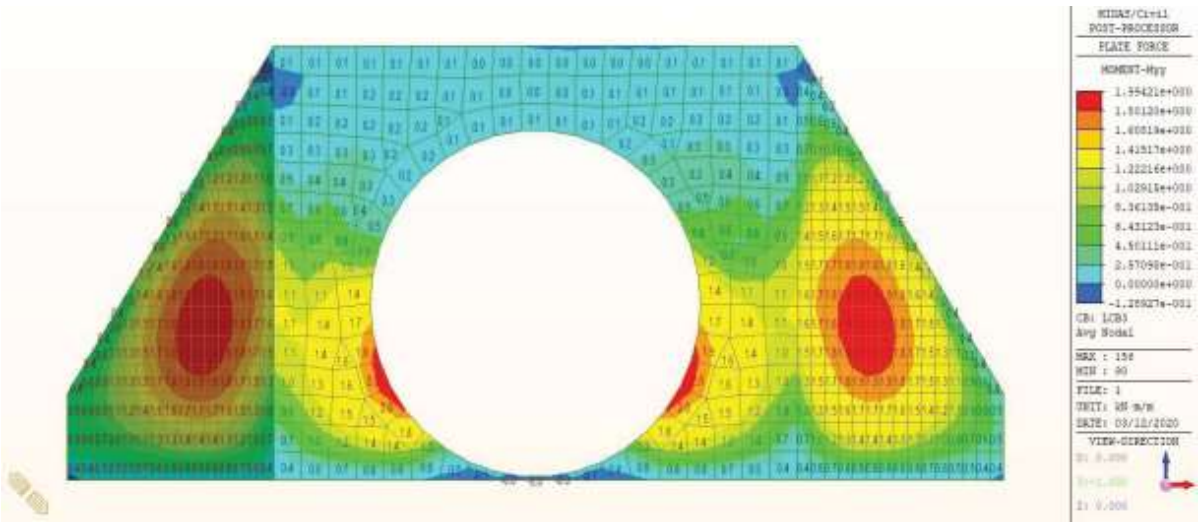


◁ LCB2



(3) B.M.D (Service Load) - Unit : kN.m

▷ LCB3



(4) Summary

Division	LCB1	LCB2	LCB3	Section
M(kN.m)	0.958	2.375	1.994	Middle of Wall
V(kN)	4.196	10.463		Bottom of Wall

□ Mu = 2.375 kN.m

Vu = 10.463 kN

Mo = 1.994 kN.m

2.4.7 Section Design

1) Middle of Wall

(1) **Section Design**

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	100.0	mm		
B	=	1000	mm	H	=	200	mm		d'	=	100.0	mm		
M_u	=	2.375	kN·m	V_u	=	10.463	kN	M_o	=		1.994	kN·m		

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (100.0 - 18.322) / 18.322

= 0.0134

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 63.138 ㎟

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎟ (8 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	2485.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	336.7	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00084	㎟	$A_{s,4/3req}$	=	84.2	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00084	㎟	$A_{s,min}$	=	84.2	㎟
P_{use}	=	A_s / (B·d)	=	0.01032	㎟	$A_{s,min}$	=	1032.0	㎟

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 36.074 kN·m > Mu = 2.375 kN·m

Ā O.K

ⓝ Shear firction Check

Øv Vn = Øv × Avf × fy × μ 325.080 > Vu = 10.463 kN Ā O.K

(2) **Crack Check**

ⓝ Calculation of stress

n = 9

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 9 × 1,032.00 / 1000 + 9 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 100 / (9 × 1,032.00)]

= 34.801 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 1.994 / [1000 × 34.801 x (100.0 - 34.801 / 3)] × 10^6

= 1.296 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 1.994 / [1032.000 x (100.0 - 34.801 / 3)] × 10^6

= 21.857 MPa

fst = fs x (H - d' - χ) / (d - χ) = 22 x (200 - 100 - 1) / (100 - 35) = 21.86 MPa

ⓝ Maximum center space of reinforcement

C_C = 100.00 - 13.00 / 2 = 93.50 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 21.86) - 2.5 x 93.50 = 4634.21 ⌟

300 x (280 / fs) = 300 x (280 / 21.86) = 3843.12 ⌟

Sa = 3843.12 ⌟ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (3843.12 mm) Ā O.K

2.4.8 Distribution Reinforcement Check

1) Wall (H = 325 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 325 = 585.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : D 13@ 125 = 1032.0 ㎟

□ = 1032.0 ㎟ > 585.0 ㎟ Ā O.K

· Bar spacing : 125 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 450 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 450 = 810.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : Upper side : D 13@ 250 = 516.0 ㎟

Bottom side : D 13@ 250 = 516.0 ㎟

□ = 1032.0 ㎟ > 810.0 ㎟ Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

3. Box Culvert Wing Wall

3.1 Wing Wall (H=3.3m)

3.1.1 Design Conditions (H=3.300m , N= 1 : 2.00 , Ho= 3.000)

1) General Items

(1)	Type of WingWall	:	Reverse T Type WingWall	
(2)	Height of WingWall	:	3.300	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	3.000	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of ground	:	γt =	18.000	kN/m³
(4)	angle of internal friction of ground	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of ground	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(=0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

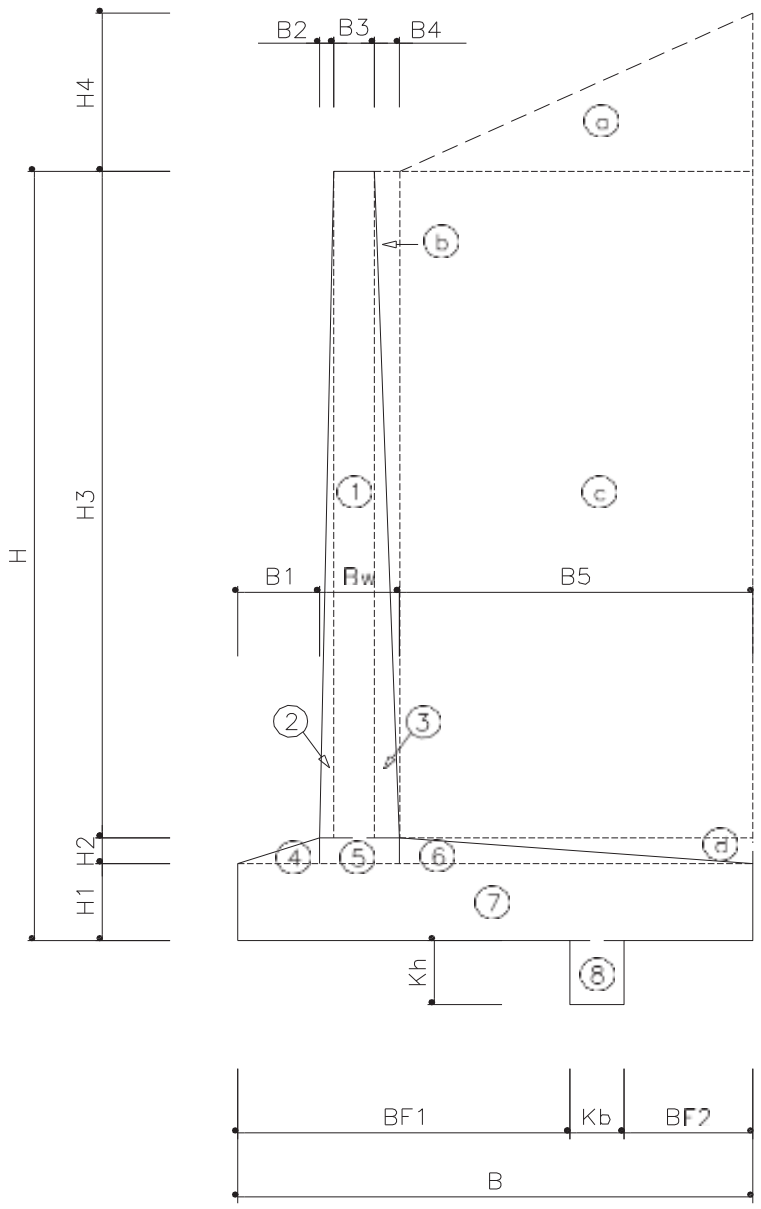
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

3.1.2 Section Assumption



※ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.300	0.058	0.300	0.042	2.200	2.900	0.400

- Height

H1	H2	H3	H4	H	Ho	
0.400	0.000	2.900	1.100	3.300	3.000	

- Shear Key

BF1	BF2	Kb	Kh			
1.900	0.600	0.400	0.500			

3.1.3 Evaluation of serviceability

1) At Nomal

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta)} \times W$$

where,

Φ(ㅈx)	β(ㅈx)	δ(ㅈx)	θ(ㅈx)
28.0	26.565	22.359	0.830

∞ (Apply δ = β')

α(ㅈx)	δ'(ㅈx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
49.0	22.276	4.400	259.179	92.904	0.505	0.467	0.192
49.1	22.318	4.400	258.025	92.909	0.505	0.467	0.192
49.2	**22.359**	**4.400**	**256.875**	**92.911**	**0.505**	**0.467**	**0.192**
49.3	22.401	4.400	255.728	92.911	0.505	0.467	0.192
49.4	22.443	4.400	254.585	92.908	0.505	0.467	0.192

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 85.927 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 35.313 kN/m³

(2) Load

Division		Calculation							Unit Weight	Vertical Force(kN)
Concrete	▷	2.900	x	0.300			=	0.870	25.00	21.750
	◁	2.900	x	0.058	x	½	=	0.084	25.00	2.103
	V	2.900	x	0.042	x	½	=	0.061	25.00	1.523
	Å	0.000	x	0.300	x	½	=	0.000	25.00	0.000
	Λ	0.000	x	0.400			=	0.000	25.00	0.000
	Λ̇	0.000	x	2.200	x	½	=	0.000	25.00	0.000
	>	0.400	x	2.900			=	1.160	25.00	29.000
	̇>	0.500	x	0.400			=	0.200	25.00	5.000
Soil	▷·	1.100	x	2.200	x	½	=	1.210	19.00	22.990
	·▷̇	2.900	x	0.042	x	½	=	0.061	19.00	1.157
	▷̇·	2.900	x	2.200			=	6.380	19.00	121.220
	▷̈	0.000	x	2.200	x	½	=	0.000	19.00	0.000
Surface load		1.638					=	1.638	10.00	16.380

(3) Moment

Division		Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
		V (kN/m)	H (kN/m)	X	Y	V·X(Mr)	H·Y(Mo)
Concrete		21.750	-	0.508	-	11.049	-
		2.103	-	0.339	-	0.712	-
		1.523	-	0.672	-	1.023	-
		0.000	-	0.200	-	0.000	-
		0.000	-	0.500	-	0.000	-
		0.000	-	1.433	-	0.000	-
		29.000	-	1.450	-	42.050	-
		5.000	-	2.100	-	10.500	-
Sub Total		**59.375**				**65.334**	
Soil		22.990	-	2.167	-	49.812	-
		1.157	-	0.686	-	0.794	-
		121.220	-	1.800	-	218.196	-
		0.000	-	2.167	-	0.000	-
Sub Total		**145.367**				**268.801**	
Earth pressure		35.313	85.927	2.900	1.467	102.407	126.027
Surface load		3.145	7.653	2.900	1.467	9.120	11.224
Total		**243.200**	**93.580**			**445.663**	**137.251**

(4) Evaluation of serviceability

▷ Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 2.900 - 2 x 0.182 = 2.536 m

C = 0.000 kN/m²

ΣV = 243.200 kN/m

ΣH = 93.580 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 121.600 kN/m

F.S = Hu / ΣH = 1.299 < 1.5 --- **N.G**

ÿ **Need Shear Key.**

◁ Shear Key

V = 243.200 kN/m

tanΦB = 0.500

Kp = 3.688

$$Kp = \frac{\cos^2(\varphi + \theta)}{\cos^2\theta \cos(\theta - \delta)\left[1 - \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi + \beta)}{\cos(\theta - \delta)\cos(\theta - \beta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	0.00	9.33	0

Pp = 41.489 kN/m

Hk = V $\tan\Phi_B$ + Pp = 163.089 kN/m

F.S = Hk / ΣH = 1.743 > 1.5 --- **O.K**

ÿ **Stability of sliding is O.K.**

¤ Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

= 2.900 / 2 - (445.663 - 137.251) / 243.2 = 0.182 m

B / 6 = 2.900 / 6 = 0.483 > e --- **O.K**

F.S = ΣMr / ΣMo = 3.247 > 2.0 --- **O.K**

ÿ **Stability of overturning is O.K.**

¥ Bearing Capacity

e = 0.182 m < B/6 = 0.483 m

x = 3·[B/2 - e] = 3 x (2.900 / 2 - 0.182) = 3.804 m > 2.900 m

□ resultant in middle one-third of base

q(max,min) = (ΣV / B) x (1 ± 6·e / B)

= 243.200 / 2.900 x (1 ± 6 x 0.182 / 2.900)

qmax = 115.416 kN/m² (Toe)

qmin = 52.308 kN/m² (Heel)

qmax = 115.416 kN/m² < qa = 270.830 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

2) At Earthquake

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta')} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)	
28.0	26.565	27.1	0.830	6.538	($\omega = \tan^{-1}Kh$)
					È(δ = β')

if β' + ω ¯Φ, δ' = Φ

α(ƒx)	δ'(ƒx)	H (m)	We (kN/m)	Pae (kN/m)	Kae	Kaeh	Kaev
42.7	27.044	4.400	342.246	126.910	0.69	0.615	0.314
42.8	27.084	4.400	340.809	126.916	0.6901	0.614	0.314
42.9	**27.125**	**4.400**	**339.376**	**126.918**	**0.6901**	**0.614**	**0.315**
43.0	27.166	4.400	337.950	126.918	0.6901	0.614	0.315
43.1	27.207	4.400	336.528	126.915	0.6901	0.614	0.316

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 112.927 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 57.935 kN/m³

(2) Load

Division	Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
	V (kN/m)	H (kN/m)	X	Y	V · X	H · Y
Concrete	59.375				65.334	
Soil	145.367				268.801	
Earth pressure	57.935	112.927	2.900	2.200	168.011	248.439
Total	**262.677**	**112.927**	1.912	2.200	**502.147**	**248.439**

(3) Moment

Load		Calculation	Horizontal seismic coefficient	Horizental Force	MOMENT (kN·m)	
				H (kN/m)	Y	H · Y
Concrete	▷	21.750	0.115	2.493	1.850	4.611
	◁	2.103	0.115	0.241	1.367	0.329
	V	1.523	0.115	0.174	1.367	0.238
	Å	0.000	0.115	0.000	0.400	0.000
	Λ	0.000	0.115	0.000	0.400	0.000
	Λ̇	0.000	0.115	0.000	0.400	0.000
	>	29.000	0.115	3.323	0.200	0.665
Sub Total		**54.375**		**6.231**	-	**5.844**
Soil	▷·	22.990	0.115	2.635	3.667	9.660
	·▷	1.157	0.115	0.133	2.333	0.309
	▷·	121.220	0.115	13.892	1.850	25.700
	▷	0.000	0.115	0.000	0.400	0.000
Sub Total		**145.367**		**16.659**	-	**35.670**
Total		**199.742**		**22.890**	1.814	**41.514**

(4) Evaluation of serviceability

▷ Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 2.900 - 2 x 0.642 = 1.616 m

C = 0.000 kN/m²

ΣV = 262.677 kN/m

ΣH = 135.817 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 131.338 kN/m --- **N.G**

F.S = Hu / ΣH = 0.967 < 1.1

ÿ **Need Shear Key.**

◁ Shear Key

V = V = 262.677 kN/m

$\tan\Phi B$ = $\tan\Phi_B$ = 0.500

Kp = Kpe = 2.138

$$Kp = \frac{\cos^2(\varphi - \omega + \theta)}{\cos\omega \cos^2\theta \cos(\delta - \theta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi - \delta)\sin(\varphi - \omega + \beta)}{\cos(\delta - \theta + \omega)\cos(\beta - \theta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.00	0.00	9.33	0	6.538

Ppe = 24.054 kN/m

Hk = V $\tan\Phi_B$ + Ppe = 155.392 kN/m

F.S = Hk / ΣH = 1.144 > 1.1 --- **O.K**

ÿ **Stability of sliding is O.K.**

¤ **Overturning**

e = B/2 - (ΣMr - ΣMo)/ΣV

2.900 / 2 - (502.147 - (248.439 + 41.514)) / 262.677 = 0.642 m

B / 3 = 2.900 / 3 = 0.967 > e --- **O.K**

F.S = ΣMr / ΣMo = 1.732 > 1.5 --- **O.K**

ÿ **Stability of overturning is O.K.**

¥ **Bearing Capacity**

e = 0.642 m < B/3 = 0.967 m

x = 3·[B/2 - e] = 3 x (2.900 / 2 -0.642) = 2.424 m < 2.900 m

▯ Pressure for resultant outside middle one-third

qmax = 2·ΣV / X

= 262.677 x 2 / 2.424 = 216.730 kN/m²

qmax = 216.730 kN/m² (Toe)

qmin = 0.000 kN/m² (Heel)

qmax = 216.730 kN/m² < qa = 270.830 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

3.1.4 Load Calculation

1) Wall

(1) Earth pressure

At Nomal

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	0.830

∵ (δ = Λ· x Φ)

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
42.7	9.333	2.900	188.527	47.991	0.601	0.591	0.106
42.8	9.333	2.900	187.275	47.996	0.601	0.591	0.106
42.9	**9.333**	**2.900**	**186.027**	**47.998**	**0.601**	**0.591**	**0.106**
43.0	9.333	2.900	184.784	47.996	0.601	0.591	0.106
43.1	9.333	2.900	183.545	47.992	0.601	0.591	0.106

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 47.218 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 8.469 kN/m³

At Earthquake

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	0.830	6.538

∵ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
38.1	0.000	2.900	252.392	73.222	0.9165	0.916	0.013
38.2	0.000	2.900	250.870	73.227	0.9165	0.916	0.013
38.3	**0.000**	**2.900**	**249.354**	**73.227**	**0.9165**	**0.916**	**0.013**
38.4	0.000	2.900	247.845	73.225	0.9165	0.916	0.013
38.5	0.000	2.900	246.342	73.220	0.9165	0.916	0.013

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 73.184 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 1.039 kN/m³

Earthquake earth pressure : Paeh' = 1/2 x kaeh' x γt x H^2 = 25.966 kN/m³

Division		Load	Horizental Force	length (m)	Mr (kN·m)
		W (kN)	H (kN/m)	Y	H x Y
Concrete	▷	21.750	2.493	1.450	3.614
	◁	2.103	0.241	0.967	0.233
	V	1.523	0.174	0.967	0.169
Bottom of Wall		**25.375**	**2.908**		**4.016**
Earthquake earth pressure			**25.966**	1.450	**37.651**
Concrete	▷	10.875	1.246	0.725	0.904
	◁	0.053	0.006	0.483	0.003
	V	0.381	0.044	0.483	0.021
Middle of Wall		**11.308**	**1.296**		**0.928**
Earthquake earth pressure			**6.491**	0.725	**4.706**

(2) Stress Resultant

At Nomal

Bottom of Wall	(H=	2.900 m	)				
V=	47.218				=	47.218	kN
M=	47.218	x	2.900	/ 3	=	45.644	kN/m
Middle of Wall	(H=	1.450 m	)				
V=	11.804				=	11.804	kN
M=	11.804	x	1.450	/ 3	=	5.706	kN/m

At Earthquake

Bottom of Wall	(H=	2.900 m	)				
Ve=	73.184				=	73.184	kN
Me=	73.184	x	2.900	/2	=	106.117	kN/m
Middle of Wall	(H=	1.450 m	)				
Ve=	18.296				=	18.296	kN
Me=	18.296	x	1.450	/ 2	=	13.265	kN/m

(3) Design Load for cross section

Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Summary

Division		Bottom of Wall		Middle of Wall	
		Horizontal earth pressure	Inertial force	Horizontal earth pressure	Inertial force
LCB1	Shear force	75.549	0.000	18.887	0.000
	Moment	73.030	0.000	9.129	0.000
LCB2	Shear force	75.549	28.874	18.887	7.787
	Moment	73.030	41.667	9.129	5.634
LCB3	Shear force	47.218	0.000	11.804	0.000
	Moment	45.644	0.000	5.706	0.000

V Design Load for cross section

-Bottom of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
75.549	73.030	104.423	114.697	47.218	45.644

-Middle of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
18.887	9.129	26.675	14.763	11.804	5.706

2) Foundation

(1) Stress resultant of Foundation

▷ Load

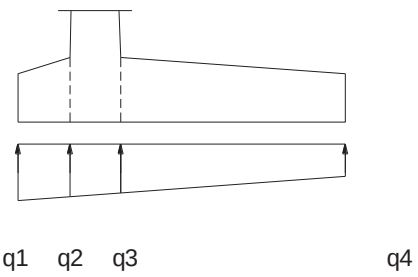
Division		V(kN)	H(kN)	Mr(kN.m)	Mo(kN.m)
At Nomal	Concrete	59.375	0.000	65.334	0.00
	Soil	145.367	0.000	268.801	0.00
	Earth pressure	35.313	85.927	102.407	126.027
	Surface load	3.145	7.653	9.120	11.224
	□	**243.200**	**93.580**	**445.663**	**137.25**
At Earthquake	Concrete	59.375	6.231	65.334	5.844
	Soil	145.367	16.659	268.801	35.670
	Earth pressure Earthquake	35.313	85.927	102.407	126.027
	earth pressure	22.622	26.999	65.604	122.412
	□	**262.677**	**135.817**	**502.147**	**289.95**

◁ Ultimate load Combination

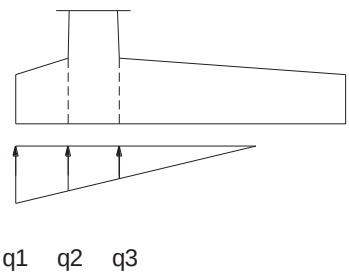
Division	□V	□Mr	□Mo	e	Load shape
LCB1	307.223	579.406	219.602	0.279	CASE1
LCB2	263.390	530.177	361.418	0.809	CASE2
LCB3	243.200	445.663	137.251	0.182	CASE1

V Stress resultant of Foundation

CASE1



CASE2



Division	q1	q2	q3	q4	e	CASE
LCB1	167.091	154.439	137.569	44.787	0.279	1
LCB2	273.937	231.201	174.220	-	0.809	2
LCB3	115.440	108.907	100.196	52.284	0.182	1

(2) Heel

▷ Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Λ'	0.000	0.733	0.000	
	> '	22.000	1.100	24.200	
	> '	5.000	1.400	7.000	
Sub Total		**27.000**	**1.156**	**31.200**	
Soil	▷·	22.990	1.453	33.397	
	▷·	121.220	1.100	133.342	
	▷	0.000	1.467	0.000	
Sub Total		**144.210**	**1.156**	**166.739**	
Total		**171.210**	**1.156**	**197.939**	

◄ Cross section force by Vertical Force

- At Nomal

V =	35.313	kN				
M =	35.313	x	2.200	=	77.688	kN·m

- At Earthquake

V =	35.313	+	22.622	=	57.935	kN
M1 =	35.313	x	2.200	=	77.688	kN·m
M2 =	22.622	x	2.200	=	49.769	kN·m

V Cross section force by Stress resultant of Foundation

Load	q3	q4	length(m)	V (kN)	M (kN·m)
LCB1	137.569	44.787	0.913	-200.592	-183.228
LCB2	174.220	0.000	0.408	-106.535	-43.431
LCB3	100.196	52.284	0.985	-167.727	-165.175

∞ V = (q4 + q3)/2 × B5

Λ Design Load for cross section

-Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Division		D	L	H	E	Stress resultant of Foundation	Total
LCB1	Vu	205.452	-	56.500	-	-200.592	61.361
	Mu	237.527	-	124.300	-	-183.228	178.599
LCB2	Vu	154.089	-	56.500	22.622	-106.535	126.676
	Mu	178.145	-	124.300	49.769	-43.431	308.783
LCB3	Vo	171.210	-	35.313	-	-167.727	38.796
	Mo	197.939	-	77.688	-	-165.175	110.451

(3) Toe

Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Å'	0.000	0.100	0.000	
	> '	3.000	0.150	0.450	
Sub Total		**3.000**	**0.150**	**0.450**	

Cross section force by Stress resultant of Foundation

Load	q1	q2	length(m)	V (kN)	M (kN·m)
LCB1	167.091	154.439	0.152	48.230	7.329
LCB2	273.937	231.201	0.154	75.771	11.686
LCB3	115.440	108.907	0.151	33.652	5.097

∞ V = (q1 + q2)/2 × B1

Design Load for cross section

-Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Division		D	H	Stress resultant of Foundation	Total
LCB1	Vu	-3.600	-	48.230	44.630
	Mu	-0.540	-	7.329	6.789
LCB2	Vu	-2.700	-	75.771	73.071
	Mu	-0.405	-	11.686	11.281
LCB3	Vo	-3.000	-	33.652	30.652
	Mo	-0.450	-	5.097	4.647

(4) Shear Key

Passive earth pressure

At Nomal : Pp = 41.489 kN/m

At Earthq : Ppe = 24.054 kN/m Ā Apply Cross section force at Nomal

Design Load for cross section

Division	qk1	qk2	H(m)	V (kN)	M (kN·m)
At Nomal	66.382	99.573	0.500	41.489	11.064

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)
Design Load for cross section	17.702	66.382	11.064

3) Summary

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)	ØMn(kN·m)	Bar	S.F
Bottom of Wall	114.697	104.423	45.644	165.610	D13 @ 125	1.44
Middle of Wall	14.763	26.675	5.706	56.513	D13 @ 250	3.83
Heel	114.697	126.676	45.644	198.822	D16 @ 125	1.73
Toe	11.281	73.071	4.647	101.157	D16 @ 250	8.97
Shear Key	17.702	66.382	11.064	66.265	D13 @ 250	3.74

3.1.5 Section Design

1) Bottom of Wall

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	114.697	kN·m	V_u	=	104.423	kN	M_o	=	45.644	kN·m			

- Check of Strength reduction factor (Φ)

a = 19.133

Because T = C , *c* = 19.133 / β1 = 19.133 / 0.821 = 23.293 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 23.293) / 23.293

= 0.0412

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ.Ţ ТФГТУКУФУZТ НПΣ ИЦ.Ţ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 901.622 ㎡

Use As = D 13 @ 250 + D 16 @ 250 = 1312.00 ㎡ (8 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	8536.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00350	㎡	$A_{s,4/3req}$	=	1202.2	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00337	㎡	$A_{s,min}$	=	1156.6	㎡
P_{use}	=	A_s / (B·d)	=	0.00382	㎡	$A_{s,min}$	=	1312.0	㎡

ⓒ Pmin ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 19.133 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 165.610 kN·m > Mu = 114.697 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 104.423 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

$\chi = -nA_s/b + nA_s/b \times \sqrt{1+2bd/nA_s}$

$= -8 \times 1{,}312.00 / 1000 + 8 \times 1{,}312.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 1{,}312.00)]}$

= 75.066 ㎜

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 45.644 / [1000 \times 75.066 \times (343.5 - 75.066 / 3)] \times 10^6$

= 3.818 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 45.644 / [1312.000 \times (343.5 - 75.066 / 3)] \times 10^6$

= 109.237 MPa

fst = fs x (H - d' - χ) / (d - χ) = 109 x (400 - 57 - 4) / (344 - 75) = 109.24 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 109.24) - 2.5 x 50.00 = 849.03 ㎜

300 x (280 / fs) = 300 x (280 / 109.24) = 768.97 ㎜

Sa = 768.97 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (768.97 mm) Ā O.K

2) Middle of Wall

(1) Section Design

ⅰ. Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	293.5	mm		
B	=	1000	mm	H	=	350	mm		d'	=	56.5	mm		
M_u	=	14.763	kN·m	V_u	=	26.675	kN	M_o	=		5.706	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (293.5 - 9.161) / 9.161

= 0.0931

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИД.Т ТФГТУКУФУZТ НПΣ ИД.Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 133.538 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ⅰ. Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	7293.5	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	988.3	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00061	㎟	$A_{s,4/3req}$	=	178.1	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00061	㎟	$A_{s,min}$	=	178.1	㎟
P_{use}	=	A_s / (B·d)	=	0.00176	㎟	$A_{s,min}$	=	516.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⅰ. Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

ØMn = 0.9 × As × fy × (d - a/2) = 56.513 kN·m > Mu = 14.763 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 207.536 kN > Vu = 26.675 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 516.00 / 1000 + 8 × 516.00 / 1000 × √[1 + 2 × 1000 × 293.5 / (8 × 516.00)]

= 45.270 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 5.706 / [1000 × 45.270 x (293.5 - 45.270 / 3)] × 10^6

= 0.905 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 5.706 / [516.000 x (293.5 - 45.270 / 3)] × 10^6

= 39.715 MPa

fst = fs x (H - d' - χ) / (d - χ) = 40 x (350 - 57 - 1) / (294 - 45) = 39.72 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 39.72) - 2.5 x 50.00 = 2554.06 ㎜

300 x (280 / fs) = 300 x (280 / 39.72) = 2115.05 ㎜

Sa = 2115.05 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (2115.05 mm) Ā O.K

3) Heel

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	342.0	mm		
B	=	1000	mm	H	=	400	mm		d'	=	58.0	mm		
M_u	=	114.697	kN·m	V_u	=	126.676	kN	M_o	=	45.644	kN·m			

- Check of Strength reduction factor (Φ)

a = 23.217

Because T = C , *c* = 23.217 / β1 = 23.217 / 0.821 = 28.264 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (342.0 - 28.264) / 28.264

= 0.0333

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИL.T ТФГТУКУФУZT НПΣ ИL.T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 905.745 ㎟

Use As = D 16 @ 250 + D 16 @ 250 = 1592.00 ㎟ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8498.7	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1151.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00353	㎟	$A_{s,4/3req}$	=	1207.7	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00337	㎟	$A_{s,min}$	=	1151.6	㎟
P_{use}	=	A_s / (B·d)	=	0.00465	㎟	$A_{s,min}$	=	1592.0	㎟

ㄴ Pmin ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 23.217 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 198.822 kN·m > Mu = 114.697 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 241.831 kN > Vu = 126.676 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,592.00 / 1000 + 8 × 1,592.00 / 1000 × √[1 + 2 × 1000 × 342 / (8 × 1,592.00)]

= 81.464 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 45.644 / [1000 × 81.464 x (342.0 - 81.464 / 3)] × 10^6

= 3.559 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 45.644 / [1592.000 x (342.0 - 81.464 / 3)] × 10^6

= 91.063 MPa

fst = fs x (H - d' - χ) / (d - χ) = 91 x (400 - 58 - 4) / (342 - 81) = 91.06 MPa

ⓐ Maximum center space of reinforcement

C_C = 58.00 - 16.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 91.06) - 2.5 x 50.00 = 1043.42 ㎜

300 x (280 / fs) = 300 x (280 / 91.06) = 922.44 ㎜

Sa = 922.44 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (922.44 mm) Ā O.K

4) Toe

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	342.0	mm		
B	=	1000	mm	H	=	400	mm		d'	=	58.0	mm		
M_u	=	11.281	kN·m	V_u	=	73.071	kN	M_o	=		4.647	kN·m		

- Check of Strength reduction factor (Φ)

a = 11.608

Because T = C , *c* = 11.608 / β1 = 11.608 / 0.821 = 14.132 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (342.0 - 14.132) / 14.132

= 0.0696

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 87.436 ㎟

Use As = D 16 @ 500 + D 16 @ 500 = 796.00 ㎟ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8498.7	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1151.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00034	㎟	$A_{s,4/3req}$	=	116.6	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00034	㎟	$A_{s,min}$	=	116.6	㎟
P_{use}	=	A_s / (B·d)	=	0.00233	㎟	$A_{s,min}$	=	796.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 11.608 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 101.157 kN·m > Mu = 11.281 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 241.831 kN > Vu = 73.071 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 796.00 / 1000 + 8 × 796.00 / 1000 × √[1 + 2 × 1000 × 342 / (8 × 796.00)]

= 59.936 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 4.647 / [1000 × 59.936 x (342.0 - 59.936 / 3)] × 10^6

= 0.482 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 4.647 / [796.000 x (342.0 - 59.936 / 3)] × 10^6

= 18.128 MPa

fst = fs x (H - d' - χ) / (d - χ) = 18 x (400 - 58 - 0) / (342 - 60) = 18.13 MPa

ⓐ Maximum center space of reinforcement

C_C = 58.00 - 16.00 / 2 = 50.00 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 18.13) - 2.5 x 50.00 = 5.7E+03 ⌟

300 x (280 / fs) = 300 x (280 / 18.13) = 4.6E+03 ⌟

Sa = 4.63E+03 ⌟ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (4.6E+03 mm) Ā O.K

5) Shear Key

(1) Section Design

ⅰ. Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	17.702	kN·m	V_u	=	66.382	kN	M_o	=		11.064	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 136.754 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ⅰ. Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00053	㎟	$A_{s,4/3req}$	=	182.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00053	㎟	$A_{s,min}$	=	182.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00150	㎟	$A_{s,min}$	=	516.0	㎟

∠ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⅰ. Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 17.702 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 66.382 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 516.00 / 1000 + 8 × 516.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 516.00)]

= 49.285 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 11.064 / [1000 × 49.285 x (343.5 - 49.285 / 3)] × 10^6

= 1.373 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 11.064 / [516.000 x (343.5 - 49.285 / 3)] × 10^6

= 65.557 MPa

fst = fs x (H - d' - χ) / (d - χ) = 66 x (400 - 57 - 1) / (344 - 49) = 65.56 MPa

ⓐ Maximum center space of reinforcement

Cc = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 65.56) - 2.5 x 50.00 = 1498.01 ㎜

300 x (280 / fs) = 300 x (280 / 65.56) = 1281.33 ㎜

Sa = 1281.33 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (1281.33 mm) Ā O.K

3.1.6 Distribution Reinforcement Check

1) Wall (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	250	=	516.0	㎟			
Compression side	D	13@	250	=	516.0	㎟			
			□	=	1032.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

3.2 Wing Wall (H=1.5m)

3.2.1 Design Conditions (H=1.500m , N= 1 : 2.00 , Ho= 2.000)

1) General Items

(1)	Type of WingWall	:	Reverse T Type WingWall	
(2)	Height of WingWall	:	1.500	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	2.000	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of ground	:	γt =	18.000	kN/m³
(4)	angle of internal friction of ground	:	$\Phi 1$ =	28.000	°
(5)	coefficient of earth pressure atrest of ground	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(=0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

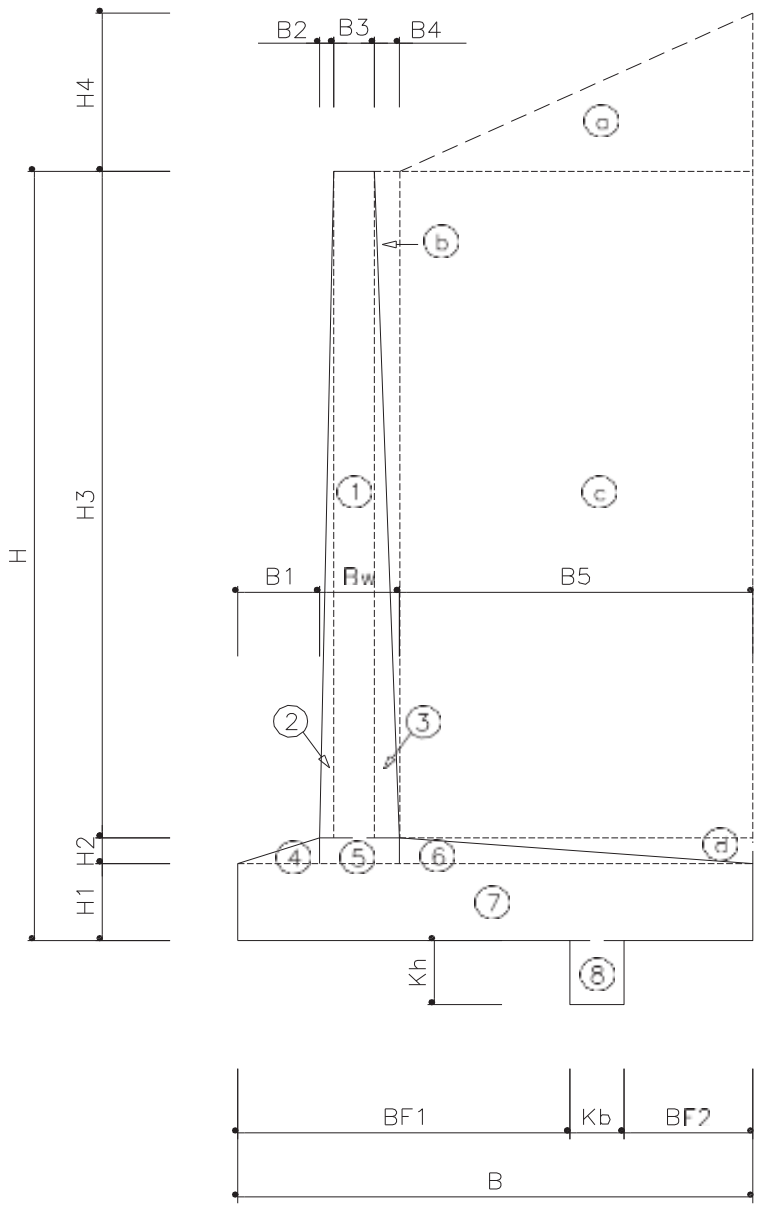
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

3.2.2 Section Assumption



※ Sectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.300	0.022	0.300	0.078	0.800	1.500	0.400

- Height

H1	H2	H3	H4	H	Ho	
0.400	0.000	1.100	0.400	1.500	2.000	

- Shear Key

BF1	BF2	Kb	Kh			
0.700	0.400	0.400	0.500			

3.2.3 Evaluation of serviceability

1) At Nomal

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta)} \times W$$

where,

Φ(rx)	β(rx)	δ(rx)	θ(rx)
28.0	26.565	25.204	4.056

∵ (Apply δ = β')

α(rx)	δ'(rx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
44.0	25.122	1.900	71.870	20.064	0.585	0.529	0.249
44.1	25.163	1.900	71.450	20.065	0.585	0.529	0.249
44.2	**25.204**	**1.900**	**71.031**	**20.064**	**0.585**	**0.529**	**0.249**
44.3	25.245	1.900	70.614	20.063	0.585	0.529	0.249
44.4	25.285	1.900	70.198	20.061	0.585	0.529	0.249

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 18.152 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 8.539 kN/m³

(2) Load

Division		Calculation							Unit Weight	Vertical Force(kN)
Concrete		1.100	x	0.300			=	0.330	25.00	8.250
		1.100	x	0.022	x	½	=	0.012	25.00	0.303
		1.100	x	0.078	x	½	=	0.043	25.00	1.073
		0.000	x	0.300	x	½	=	0.000	25.00	0.000
		0.000	x	0.400			=	0.000	25.00	0.000
		0.000	x	0.800	x	½	=	0.000	25.00	0.000
		0.400	x	1.500			=	0.600	25.00	15.000
		0.500	x	0.400			=	0.200	25.00	5.000
Soil		0.400	x	0.800	x	½	=	0.160	19.00	3.040
		1.100	x	0.078	x	½	=	0.043	19.00	0.815
		1.100	x	0.800			=	0.880	19.00	16.720
		0.000	x	0.800	x	½	=	0.000	19.00	0.000
Surface load		0.399					=	0.399	10.00	3.991

(3) Moment

Division		Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
		V (kN/m)	H (kN/m)	X	Y	V·X(Mr)	H·Y(Mo)
Concrete		8.250	-	0.472	-	3.894	-
		0.303	-	0.315	-	0.095	-
		1.073	-	0.648	-	0.695	-
		0.000	-	0.200	-	0.000	-
		0.000	-	0.500	-	0.000	-
		0.000	-	0.967	-	0.000	-
		15.000	-	0.750	-	11.250	-
		5.000	-	0.900	-	4.500	-
Sub Total		**29.625**				**20.434**	
Soil		3.040	-	1.233	-	3.749	-
		0.815	-	0.674	-	0.549	-
		16.720	-	1.100	-	18.392	-
		0.000	-	1.233	-	0.000	-
Sub Total		**20.575**				**22.691**	
Earth pressure		8.539	18.152	1.500	0.633	12.809	11.496
Surface load		0.994	2.113	1.500	0.633	1.491	1.338
Total		**59.733**	**20.265**			**57.425**	**12.834**

(4) Evaluation of serviceability

▷ Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.500 - 2 x 0.004 = 1.493 m

C = 0.000 kN/m²

ΣV = 59.733 kN/m

ΣH = 20.265 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 29.867 kN/m

F.S = Hu / ΣH = 1.474 < 1.5 --- **N.G**

ÿ **Need Shear Key.**

◁ Shear Key

V = 59.733 kN/m

tanΦB = 0.500

Kp = 3.688

$$Kp = \frac{\cos^2(\varphi + \theta)}{\cos^2\theta \cos(\theta - \delta)\left[1 - \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi + \beta)}{\cos(\theta - \delta)\cos(\theta - \beta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	0.00	9.33	0

Pp = 41.489 kN/m

Hk = V $\tan\Phi_B$ + Pp = 71.356 kN/m

F.S = Hk / ΣH = 3.521 > 1.5 --- **O.K**

ÿ **Stability of sliding is O.K.**

¤ Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

= 1.500 / 2 - (57.425 - 12.834) / 59.733 = 0.004 m

B / 6 = 1.500 / 6 = 0.250 > e --- **O.K**

F.S = ΣMr / ΣMo = 4.474 > 2.0 --- **O.K**

ÿ **Stability of overturning is O.K.**

¥ Bearing Capacity

e = 0.004 m < B/6 = 0.250 m

x = 3·[B/2 - e] = 3 x (1.500 / 2 - 0.004) = 2.239 m > 1.500 m

□ resultant in middle one-third of base

q(max,min) = (ΣV / B) x (1 ± 6·e / B)

= 59.733 / 1.500 x (1 ± 6 x 0.004 / 1.500)

qmax = 40.381 kN/m² (Toe)

qmin = 39.263 kN/m² (Heel)

qmax = 40.381 kN/m² < qa = 270.830 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

2) At Earthquake

(1) Earth pressure

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta')} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)	
28.0	26.565	28.0	4.056	6.538	(ω = $\tan^{-1}$Kh)
					(δ = Φ)

if β' + ω ¯Φ, δ' = Φ

α(ƒx)	δ'(ƒx)	H (m)	We (kN/m)	Pae (kN/m)	Kae	Kaeh	Kaev
37.7	28.000	1.900	102.599	30.218	0.8811	0.778	0.414
37.8	28.000	1.900	102.053	30.220	0.8812	0.778	0.414
37.9	**28.000**	**1.900**	**101.510**	**30.221**	**0.8812**	**0.778**	**0.414**
38.0	28.000	1.900	100.970	30.220	0.8812	0.778	0.414
38.1	28.000	1.900	100.432	30.219	0.8811	0.778	0.414

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 26.682 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 14.198 kN/m³

(2) Load

Division	Vertical Force	Horizental Force	length (m)		MOMENT (kN·m)	
	V (kN/m)	H (kN/m)	X	Y	V · X	H · Y
Concrete	29.625				20.434	
Soil	20.575				22.691	
Earth pressure	14.198	26.682	1.500	0.950	21.297	25.347
Total	**64.398**	**26.682**	1.000	0.950	**64.422**	**25.347**

(3) Moment

Load		Calculation	Horizontal seismic coefficient	Horizental Force	MOMENT (kN·m)	
				H (kN/m)	Y	H · Y
Concrete		8.250	0.115	0.945	0.950	0.898
		0.303	0.115	0.035	0.767	0.027
		1.073	0.115	0.123	0.767	0.094
		0.000	0.115	0.000	0.400	0.000
		0.000	0.115	0.000	0.400	0.000
		0.000	0.115	0.000	0.400	0.000
		15.000	0.115	1.719	0.200	0.344
Sub Total		**24.625**		**2.822**	-	**1.363**
Soil		3.040	0.115	0.348	1.633	0.569
		0.815	0.115	0.093	1.133	0.106
		16.720	0.115	1.916	0.950	1.820
		0.000	0.115	0.000	0.400	0.000
Sub Total		**20.575**		**2.358**	-	**2.495**
Total		**45.200**		**5.180**	0.745	**3.858**

(4) Evaluation of serviceability

▷ Sliding

$\tan\Phi_B$ = 0.500

B' = B - 2 x e = 1.500 - 2 x 0.203 = 1.094 m

C = 0.000 kN/m²

ΣV = 64.398 kN/m

ΣH = 31.861 kN/m

Hu = C x A' + V x $\tan\Phi_B$ = 32.199 kN/m --- **N.G**

F.S = Hu / ΣH = 1.011 < 1.1

ÿ **Need Shear Key.**

◁ Shear Key

V = V = 64.398 kN/m

tanΦB = $\tan\Phi_B$ = 0.500

Kp = Kpe = 2.138

$$Kp = \frac{\cos^2(\varphi - \omega + \theta)}{\cos\omega \cos^2\theta \cos(\delta - \theta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi - \delta)\sin(\varphi - \omega + \beta)}{\cos(\delta - \theta + \omega)\cos(\beta - \theta)}}\right]^2}$$

Where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.00	0.00	9.33	0	6.538

Ppe = 24.054 kN/m

Hk = V $\tan\Phi_B$ + Ppe = 56.253 kN/m

F.S = Hk / ΣH = 1.766 > 1.1 --- **O.K**

ÿ **Stability of sliding is O.K.**

¤ Overturning

e = B/2 - (ΣMr - ΣMo)/ΣV

1.500 / 2 - (64.422 - (25.347 + 3.858)) / 64.398 = 0.203 m

B / 3 = 1.500 / 3 = 0.500 > e --- **O.K**

F.S = ΣMr / ΣMo = 2.206 > 1.5 --- **O.K**

ÿ **Stability of overturning is O.K.**

¥ Bearing Capacity

e = 0.203 m < B/3 = 0.500 m

x = 3·[B/2 - e] = 3 x (1.500 / 2 -0.203) = 1.641 m > 1.500 m

□ resultant in middle one-third of base

q(max,mi = (ΣV / B) x (1 ± 6·e / B)

= 64.398 / 1.500 x (1 ± 6 x 0.203 / 1.500)

qmax = 77.793 kN/m² (Toe)

qmin = 8.071 kN/m² (Heel)

qmax = 77.793 kN/m² < qa = 270.830 kN/m² --- **O.K**

ÿ **Stability of bearing Capacity is O.K.**

3.2.4 Load Calculation

1) Wall

(1) Earth pressure

▷ **At Nomal**

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	4.056

∽ (δ = Λ· x Φ)

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
37.7	9.333	1.100	42.937	7.249	0.631	0.614	0.146
37.8	9.333	1.100	42.512	7.250	0.631	0.614	0.146
37.9	**9.333**	**1.100**	**42.089**	**7.250**	**0.631**	**0.614**	**0.146**
38.0	9.333	1.100	41.667	7.248	0.631	0.613	0.146
38.1	9.333	1.100	41.248	7.245	0.630	0.613	0.146

Horizontal earth pressure : Pah = 1/2 x kah x γt x H^2 = 7.058 kN/m³

Vertical earth pressure : Pav = 1/2 x kav x γt x H^2 = 1.678 kN/m³

◁ **At Earthquake**

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	4.056	6.538

∽ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
33.1	0.000	1.100	64.465	13.007	1.1315	1.129	0.080
33.2	0.000	1.100	63.929	13.008	1.1316	1.129	0.080
33.3	**0.000**	**1.100**	**63.395**	**13.008**	**1.1316**	**1.129**	**0.080**
33.4	0.000	1.100	62.865	13.007	1.1316	1.129	0.080
33.5	0.000	1.100	62.337	13.005	1.1314	1.129	0.080

Horizontal earth pressure : Paeh = 1/2 x kaeh x γt x H^2 = 12.978 kN/m³

Vertical earth pressure : Paev = 1/2 x kaev x γt x H^2 = 0.920 kN/m³

Earthquake earth pressure : Paeh' = 1/2 x kaeh' x γt x H^2 = 5.920 kN/m³

Division		Load	Horizental Force	length (m)	Mr (kN·m)
		W (kN)	H (kN/m)	Y	H x Y
Concrete	▷	8.250	0.945	0.550	0.520
	◁	0.303	0.035	0.367	0.013
	V	1.073	0.123	0.367	0.045
Bottom of Wall		**9.625**	**1.103**		**0.578**
Earthquake earth pressure			**5.920**	0.550	**3.256**
Concrete	▷	4.125	0.473	0.275	0.130
	◁	0.008	0.001	0.183	0.000
	V	0.268	0.031	0.183	0.006
Middle of Wall		**4.401**	**0.504**		**0.136**
Earthquake earth pressure			**1.480**	0.275	**0.407**

(2) Stress Resultant

At Nomal

Bottom of Wall	(H=	1.100 m	)				
V=	7.058				=	7.058	kN
M=	7.058	x	1.100	/ 3	=	2.588	kN/m
Middle of Wall	(H=	0.550 m	)				
V=	1.764				=	1.764	kN
M=	1.764	x	0.550	/ 3	=	0.323	kN/m

At Earthquake

Bottom of Wall	(H=	1.100 m	)				
Ve=	12.978				=	12.978	kN
Me=	12.978	x	1.100	/2	=	7.138	kN/m
Middle of Wall	(H=	0.550 m	)				
Ve=	3.244				=	3.244	kN
Me=	3.244	x	0.550	/ 2	=	0.892	kN/m

(3) Design Load for cross section

Load Combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

Summary

Division		Bottom of Wall		Middle of Wall	
		Horizontal earth pressure	Inertial force	Horizontal earth pressure	Inertial force
LCB1	Shear force	11.293	0.000	2.823	0.000
	Moment	4.141	0.000	0.518	0.000
LCB2	Shear force	11.293	7.023	2.823	1.984
	Moment	4.141	3.834	0.518	0.543
LCB3	Shear force	7.058	0.000	1.764	0.000
	Moment	2.588	0.000	0.323	0.000

V Design Load for cross section

-Bottom of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
11.293	4.141	18.316	7.975	7.058	2.588

-Middle of Wall

LCB1		LCB2		LCB3	
Shear force	Moment	Shear force	Moment	Shear force	Moment
2.823	0.518	4.807	1.061	1.764	0.323

2) Foundation

(1) Stress resultant of Foundation

▷ Load

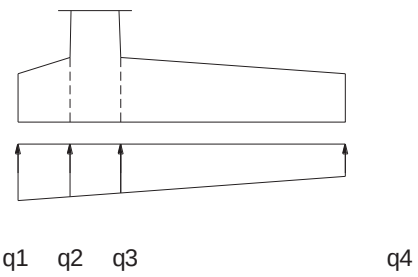
Division		V(kN)	H(kN)	Mr(kN.m)	Mo(kN.m)
At Nomal	Concrete	29.625	0.000	20.434	0.00
	Soil	20.575	0.000	22.691	0.00
	Earth pressure	8.539	18.152	12.809	11.496
	Surface load	0.994	2.113	1.491	1.338
	□	**59.733**	**20.265**	**57.425**	**12.83**
At Earthquake	Concrete	29.625	2.822	20.434	1.363
	Soil	20.575	2.358	22.691	2.495
	Earth pressure Earthquake	8.539	18.152	12.809	11.496
	earth pressure	5.659	8.529	8.488	13.851
	□	**64.398**	**31.861**	**64.422**	**29.21**

◁ Ultimate load Combination

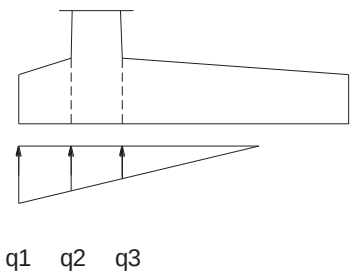
Division	□V	□Mr	□Mo	e	Load shape
LCB1	75.493	74.630	20.535	0.033	CASE1
LCB2	64.502	67.795	35.718	0.253	CASE2
LCB3	59.733	57.425	12.834	0.004	CASE1

V Stress resultant of Foundation

CASE1



CASE2



Division	q1	q2	q3	q4	e	CASE
LCB1	56.972	54.315	50.772	43.685	0.033	1
LCB2	86.522	69.113	45.901	-	0.253	2
LCB3	40.459	40.205	39.865	39.185	0.004	1

(2) Heel

▷ Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Λ'	0.000	0.267	0.000	
	>	8.000	0.400	3.200	
	>	5.000	0.200	1.000	
Sub Total		**13.000**	**0.323**	**4.200**	
Soil	▷	3.040	0.507	1.542	
	▷	16.720	0.400	6.688	
	▷	0.000	0.533	0.000	
Sub Total		**19.760**	**0.417**	**8.230**	
Total		**32.760**	**0.379**	**12.430**	

◁ Cross section force by Vertical Force

- At Nomal

V = 8.539 kN

M = 8.539 x 0.800 = 6.832 kN·m

- At Earthquake

V = 8.539 + 5.659 = 14.198 kN

M1 = 8.539 x 0.800 = 6.832 kN·m

M2 = 5.659 x 0.800 = 6.832 kN·m

V Cross section force by Stress resultant of Foundation

Load	q3	q4	length(m)	V (kN)	M (kN·m)
LCB1	50.772	43.685	0.390	-37.783	-14.735
LCB2	45.901	0.000	0.264	-18.154	-4.787
LCB3	39.865	39.185	0.399	-31.620	-12.612

∞ V = (q4 + q3)/2 × B5

Λ Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	L	H	E	Stress resultant of Foundation	Total
LCB1	Vu	39.312	-	13.663	-	-37.783	15.192
	Mu	14.916	-	10.931	-	-14.735	11.112
LCB2	Vu	29.484	-	13.663	5.659	-18.154	30.652
	Mu	11.187	-	10.931	4.527	-4.787	21.858
LCB3	Vo	32.760	-	8.539	-	-31.620	9.680
	Mo	12.430	-	6.832	-	-12.612	6.650

(3) Toe

▷ Cross section force by Concrete & Soil

Load		Vertical Force	length (m)	Mr (kN·m)	
		V (kN/m)	X	V · X	
Concrete	Å'	0.000	0.100	0.000	
	> '	3.000	0.150	0.450	
Sub Total		**3.000**	**0.150**	**0.450**	

◁ Cross section force by Stress resultant of Foundation

Load	q1	q2	length(m)	V (kN)	M (kN·m)
LCB1	56.972	54.315	0.151	16.693	2.524
LCB2	86.522	69.113	0.156	23.345	3.632
LCB3	40.459	40.205	0.150	12.100	1.817

∞ V = (q1 + q2)/2 × B1

V Design Load for cross section

-Load Combination

LCB 1 : Ultimate Load at nomal (1.2 D + 1.6 L + 1.6 H)

LCB 2 : Ultimate Load at earthquake (0.9 D + 1.6 H + 1.0 E)

LCB 3 : Service Load at nomal (1.0 D + 1.0 L + 1.0 H)

Division		D	H	Stress resultant of Foundation	Total
LCB1	Vu	-3.600	-	16.693	13.093
	Mu	-0.540	-	2.524	1.984
LCB2	Vu	-2.700	-	23.345	20.645
	Mu	-0.405	-	3.632	3.227
LCB3	Vo	-3.000	-	12.100	9.100
	Mo	-0.450	-	1.817	1.367

(4) Shear Key

▷ Passive earth pressure

At Nomal : Pp = 41.489 kN/m

At Earthq : Ppe = 24.054 kN/m Ā Apply Cross section force at Nomal

◁ Design Load for cross section

Division	qk1	qk2	H(m)	V (kN)	M (kN·m)
At Nomal	66.382	99.573	0.500	41.489	11.064

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)
Design Load for cross section	17.702	66.382	11.064

3) Summary

Division	Mu(kN·m)	Vu(kN)	Mo(kN·m)	ØMn(kN·m)	Bar	S.F
Bottom of Wall	7.975	18.316	2.588	165.610	D13 @ 125	20.77
Middle of Wall	1.061	4.807	0.323	56.513	D13 @ 250	53.28
Heel	7.975	30.652	2.588	198.822	D16 @ 125	24.93
Toe	3.227	20.645	1.367	101.157	D16 @ 250	31.34
Shear Key	17.702	66.382	11.064	66.265	D13 @ 250	3.74

3.2.5 Section Design

1) Bottom of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	7.975	kN·m	V_u	=	18.316	kN	M_o	=		2.588	kN·m		

- Check of Strength reduction factor (Φ)

a = 19.133

Because T = C , *c* = 19.133 / β1 = 19.133 / 0.821 = 23.293 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 23.293) / 23.293

= 0.0412

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□.T TΦΓTYKYΦYZT HΠΣ И□.T

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$ □ Req. As = 61.502 ㎟

Use As = D 13 @ 250 + D 16 @ 250 = 1312.00 ㎟ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00024	㎟	$A_{s,4/3req}$	=	82.0	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00024	㎟	$A_{s,min}$	=	82.0	㎟
P_{use}	=	A_s / (B·d)	=	0.00382	㎟	$A_{s,min}$	=	1312.0	㎟

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 19.133 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 165.610 kN·m > Mu = 7.975 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 242.891 kN > Vu = 18.316 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 1,312.00 / 1000 + 8 × 1,312.00 / 1000 × √[1 + 2 × 1000 × 343.5 / (8 × 1,312.00)]

= 75.066 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 2.588 / [1000 × 75.066 x (343.5 - 75.066 / 3)] × 10^6

= 0.216 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 2.588 / [1312.000 x (343.5 - 75.066 / 3)] × 10^6

= 6.193 MPa

fst = fs x (H - d' - χ) / (d - χ) = 6 x (400 - 57 - 0) / (344 - 75) = 6.19 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 6.19) - 2.5 x 50.00 = 17054.33 ㎜

300 x (280 / fs) = 300 x (280 / 6.19) = 13562.63 ㎜

Sa = 13562.63 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (13562.63 mm) Ā O.K

2) Middle of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82				
Øf	=	0.90		Øv	=	0.75			d	=	293.5	mm			
B	=	1000	mm	H	=	350	mm		d'	=	56.5	mm			
M_u	=	1.061	kN·m	V_u	=	4.807	kN	M_o	=		0.323	kN·m			

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (293.5 - 9.161) / 9.161

= 0.0931

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 9.562 ㎠

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎠ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313				
P_{max}	=	0.75 · Pb	=	0.02485	㎠	$A_{s,max}$	=	7293.5 ㎠
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎠	$A_{s,min}$	=	988.3 ㎠
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00004	㎠	$A_{s,4/3req}$	=	12.7 ㎠
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00004	㎠	$A_{s,min}$	=	12.7 ㎠
P_{use}	=	A_s / (B·d)	=	0.00176	㎠	$A_{s,min}$	=	516.0 ㎠

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

ØMn = 0.9 × As × fy × (d - a/2) = 56.513 kN·m > Mu = 1.061 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 207.536 kN > Vu = 4.807 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 516.00 / 1000 + 8 × 516.00 / 1000 × √[1 + 2 × 1000 × 293.5 / (8 × 516.00)]

= 45.270 ㎜

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 0.323 / [1000 × 45.270 x (293.5 - 45.270 / 3)] × 10^6

= 0.051 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 0.323 / [516.000 x (293.5 - 45.270 / 3)] × 10^6

= 2.252 MPa

fst = fs x (H - d' - χ) / (d - χ) = 2 x (350 - 57 - 0) / (294 - 45) = 2.25 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 2.25) - 2.5 x 50.00 = 47126.69 ㎜

300 x (280 / fs) = 300 x (280 / 2.25) = 37303.97 ㎜

Sa = 37303.97 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (37303.97 mm) Ā O.K

3) Heel

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	342.0	mm		
B	=	1000	mm	H	=	400	mm		d'	=	58.0	mm		
M_u	=	7.975	kN·m	V_u	=	30.652	kN	M_o	=		2.588	kN·m		

- Check of Strength reduction factor (Φ)

a = 23.217

Because T = C , *c* = 23.217 / β1 = 23.217 / 0.821 = 28.264 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (342.0 - 28.264) / 28.264

= 0.0333

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИД.Т ТФГТУКУФУZT НПΣ ИД.Т

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 61.773 ㎟

Use As = D 16 @ 250 + D 16 @ 250 = 1592.00 ㎟ (8 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8498.7	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1151.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00024	㎟	$A_{s,4/3req}$	=	82.4	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00024	㎟	$A_{s,min}$	=	82.4	㎟
P_{use}	=	A_s / (B·d)	=	0.00465	㎟	$A_{s,min}$	=	1592.0	㎟

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 23.217 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 198.822 kN·m > Mu = 7.975 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 241.831 kN > Vu = 30.652 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

$\chi = -nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

$= -8 \times 1{,}592.00 / 1000 + 8 \times 1{,}592.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 342 / (8 \times 1{,}592.00)]}$

= 81.464 ㎜

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 2.588 / [1000 \times 81.464 \times (342.0 - 81.464 / 3)] \times 10^6$

= 0.202 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 2.588 / [1592.000 \times (342.0 - 81.464 / 3)] \times 10^6$

= 5.163 MPa

fst = fs x (H - d' - χ) / (d - χ) = 5 x (400 - 58 - 0) / (342 - 81) = 5.16 MPa

ⓐ Maximum center space of reinforcement

C_C = 58.00 - 16.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 5.16) - 2.5 x 50.00 = 20482.87 ㎜

300 x (280 / fs) = 300 x (280 / 5.16) = 16269.37 ㎜

Sa = 16269.37 ㎜ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (16269.37 mm) Ā O.K

4) Toe

(1) Section Design

ⓐ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	342.0	mm		
B	=	1000	mm	H	=	400	mm		d'	=	58.0	mm		
M_u	=	3.227	kN·m	V_u	=	20.645	kN	M_o	=		1.367	kN·m		

- Check of Strength reduction factor (Φ)

a = 11.608

Because T = C , *c* = 11.608 / β1 = 11.608 / 0.821 = 14.132 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (342.0 - 14.132) / 14.132

= 0.0696

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

ИЦ ТФГТУКУФУZТ НПΣ ИЦ

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 24.979 ㎟

Use As = D 16 @ 500 + D 16 @ 500 = 796.00 ㎟ (4 ea/m)

ⓐ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎟	$A_{s,max}$	=	8498.7	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎟	$A_{s,min}$	=	1151.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00010	㎟	$A_{s,4/3req}$	=	33.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00010	㎟	$A_{s,min}$	=	33.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00233	㎟	$A_{s,min}$	=	796.0	㎟

ⓒ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ⓐ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 11.608 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 101.157 kN·m > Mu = 3.227 kN·m

Ā O.K

ⓐ Shear Check

ØVc = 0.75 × 1/6 × √ fc' × B × d = 241.831 kN > Vu = 20.645 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 8 × 796.00 / 1000 + 8 × 796.00 / 1000 × √[1 + 2 × 1000 × 342 / (8 × 796.00)]

= 59.936 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 1.367 / [1000 × 59.936 x (342.0 - 59.936 / 3)] × 10^6

= 0.142 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 1.367 / [796.000 x (342.0 - 59.936 / 3)] × 10^6

= 5.332 MPa

fst = fs x (H - d' - χ) / (d - χ) = 5 x (400 - 58 - 0) / (342 - 60) = 5.33 MPa

ⓐ Maximum center space of reinforcement

C_C = 58.00 - 16.00 / 2 = 50.00 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 5.33) - 2.5 x 50.00 = 2.0E+04 ⌟

300 x (280 / fs) = 300 x (280 / 5.33) = 1.6E+04 ⌟

Sa = 1.58E+04 ⌟ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (1.6E+04 mm) Ā O.K

5) Shear Key

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82			
Øf	=	0.90		Øv	=	0.75			d	=	343.5	mm		
B	=	1000	mm	H	=	400	mm		d'	=	56.5	mm		
M_u	=	17.702	kN·m	V_u	=	66.382	kN	M_o	=		11.064	kN·m		

- Check of Strength reduction factor (Φ)

a = 7.525

Because T = C , *c* = 7.525 / β1 = 7.525 / 0.821 = 9.161 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (343.5 - 9.161) / 9.161

= 0.1095

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

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$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 136.754 ㎟

Use As = D 13 @ 500 + D 13 @ 500 = 516.00 ㎟ (4 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	ㅿ	$A_{s,max}$	=	8536.0	㎟
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	ㅿ	$A_{s,min}$	=	1156.6	㎟
$P_{4/3req}$	=	4/3 · $A_{s.req}$ / (B·d)	=	0.00053	ㅿ	$A_{s,4/3req}$	=	182.3	㎟
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00053	ㅿ	$A_{s,min}$	=	182.3	㎟
P_{use}	=	A_s / (B·d)	=	0.00150	ㅿ	$A_{s,min}$	=	516.0	㎟

∠ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 7.525 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 66.265 kN·m > Mu = 17.702 kN·m

Ā O.K

ⓐ Shear Check

$\varnothing V_c = 0.75 \times 1/6 \times \sqrt{f_c'} \times B \times d$ = 242.891 kN > Vu = 66.382 kN

Ā No shear reinforcement is required

(2) Crack Check

ⓐ Calculation of stress

n = 8

$\chi = -nA_s/b + nA_s/b \times \square\{1+2bd/nA_s\}$

$= -8 \times 516.00 / 1000 + 8 \times 516.00 / 1000 \times \sqrt{[1 + 2 \times 1000 \times 343.5 / (8 \times 516.00)]}$

= 49.285 ㎜

$f_c = 2 \times M_o / [B \times \chi \times (d - \chi/3)]$

$= 2.0 \times 11.064 / [1000 \times 49.285 \times (343.5 - 49.285 / 3)] \times 10^6$

= 1.373 MPa

$f_s = M_o / [A_s \times (d - \chi/3)]$

$= 11.064 / [516.000 \times (343.5 - 49.285 / 3)] \times 10^6$

= 65.557 MPa

fst = fs x (H - d' - χ) / (d - χ) = 66 x (400 - 57 - 1) / (344 - 49) = 65.56 MPa

ⓐ Maximum center space of reinforcement

C_C = 56.50 - 13.00 / 2 = 50.00 ㎜

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 65.56) - 2.5 x 50.00 = 1498.01 ㎜

300 x (280 / fs) = 300 x (280 / 65.56) = 1281.33 ㎜

Sa = 1281.33 ㎜ Applying Minimum value

S = 1,000 / 4 Ea = 250.0 < Sa (1281.33 mm) Ā O.K

3.2.6 Distribution Reinforcement Check

1) Wall (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As :

Tension side	D	13@	200	=	645.0	㎟			
Compression side	D	13@	200	=	645.0	㎟			
			□	=	1290.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 200 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 400 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 400 = 720.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

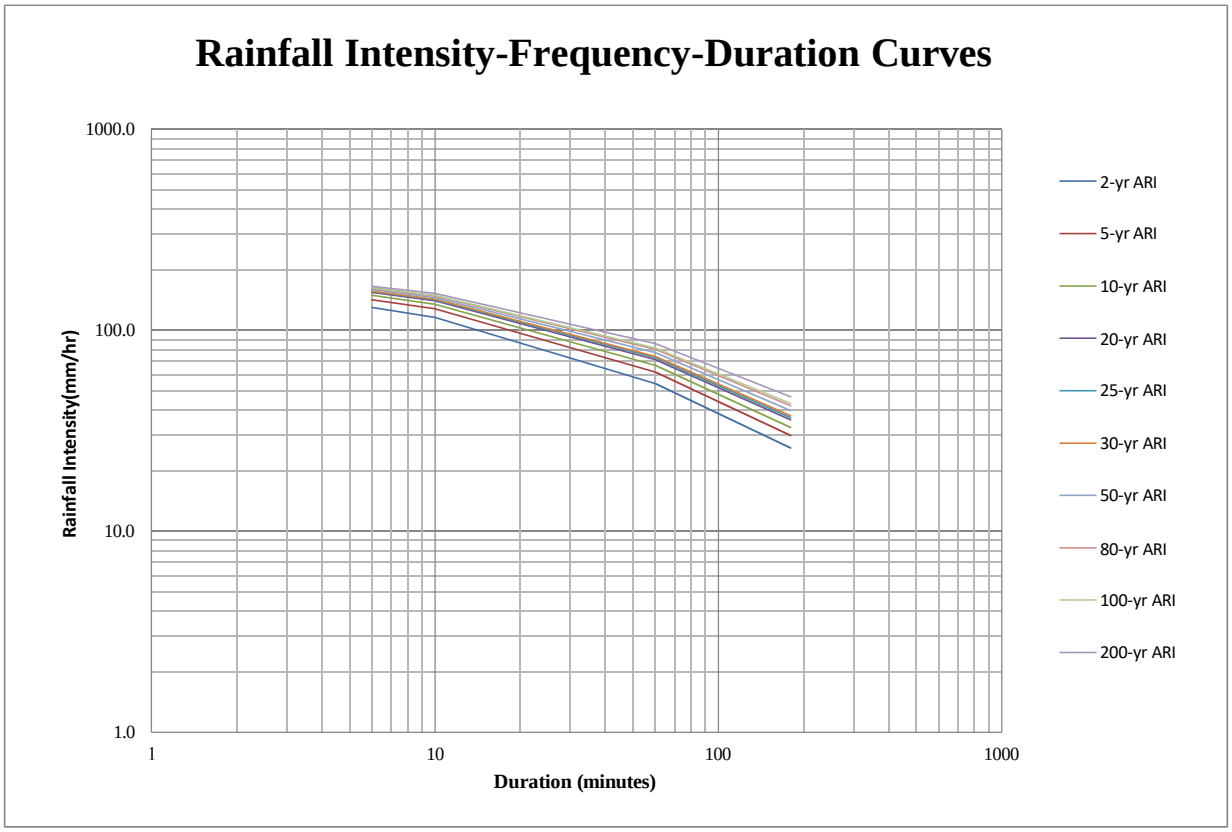
· Used As :

Tension side	D	13@	250	=	516.0	㎟			
Compression side	D	13@	250	=	516.0	㎟			
			□	=	1032.0	㎟	>	720.0 ㎟	Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

4 .HYDRAULIC CALCULATION

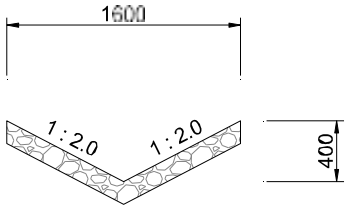
4.1 U-Ditch Calculation



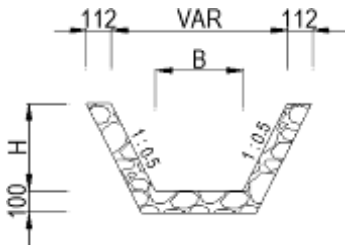
Duration (mins) \ Rainfall Intensity (mm/hr)	2-yr ARI	5-yr ARI	10-yr ARI	20-yr ARI	25-yr ARI	30-yr ARI	50-yr ARI	80-yr ARI	100-yr ARI	200-yr ARI
6	129.7	**142.1**	149.2	154.2	**155.8**	156.8	159.3	161.9	162.9	165.6
10	115.9	127.9	134.9	140.1	141.8	142.9	145.7	148.4	149.5	152.6
60	54.3	62.0	66.9	71.5	73.0	74.2	77.4	80.3	81.7	85.9
180	25.9	30.0	32.9	35.8	36.8	37.6	39.9	42.1	43.2	46.6
360	14.5	16.9	18.7	20.7	21.4	22.0	23.8	25.7	26.7	30.7
540	10.5	12.7	14.7	17.0	17.8	18.4	20.6	22.8	23.9	28.1
720	8.2	10.4	12.3	14.6	15.4	16.2	18.4	20.7	21.9	26.3
900	6.8	8.9	10.7	13.0	13.8	14.5	16.8	19.1	20.4	24.8
1080	5.8	7.8	9.6	11.8	12.6	13.3	15.5	17.9	19.2	23.6
1440	4.6	6.3	8.0	10.0	10.8	11.5	13.7	16.0	17.3	21.7

A. Calculation of Earth Drain & U-type Ditch

1. EARTH DRAIN- LINED



2.U-type ditch



1) Flow rate in Earth drain

① Earth Drain Q= A X V = A × (1/n) × $R^{2/3}$ × $I^{1/2}$ (n=0.020)

TYPE	Depth of Flow (0.8H)	Width(D) (m)	Area of the flow (m^2)	Wetted perimeter (m)	Hydraulic Radius (m)	$R^{2/3}$	Q m^3/sec
LINED DITCH(H=0.9M)	0.32	1.280	0.2048	0.716	0.2860	0.4341	4.4452 × $I^{1/2}$

① Utype- ditch(RIPRAP) Q= A X V = A × (1/n) × $R^{2/3}$ × $I^{1/2}$ (n=0.025)

TYPE	Depth of Flow (0.8H)	Width(D) (m)	Area of the flow (m^2)	Wetted perimeter (m)	Hydraulic Radius (m)	$R^{2/3}$	Q m^3/sec
TYPE-4(B,H=0.40M)	0.32	0.720	0.1792	1.116	0.1606	0.2955	2.1181 × $I^{1/2}$
TYPE-6(B,H=0.60M)	0.48	1.080	0.4032	1.673	0.2410	0.3873	6.2464 × $I^{1/2}$

2) The Rational Method formula for flow estimation Q(m^3/sec)

① Plain farming area

$Qd_1 = 0.278 \times C_1 \times I_1 \times A_1$

where, C_1, C_2 : Run-off Coefficient

I_1, I_2 : Rainfall intensity(mm/hr)

A_1, A_2 : Catchment Area(km^2)

② Paved surface

$Qd_2 = 0.278 \times C_2 \times I_2 \times A_2$

C_1 : 0.8 C_2 : 0.9

I_1(5yr) : 142.1 I_2(5yr) : 142.1

③ Peak Catchment Discharge $Qd = Qd_1 + Qd_2$

3) Determination of Cross section

Compare Flow rate with the Peak Catchment Discharge according to Runoff Coefficient and Catchment Area

※ **Q > Qd (OK)**

U-Ditch Hydrological Caculation (1)

NO.	Station(Lot-2-1) BP	Station(Lot-2-1) EP	Direction	Catchment Area (㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope: LINED DITCH	Flow rate according to slope: TYPE-4	Flow rate according to slope: TYPE-6	Peak Catchment Discharge (㎥/sec) (Qd_1,Qd_2)	Peak Catchment Discharge (㎥/sec) Qd	Determination: Check Capacity	Determination: Apply	Remark
1	0+000	0+090	R	0.0007	0.8	6.81	1.160	0.553	1.630	0.022	0.033	L/D	L/D	LINED DITCH
				0.0003	0.9					0.011				
2	0+150	0+210	R	0.0002	0.8	4.35	0.927	0.442	1.303	0.006	0.013	U4	U4	U-TYPE
				0.0002	0.9					0.007				
3	0+220	0+320	R	0.0010	0.8	16.35	1.797	0.856	2.526	0.032	0.138	U4	U4	U-TYPE
				0.0030	0.9					0.107				
4	0+320	0+662	R	0.0046	0.8	11.32	1.496	0.713	2.102	0.145	0.188	U4	U4	U-TYPE
				0.0012	0.9					0.043				
5	0+670	0+960	R	0.0037	0.8	10.75	1.457	0.694	2.048	0.117	0.156	U4	U4	U-TYPE
				0.0011	0.9					0.039				
6	0+960	1+185	R	0.0022	0.8	1.94	0.619	0.295	0.870	0.070	0.098	U4	U4	U-TYPE
				0.0008	0.9					0.028				
7	1+195	1+310	R	0.0002	0.8	0.91	0.424	0.202	0.596	0.006	0.021	U4	U4	U-TYPE
				0.0004	0.9					0.014				
8	1+590	1+645	R	0.0002	0.8	6.05	1.093	0.521	1.536	0.006	0.013	U4	U4	U-TYPE
				0.0002	0.9					0.007				
9	1+645	1+810	R	0.0006	0.8	9.50	1.370	0.653	1.925	0.019	0.040	U4	U4	U-TYPE
				0.0006	0.9					0.021				
10	1+870	1+970	R	0.0004	0.8	12.55	1.575	0.750	2.213	0.013	0.030	U4	U4	U-TYPE
				0.0005	0.9					0.018				
11	2+150	2+310	R	0.0010	0.8	11.62	1.515	0.722	2.129	0.032	0.053	U4	U4	U-TYPE
				0.0006	0.9					0.021				
12	2+350	2+504	R	0.0005	0.8	11.59	1.513	0.721	2.127	0.016	0.034	U4	U4	U-TYPE
				0.0005	0.9					0.018				

U-Ditch Hydrological Caculation (2)

NO.	Station(Lot-2-1)		Direction	Catchment Area (㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope			Peak Catchment Discharge (㎥/sec)		Determination		Remark
	BP	EP					LINED DITCH	TYPE-4	TYPE-6	(Qd_1,Qd_2)	Qd	Check Capacity	Apply	
13	0+000	0+110	L	0.0002	0.8	6.08	1.096	0.522	1.540	0.006	0.017	L/D	L/D	LINED DITCH
				0.0003	0.9					0.011				
14	0+250	0+310	L	0.0003	0.8	11.40	1.501	0.715	2.109	0.009	0.017	U4	U4	U-TYPE
				0.0002	0.9					0.007				
15	0+330	0+395	L	0.0020	0.8	10.62	1.449	0.690	2.036	0.063	0.102	U4	U4	U-TYPE
				0.0011	0.9					0.039				
16	0+418	0+630	L	0.0020	0.8	11.39	1.500	0.715	2.108	0.063	0.102	U4	U4	U-TYPE
				0.0011	0.9					0.039				
17	0+710	0+970	L	0.0012	0.8	10.05	1.409	0.671	1.980	0.038	0.070	U4	U4	U-TYPE
				0.0009	0.9					0.032				
18	0+990	1+050	L	0.0001	0.8	2.17	0.655	0.312	0.920	0.003	0.010	U4	U4	U-TYPE
				0.0002	0.9					0.007				
19	1+070	1+130	L	0.0002	0.8	2.17	0.655	0.312	0.920	0.006	0.013	U4	U4	U-TYPE
				0.0002	0.9					0.007				
20	1+230	1+310	L	0.0009	0.8	0.59	0.341	0.163	0.480	0.028	0.039	U4	U4	U-TYPE
				0.0003	0.9					0.011				
21	1+395	1+644	L	0.0003	0.8	7.76	1.238	0.590	1.740	0.009	0.027	U4	U4	U-TYPE
				0.0005	0.9					0.018				
22	1+644	1+842	L	0.0017	0.8	8.91	1.327	0.632	1.865	0.054	0.079	U4	U4	U-TYPE
				0.0007	0.9					0.025				
23	1+858	2+010	L	0.0010	0.8	11.53	1.509	0.719	2.121	0.032	0.053	U4	U4	U-TYPE
				0.0006	0.9					0.021				
24	2+090	2+340	L	0.0014	0.8	11.59	1.513	0.721	2.127	0.044	0.076	U4	U4	U-TYPE
				0.0009	0.9					0.032				
25	2+340	2+504	L	0.0025	0.8	11.66	1.518	0.723	2.133	0.079	0.100	U4	U4	U-TYPE
				0.0006	0.9					0.021				

U-Ditch Hydrological Caculation (3)

No.	Station(Lot-2-2)		Direction	Catchment Area(㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope		Peak Catchment Discharge (㎥/sec)		Determination		Remark
	BP	EP					TYPE-4	TYPE-6	(Qd_1,Qd_2)	Qd	Check Capacity	Apply	
1	0+000.0	0+087.6	R	0.0009	0.8	8.11	0.603	1.779	0.028	0.035	U4	U4	
				0.0002	0.9				0.008				
2	0+087.6	0+410	R	0.0069	0.8	10.13	0.674	1.988	0.218	0.261	U4	U4	
				0.0012	0.9				0.043				
3	0+468	0+590	R	0.0021	0.8	11.41	0.715	2.110	0.066	0.087	U4	U4	
				0.0006	0.9				0.021				
4	0+610	0+830	R	0.0031	0.8	11.45	0.717	2.114	0.099	0.125	U4	U4	
				0.0007	0.9				0.026				
5	0+870	0+910	R	0.0002	0.8	12.00	0.734	2.164	0.006	0.011	U4	U4	
				0.0002	0.9				0.006				
5	0+930	1+210	R	0.0044	0.8	10.91	0.700	2.063	0.139	0.174	U4	U4	
				0.0010	0.9				0.035				
6	1+230	1+540	R	0.0160	0.8	12.02	0.734	2.166	0.505	0.563	U4	U4	
				0.0016	0.9				0.057				
7	1+540	1+810	R	0.0067	0.8	10.47	0.685	2.021	0.211	0.250	U4	U4	
				0.0011	0.9				0.039				
8	1+836	1+985	R	0.0051	0.8	4.66	0.457	1.348	0.162	0.182	U4	U4	
				0.0006	0.9				0.021				
9	1+995	2+400	R	0.0169	0.8	7.96	0.598	1.762	0.535	0.573	U4	U4	
				0.0011	0.9				0.038				
10	2+620	2+732	R	0.0022	0.8	12.98	0.763	2.250	0.070	0.070	U4	U4	
				0.0000	0.9				0.000				
11	0+000.0	0+070	L	0.0003	0.8	19.31	0.931	2.745	0.011	0.023	U4	U4	
				0.0003	0.9				0.012				
12	0+270	0+510	L	0.0028	0.8	12.81	0.758	2.236	0.087	0.132	U4	U4	
				0.0013	0.9				0.045				

U-Ditch Hydrological Caculation (4)

No.	Station(Lot-2-2)		Direction	Catchment Area(㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope		Peak Catchment Discharge (㎥/sec)		Determination		Remark
	BP	EP					TYPE-4	TYPE-6	(Qd_1,Qd_2)	Qd	Check Capacity	Apply	
13	0+528	0+710	L	0.0030	0.8	13.73	0.785	2.315	0.093	0.130	U4	U4	
				0.0010	0.9				0.037				
14	0+840	1+010	L	0.0035	0.8	8.55	0.619	1.826	0.109	0.171	U4	U4	
				0.0017	0.9				0.062				
15	1+090	1+310	L	0.0022	0.8	14.14	0.796	2.349	0.071	0.104	U4	U4	
				0.0009	0.9				0.033				
16	2+070	2+360	L	0.0014	0.8	4.85	0.466	1.376	0.044	0.069	U4	U4	
				0.0007	0.9				0.025				
17	2+340	2+400	R	0.0006	0.8	2.64	0.344	1.015	0.019	0.071	U4	U4	
				0.0015	0.9				0.052				
18	2+398	2+460	L	0.0002	0.8	11.37	0.714	2.106	0.007	0.033	U4	U4	
				0.0007	0.9				0.025				
19	2+486	2+660	R	0.0016	0.8	4.48	0.448	1.322	0.052	0.052	U4	U4	
				0.0000	0.9				0.000				
20	2+660	2+696	R	0.0005	0.8	0.50	0.150	0.442	0.017	0.017	U4	U4	
				0.0000	0.9				0.000				

4.2 Calculation of RC open channel

B. Calculation of Reinforced concrete open channel

Calculation of Q

Qd = 0.278CIA

Qd : Design flow rate (m^3/sec)

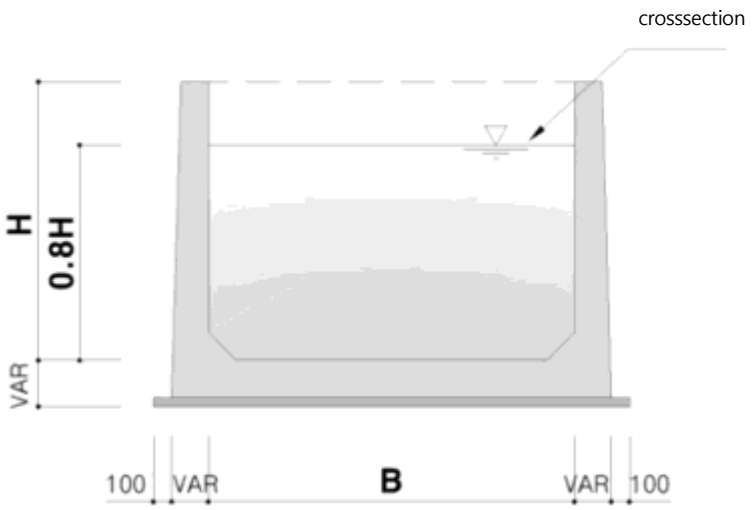
I : Rainfall intensity with rainfall duration for Tc (mm/h)

C : Runoff coefficient

A : Catchment area (km^2)

t = 6 min

The rainfall intensity (5years frequency) I= 142.10 mm/h



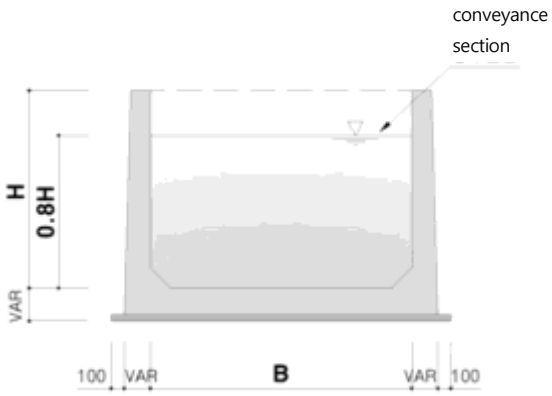
- A = B x 0.8 H = 0.8 x B H
- P(Wetted perimeter) = 2 x 0.8 H + B = 1.6 H +B
- R(Hydraulic Radius) = A/P = 0.8 x B H / 1.6 H + B

Q = A x 1/n x R ⅔ x I ½

Calculation of Reinforced Concrete Open Channel

♣ LOT 2-3 : STA. 2+430.0 ~ STA.2+610.0 (Right, L=194m)

◎ **Hydrological Calculation by rational equation**



Qd = 0.278 x C x I x A

Therefore, C : Runoff coefficient

Steep mountain and slope surface : 0.8

I : Rainfall intensity

-Distance of arrival : 1,255.0m 194.0m

-Height difference : 268.0m, 20.7m

-Rainfall intensity 5 years applied

I= 142.10 mm/hr

A : Catchment Area

0.411 ㎢

Qd = 0.278 x 0.8 x 142.1 x 0.411

= 13 ㎥/sec

◎ **Calculation of Q**

◈ A(Cross Section(80%)) = B x 0.8 H = 0.8 x B H---------- ①

◈ P(Wetted perimete)

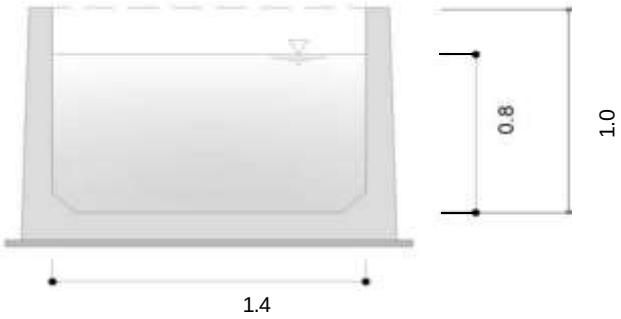
P = 2 x 0.8 H + B = 1.6 H +B----------②

◈ R(Hydraulic Radius) = A/P = 0.8 x B H / 1.6 H + B----------③

Q = A x 1/n x R $^{2/3}$ x I ½

n = 0.012(Concrete applied)

♣ Determination of R.C Open channel (1.4 X 1.0)



B : 1.4 H : 1.0

1 A

A = B x 0.8 H

= 1.4 x 0.8 = 1.12 m^2

2 P

P = 0.8(2* H) + B

= 1.6 + 1.4 = 3.00

3 R

R = A / P

= 1.12 ÷ 3.00 = 0.373

4 n = 0.012

5 I = 10.68 % (Slope = 10.68)

I½ = 0.107 ½ = 0.3268

※ Q = A x V

V = 1/n x R ⅔ x I ½

= 83.333 x 0.518 x 0.33 = 14.1

A = 1.12

Q = 1.12 X 14.119 = 15.81

♥ Qd(13.00) < Q (15.81) OK

※ Determinated by 1.4 X 1.0

4.3 Culvert Design Calculation

Culvert Design Calculations

STATION : (Access Road LOT 2-1) 0+320.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-1 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

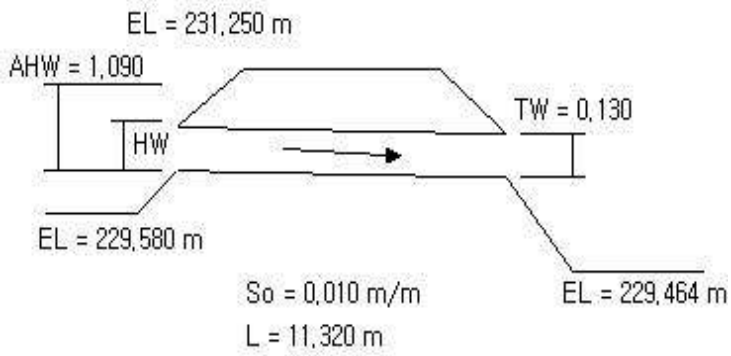
Catchment Area(A) =0.0063 ㎢
Qd = 0.218 ㎥/sec
Arrival Distance(L) = 327.300 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 269.76 m - 229.58 m
= 40.17 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0063 ㎢ Qd = 0.218 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.267

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.218	Φ900	0.36	0.33	0.5	0.0116	0.268	0.584	0.13	0.67	0.12	0.57	0.33	1.38	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 1.38 m/sec
2. Qd = AVo, Qd = 0.2188 < AVo= (πd²/4) × 0.75 × 1.38 = 0.658 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 0+320.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0063 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.218 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 40.17 m**

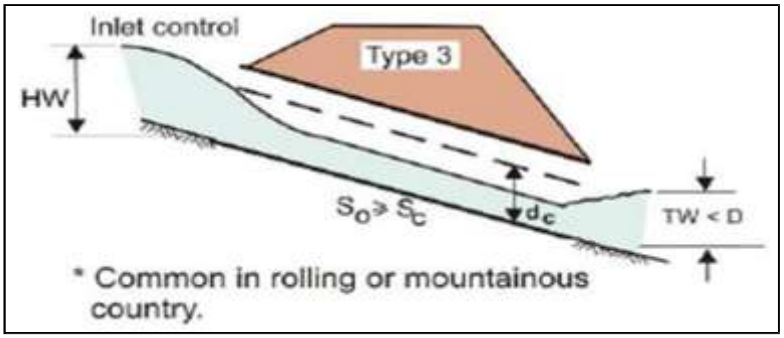
1. Hydrological Analysis

HW (0.33) ≤ 1.2 D (1.08) , So (0.0102) ≥ Sc (0.0102)

TW (0.13) ≤ dc (0.268) < D (0.9) , dn (0.267) < dc (0.268) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 1.380 m / sec.

2. Analysis of the discharge area

Qd =0.218㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.218) < Q = AVo = (πd²/4) × 0.75 × 1.380 = 0.658 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 2-1) 2+340.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-1 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

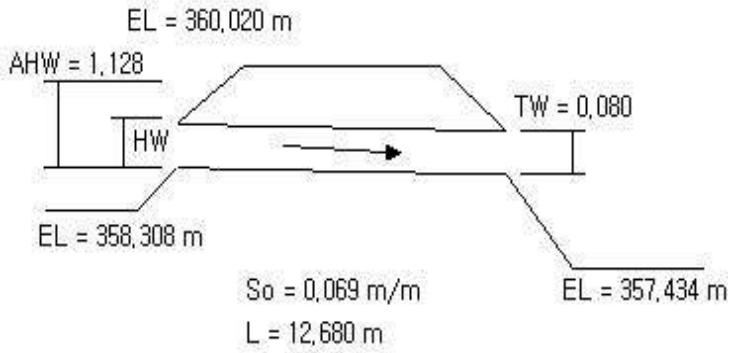
Catchment Area(A) =0.0053 ㎢
Qd = 0.184 ㎥/sec
Arrival Distance(L) = 78.400 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 385.50 m - 358.31 m
= 27.19 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0053 ㎢ Qd = 0.184 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.153

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.184	Φ900	0.33	0.29	0.5	0.0281	0.245	0.573	0.08	0.67	0.87	-0.17	0.29	2.58	Inlet	O.K

Summary and Recommndations :

1. Outlet velocity is 2.58 m/sec, will require a rock apron protection Outlet.
2. Qd = AVo, Qd = 0.184 < AVo= (πd²/4) × 0.75 × 2.58 = 1.231 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 2+340.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0053 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.184 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 27.19 m**

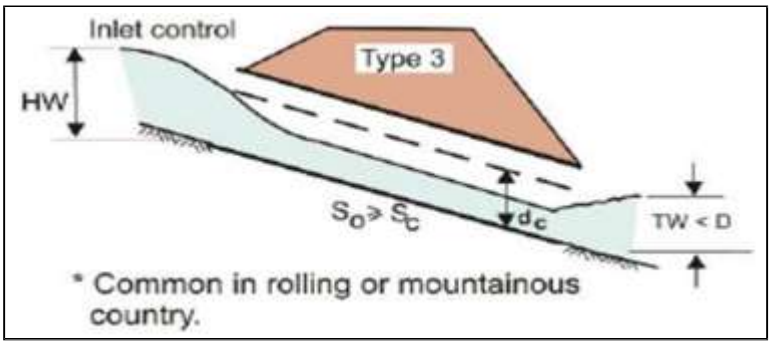
1. Hydrological Analysis

HW (0.29) ≤ 1.2 D (1.08) , So (0.0689) ≥ Sc (0.0102)

TW (0.08) ≤ dc (0.245) < D (0.9) , dn (0.153) < dc (0.245) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 2.580 m / sec.

2. Analysis of the discharge area

Qd =0.184㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.184) < Q = AVo = (πd²/4) × 0.75 × 2.580 = 1.231 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 2-2) 1+540.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-2 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

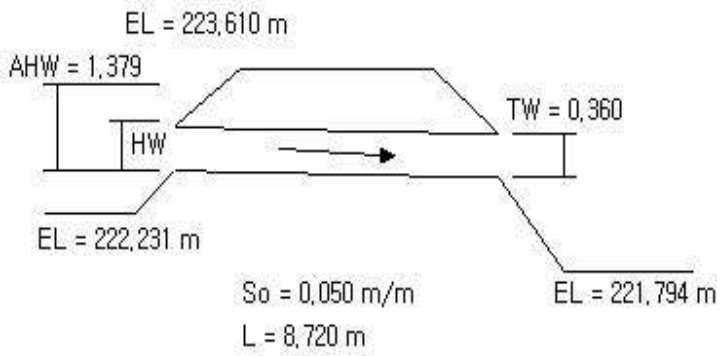
Catchment Area(A) =0.0386 km²
Qd = 1.337 m³/sec
Arrival Distance(L) = 244.50 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 345.00m - 222.23 m
= 115.77 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.80 mm/h C = 0.800 A = 0.0386 km² Qd = 1.337 m³/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m³/sec Qd = 0.000 m³/sec	Chupu Karma gauging station

Water level in Culvert = 0.441

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m³/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	1.337	Φ900	1.86	1.68	0.5	1.126	0.684	0.792	0.36	0.67	0.44	1.36	1.68	4.00	Inlet	N.G
PIPE	1.337	Φ1000	1.01	1.01	0.5	0.671	0.666	0.833	0.36	0.75	0.44	0.98	1.01	4.00	Inlet	O.K

Summary and Recommndations : 1. Qd = AVo, Qd = 1.337 < AVo= (πd²/4) × 0.75 × 4.00 = 2.356 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 1+540.00

⊙ **Input Sectional Shape (Φ1000)**

·Catchment Area(A) = 0.0386 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 1.337 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 115.77 m**

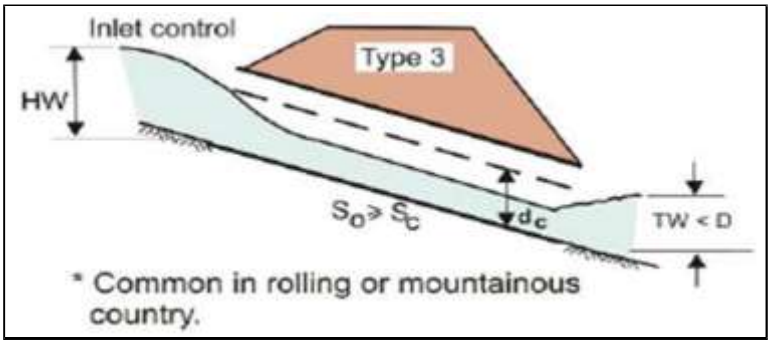
1. Hydrological Analysis

HW (1.01) ≤ 1.2 D (1.20) , So (0.5000) ≥ Sc (0.0132)

TW (0.36) ≤ dc (0.666) < D (1.0) , dn (0.441) < dc (0.666) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of the outlet

and Vc = 4.000 m / sec.

2. Analysis of the discharge area

Qd =1.337㎥/sec of the drainage area at the current STA., and the Pipe size is φ1000

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (1.337) < Q = AVo = (πd²/4) × 0.75 × 4.00 = 2.356 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 2-2) 1+820.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-2 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

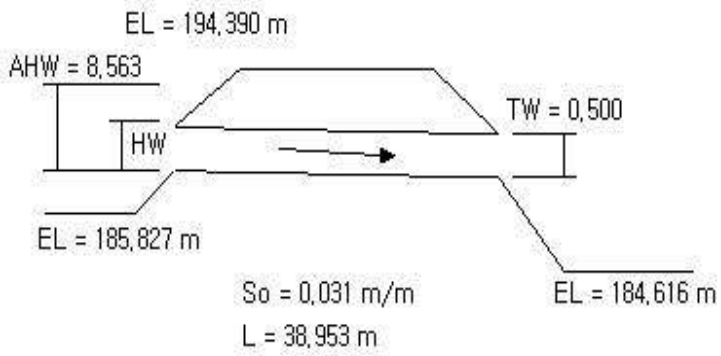
Catchment Area(A) =0.1150 ㎢
Qd = 3.985 ㎥/sec
Arrival Distance(L) = 443.55 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 415.00m - 185.83 m
= 229.17 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.115 ㎢ Qd = 3.985 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.978

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	3.985	Φ1000			0.5	19.807	0.978	0.989	0.50	0.75	1.21					N.G
PIPE	3.985	Φ1200	1.67	2.00	0.5	1.964	1.070	1.135	0.50	0.90	1.21	1.65	2.00	4.28	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd = 4.28 > AVo= (πd²/4) × 0.75 × 4.28 = 4.28 therefore,"N.G"
2. Qd = Avo "N.G",Type-5 hydraulic flow in which the inlet submerge in water. therefore, AHW=8.563 > HW=2.0 "O.K"

Review the Hydrological Calculation Results

STA. 1+820.00

⊙ **Input Sectional Shape (Φ1200)**

·Catchment Area(A) = 0.1150 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 3.985 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 229.17 m**

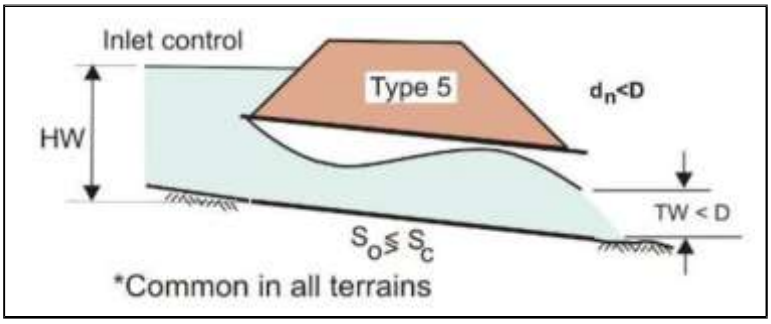
1. Hydrological Analysis

HW (2.00) ≤ 1.2 D (1.44) , TW (0.50) < D (1.2)

dn (0.978) < D (1.2) therefore,

The shape proposed is 5rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the inlet, and the applied energy equation is

$$HW = \frac{D}{2} + \frac{V_d^2}{2g} \cdot (1 + C_e)$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the nomal depth of the outlet

and VD = 4.280 m / sec.

2. Analysis of the discharge area

Qd =3.985㎥/sec of the drainage area at the current STA., and the Pipe size is φ1200

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (3.985) < Q = AVo = (πd²/4) × 0.75 × 4.280 = 3.630 : N.G

Culvert Design Calculations

STATION : (Access Road LOT 2-2) 1+990.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-2 C-3

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

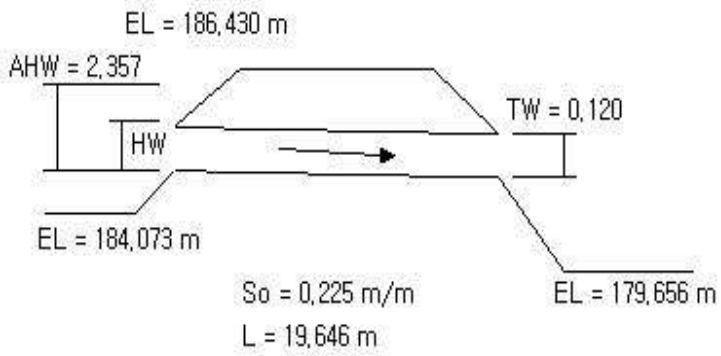
Catchment Area(A) =0.136 ㎢
Qd = 4.712 ㎥/sec
Arrival Distance(L) = 605.00 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 438.00m - 184.07 m
= 254.43 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.1360 ㎢ Qd = 4.712 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.260

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	2.356	2@ Φ1000	3.89	3.89	0.5	1.9365	0.624	0.812	0.12	0.75	4.42	-1.73	3.89	6.65	Inlet	N.G
PIPE	2.356	2@ Φ1200	0.90	1.08	0.5	0.8040	0.590	0.895	0.12	0.90	4.42	-2.71	1.08	6.53	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd =4.712 < AVo= 2 × (πd²/4) × 0.75 × 6.53 = 11.078 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 1+990.00

⊙ **Input Sectional Shape (2@Φ1200)**

·Catchment Area(A) = 0.1360 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 4.712 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 254.43m**

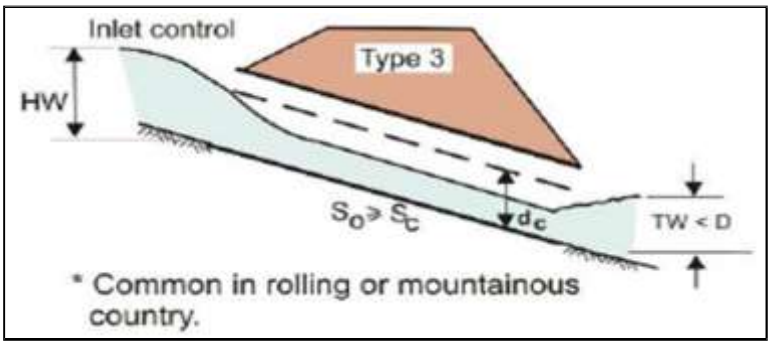
1. Hydrological Analysis

HW (1.08) ≤ 1.2 D (1.44) , So (0.2248) ≥ Sc (0.0101)

TW (0.12) ≤ dc (0.590) < D (1.2) , dn (0.260) < dc (0.590) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_e) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of the outlet

and Vc = 6.530 m / sec.

2. Analysis of the discharge area

Qd =1.337㎥/sec of the drainage area at the current STA., and the Pipe size is φ1000

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (4.712) < Q = AVo = (πd²/4) × 0.75 × 2 × 6.53 = 11.078 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 2-2) 2+423.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-2 C-4

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

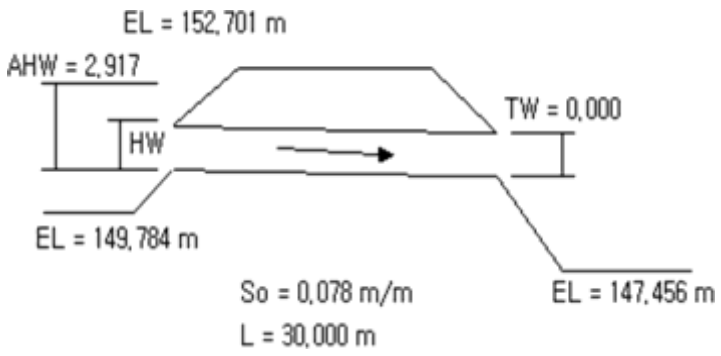
Catchment Area(A) =0.6199 ㎢
Qd = 22.941 ㎥/sec
Arrival Distance(L) = 1250.00 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 438.50m - 149.78 m
= 288.72 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.620 ㎢ Qd = 21.479 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.322

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
BOX	10.740	2@ 1.5 x 1.5	3.66	5.50	0.2	1.5645	1.093	1.296	0.32	1.20	2.33	0.44	5.50	8.81	Inlet	N.G
BOX	10.740	2@ 2.0 x 1.5	3.35	5.03	0.2	0.8645	0.902	1.201	0.32	1.20	2.33	-0.26	5.03	8.36	Inlet	N.G
BOX	10.740	2@ 2.0 x 1.6	3.17	5.08	0.2	0.7565	0.902	1.251	0.32	1.28	2.33	-0.29	5.08	8.36	Inlet	N.G
BOX	10.740	2@ 2.0 x 2.0	0.91	1.82	0.2	0.4779	0.902	1.451	0.32	1.60	2.33	-0.25	1.82	8.36	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd = 21.479 < AVo= (B × H) × 0.8 × 2 × 8.360 = 53.504 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 2+423.00

⊙ **Input Sectional Shape (2Φ2.0x2.0)**

·Catchment Area(A) = 0.6199 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 22.479 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 288.72 m**

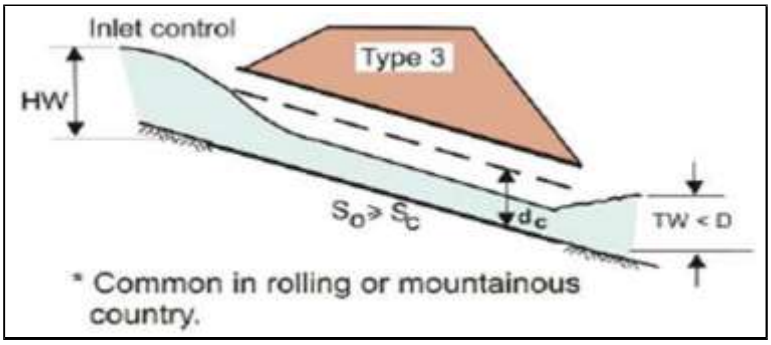
1. Hydrological Analysis

HW (1.82) ≤ 1.2 D (2.4) , So (0.0776) ≥ Sc (0.0040)

TW (0.32) ≤ dc (0.902) < D (2.0) , dn (0.322) < dc (0.902) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = (0.701 + 0.234 \cdot C_e) \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}}$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in Normal depth of the outlet

and Vn = 8.360 m / sec.

2. Analysis of the discharge area

Qd =21.479㎥/sec of the drainage area at the current STA., and the Box size is 2@2.0X2.0

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (21.497) < Q = AVo = (B × H) × 0.8 × 2 × 8.360 = 53.504 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 2-3) 2+660.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-3 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

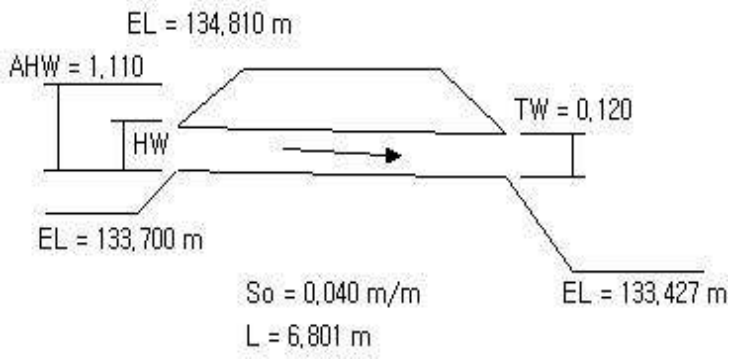
Catchment Area(A) =0.0138 km^2
Qd = 0.478 m^3/sec
Arrival Distance(L) = 383.00 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 164.00m - 133.70 m
= 30.30 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.014 km^2 Qd = 0.478 m^3/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 m^3/sec Qd = 0.000 m^3/sec	Chupu Karma gauging station

Water level in Culvert = 0.282

SKETCH



CULVERT DESCRIPTION	Qd/ cell (m^3/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.478	Φ900	0.60	0.54	5.0	0.1218	0.402	0.651	0.16	0.67	0.27	0.52	0.54	2.80	Inlet	O.K

Summary and Recommndations : 1. Qd = AVo, Qd = 0.478 < AVo= (πd²/4) × 0.75 × 2.80 = 1.336 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 2+660.00

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0138 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.478 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 30.30 m**

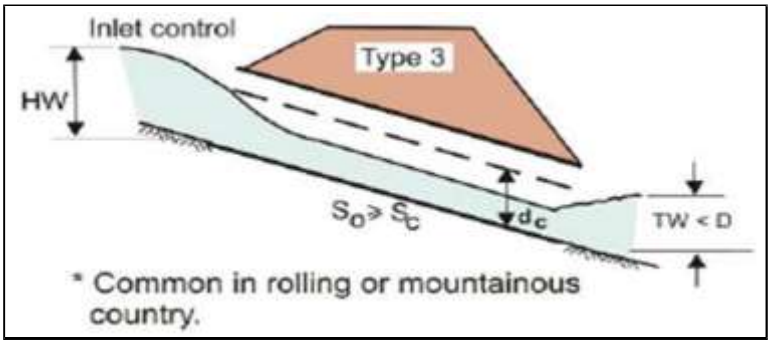
1. Hydrological Analysis

HW (0.54) ≤ 1.2 D (1.08) , So (0.0401) ≥ Sc (0.0107)

TW (0.16) ≤ dc (0.402) < D (0.9) , dn (0.282) < dc (0.402) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of the outlet

and Vc = 2.800 m / sec.

2. Analysis of the discharge area

Qd =0.478㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.478) < Q = AVo = (πd²/4) × 0.75 × 2.800 = 1.336 : O.K

Culvert Design Calculations

STATION : (Temp Road 3) 0+141.13
PROJECT : Tina River Hydropower Development Project
Designation : LOT 2-3 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

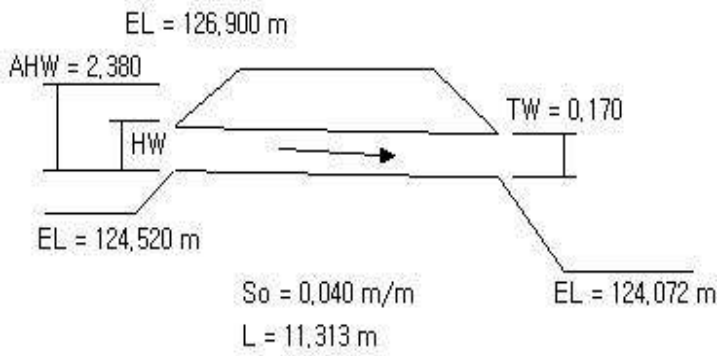
Catchment Area(A) =0.0138 ㎢
Qd = 0.478 ㎥/sec
Arrival Distance(L) = 389.00 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 164.00m - 124.52 m
= 39.48 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0138 ㎢ Qd = 0.478 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.283

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.478	Φ900	0.60	0.54	0.5	0.1739	0.402	0.651	0.17	0.67	0.45	0.40	0.54	2.79	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd = 0.478 < AVo= (πd²/4) × 0.75 × 2.79 = 1.331 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 0+141.13 (Temp Road 3)

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0138 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.478 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 30.30 m**

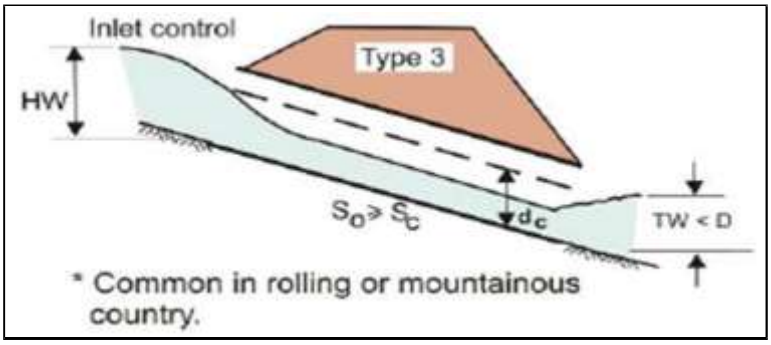
1. Hydrological Analysis

HW (0.54) ≤ 1.2 D (1.08) , So (0.0400) ≥ Sc (0.0107)

TW (0.17) ≤ dc (0.402) < D (0.9) , dn (0.283) < dc (0.402) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of the outlet

and Vc = 2.790 m / sec.

2. Analysis of the discharge area

Qd =0.478㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.478) < Q = AVo = (πd²/4) × 0.75 × 2.790 = 1.331 : O.K

Tina River Hydropower Development P oject (TRHDP)

CALCULATION
(ACCESS ROAD LOT 3)

24 - JUN -2020

Tina Hydropower Limited

HYUNDAI ENGINEERING CO., LTD

Content

1. Corrugated Steel Pipe

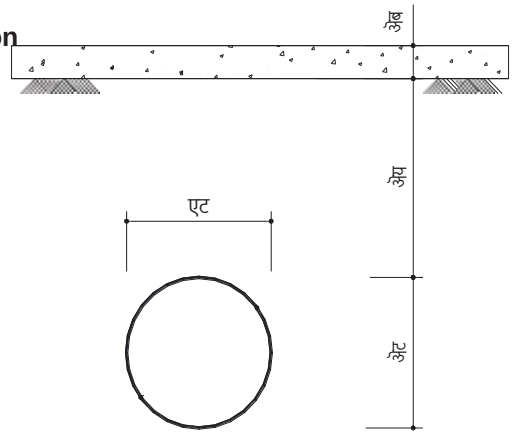
1.1 Corrugated Steel Pipe (D=900)

D=0.9M

H=1.14 m [SI UNIT]

1.1.1 Design Condition

1) Section Assumption



- Unit density of pavement : γp = 23.000 kN/m³
- Unit density of soil : γs = 20.000 kN/m³
- Thickness of pavement : Hp = 0.320 m
- Depth of soil : Hs = 0.820 m
- Pipe span : Sc = 0.900 m
- Corrugated steel pipe specifications
  - pitch x depth : 68x13 mm
  - thickness : 1.6 mm
  - yield strength : fy = 230 Mpa
  - modulus of elasticity : E = 200000 Mpa

2) Reference

Corrugated steel pipe institute – Handbook of Steel Drainage Highway Construction Products

1.1.2 Section Properties for corrugated steel pipe

type	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, mm4/mm											
38x6.5	3.7	5.1	6.5	8.6								
68x13	16.5	22.6	28.4	37.1	54.6		70.2		86.7			
76x25	75.8	104.0	130.4	170.4	249.7		319.8		393.1			
125x25			133.3	173.7	253.2		322.7		394.8			
152x51						1057.3		1457.6		1867.1	2278.3	2675.1
	Cross section Wall area, mm2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
	Radius of Gyration, mm											
38x6.5	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523

1.1.3 Loads

1) Dead Load

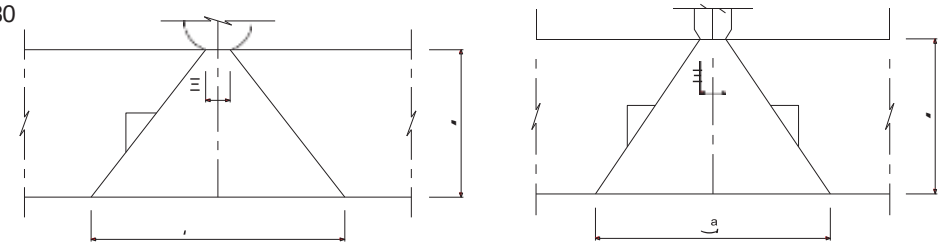
The dead load is considered to be the soil prism over the pipe:

DL = gH = 23.000 x 0.320 + 20.000 x 0.820 = 23.760 kN/□

DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pipe
g = unit weight of the soil
H = height of cover over the pipe

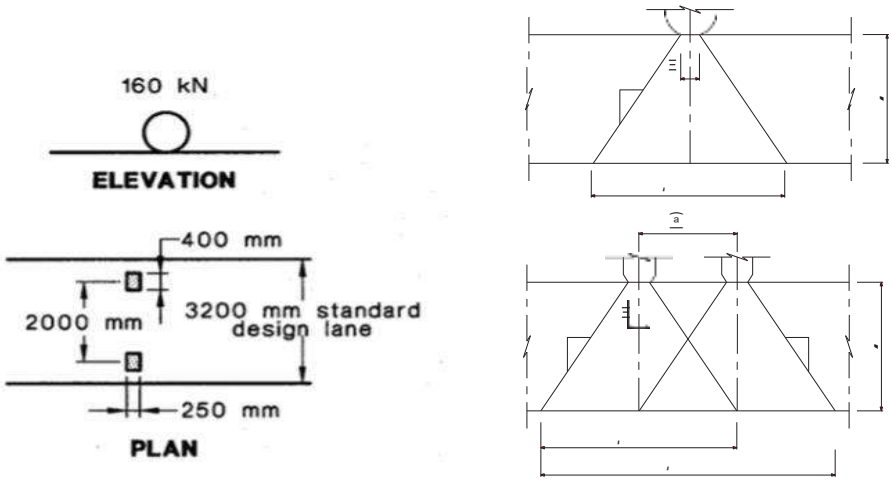
2) Live Load

(1) W80



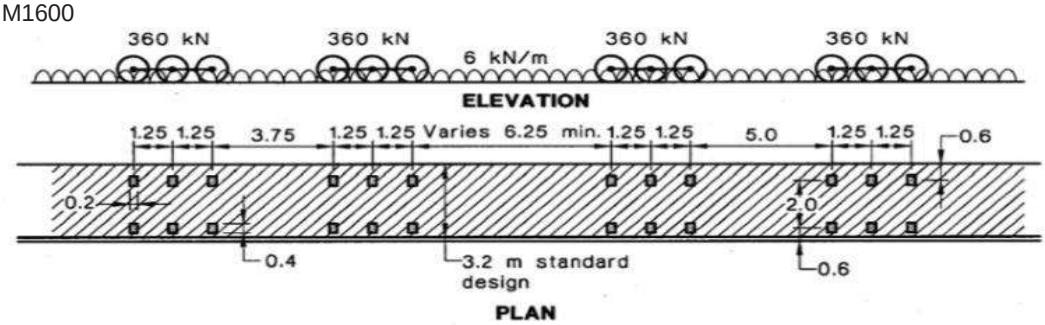
$$P_{vl} = \frac{80}{(0.25 + 2D) \times (0.4 + 2D)} = \frac{80}{(0.25 + 2.28) \times (0.4 + 2.28)} = 11.799 \text{ kN/m}^2$$

(2) A160

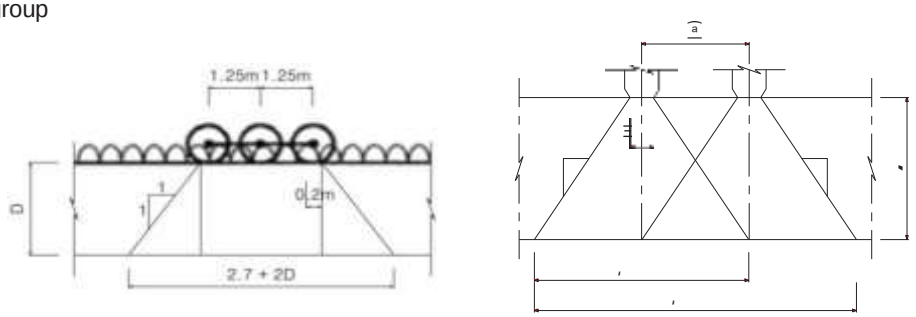


$$P_{vl} = \frac{2 \times 80}{(0.25 + 2D) \times (2.4 + 2D)} = \frac{160}{(0.25 + 2.28) \times (2.4 + 2.28)} = 13.513 \text{ kN/m}^2$$

(3) M1600



- Axle group

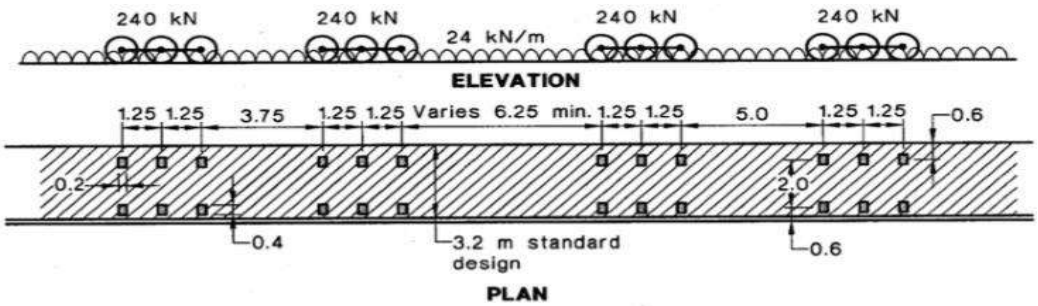


$$P_{v1} = \frac{6\times60}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{360}{(2.7 + 2.28) \times (2.4 + 2.28)} = 15.446 \text{ kN/m}^2$$

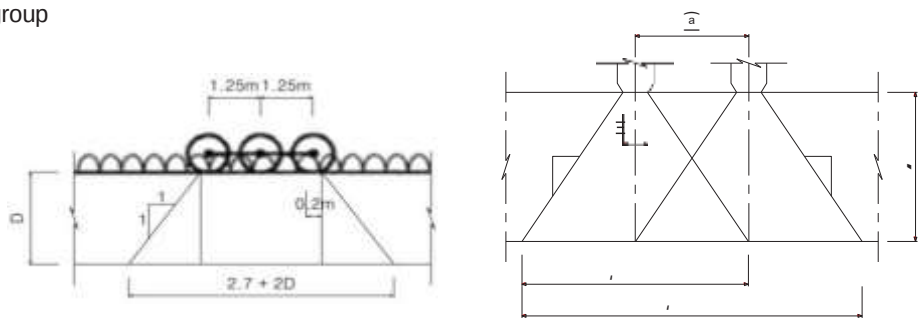
- Lane uniformly distributed loads : 6.000 kN/m² / 3.2 m = 1.875 kN/m²

- P_{vl} = 15.446 + 1.875 = 17.321 kN/m²

(4) S1600



- Axle group

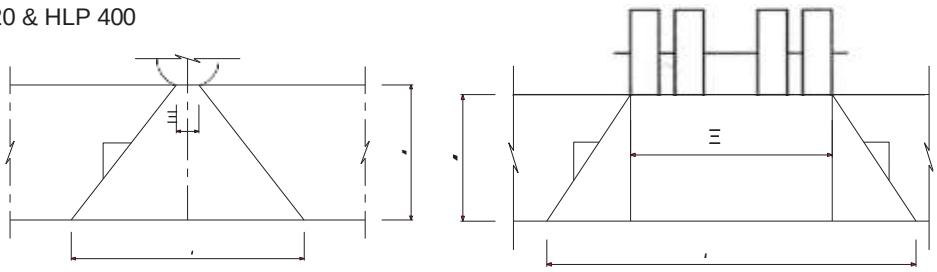


$$P_{v1} = \frac{6 \times 40}{(2.7 + 2D) \times (2.4 + 2D)} = \frac{240}{(2.7 + 2.28) \times (2.4 + 2.28)} = 10.298 \text{ kN/m}^2$$

- Lane uniformly distributed loads : 24.000 kN/m² / 3.2 m = 7.500 kN/m²

- P_{vl} = 10.298 + 7.500 = 17.798 kN/m²

(5) HLP 320 & HLP 400



$$P_{vl} = \frac{125}{(0.2 + 2D) \times (1.4 + 2D)} = \frac{125}{(0.2 + 2.28) \times (1.4 + 2.28)} = 13.697 \text{ kN/m}^2$$

(6) Live Load

TYPE	Load	Dynamic Load Allowance (α)	(1 + α) x Load
W80	11.799	0.23	14.501
A160	13.513	0.23	16.608
M1600	17.321	0.19	20.543
S1600	17.798	0.00	17.798
HLP	13.697	0.10	15.066

□ LL = 20.543 kN/m² = 20.543 kN/m²

1.1.4 Minimum cover

pipe span (mm)	Minimum cover for indicted Axle Loads (tonnes)			
	8~22	22~34	34~50	50~68
300-1050	600	760	900	900
1200-1830	900	900	1050	1200
1980-3050	900	1050	1200	1200
3200~3660	1050	1200	1370	1370

□ H_{min}= 600 ㎜ < H = 1140 ㎜ ĀO.K

1.1.5 Backfill compaction

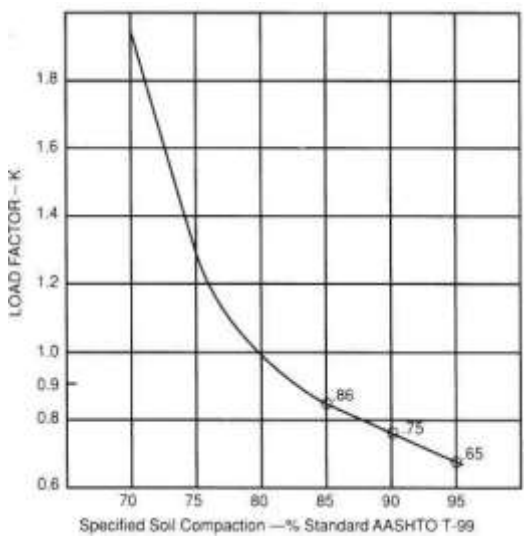
The value chosen should reflect the importance and size of the structure, and quality of backfill material and its installation that cn reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density.
Therefore, the design assumes a backfill compaction density 85%

1.1.6 Design Pressure

H = 1.1 m > S = 0.9 m

Pv = K (DL + LL), when H > S
= 0.86 x (23.76 + 20.543) = 38.101 kN/m²

where:
Pv = design pressure, kPa
K = 0.86 , load factor
DL = dead load
LL = live load
H = height of cover
S = span or diameter



1.1.7 Ring compression

C = Pv x S / 2 = 38.101 x 0.900 / 2 = 17.145 kN/m

where: C = ring compression
Pv = design pressure
S = span or diameter

1.1.8 Allowable wall stress

The ultimate compressive stress in the pipe wall is expressed by the following equation

fb = fy, when D/r = 208 < 294
fb = 230 MPa

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress

fc = fb / 2 = 115.00 MPa

where: fb = ultimate compressive stress
fc = allowable stress
fy = 230 MPa , yield strength
D = 900 mm span or diameter
r = 4.332 ฟ , radius of gyration of the pipe wall

1.1.9 Wall thickness

A required wall area A, is computed using the calculated compression in the pipe wall, C, and allowable stress, fc

ฦ = C / fc = 17.145 / 115.000 = 0.149 ฯ/ฟ

where: A = required area in the pipe wall
C = ring compression
fc = allowable stress

Cross section Wall area = 1.512 ฯ/ฟ > 0.149 ฯ/ฟ ĀO.K

1.1.10 Handling stiffness

The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness

FF = D^2 / EI = 0.143 ฟ/N < 0.245 ฟ/N ĀO.K

where: E = 200000 MPa , modulus of elasticity
D = diameter or span

I = 28.37 ,moment of inertia of the pipewall

Recommended maximum allowable values of FF for ordinary round and underpass pipe installations are as follows:

68x13	mm corrugation, FF	0.245 ฟ/N
125x25	mm corrugation,	0.188 ฟ/N
76x25	FF mm	0.188 ฟ/N
152x51	corrugation, FF	0.114 ฟ/N
	mm corrugation, FF	

1.1.11 Seam strength

The allowable ring compression accounting for the seam strength consideration, is the ultimate seam strength, show in tables below, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for the types of pipe

1.2 Head/Wing Wall (H=1.5m)

1.2.1 Design Conditions (H=1.500m , N= 1 : 2.00 , Ho= 6.370)

1) General Items

(1)	Type of WingWall	:	Cantilever Type	
(2)	Height of WingWall	:	1.500	m
(3)	Slope of Backfill	:	1 : 2.00	
(4)	Height of Backfill	:	6.370	m

2) Soil

(1)	Unit Weight of Backfill	:	γt =	19.000	kN/m³
(2)	angle of internal friction of Backfill	:	Φ =	28.000	°
(3)	Unit Weight of filler	:	γt =	18.500	kN/m³
(4)	angle of internal friction of filler	:	$\Phi 1$ =	32.000	°
(5)	coefficient of earth pressure atrest of filler	:	Φ_B =	0.500	
(6)	Cohesion of Soil	:	C =	0.000	kN/m²

3) Load

(1)	Surface load	:	qL =	10.000	kN/m²
(2)	horizontal seismic coefficient	:	Kh =	0.115	(0.191 x 0.5 x 1.2)

4) Design Material

(1)	Reinforced Concrete Weight	:	γc =	25.00	kN/m³
(2)	Strength of Concrete	:	fck =	32.00	MPa
(3)	Yield Strength of Reinforcement	:	fy =	420.00	MPa

5) Coefficient of Earth Pressure

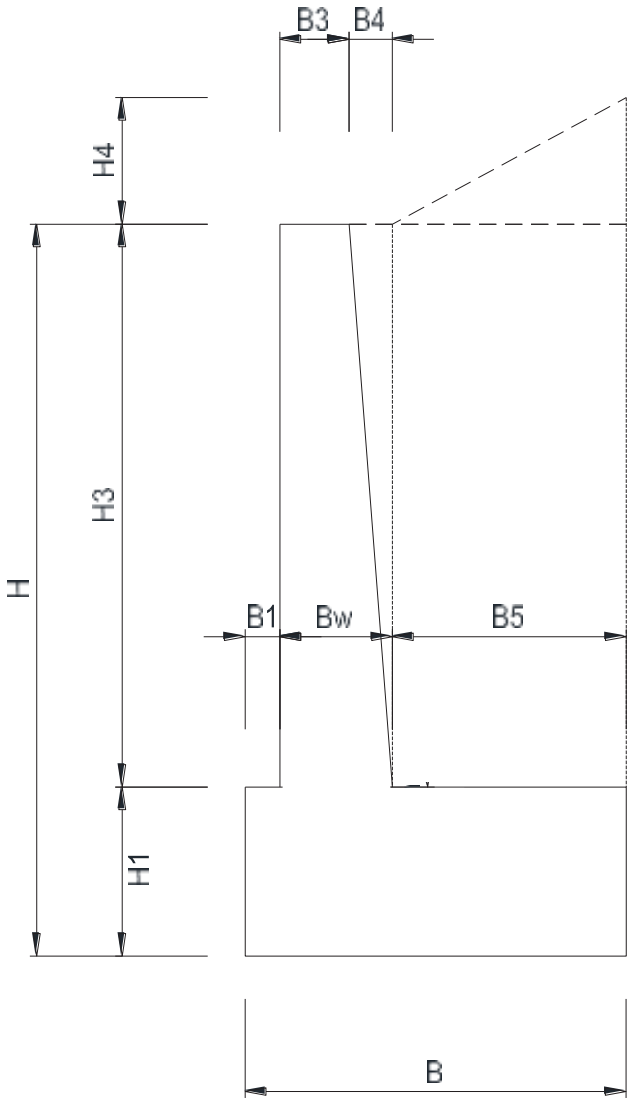
(1)	Evaluation of serviceability	:	Wedge of Soil pressure
(2)	Evaluation of section	:	Wedge of Soil pressure

6) Reference

(1) American Concrete Institute – Code for the design of concrete structure, USA

(2) AS 3600 Concrete structures / 5100 Bridge design - Australian Standard

1.2.2 Section Assumption



ŠSectional specification

- Width

B1	B2	B3	B4	B5	B	Bw
0.100	0.000	0.200	0.125	0.675	1.100	0.325

- Height

H1	H2	H3	H4	H	Ho	
0.450	0.000	1.500	0.338	1.950	6.370	

1.2.3 Load Calculation

1) Self weight (D)

Automatic consideration in program

2) Earth pressure

▷ At Normal (H)

$$Pa = \frac{SIN(\alpha - \Phi)}{COS(\alpha - \Phi - \delta - \theta)} \times W$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)
28.00	26.565	9.33	4.764

∵ (δ = Λ· x Φ)

α(ƒx)	δ'(ƒx)	H (m)	W (kN/m)	Pa (kN/m)	Ka	Kah	Kav
32.2	9.333	1.500	165.183	12.280	0.575	0.557	0.140
32.3	9.333	1.500	161.576	12.294	0.575	0.558	0.140
32.4	**9.333**	**1.500**	**157.989**	**12.296**	**0.575**	**0.558**	**0.140**
32.5	9.333	1.500	154.422	12.288	0.575	0.558	0.140
32.6	9.333	1.500	150.875	12.268	0.574	0.557	0.140

Coefficient of earth pressure : Kah = 0.558

Horizontal earth pressure Pah = Kah x γt x H

Pah1 = 0.558 × 19 × 0.000 = 0.000 kN/m³

Pah2 = 0.558 × 19 × 1.500 = 15.903 kN/m³

◁ At Earthquake (E)

$$Pa = \frac{SIN(\alpha - \Phi + \omega)}{COS(\alpha - \Phi - \delta - \theta)} \times \frac{W}{COS(\omega)}$$

where,

Φ(ƒx)	β(ƒx)	δ(ƒx)	θ(ƒx)	ω(ƒx)
28.000	26.565	0.000	4.764	6.538

∵ $\omega = \tan^{-1}Kh$

α(ƒx)	δ(ƒx)	H (m)	We (kN/m)	Pa (kN/m)	Kae	Kaeh	Kaev
28.2	0.000	1.500	328.536	38.670	1.8091	1.803	0.150
28.3	0.000	1.500	323.922	38.684	1.8098	1.804	0.150
28.4	**0.000**	**1.500**	**319.338**	**38.687**	**1.8099**	**1.804**	**0.150**
28.5	0.000	1.500	314.783	38.677	1.8094	1.803	0.150
28.6	0.000	1.500	310.258	38.655	1.8084	1.802	0.150

Coefficient of earthquake earth pressure : Kaeh' = Kaeh - Kah = 1.246

Earthquake earth pressure Paeh' = Kaeh' x γt x H

Paeh1' = 1.246 × 19 × 0.000 = 0.000 kN/m³

Paeh2' = 1.246 × 19 × 1.500 = 35.511 kN/m³

3) Inertia force at earthquake (E)

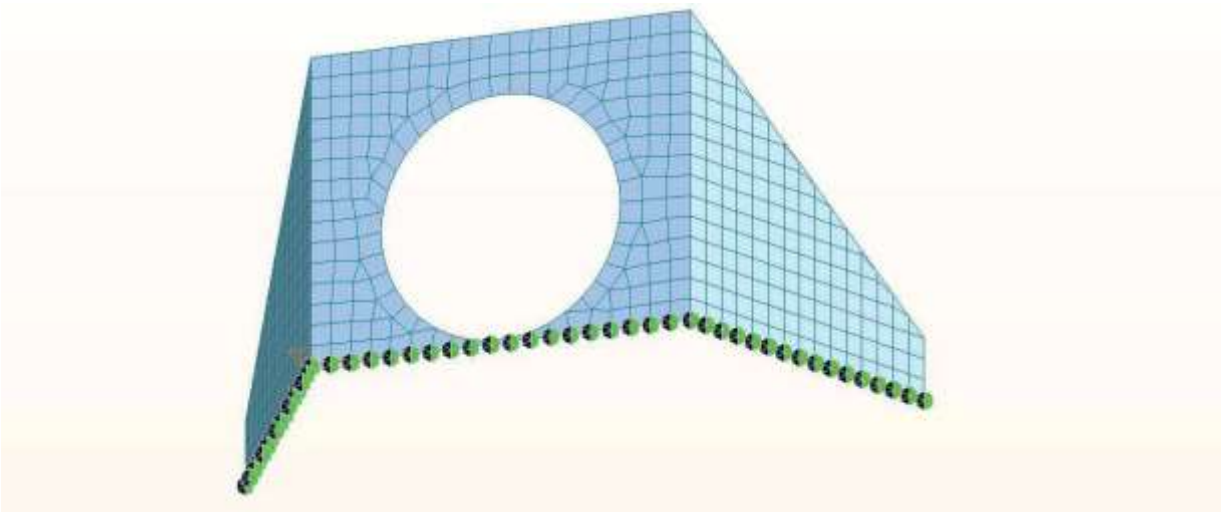
P = W x Kh

W : Weight of structure

Kh : 0.115 horizontal seismic coefficient

1.2.4 Modeling & Loading

1) Anaysis Model

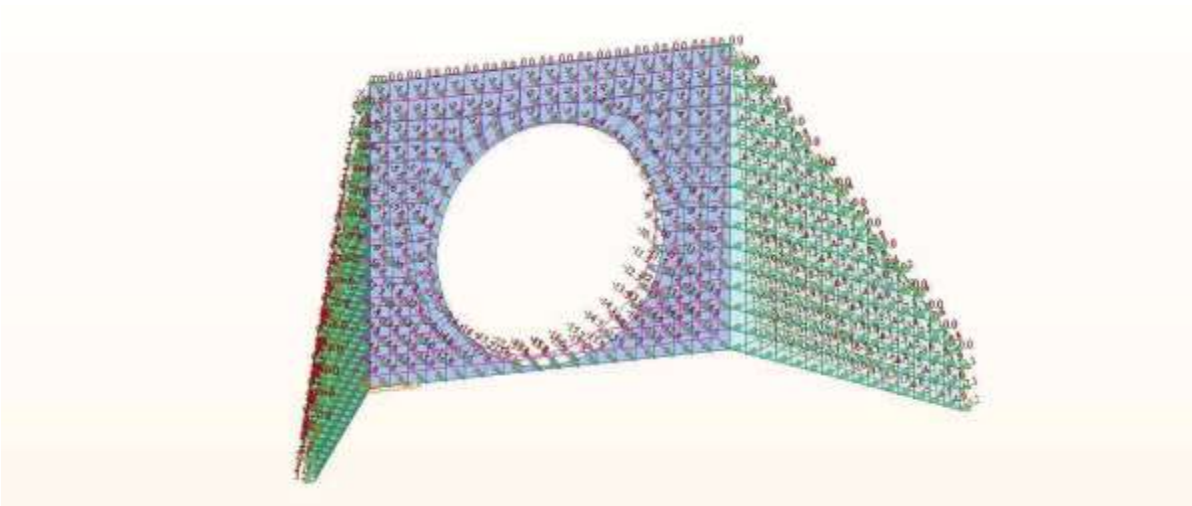


Boundary condition : Hinge

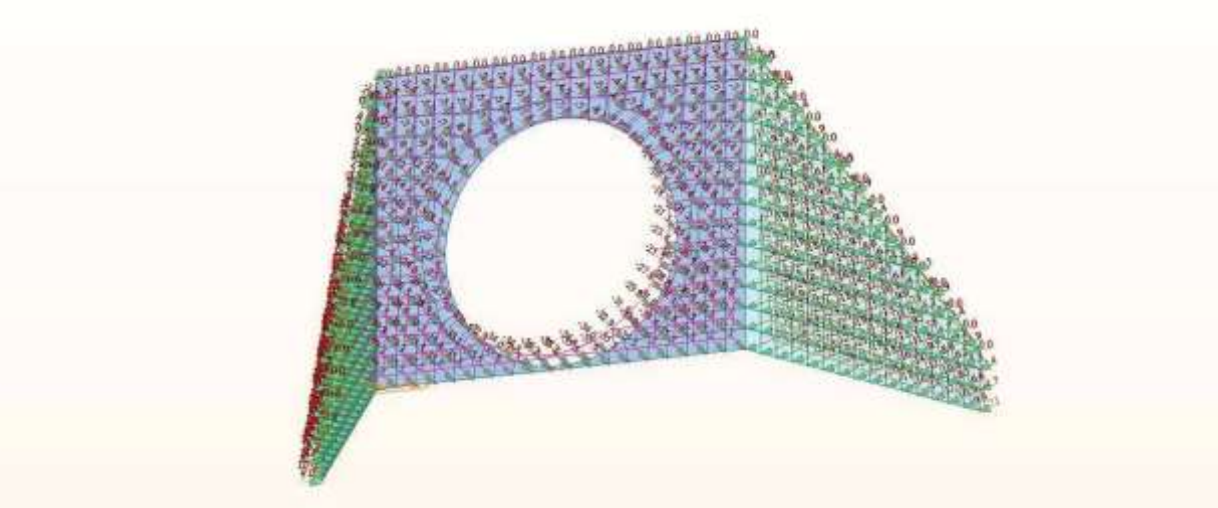
2) Loading

(1) Self weight - Automatic consideration in program (D)

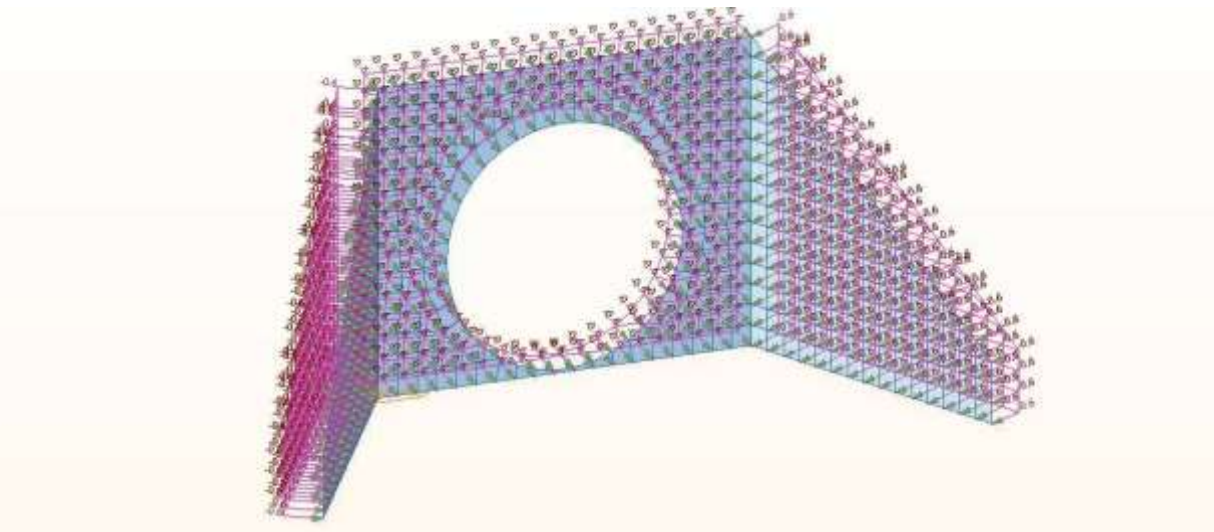
(2) Horizontal Earth Pressure at normal (H)



(3) Horizontal Earth Pressure at earthquake (E)



(4) Inertia force at earthquake (E)



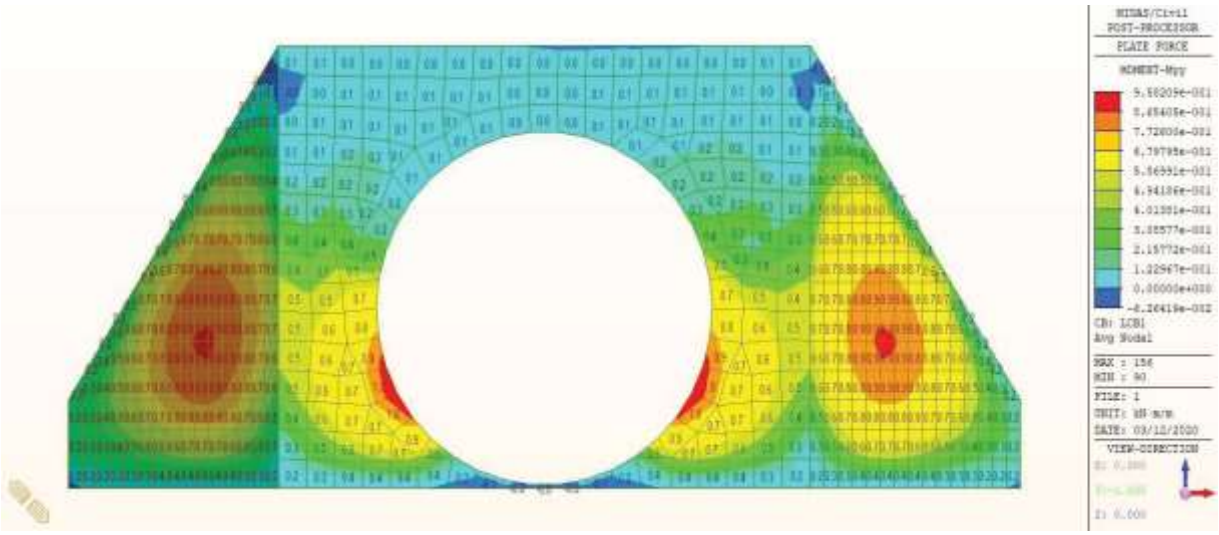
1.2.5 Load combination

LCB 1	:	Ultimate Load at nomal	(	1.2 D	+	1.6 L	+	1.6 H	)
LCB 2	:	Ultimate Load at earthquake	(	0.9 D	+	1.6 H	+	1.0 E	)
LCB 3	:	Service Load at nomal	(	1.0 D	+	1.0 L	+	1.0 H	)

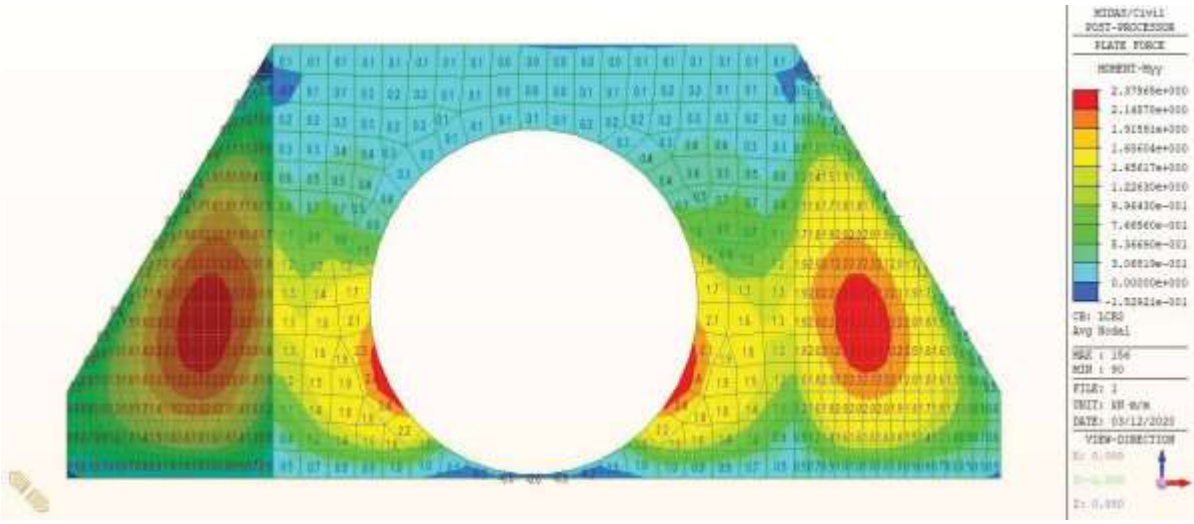
1.2.6 Summary of Analysis Results

(1) B.M.D (Ultimate Load) - Unit : kN.m

- LCB1

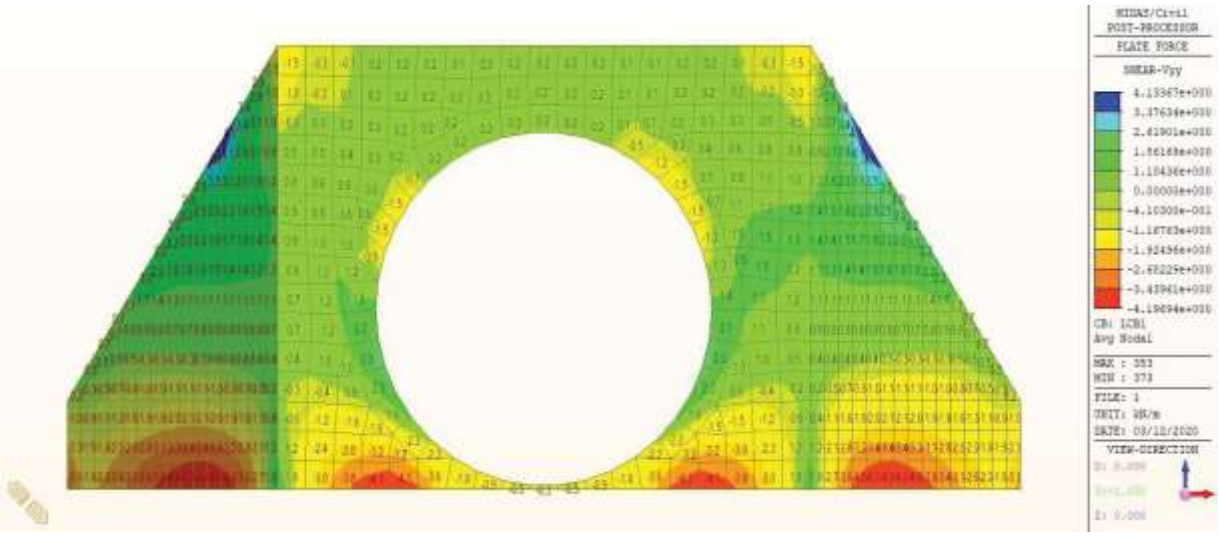


- LCB2

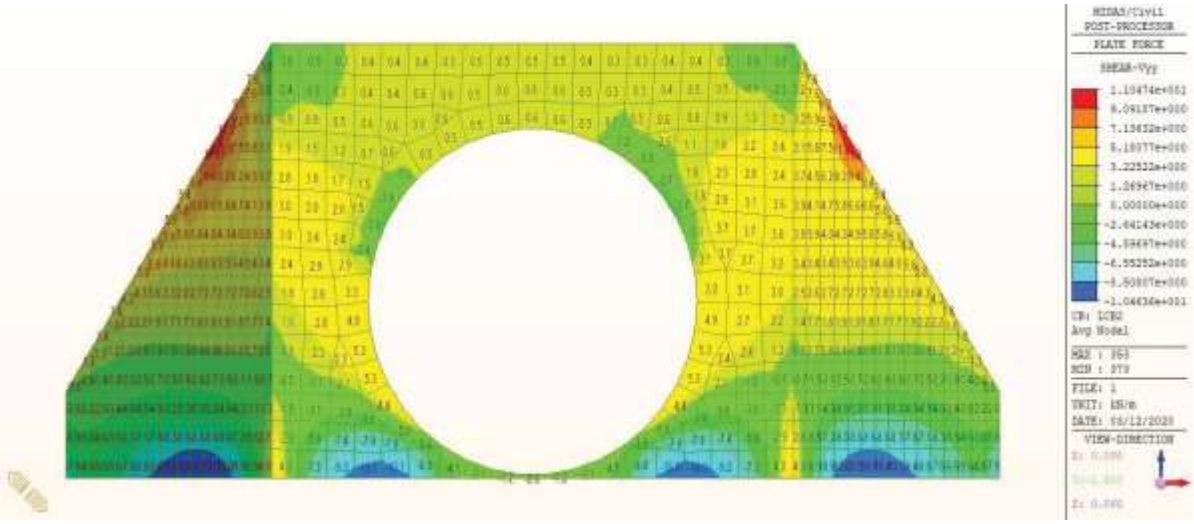


(2) S.F.D (Ultimate Load) - Unit : kN

▷ LCB1

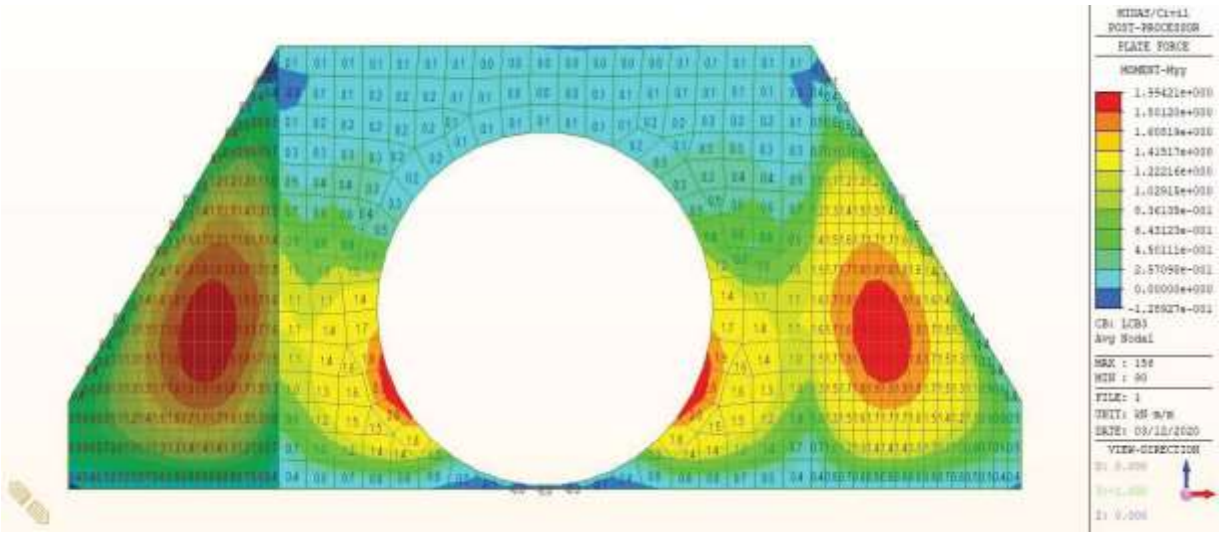


◁ LCB2



(3) B.M.D (Service Load) - Unit : kN.m

▷ LCB3



(4) Summary

Division	LCB1	LCB2	LCB3	Section
M(kN.m)	0.958	2.375	1.994	Middle of Wall
V(kN)	4.196	10.463		Bottom of Wall

□ Mu = 2.375 kN.m

Vu = 10.463 kN

Mo = 1.994 kN.m

1.2.7 Section Design

1) Middle of Wall

(1) Section Design

ㅿ Section specification and design condition

$f_{c'}$	=	32	MPa	f_y	=	420	MPa		k_1	=	0.82		
Øf	=	0.90		Øv	=	0.75			d	=	100.0	mm	
B	=	1000	mm	H	=	200	mm		d'	=	100.0	mm	
M_u	=	2.375	kN·m	V_u	=	10.463	kN	M_o	=		1.994	kN·m	

- Check of Strength reduction factor (Φ)

a = 15.050

Because T = C , *c* = 15.050 / β1 = 15.050 / 0.821 = 18.322 mm

εy = fy / Es = 420.000 / 200000 = 0.0021

εt = 0.0030 x (dt - c) / c = 0.003 x (100.0 - 18.322) / 18.322

= 0.0134

εt > 0.0050 Tension-controlled sections Φf = 0.900

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ ---------- (1)

$M_u / Ø = A_s \times f_y \times (d - a / 2)$ ---------- (2)

И□ ТФГТУКУФУZТ НПΣ И□

$$\frac{f_y^2}{2 \times 0.85 \times f_{c'} \times b} As^2 - f_y \times d \times A_s + \frac{M_u}{Ø} = 0$$

□ Req. As = 63.138 ㎡

Use As = D 13 @ 250 + D 13 @ 250 = 1032.00 ㎡ (8 ea/m)

ㅿ Evaluation of reinforcement

P_b	=	$k_1 \times Ø \times (f_{c'}/f_y) \times \{600/(600 + f_y)\}$	=	0.03313					
P_{max}	=	0.75 · Pb	=	0.02485	㎡	$A_{s,max}$	=	2485.0	㎡
P_{min}	=	max (1.4 / fy , 0.25 □ fc' / fy)	=	0.00337	㎡	$A_{s,min}$	=	336.7	㎡
$P_{4/3req}$	=	$4/3 \cdot A_{s.req} / (B \cdot d)$	=	0.00084	㎡	$A_{s,4/3req}$	=	84.2	㎡
P_{min}	=	min (P_{min}, $P_{4/3req}$)	=	0.00084	㎡	$A_{s,min}$	=	84.2	㎡
P_{use}	=	$A_s / (B \cdot d)$	=	0.01032	㎡	$A_{s,min}$	=	1032.0	㎡

ㄴ 4/3 x Preq ≤ Puse ≤ Pmax Ā O.K

ㅿ Bending Check

$a = A_s \times f_y / (Ø \times f_{c'} \times b)$ = 15.050 mm

$ØMn = 0.9 \times As \times fy \times (d - a/2)$ = 36.074 kN·m > Mu = 2.375 kN·m

Ā O.K

ᵢ Shear firction Check

Øv Vn = Øv × Avf × fy × μ 325.080 > Vu = 10.463 kN Ā O.K

(2) Crack Check

ᵢ Calculation of stress

n = 9

χ = - nA_s/b + nA_s/b × □{1+2bd/nA_s}

= - 9 × 1,032.00 / 1000 + 9 × 1,032.00 / 1000 × √[1 + 2 × 1000 × 100 / (9 × 1,032.00)]

= 34.801 ⌟

f_c = 2 × M_o / [B × χ × (d - χ/3)]

= 2.0 × 1.994 / [1000 × 34.801 x (100.0 - 34.801 / 3)] × 10^6

= 1.296 MPa

f_s = M_o / [A_s × (d - χ/3)]

= 1.994 / [1032.000 x (100.0 - 34.801 / 3)] × 10^6

= 21.857 MPa

fst = fs x (H - d' - χ) / (d - χ) = 22 x (200 - 100 - 1) / (100 - 35) = 21.86 MPa

ᵢ Maximum center space of reinforcement

C_C = 100.00 - 13.00 / 2 = 93.50 ⌟

Cc ; Thickness between reinforcement or surface of stress member and concrete surface(mm)

Smin : 380 x (280 / fs) - 2.5 x Cc = 380 x (280 / 21.86) - 2.5 x 93.50 = 4634.21 ⌟

300 x (280 / fs) = 300 x (280 / 21.86) = 3843.12 ⌟

Sa = 3843.12 ⌟ Applying Minimum value

S = 1,000 / 8 Ea = 125.0 < Sa (3843.12 mm) Ā O.K

1.2.8 Distribution Reinforcement Check

1) Wall (H = 325 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 325 = 585.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : D 13@ 125 = 1032.0 ㎟

□ = 1032.0 ㎟ > 585.0 ㎟ Ā O.K

· Bar spacing : 125 ㎜ < 450 ㎜ Ā O.K

2) Bottom Slab (H = 450 mm)

· $A_{s,\,min}$ = (0.0018 × 420 / fy) × B × H = (0.0018 × 420 / 420) × 1000 × 450 = 810.0 ㎟

· The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5h and 450mm. = 450 ㎜

· Used As : Upper side : D 13@ 250 = 516.0 ㎟

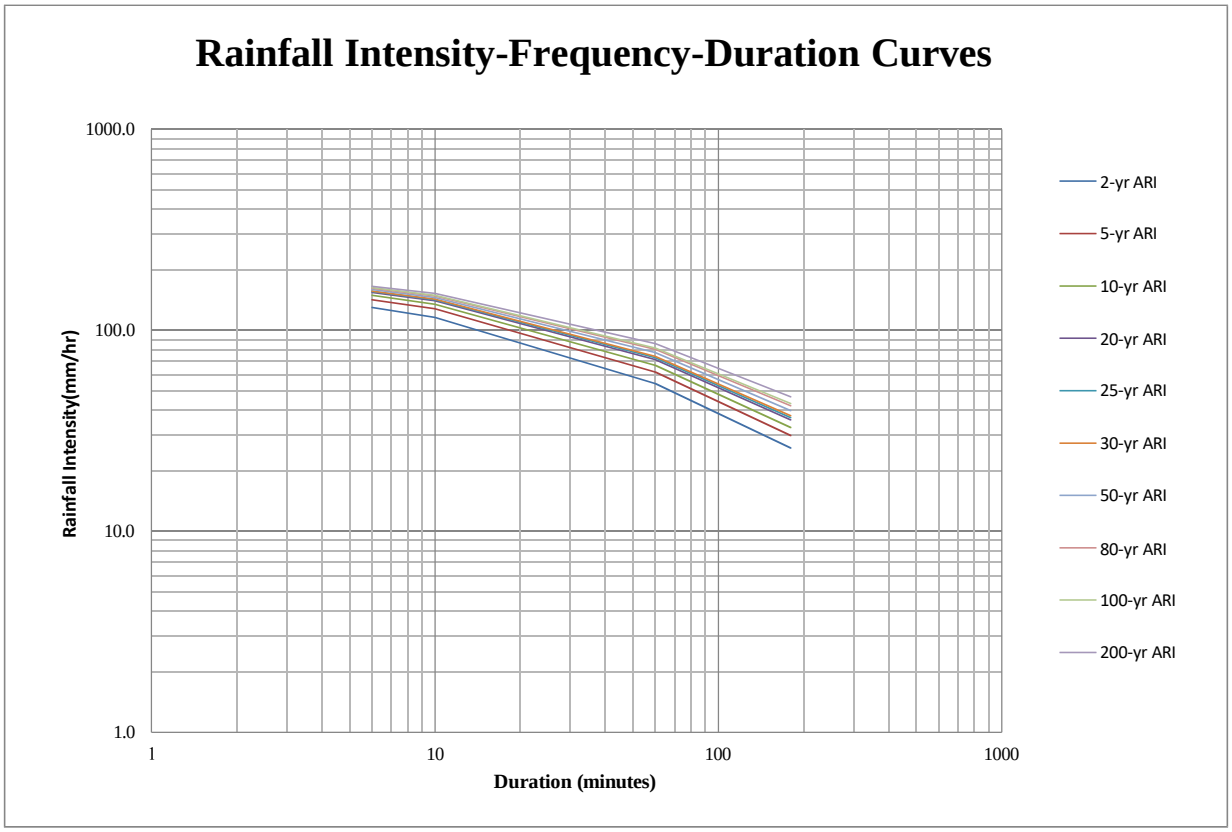
Bottom side : D 13@ 250 = 516.0 ㎟

□ = 1032.0 ㎟ > 810.0 ㎟ Ā O.K

· Bar spacing : 250 ㎜ < 450 ㎜ Ā O.K

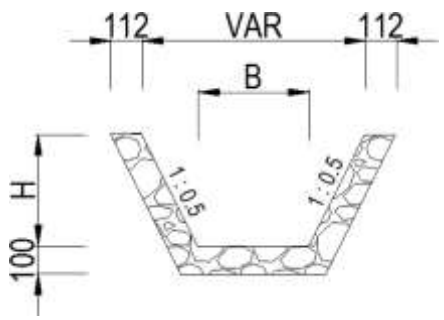
2 .HYDRAULIC CALCULATION

2.1 U-Ditch Calculation



Duration (mins) \ Rainfall Intensity (mm/hr)	2-yr ARI	5-yr ARI	10-yr ARI	20-yr ARI	25-yr ARI	30-yr ARI	50-yr ARI	80-yr ARI	100-yr ARI	200-yr ARI
6	129.7	**142.1**	149.2	154.2	**155.8**	156.8	159.3	161.9	162.9	165.6
10	115.9	127.9	134.9	140.1	141.8	142.9	145.7	148.4	149.5	152.6
60	54.3	62.0	66.9	71.5	73.0	74.2	77.4	80.3	81.7	85.9
180	25.9	30.0	32.9	35.8	36.8	37.6	39.9	42.1	43.2	46.6
360	14.5	16.9	18.7	20.7	21.4	22.0	23.8	25.7	26.7	30.7
540	10.5	12.7	14.7	17.0	17.8	18.4	20.6	22.8	23.9	28.1
720	8.2	10.4	12.3	14.6	15.4	16.2	18.4	20.7	21.9	26.3
900	6.8	8.9	10.7	13.0	13.8	14.5	16.8	19.1	20.4	24.8
1080	5.8	7.8	9.6	11.8	12.6	13.3	15.5	17.9	19.2	23.6
1440	4.6	6.3	8.0	10.0	10.8	11.5	13.7	16.0	17.3	21.7

A. Calculation of U-type Ditch



U-type ditch

1) Flow rate in U-type ditches

① Utype- ditch(Lined) Q= A X V = $A \times (1/n) \times R^{2/3} \times I^{1/2}$ (n=0.025)

TYPE	Depth of Flow (0.8H)	Width(D) (m)	Area of the flow (m^2)	Wetted perimeter (m)	Hydraulic Radius® (m)	$R^{2/3}$	Q m^3/sec
TYPE-4(H=0.40M)	0.32	0.720	0.1792	1.116	0.1606	0.2955	$2.1181 \times I^{1/2}$
TYPE-6(H=0.60M)	0.48	1.080	0.4032	1.673	0.2410	0.3873	$6.2464 \times I^{1/2}$

2) The Rational Method formula for flow estimation Q(m^3/sec)

① Steep Mountain and Slope Surface

$Qd_1 = 0.278 \times C_1 \times I_1 \times A_1$

where, C_1, C_2 : Run-off Coefficient

I_1, I_2 : Rainfall intensity(mm/hr)

A_1, A_2 : Catchment Area(km^2)

② Paved surface

$Qd_2 = 0.278 \times C_2 \times I_2 \times A_2$

C_1 : 0.8 C_2 : 0.9

I_1(5yr) : 142.1 I_2(5yr) : 142.1

③ Peak Catchment Discharge $Qd = Qd_1 + Qd_2$

3) Determination of Cross section

Compare Flow rate with the Peak Catchment Discharge according to Runoff Coefficient and Catchment Area

※ **Q > Qd (OK)**

U-Ditch Hydrological Caculation (1)

No.	Station(Lot-3-1)		Direction	Catchment Area(㎢) (A1,A2)	Run-off Coeffiecient (C1,C2)	Slope (%)	Flow rate according to slope		Peak Catchment Discharge (㎥/sec)		Determination		Remark
	BP	EP					TYPE-4	TYPE-6	(Qd_1,Qd_2)	Qd	Check Capacity	Apply	
1	0+115	0+255	L	0.0042	0.8	11.48	0.718	2.116	0.133	0.180	U4	U4	
				0.0013	0.9				0.048				
2	0+380	0+495	L	0.0021	0.8	10.27	0.679	2.002	0.066	0.097	U4	U4	
				0.0009	0.9				0.030				
3	0+645	0+955	L	0.0041	0.8	10.50	0.686	2.024	0.130	0.194	U4	U4	
				0.0018	0.9				0.064				
4	1+100	1+480	L	0.0077	0.8	10.22	0.677	1.997	0.243	0.347	U4	U4	
				0.0029	0.9				0.103				
5	0+020	0+115	R	0.0035	0.8	14.27	0.800	2.360	0.110	0.135	U4	U4	
				0.0007	0.9				0.026				
6	0+240	0+500	R	0.0032	0.8	9.46	0.651	1.921	0.101	0.145	U4	U4	
				0.0012	0.9				0.044				
7	0+500	0+855	R	0.0037	0.8	10.78	0.695	2.051	0.117	0.160	U4	U4	
				0.0012	0.9				0.043				
8	0+925	1+040	R	0.0011	0.8	10.30	0.680	2.005	0.036	0.062	U4	U4	
				0.0007	0.9				0.026				
9	1+195	1+395	R	0.0143	0.8	11.85	0.729	2.150	0.451	0.476	U4	U4	
				0.0007	0.9				0.025				

2.2 Culvert Design Calculation

Culvert Design Calculations

STATION : (Access Road LOT 3-1) 0+500.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 3-1 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

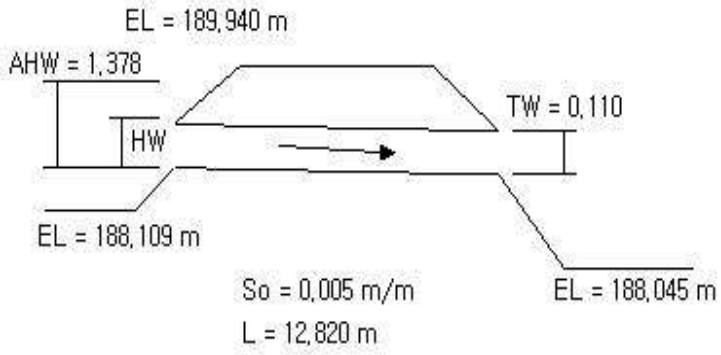
Catchment Area(A) =0.0044 ㎢
Qd = 0.152 ㎥/sec
Arrival Distance(L) = 337.000 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 226.75 m - 188.11 m
= 38.64 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0044 ㎢ Qd = 0.152 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.267

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.52	Φ900	0.29	0.26	0.5	0.4575	0.222	0.561	0.11	0.67	0.06	1.07	1.07	1.24	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd = 0.152 < AVo= (πd²/4) × 0.75 × 1.24 = 0.592 therefore, "O.K"

Review the Hydrological Calculation Results

STA. 0+500.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0044 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.152 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 38.64 m**

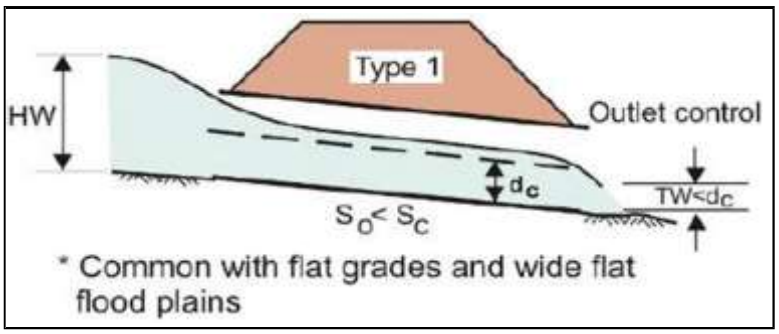
1. Hydrological Analysis

HW (1.07) ≤ 1.2 D (1.00) , So (0.005) < Sc (0.0103)

TW (0.11) < dc (0.222) , dn (0.267) > dc (0.222) therefore,

The shape proposed is 1rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = h_e + h_f + d_c - \frac{V_c^2}{2g} \cdot S_0 L$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 1.240 m / sec.

2. Analysis of the discharge area

Qd =0.152㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.152) < Q = AVo = (πd²/4) × 0.75 × 1.240 = 0.592 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 3-1) 0+955.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 3-1 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

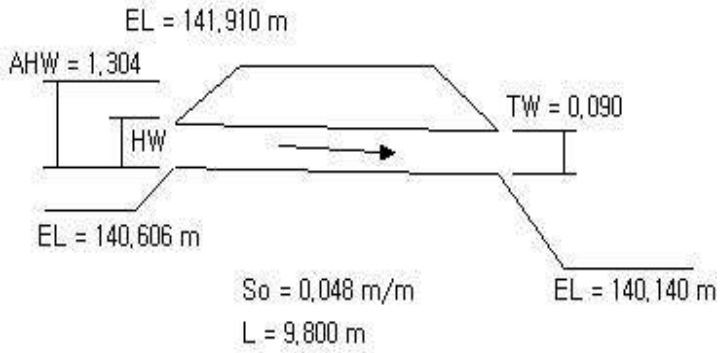
Catchment Area(A) =0.0059 ㎢
Qd = 0.204 ㎥/sec
Arrival Distance(L) = 403 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 184.92 m - 140.61 m
= 44.31 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0059 ㎢ Qd = 0.204 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.176

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.204	Φ900	0.35	0.31	0.5	0.0285	0.259	0.58	0.09	0.67	0.47	0.24	0.31	2.33	Inlet	O.K

Summary and Recommndations :

1. Qd = AVo, Qd = 0.204 < AVo= (πd²/4) × 0.75 × 2.33 = 1.112 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 1+480.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0059 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.204 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 44.31 m**

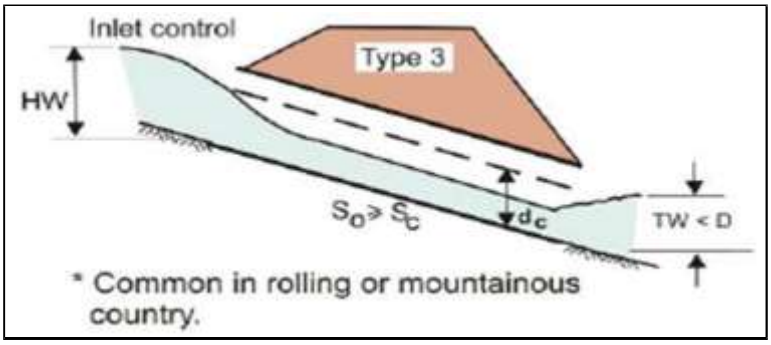
1. Hydrological Analysis

HW (0.31) ≤ 1.2 D (1.08) , So (0.0476) ≥ Sc (0.0102)

TW (0.09) ≤ dc (0.259) < D (0.9) , dn (0.176) < dc (0.259) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 2.330 m / sec.

2. Analysis of the discharge area

Qd =0.204㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.204) < Q = AVo = (πd²/4) × 0.75 × 2.330 = 1.112 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 3-1) 1+480.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 3-1 C-2

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

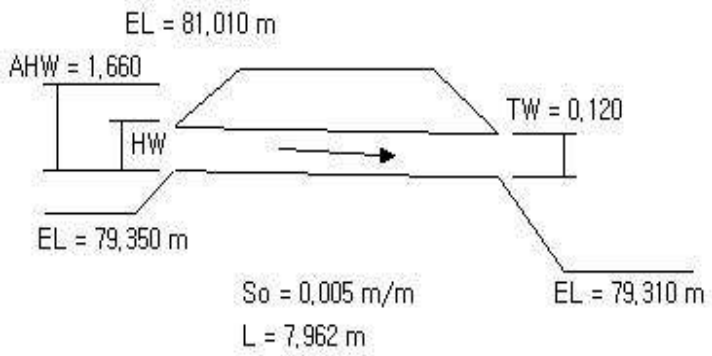
Catchment Area(A) =0.0087 ㎢
Qd = 0.301 ㎥/sec
Arrival Distance(L) = 500.0m

Recurrence Interval for Culv.25yr
Elev. Diffence: 138.94 m - 79.35 m
= 59.59 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0087 ㎢ Qd = 0.301 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.284

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.301	Φ900	0.44	0.4	0.5	0.0118	0.316	0.608	0.12	0.67	0.04	0.65	0.4	1.74	Inlet	O.K

Summary and Recommndations : 1. Qd = AVo, Qd = 0.301 < AVo= (πd²/4) × 0.75 × 1.74 = 0.830 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 1+480.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0087 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.301 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 59.59 m**

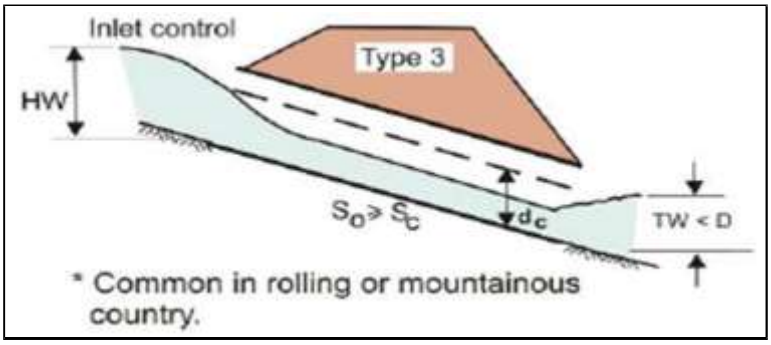
1. Hydrological Analysis

HW (0.40) ≤ 1.2 D (1.08) , So (0.0050) ≥ Sc (0.0034)

TW (0.12) ≤ dc (0.316) < D (0.9) , dn (0.284) < dc (0.316) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 1.740 m / sec.

2. Analysis of the discharge area

Qd =0.301㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.301) < Q = AVo = (πd²/4) × 0.75 × 1.740 = 0.830 : O.K

Culvert Design Calculations

STATION : (Access Road LOT 3-2) 0+010.00
PROJECT : Tina River Hydropower Development Project
Designation : LOT 3-2 C-1

Desinger :
Checker / Reviewer :
Date :

HYDROLOGICAL AND DESIGN INFORMATION

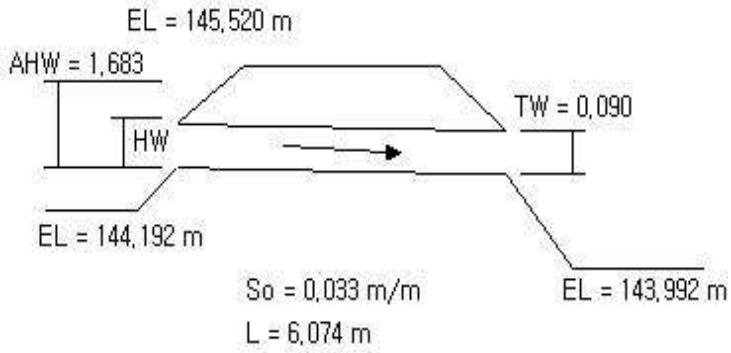
Catchment Area(A) =0.0045 ㎢
Qd = 0.156 ㎥/sec
Arrival Distance(L) = 360.46 m

Recurrence Interval for Culv.25yr
Elev. Diffence: 186.42m - 143.19 m
= 43.23 m

Rational Formula Qd = 0.278 C I A	STD Runoff Formula Qd = RFxLFxFFxQ	Rainfall Area
Tc = 0.100 hr I = 155.800 mm/h C = 0.800 A = 0.0045 ㎢ Qd = 0.156 ㎥/sec	RF = 0.000 LF = 0.000 FF = 0.000 Q = 0.000 ㎥/sec Qd = 0.000 ㎥/sec	Chupu Karma gauging station

Water level in Culvert = 0.169

SKETCH



CULVERT DESCRIPTION	Qd/ cell (㎥/sec)	Size D or Ht	HEADWATER COMPUTATION										Adjustment HW	OUTLET VELOCITY Vo	CONTROLLING HEADWATER	Comments
			INLET CONTROL		OUTLET CONTROL = HW = H + ho - SoL											
			HW/D	HW	Ce	H	dc	(dc+D)/2	TW	ho	SoL	HW				
PIPE	0.156	Φ900	0.29	0.27	0.5	0.0105	0.225	0.563	0.09	0.67	0.2	0.49	0.27	1.89	Inlet	O.K

Summary and Recommndations : 1. Qd = AVo, Qd = 0.156 < AVo= (πd²/4) × 0.75 × 1.89 = 0.902 therefore,"O.K"

Review the Hydrological Calculation Results

STA. 0+010.0000

⊙ **Input Sectional Shape (Φ900)**

·Catchment Area(A) = 0.0045 ㎢	**·Rainfall Intensity(I) = 155.8mm/hr**
·Peak Discharge(Qd) = 0.156 ㎥/sec	**·Rain fall Area = Chupu Karma**
·Recurrence Interval = 25 years	**·Elevation Diffence = 43.23 m**

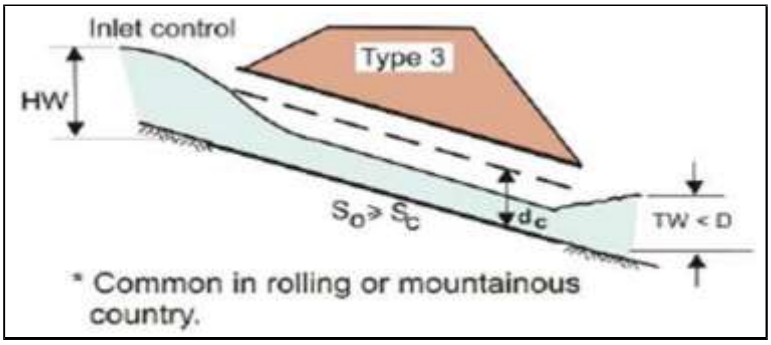
1. Hydrological Analysis

HW (0.27) ≤ 1.2 D (1.08) , So (0.0329) ≥ Sc (0.0103)

TW (0.09) ≤ dc (0.225) < D (0.9) , dn (0.169) < dc (0.225) therefore,

The shape proposed is 3rd - type as follows.

Hydrological diagram



In the above hydraulic diagram, Control section is the outlet, and the applied energy equation is

$$HW = 0.467 \cdot \left(\frac{Q}{D}\right)^{\frac{2}{3}} + 0.051 \cdot (1 + C_d) \cdot V_c^2$$

HW can be found from the equation.

The flow velocity at this time is the flow velocity in the critical depth of th outlet

and Vc = 1.890 m / sec.

2. Analysis of the discharge area

Qd =0.156㎥/sec of the drainage area at the current STA., and the Pipe size is φ900

Therefore, compared with the design discharge and the capacity of drainage are as follows.

Qd (0.156) < Q = AVo = (πd²/4) × 0.75 × 1.890 = 0.902 : O.K

ANNEX C-10-III RIVER DIVERSION CONSTRUCTION METHOD

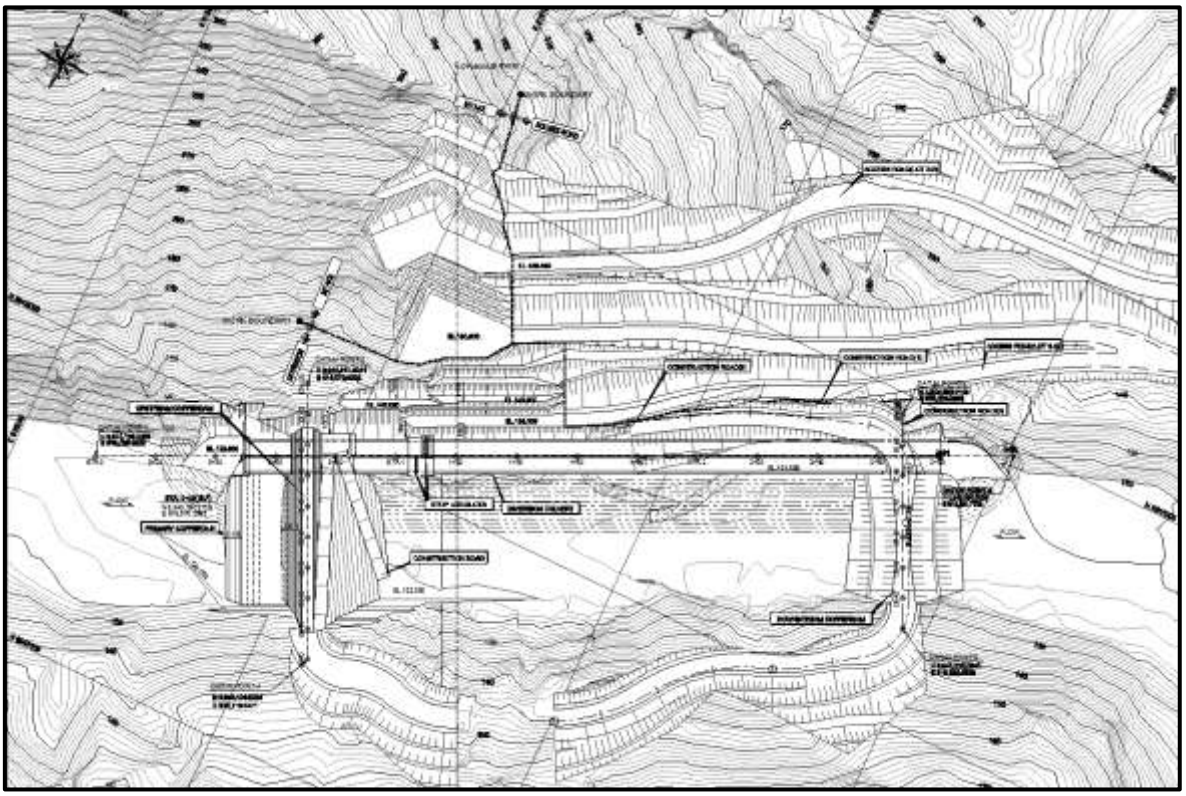
1.1. River Diversion

1.1.1. Priority Management Plan

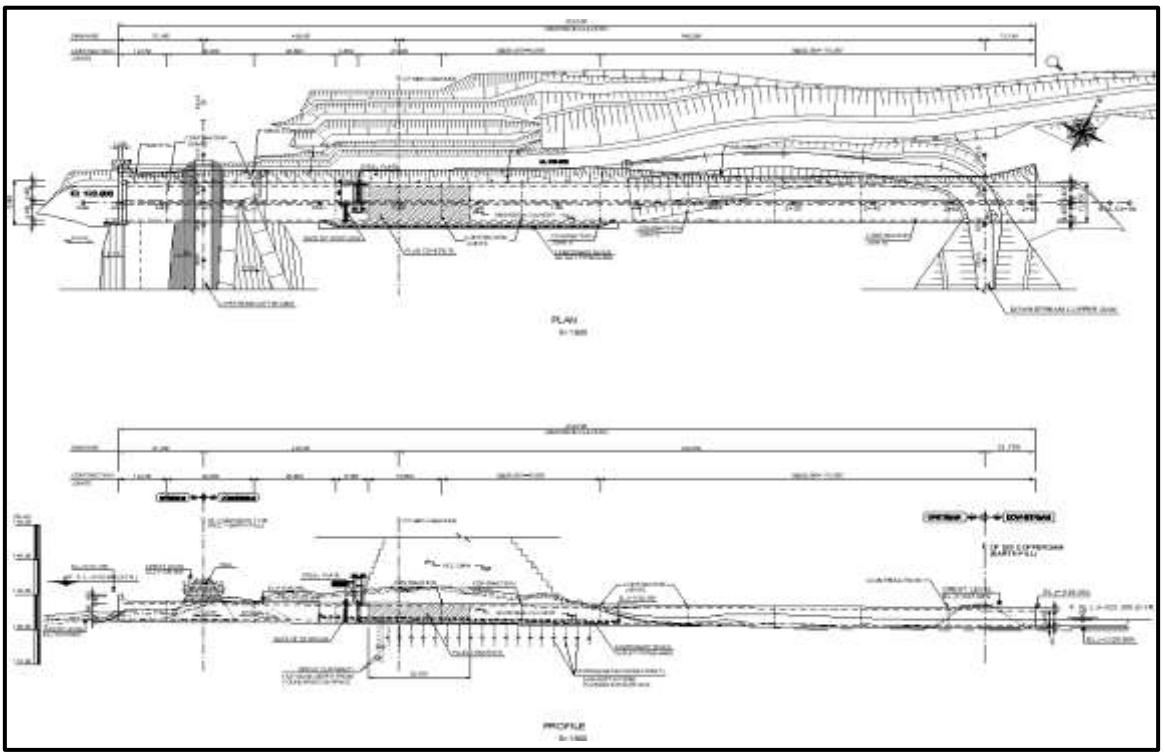
1) General

The river diversion for dam construction consists of 3 components – diversion culvert, upstream Composite type(RCC + Earth fill) and downstream cofferdam. The design flood for longitudinal cofferdam, upstream and downstream cofferdams and diversion culverts is 360 m³/s, which is 2-year frequency flood.

Layout and profile of diversion facilities are shown in Figure 1-1 and Figure 1-2.



<Figure 1-1> Layout of Diversion Facilities



<Figure 1-2> Profile of River Diversion

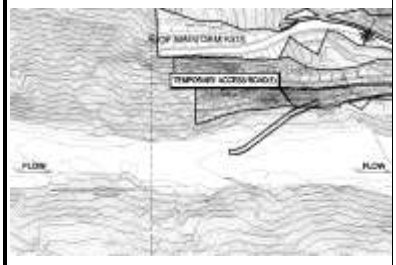
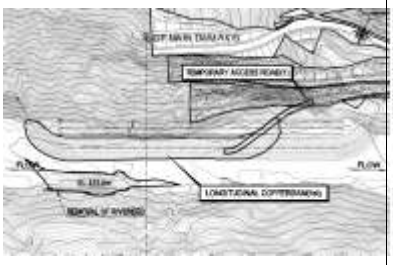
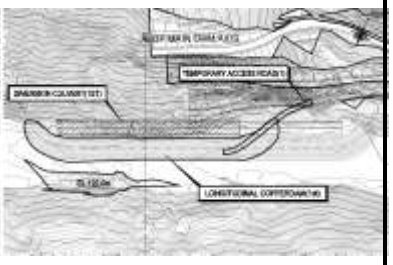
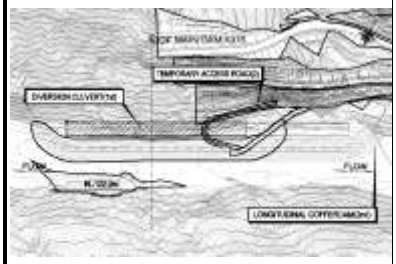
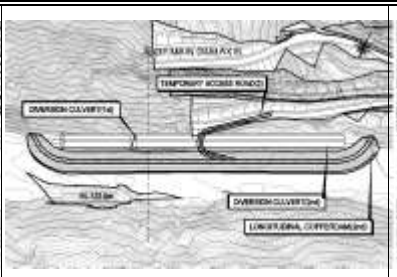
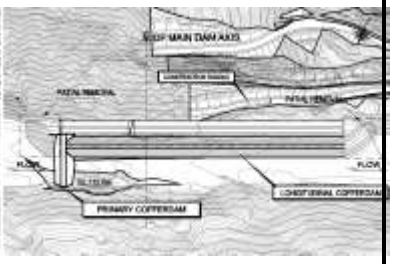
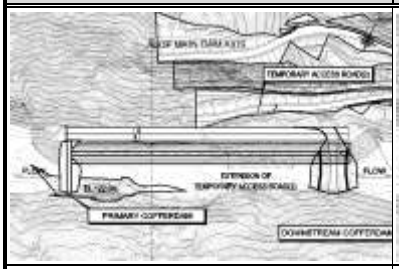
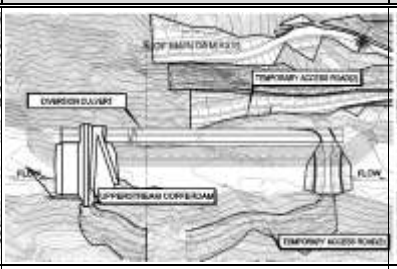
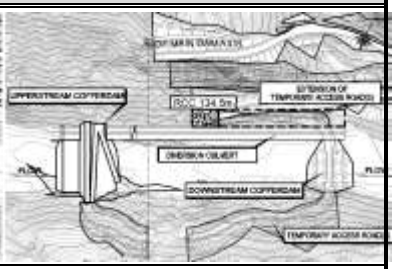
Specifications of river diversion works are described in table 1-1.

<Table 1-1> > Comparison of River Diversion Works

Classification		F/S	Basic Design	Detail Design
Design Discharge		360 m^3/s (2-year)	(Same)	(Same)
Longitudinal Cofferdam	Type	Earth Fill U/S: EL.127.0 m, D/S: EL.126.0 m	Earth Fill	(Same)
	Crest EL.		U/S: EL.127.0 m, D/S: EL.126.0 m	(Same)
	Crest Width		3.0 m	(Same)
Upstream Cofferdam	Type	RCC	RCC	RCC + Earth Fill
	Crest EL.	EL.134.0 m	EL.134.5 m	(Same)
	Height/Length	-	13.0 m / 69.8 m	(Same)
	Gradient	1:1.0	1:0.5	1:0.5 / 1:1.5
	Grouting Method	Cut-off wall or Grout curtain	Permeation grouting	(Same)
Downstream Cofferdam	Type	RCC	Earth Fill	(Same)
	Crest EL.	EL.126.0 m	EL.127.5 m	(Same)
	H/L	-	6.5 m / 62.1 m	7.7~9.5 m / 42.2 m
	Gradient	1:1.0	1:1.2 / 1:1.5	1:1.5 / 1:1.8
	Grouting Method	Cutoff wall or Grout curtain	-	-
Diversion Culverts	Location	Right bank	(Same)	(Same)
	Type	Reinforced concrete (Precast)	(Same)	(Same)
	Size	B3.6 m×H3.6 m×3row	B4.4 m×H4.4 m×2row	(Same)
	Length	450 m (150 m×3)	332 m (166m×2)	464m (232m×2)

2) Sequence of the Works

River diversion will be executed in 9 stages.

Stage-1	Stage-2	Stage-3
		
• Embankment of temporary access road (1)	• Using temporary access road (1) • Riverbed excavation (EL.122.0m) • Embankment of longitudinal cofferdam (1st) • Foundation excavation of main dam section • 1st Foundation treatment (Consolidation grouting)	• Using temporary access road (1) • Construction of diversion culverts (1st)
Stage-4	**Stage-5**	**Stage-6**
		
• Embankment of construction road (2) • Removal of construction road (1)	• Using temporary access road (2) • Embankment of longitudinal cofferdam (2nd) • 2nd Foundation treatment (Consolidation grouting) • Construction of diversion culvert (2nd)	• Using temporary access road (2) • Embankment of upstream primary cofferdam • Partial removal of longitudinal cofferdam
Stage-7	**Stage-8**	**Stage-9**
		
• Using temporary access road (2) • Extension of temporary access road (2) • Embankment of downstream cofferdam	• Using temporary access road (2) • Embankment of temporary access road (3) • Embankment of upstream cofferdam	• Constructed to EL.134.5m RCC • Extension of temporary access road (3) • Removal of temporary access road (2)

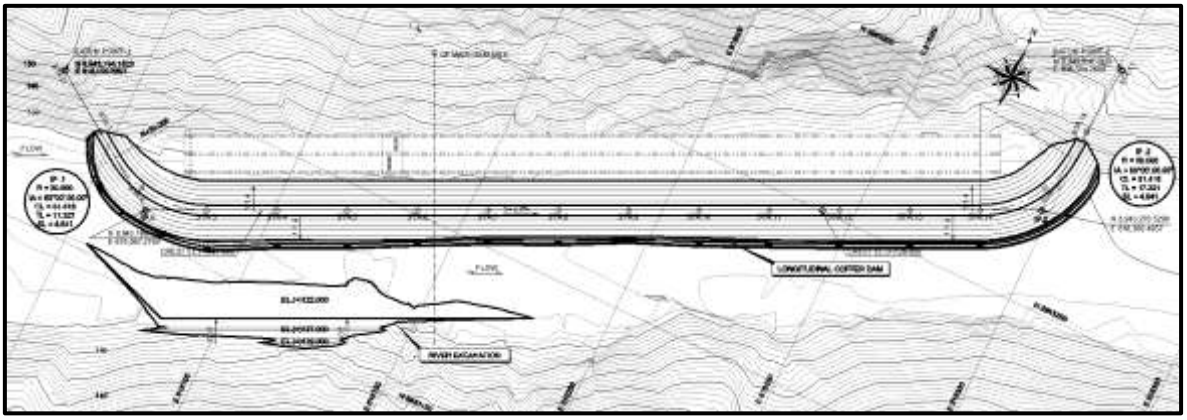
<Figure 1-3 > Sequence of River Diversion Works

1.1.2. Longitudinal Cofferdam

The longitudinal cofferdam is temporarily installed during the construction of diversion culvert. Longitudinal cofferdams will be installed prior before diversion culvert to block water flow during diversion culvert construction.

The total length is 296.85m and the width of the Crest Level is 3.0m.

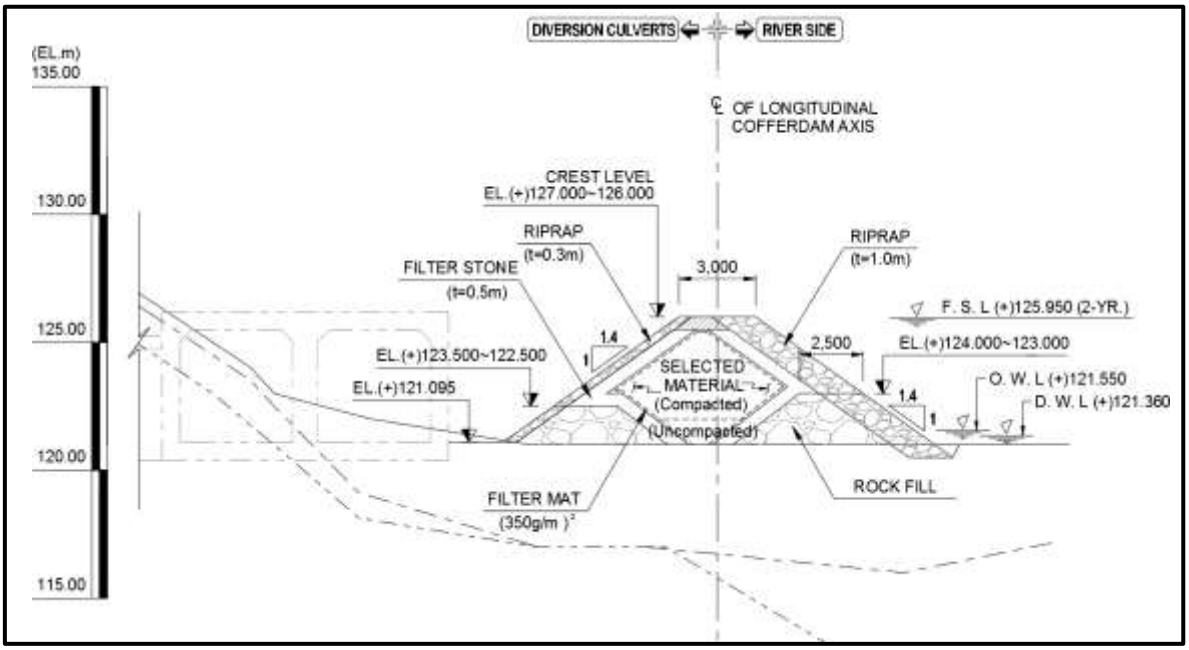
Layout of longitudinal cofferdam are shown in figure 1-4



<Figure 1-4> Layout of Longitudinal Cofferdam

On the lower part of the longitudinal cofferdam, filter mat is laid after rock filling, and the central part is compacted and filled with random material. The exterior is arranged at a slope of 1.4:1.0 and a riprap is installed with 1.0m thickness to protect the slope.

Cross section of longitudinal cofferdam is shown in figure 1-5



<Figure 1-5> Cross section of primary cofferdam

The longitudinal cofferdam will be constructed in several stages. The division of stage is adjusted according to interference with the main dam or the schedule of diversion culvert.

The longitudinal cofferdam should be removed after the division culvert is completed and the river diversion.

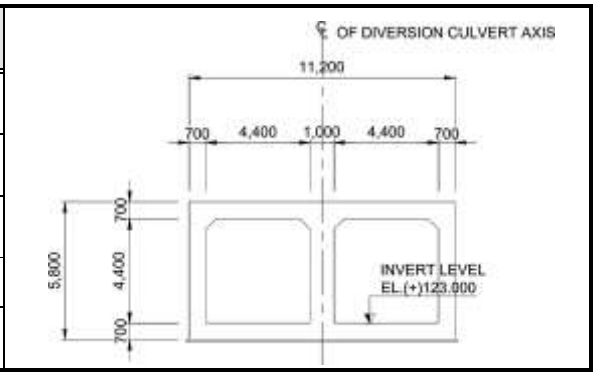
1.1.3. Diversion Culvert

1) General

Specification of diversion culvert is as follows.

<Table 1-2> Specifications and Typical Section of Diversion Culvert

Specifications		
Design Discharge		360 m³/s
Bottom EL.	Inlet	EL.123.00 m
	Outlet	EL.120.90 m
Max. Water Level		EL.133.55 m
Size		B4.4 m×H4.4 m×2row



The bottom elevations of the diversion culverts have been set higher (EL.123.0m for the inlet and 120.9m for the outlet) than the annual mean water level in order that the ~~1st~~ diversion culvert can be constructed under dry condition.

The water level corresponding to the design flood of 360 m³/s for upstream cofferdam is EL.133.55 m, to which 0.66m freeboard has been added to set the crest level of upstream cofferdam at EL.134.50 m.

The diversion culvert has a section of two rows and B=4.4m by H=4.4m.

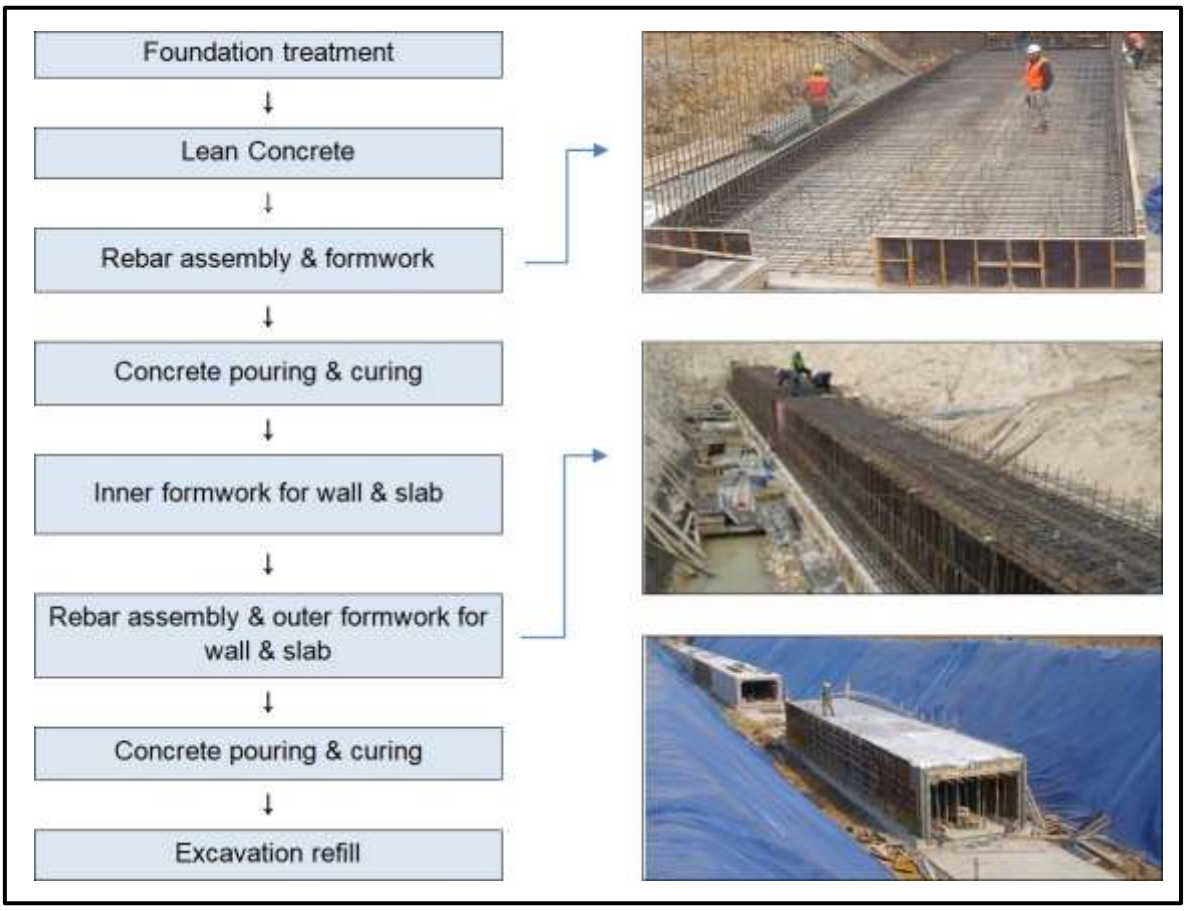
Cast-in-place concrete will be used in consideration of the site conditions and constructability.

Materials in <Table 1-3> are used for diversion culvert.

<Table 1-3> Materials of Diversion Culvert

Division	Specifications	Remark
Concrete	fc=30Mpa	Compressive Strength at 28days
Lean Concrete	fc=18Mpa	Compressive Strength at 28days
Steel Bar	fc=420Mpa	Grade 60

2) Sequence of the Works

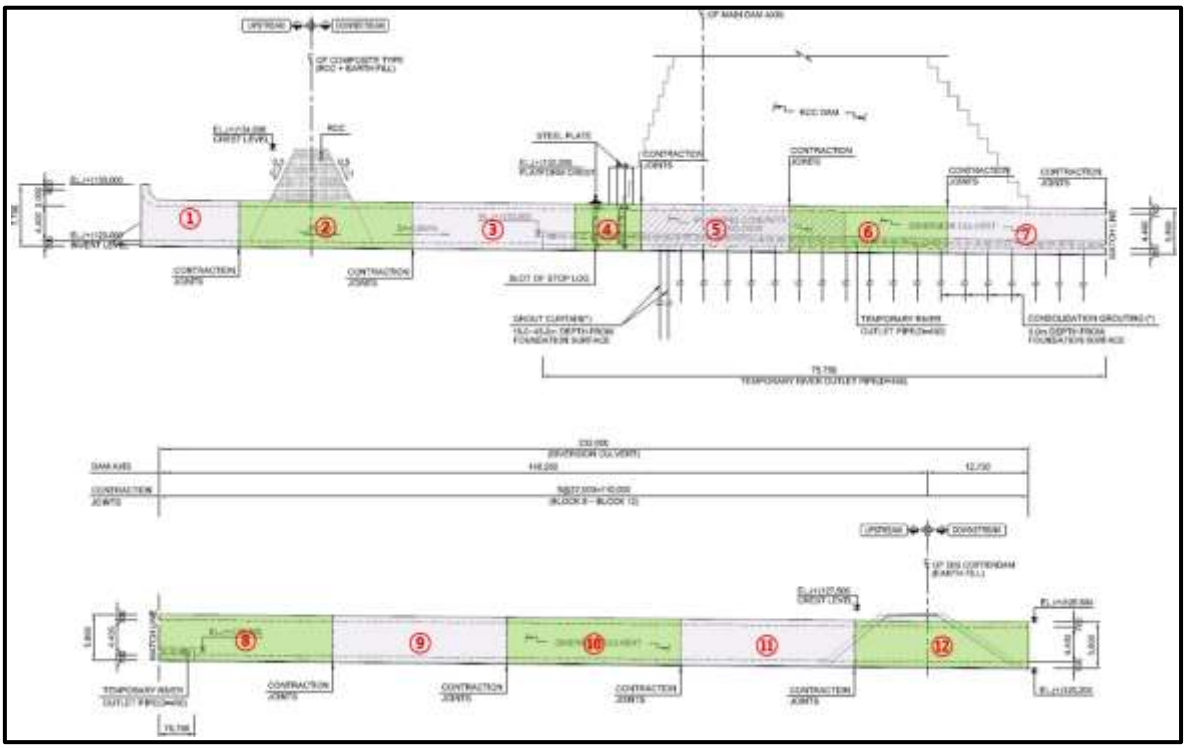


<Figure 1-6 > Sequence of Diversion Culvert Construction

Diversion culvert needs to be considered because the main dam, upstream and coffer dams are interfered.

Multiple work crews are required to be completed within construction period.

<Figure 1-7> shows the location of the Contraction Joint.



<Figure 1-7 > Contraction Joint Plan

Interferences in the main section are as follows.

Lot ② proceeds with upstream dam. When diversion culvert is completed, RCC is poured over slab level of the diversion culvert.

Lot ④ is a section where the slot of the stop log for plugging of the diversion culvert is installed and has a complex shape, so procurement management of frame is important.

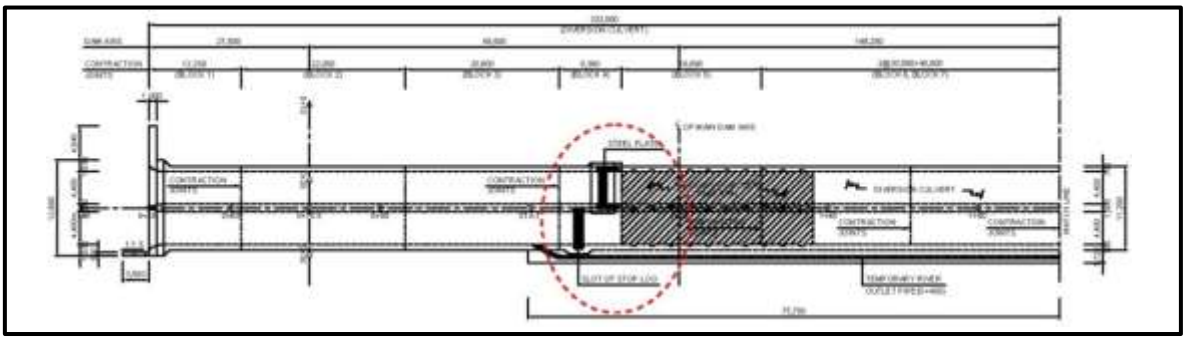
Lot ⑤, ⑥ and ⑦ is the interference section with main dam. They can be started after foundation grouting is completed after excavation.

Lot ⑪ and ⑫ are a section where downstream is constructed and does not affect the schedule compared to other sections.

The working group is operated by 2~3 work crews considering the continuity and efficiency of work.

3) Installation of Stop log

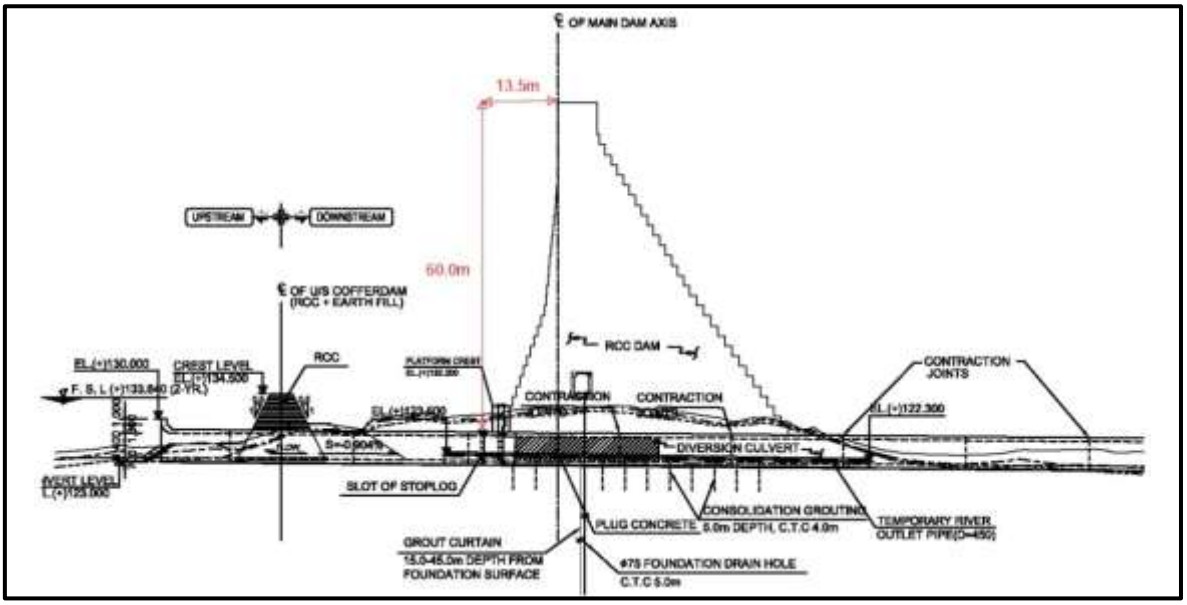
Once the main dam has been completed, take down all equipment in the dam. Then, stoplog and temporary river outlet pipe (for environmental flow during the initial reservoir filling) will be installed before constructing 1st plug concrete.



<Figure 1-8> Stop log Location

Stop log is made of steel. It is installed using 100ton mobile crane.

The following figure is a concept drawing for stop log installation.



<Figure 1-9 > Stop log Installation

As shown in upper picture, the installation height is about 60m, and horizontal distance is 13.5m from the center of crest road. Therefore, maximum lifting height including crane height is 75.0m and the maximum horizontal distance is 17.0m.

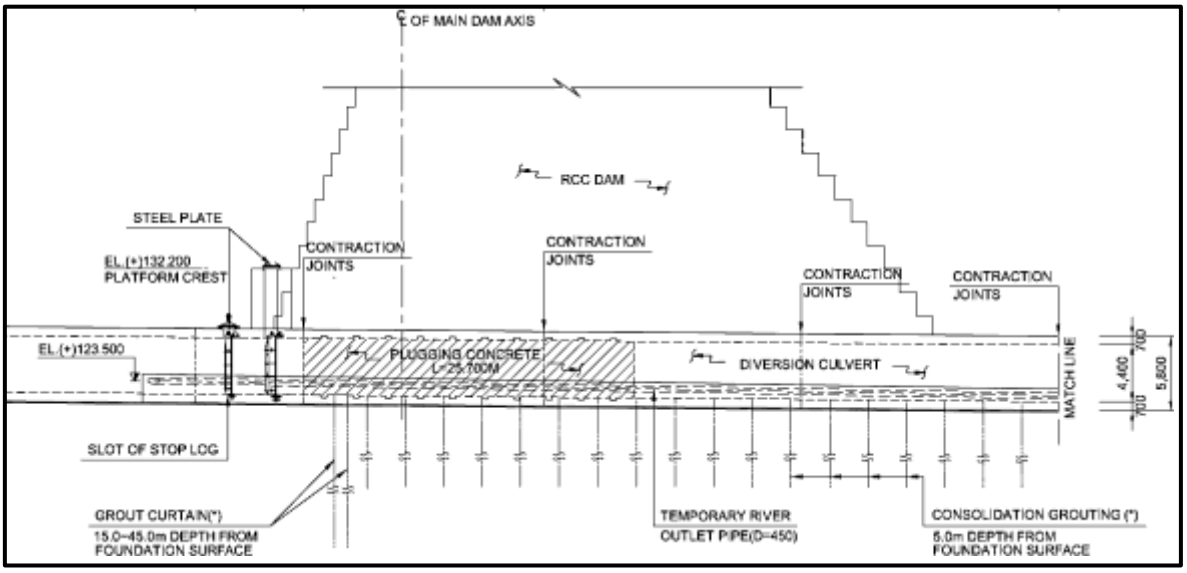
Since the working time is very short due to the rise of the water level after installation of stop log, the stop log will not divided (or one per location) considering the installation time and operator's evacuation time.

To reduce the stop log installation time, each stop log will be mounted on the diversion culvert temporarily.

After the completion of stop log installation, the operator will evacuate by riding the elevating cage installed on the crane.

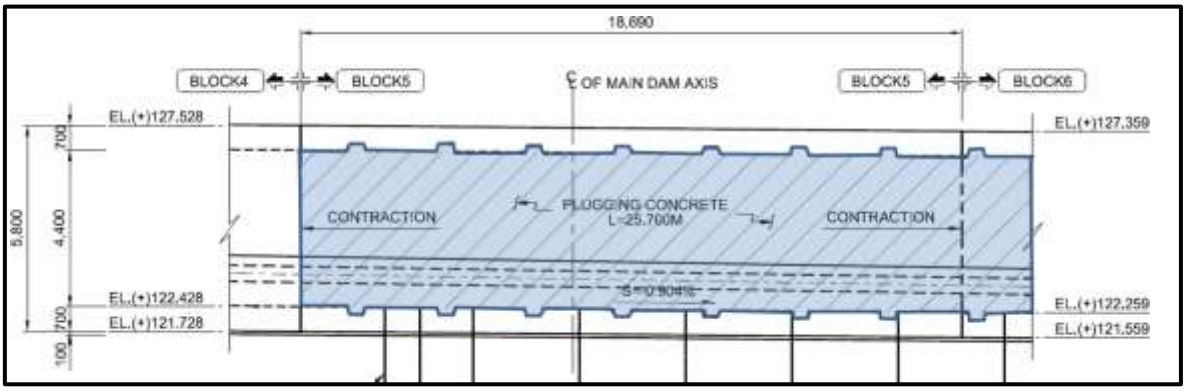
4) Concrete Plug of Diversion Culvert

Concrete plug is installing a stopper on the diversion culvert to fill the reservoir with water. Concrete plug is installed at 95m from upstream of diversion culvert and its length is 25.7m.



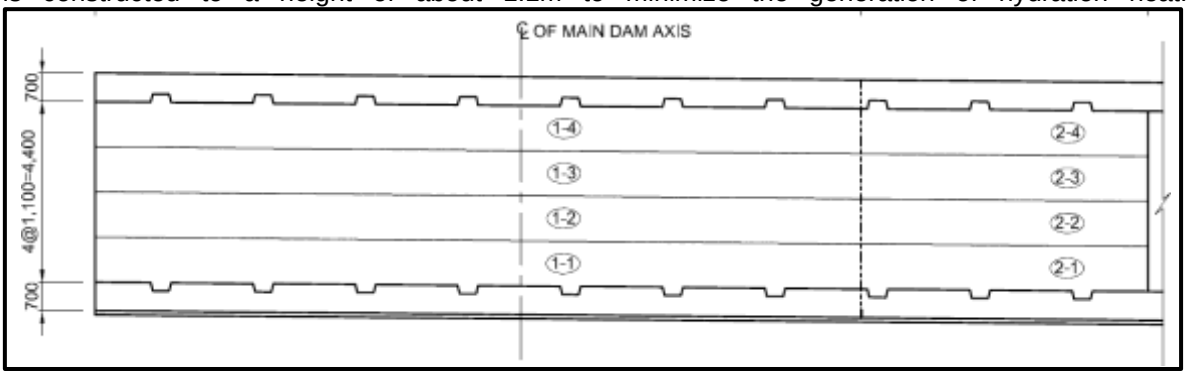
<Figure 1-10> Profile of Concrete Plug

The plug concrete shall be constructed to ensure shear resistance during the dry season with shear keys installed every 2.5m to secure shearing resistance.



<Figure 1-11> Shear Key Plan

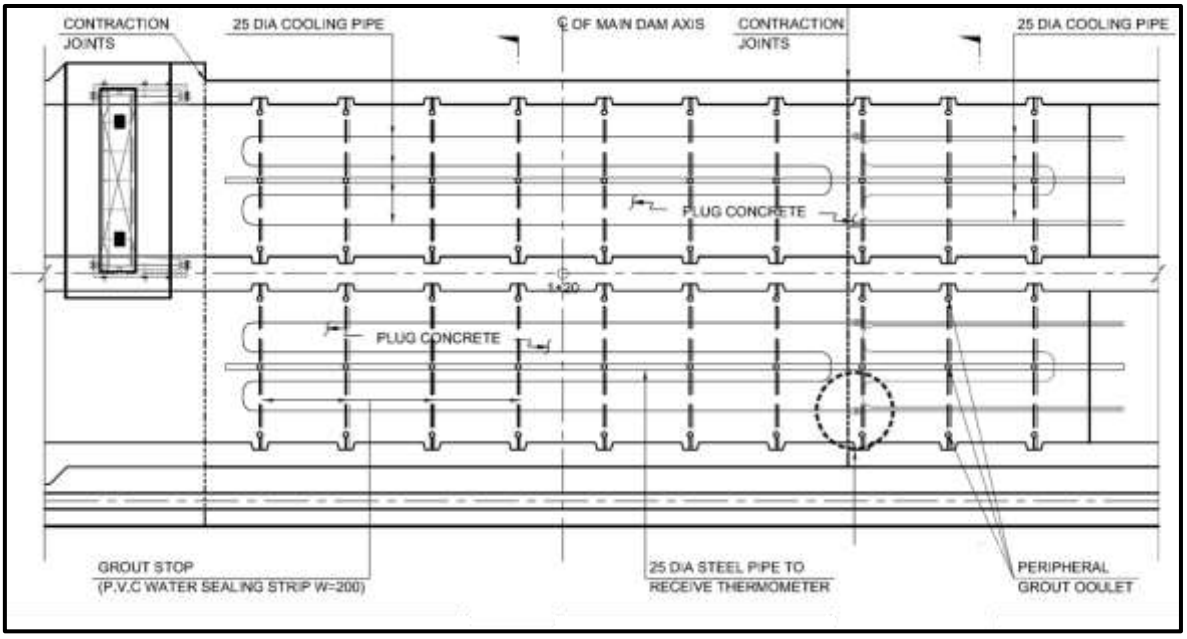
Concrete will be placed in four stages to prevent cracking by hydration heat. The height of one stage is constructed to a height of about 1.1m to minimize the generation of hydration heat.



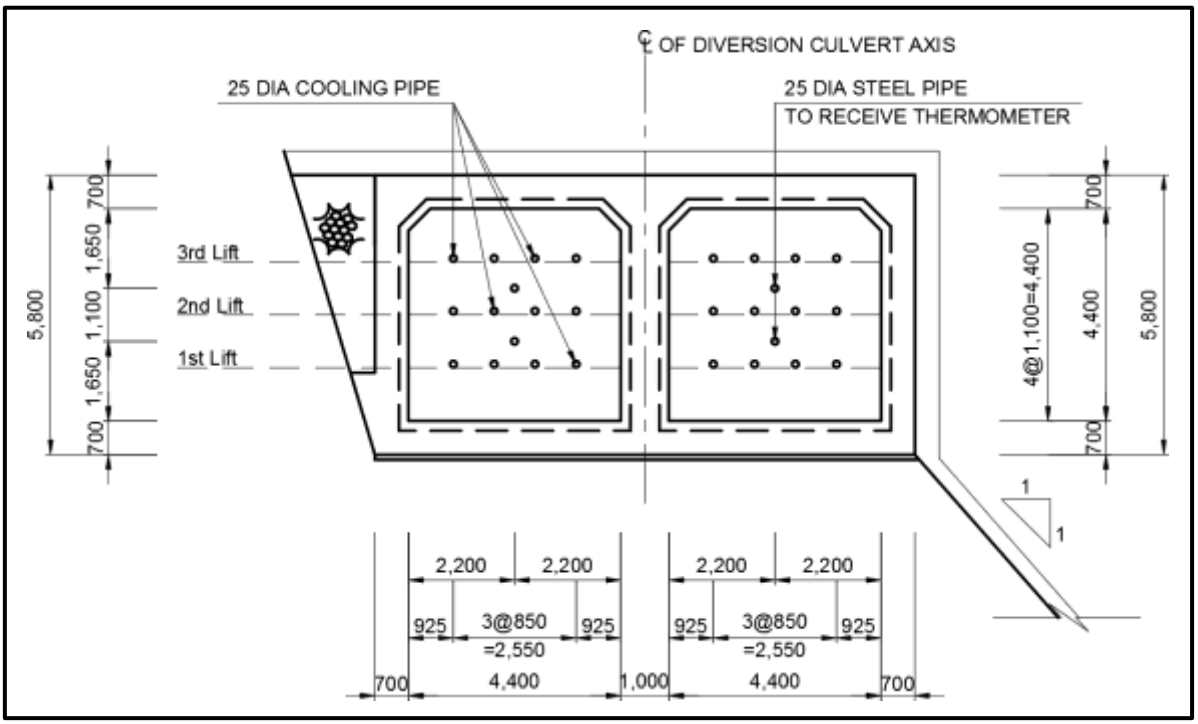
<Figure 1-12> Plugging Concrete Pouring Plan

Cooling pipe is installed to control the hydration heat after cleaning the surface before next stage.

The layout of the cooling pipe is shown in <Figure 1-13> and <Figure 1-14>.



<Figure 1-13> Cooling Pipe Plan



<Figure 1-14> Cooling Pipe Section

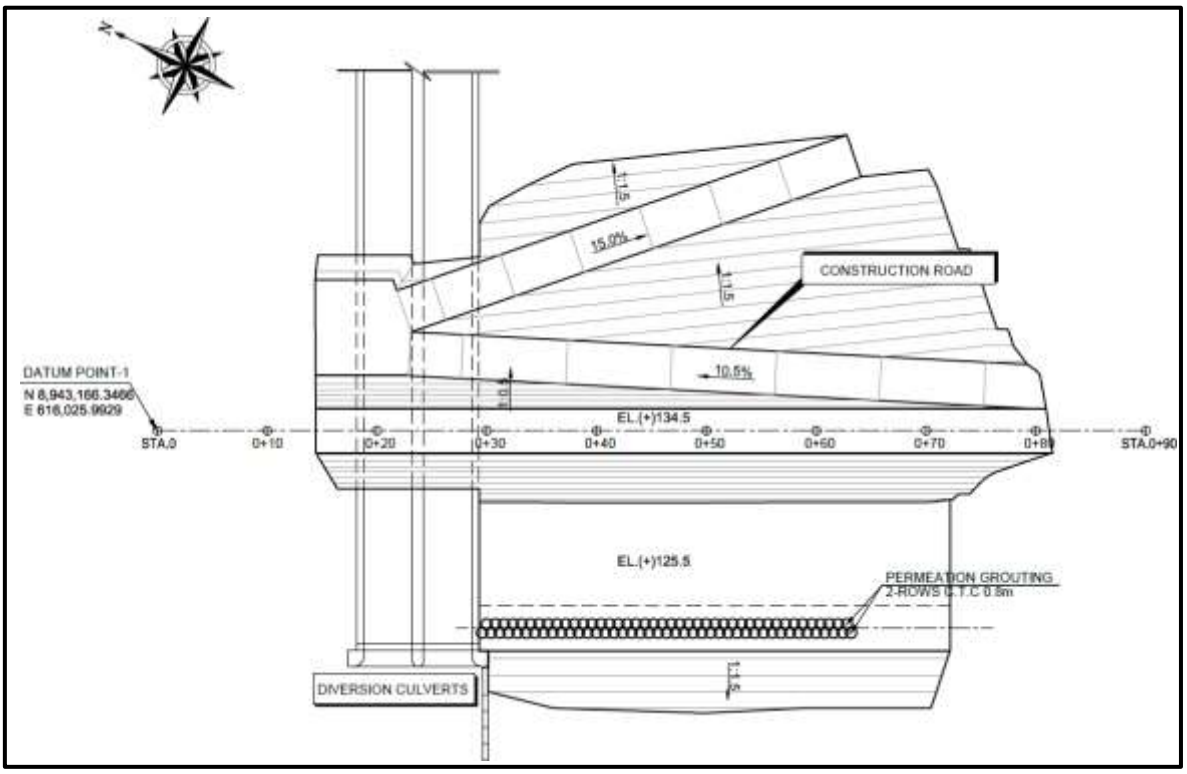
In the final pouring, grouting pipes or non-contraction concrete can be used to fill the empty space due to the contraction of concrete.

Peripheral grouting outlet is shown in figure 1-15.

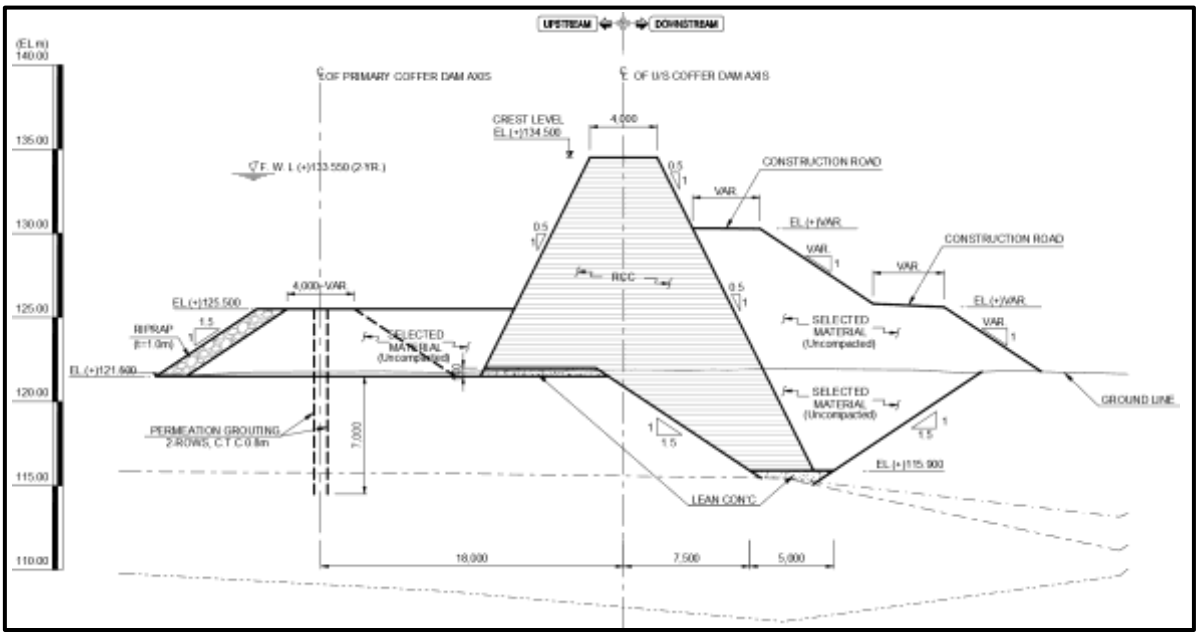
1.1.4. Upstream Coffer(RCC)dam

The upstream cofferdam will be constructed as Roller Compacted Concrete (hereinafter called “RCC”) type, same condition to the main dam excluding form work, to allow trial construction of the main dam.

Typical section of upstream cofferdam is shown in figure 1-16.



<Figure 1-15> Plan of Upstream Cofferdam



<Figure 1-16> Typical Section of Upstream Cofferdam

The construction will be carried out in the following sequence:

① River bed excavation (soil and rock)

② Primary Coffer Dam Embanking & Permeation Grouting

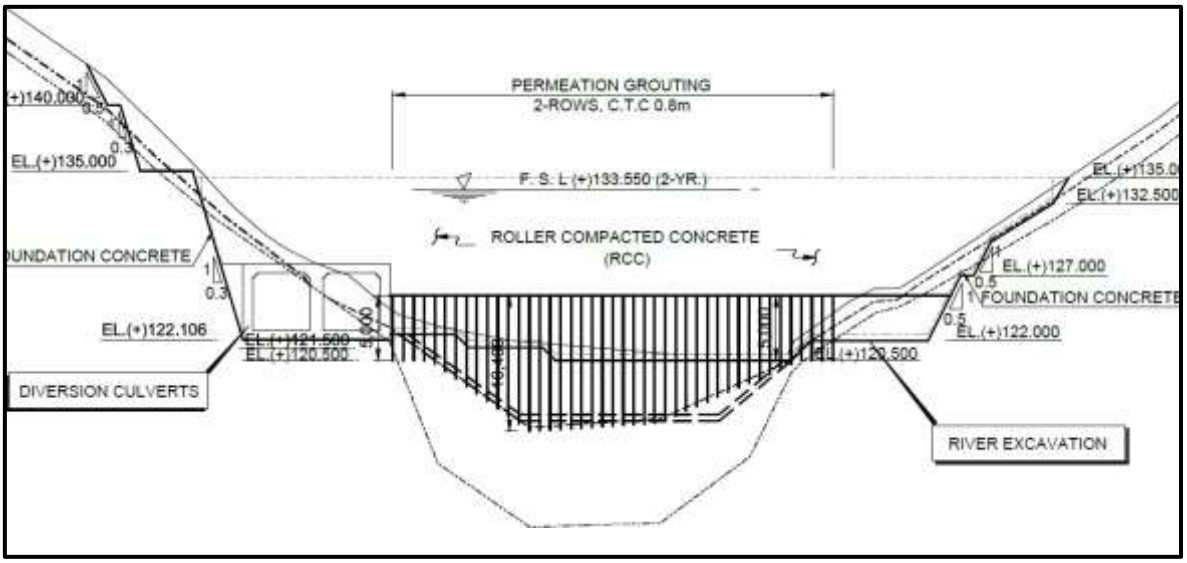
③ Primary Coffer Dam Riprap & Base Concrete

④ Under EL(+) 125.500 RCC cofferdam construction (placed and compacted every 0.3m, same as the main dam)

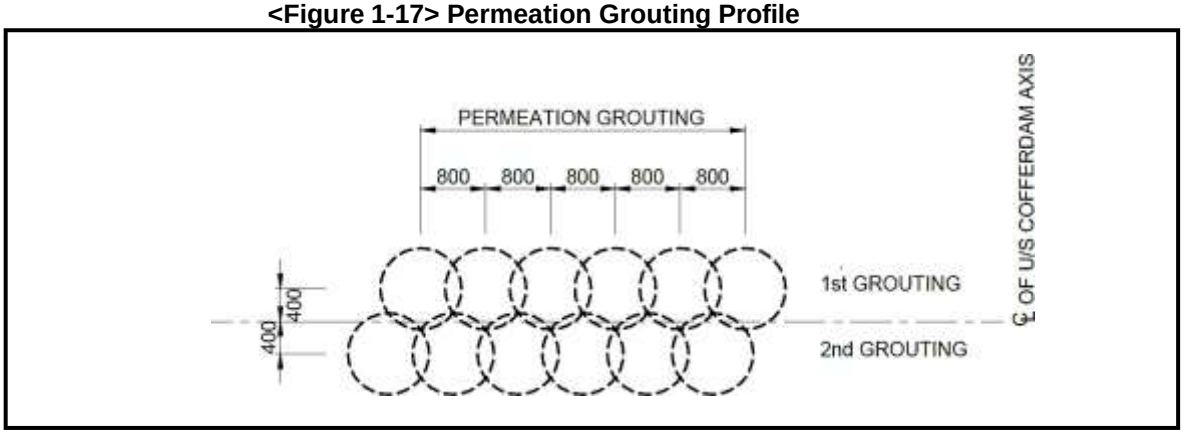
⑤ Foundation Concrete

⑥ EL(+) 125.500~ EL(+) 134.500 RCC cofferdam construction (placed and compacted every 0.3m, same as the main dam)

After excavation, the primary coffer dam will be built. And permeation grouting to block the inflow of groundwater will be executed as shown in the following figure.



<Figure 1-17> Permeation Grouting Profile



<Figure 1-18> Permeation Grouting arrangement

Three types of concrete are used for the upstream dam, and specifications and required quantities are as follows.

<Table 1-4> Concrete Specifications of Upstream Dam

Division	Compressive Strength(f'c)	Remark
Foundation Concrete	18MPa	AT 28 days
Mass Concrete	18MPa	AT 28 days
RCC	12MPa	AT 90 days

Mass concrete and foundation concrete will be transported by mixer truck and poured using a concrete pump car or chute, and RCC is constructed using the same equipment used in the main dam during test construction.

The external slope of upstream dam is finished by vibrating plate compaction.

The upstream and downstream of RCC will be filled earth to improve the stability and use the main dam construction road.

Riprap will be installed 1.0m-thick on the slope of construction road on upstream to control erosion.

<Figure 1-19> RCC Dam Slope Forming & Riprap

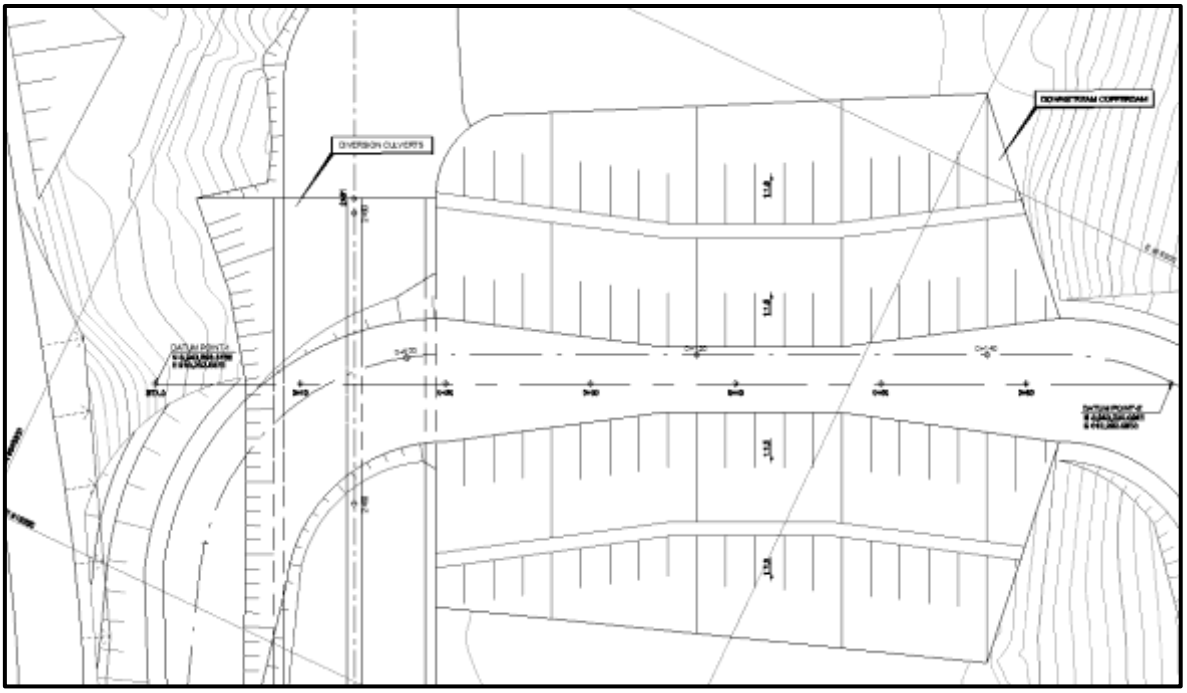
Upstream dam will be constructed before main dam in the dry season.

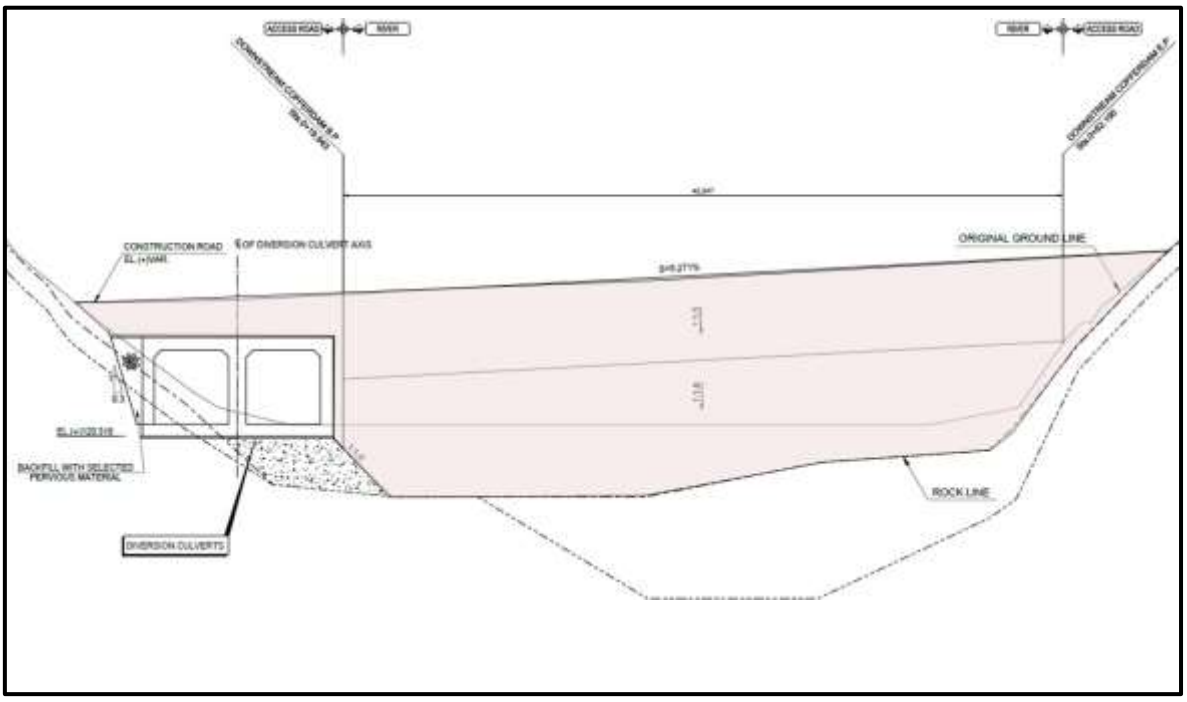
1.1.5. Downstream Cofferdam

The downstream cofferdam will be constructed to prevent back flow of river during the construction of the main dam. Therefore, in order to maintain flow of the river until river diversion will be completed, it will be partially executed concurrently with diversion culvert or started after completion.

The downstream cofferdam will be a central core earth dam which allows immediate utilization of excavated materials, prompting economical and rapid construction.

Construction road of 4.0m-wide will be installed on the upper part of downstream cofferdam, which is used as a connecting road for excavation of main dam and plunge pool





<Figure 1-20> Profile of Downstream Cofferdam

The construction will be carried out in the following sequence:

① Riverbed excavation

② Simultaneous construction of core and Selected materials (layer compaction)

③ Filter Mat Laying & Rip-rap (upstream and downstream)

The equipment mobilization plan for the construction of Downstream Cofferdam is shown in the table below.

<Table 1-5> Downstream Cofferdam equipment input plan

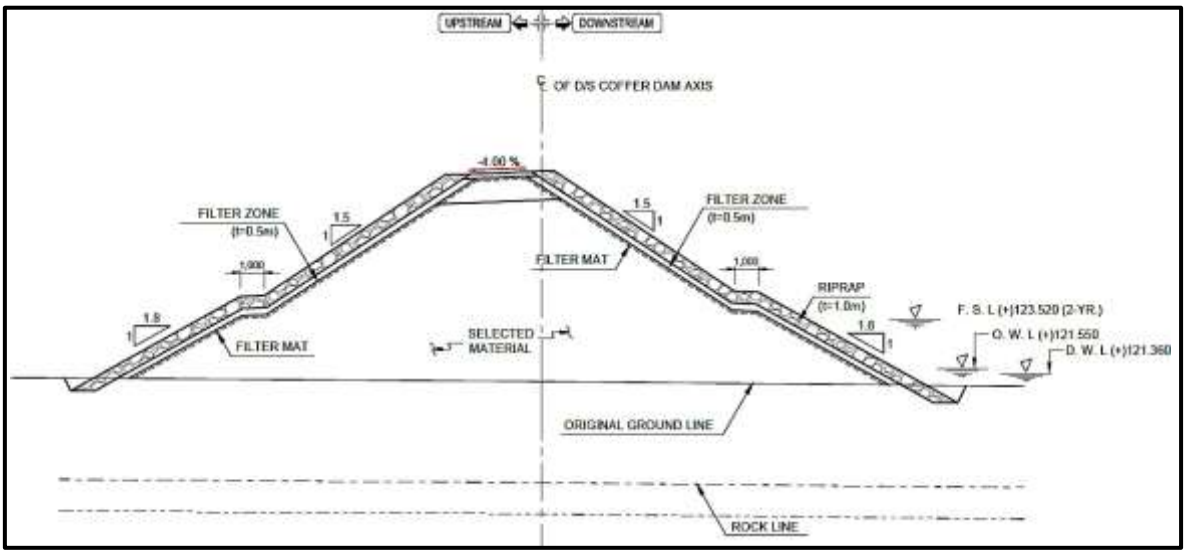
Division	Equipment	Capability	Number	Remark
Excavation	Dozer Backhoe	19ton 0.7~1.0m³	1 1	Cutting Loading
Embanking	Dozer Vibrating Roller Dump Truck	19ton 10.5ton 15~25.5ton	1 1 -	Spreading Compaction Transportation
Riprap	Backhoe	0.7~1.0m³	1	Placing

Embanking of cofferdam will be executed according to regular compactness to a thickness of 0.3~0.6m using foundation or soil excavated from the nearby working area. The surface is inclined for drainage.

Downstream cofferdam will be maintain a slope of 1:1.8 in the lower part and 1: 1.5 in the upper part, and create 1.0m-wide berm in the middle part to improve the stability of the slope.

After installing a filter mat on the slope to prevent leakage of soil, a 0.5m thick filter zone and 1.0m riprap will be installed.

Typical section of downstream cofferdam is as shown below.



<Figure 1-21> Typical Section of Downstream Cofferdam

The downstream section of the culverts shall be partially demolished.

1.1.6. Wastewater Management from Tunnelling

Wastewater generated from shield TBM works shall be treated and discharged to meet environmental standards. For this, the wastewater treatment plant (Q=1,000ton/day, W7.5m x D16.5m) is proposed.

The wastewater treatment facility consists of a sedimentation tank, a flow control tank, a chemical injection facility, a settling tank and a sludge dewatering device (filter press).

Wastewater collected through the drainage system in the tunnel is transported to the wastewater treatment facility through piping. After treatment, a treated wastewater is discharged to a waterway, and the remaining sludge is collected by a local treatment company (faith holdings).