APPENDIX - 1

FACILITY PLAN AND DESIGN

CHAPTER 1 GENERAL

1.1 Outline of Main Facilities

Main facilities planed in Wau Rice Scheme are as follows,

- Command area: A=500ha
- Dam: 1 place
- Pump station: 1 place
- Distribution canal: L=6.2km
- Main canal (command area): L=7.1km
- Secondary canal, drainage, road, etc. .in command area: 1 L.S
- Main drainage Canal: L=7.3km
- Flood Protection Dike: L=9.7km

Pump facility are operated during the rice cropping term in rainy season, and the reserved water in the dam is used during vegetable cropping term in dry season, considering the operation cost of the pump and the hydrology condition in the site.

1.2 Command Area

Command area is located beside Wau town, and has the feature of bare land without planting in the flood plain. The land is approximately flat and the land gradient toward R, Jur shows around 0.2%. Dam site is located 9.5km from Wau town. The land cover in the site is bushes and grasses. Pump station and canal line are located between the command area and dam site. There are trees, small communities, farms, etc. along the line.

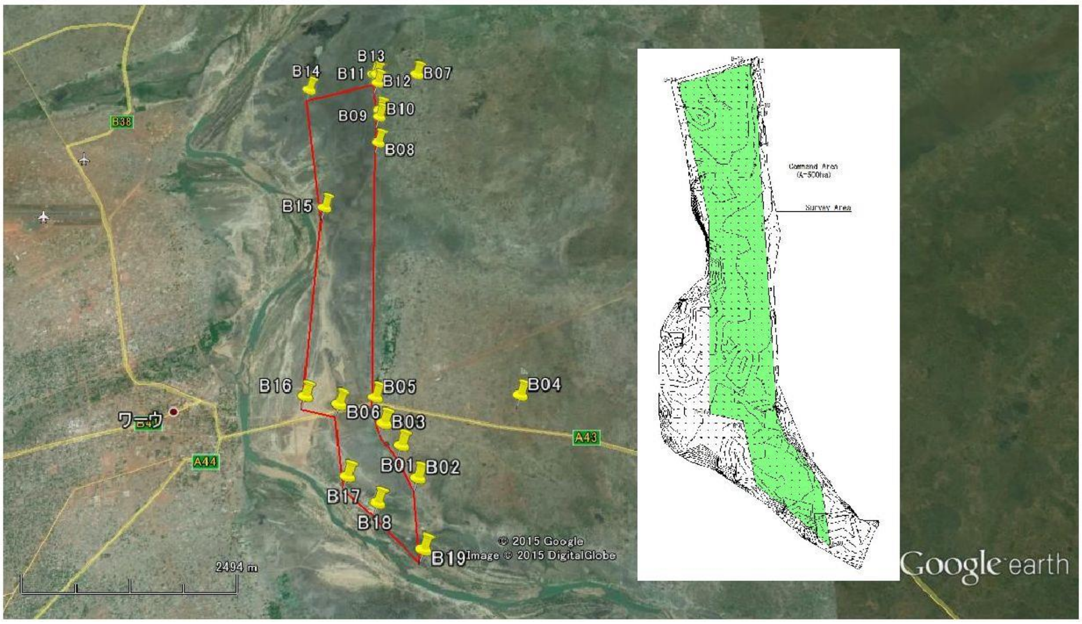


Figure1.2.1 Command Area

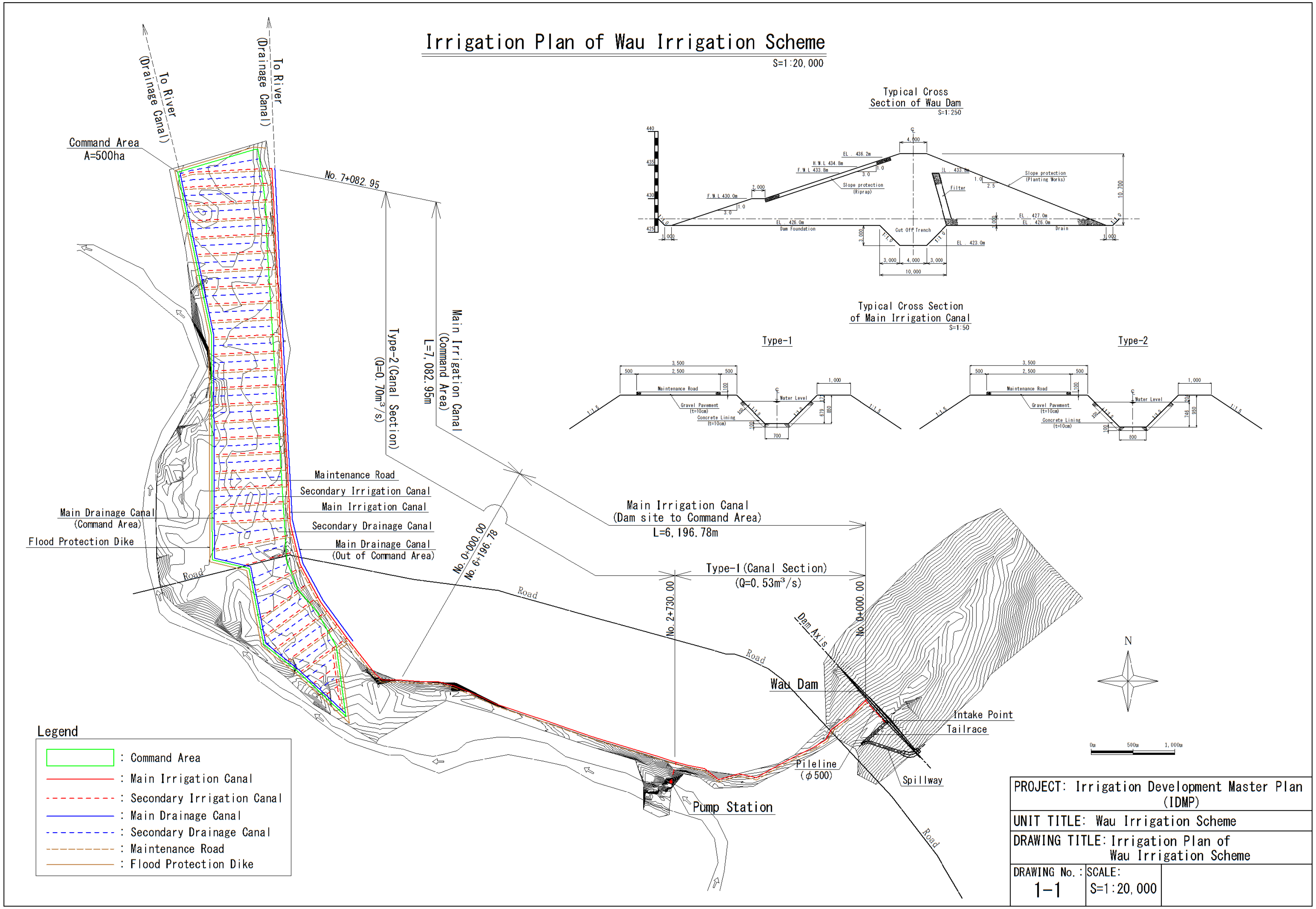


Figure1.2.2 Location of Map

CHAPTER 2 DAM

Specifications of Dam at Wau

Facility	Items	Specification	Remarks
General	Location	7 km upstream from Wau	
	River name	Tributary of Swe River	
	Foundation geology	River Gravel & Sand Base Rock: Gneiss (Granite?)	
	Purpose	Irrigation	
Reservoir	Catchment area	51 km^2	
	Average annual inflow	5,340,000 m^3	
	Reservoir area at FWL	1.8 km^2	
	Total storage capacity	5,300,000 m^3	
	High water level	H.W.L. 434.8 m	
	Full water level	F.W.L. 433.8 m	
	Minimum operation water level	L.W.L. 427.0 m	
	Available depth	7.8 m	
Dam	Dam type	Fill type	
	Dam height	10.7 m	
	Dam length	1,500 m	
	Dam crest width	4 m	
	Dam crest level	E.L. 436.7 m	
	Foundation treatment	Cutoff	
	Dam volume	270,000 m^3	
Spillway	Spillway type	Weir type	
	Dissipater type	End-sill	
	Discharge (200 year return period)	108 m^3/sec	
	Weir crest water depth	1.0 m	
	Weir crest length	60.0 m	
Intake	Intake type	Drop inlet	
	Emergency Outlet discharge	Maximum 7.7 m^3/sec	
	Irrigation Intake	0.53 m^3/sec	
	Penstock	ϕ1100	
	Irrigation	ϕ700	
	River maintaining	ϕ500	
Diversion	Type	Half closure of river	
	Construction	Dry season work	

2.1 General

2.1.1 Location of Wau

Wau is a city in northwestern South Sudan, on the western bank of the Jur River, in Wau County, Western Bahr el Ghazal State. It lies approximately 650 km, northwest of Juba, the capital and largest city in that country.

Figure 2.1.1 Location Map

Wau was initially established as a zariba (fortified base) by slave-traders in the 19th century. During the time of condominium rule, the city became an administrative center.

The city of Wau is the headquarters of Wau County. It also serves as the capital of Western Bahr el Ghazal State, one of the ten (10) states which constitute the Republic of South Sudan. The city is a culturally, ethnically and linguistically diverse urban center. Its residents include peoples of Fertit, Dinka, Luo and Arab ethnicity.

In 2008, Wau was the third-largest city in South Sudan, by population, behind Juba the capital and Malakal, in Upper Nile State. At that time, the estimated population of the city of Wau was about 128,100. In2011, the city's population was estimated at about 151, 320.

Year	Population
1973	52,800
1983	58,000
1993	84,000
2010	128,100
2011	151,320

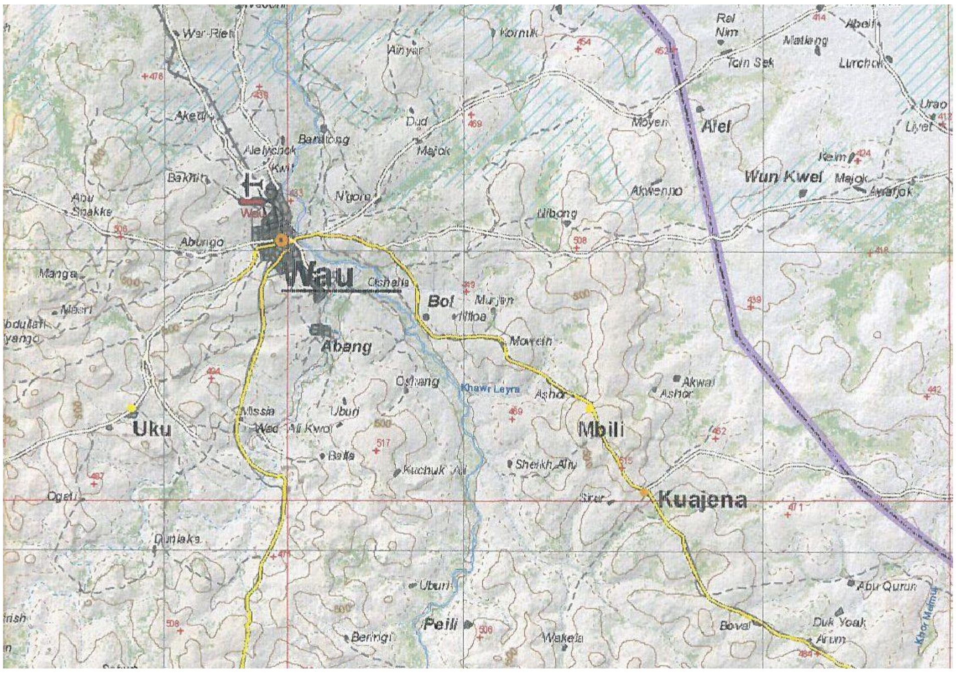


Figure 2.1.2 Location Map at Wau

Wau has two seasons: a dry season from November to March and a rainy season of the rest of the yea, as depicted in the reference table below:

Table 2.1.1 Climate Data for Wau in South Sudan

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Record High ℃	41.1	42.2	43.5	42.0	41.5	38.5	36.5	36.7	40.0	39.2	38.5	39.5	43.5
Average High ℃	35.5	37.1	38.1	37.7	35.3	32.9	31.4	31.4	32.6	33.8	35.2	35.2	34.7
Daily mean ℃	26.8	28.5	30.4	30.6	29.3	27.5	26.3	26.2	26.8	27.4	27.4	26.5	27.8
Average Low ℃	19.1	19.9	22.7	23.8	23.2	22.0	21.2	21.0	21.0	21.0	19.6	17.9	20.9
Record Low ℃	9.3	12.5	14.9	16.5	19.5	17.7	18.0	18.6	17.0	16.4	11.4	10.3	9.3
Precipitation mm	1.3	3.6	18.6	68.3	118.8	177.4	176.0	192.3	179.4	123.8	14.9	0.1	1,074.5
Average Precipitation days (≧0.1mm)	0.2	0.3	3.4	6.3	11.4	12.7	15.9	15.5	23.7	11.2	1.7	0.1	102.4
Humidity %	29.0	26.0	35.0	48.0	62.0	71.0	76.0	77.0	74.0	69.0	48.0	35.0	54.0
Mean Monthly Sunshine Hours	288.3	246.4	229.4	228.0	220.1	204.0	182.9	192.2	204.0	223.2	264.0	294.5	2,777.0
Mean Daily Sunshaine Hours	9.3	8.8	7.4	7.6	7.1	6.8	5.9	6.2	6.8	7.2	8.8	9.5	7.6
Percent Possible Sunsyine	79	74	62	61	60	54	47	50	56	60	75	82	63

2.1.2 Outline of Irrigation Plan at Wau

The Wau Irrigation system consists of the reservoir, the main canal and the irrigation area. The water resource is planed the small scale reservoir which the height is 10.7 m and the dam length is about 1,500 m. The reservoir has approximate 5.3 million m^3 of the capacity and 51 km^2 of the catchment area.

The main canal between the reservoir and the irrigation area is laid along the Jur River and the total length is about 12 km through the irrigation area.

The width of the irrigation area is 500 ha and is located the east of the Jur river and Wau city.

The map of the irrigation area is shown at Figure 2.2.1.

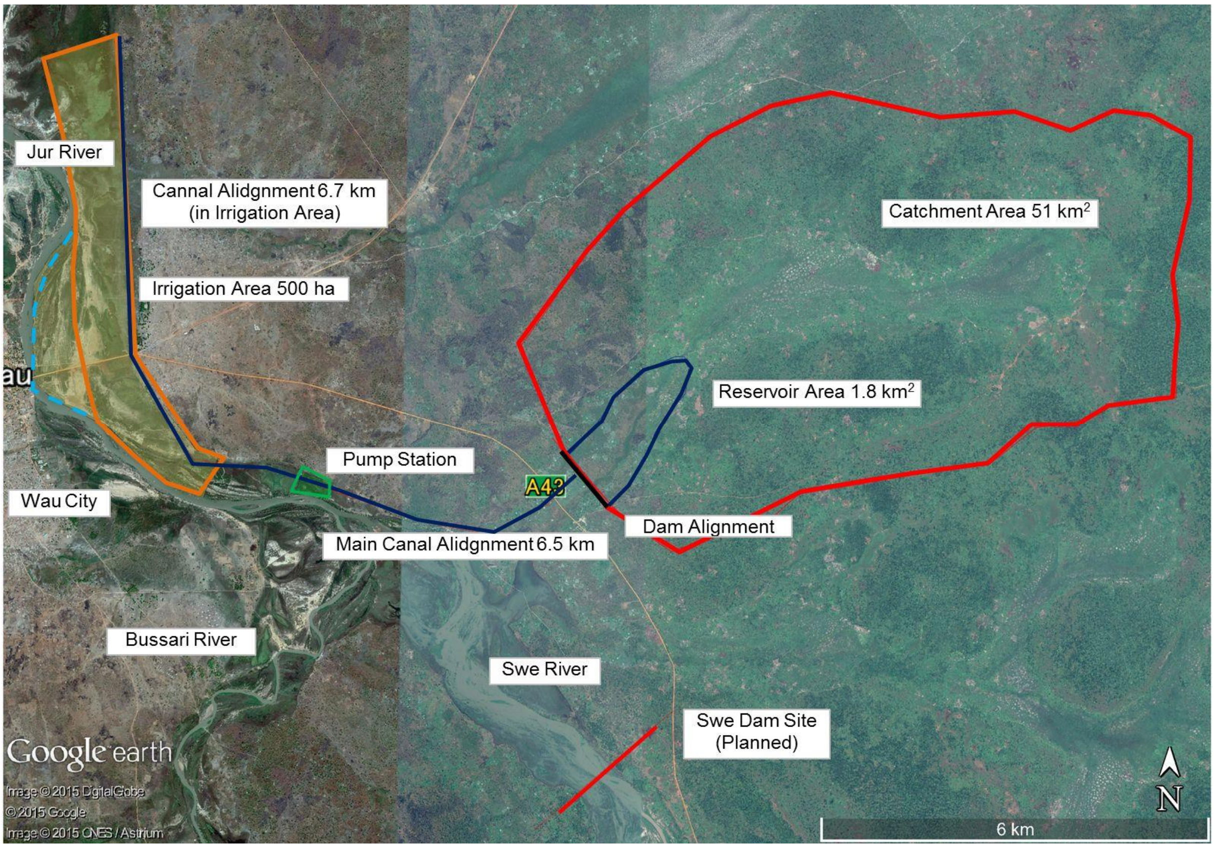


Figure 2.1.3 Location Map

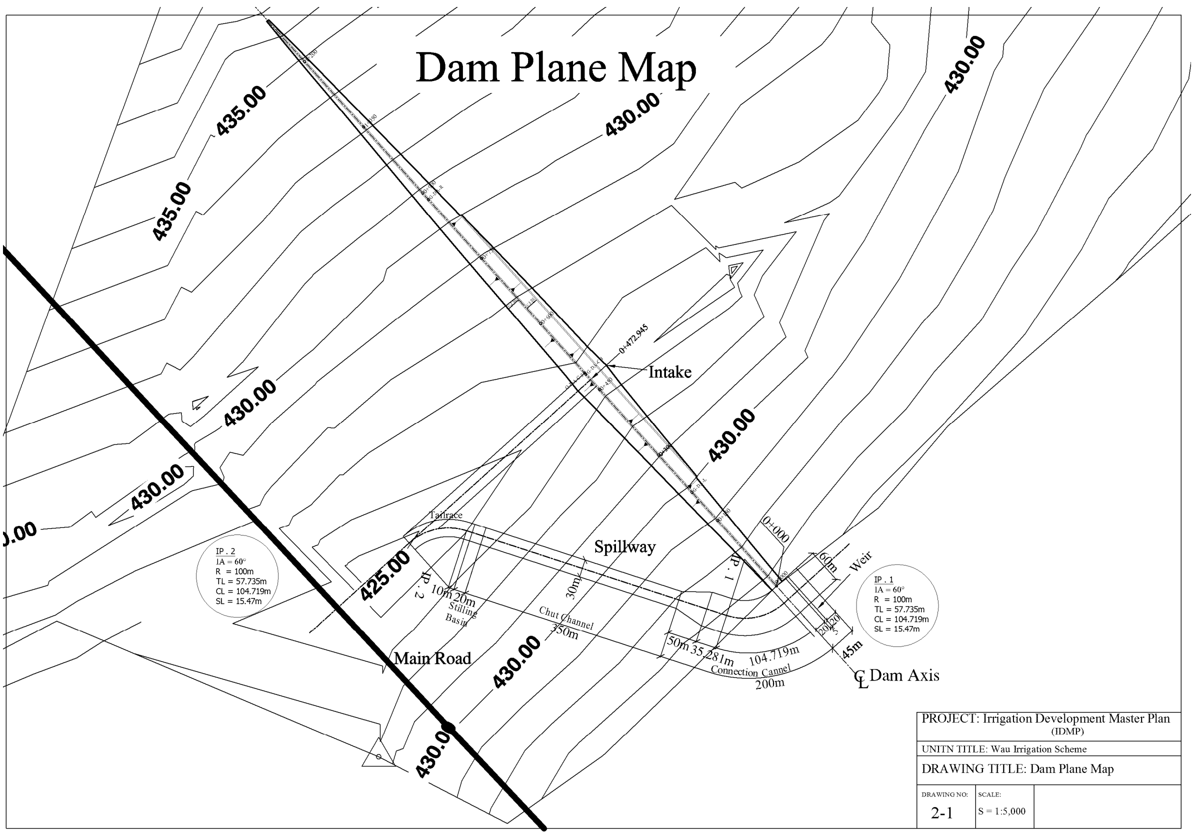


Figure 2.1.4 Dam Plane Map

2.2 Geology

2.2.1 General Geology

Geological condition of South Sudan is rather simple, especially its surface geology is consisted of only two major units basically; Basement Complex of mainly Pre-Cambrian age including several intrusive rock bodies from Pre-Cambrian to Tertiary, and some unconsolidated sediments filling up the vast Sudd Basin.

Pre-Cambrian Basement complex associated with some young intrusive rocks expose in the southwest, south, southeast and east to northeast hedge of the country just surrounding the Sudd Basin occupying around one thirds of the territory. Basement Complex is consisted of mainly "Granitic Gneiss", normally massive and hard. However, the Granites form weathered zone on its surface, and regular joints and fissures inside. Intrusive rocks associated with the basement are mainly Basalt, very hard and impervious. Unconsolidated sediments occupy remaining two thirds of the country area, and the Nile run through the basin from south to north. The sediments are classified into two formations; old sediments formed through Tertiary to Quaternary, and young one of recent. The old sediments are called as "Umm Ruwaba Formation", the most famous aquifer in South Sudan.

Thus geological setting of South Sudan is summarized as Table 2.2.1 shown below and a geological map of South Sudan in Figure 2.2.1.

Table 2.2.1 Geological Setting of South Sudan

Era	Period	Common Name in Africa	Local Name
Ceozoic	Quaternary	Alluvium	Alluvium
	Tertiary	Continental Terminal	Umm Ruwaba Formation
Mesizoic Paleozoic		Continental Intercalary	Nubian Sandstone
Proterozoic	Pre-Cambrian	Basement Complex	Basement Complex

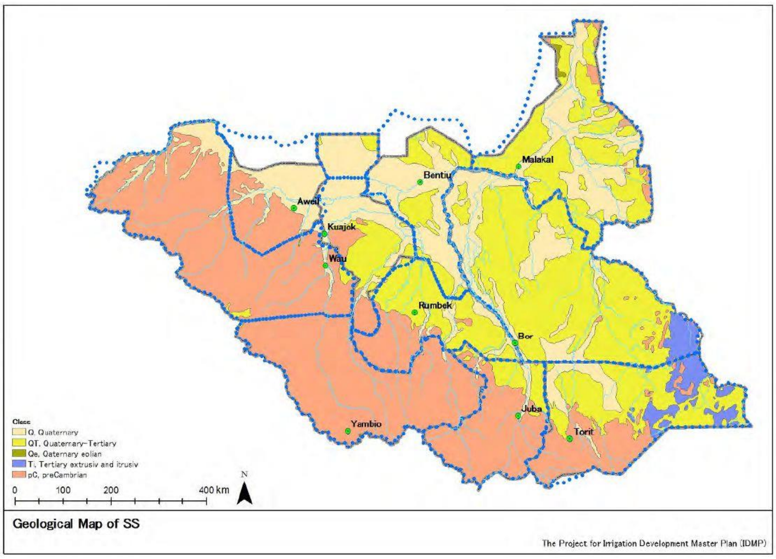


Figure 2.2.1 Geological Map

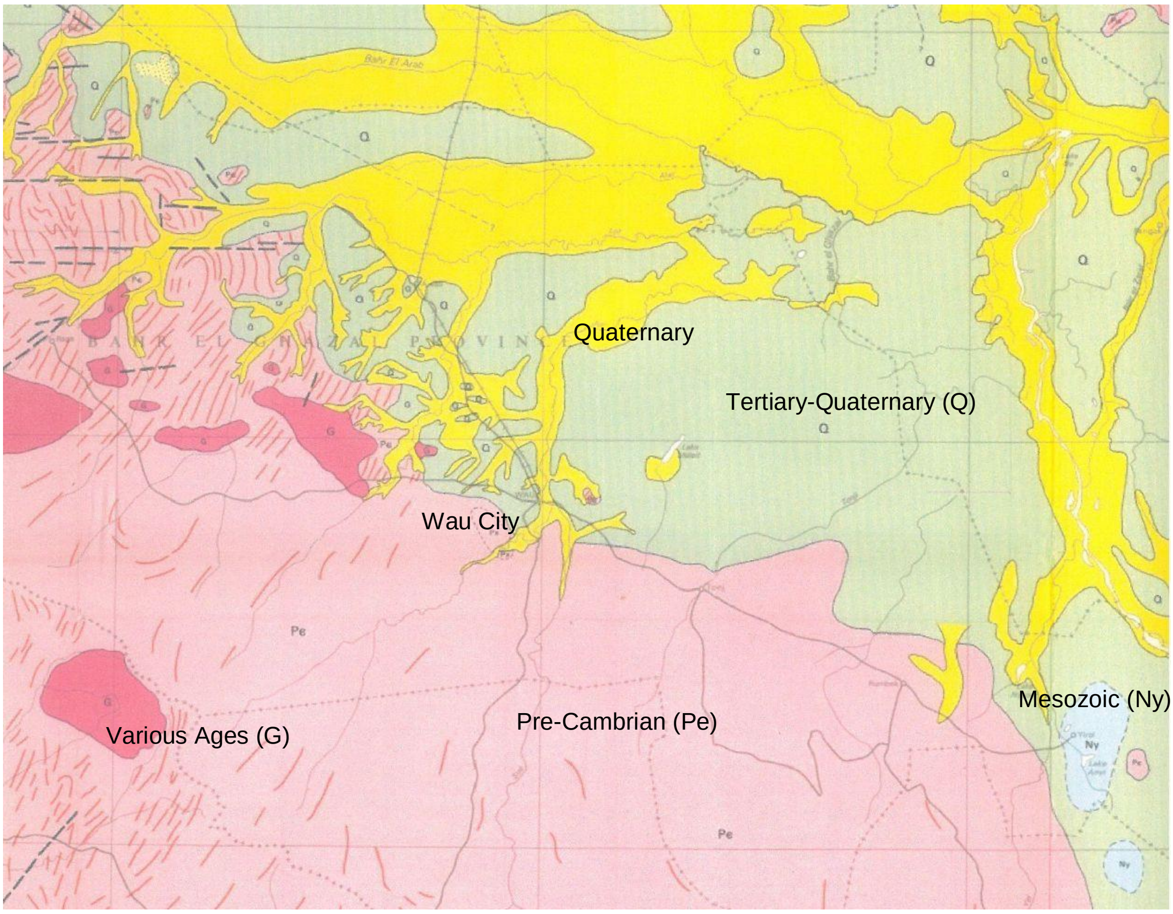


Figure 2.2.2 Geological Map

*Quanternary: Alluvium, wadi fill and swamp deposits

Tertiary-Quanternary (Q): Unconsolidated superficial sediments mainly sands, gravels and clays; precise age uncertain

Mesozoic (Ny) (Lower? Cretaceous): Continental clastic sediments including sandstones, mudstones, pebbles beds and conglomerates

Pre-Cambrian (Pe): Basement Complex Undifferentiated

Various Ages (G): Granite, mainly late or post-tectonic

2.2.2 Site Geology

The dam project area is covered by the sedimentary layer, the silty clay and the gravel sand which are thick layer and their thicknesses are 8m ~ 10 m, and the maximum thickness is 14 m at the river portion of the dam site (Borehole No. G-DA-C (2)). The foundation of the base rock is gneiss but the depth of the layer is deeper than 10 m.

The N value of the sedimentary layer is about 30 at the depths of 3m ~ 4m and the permeability is less than $K = 5 \times 10^{-5}$ cm/sec. The layer of the silty clay and the gravel sand is firm for the low dam as the 10 m height and impervious for the dam foundation.

2.2.3 Boreholes at Dam Site

There are three borings at the dam alignment, the right abutment (10 m in depth), the river portion (11 m and 14 m in depth) and the left abutment (10 m in depth) as Table 2.3.1:

Table 2.2.2 Boreholes at Dam Site

Borehole	Coordinates		Drilling Depth (m)	Dam Axis
	Northing	Easting		
G-DA-L	7° 40′ 48″	28° 5′ 24″	10.0	0+215
G-DA-C (1)	7° 40′ 59″	28° 5′ 13″	11.0	0+485
G-DA-C (2)			14.0	0+488
G-DA-R	7° 41′ 13″	28° 5′ 4″	10.0	0+885

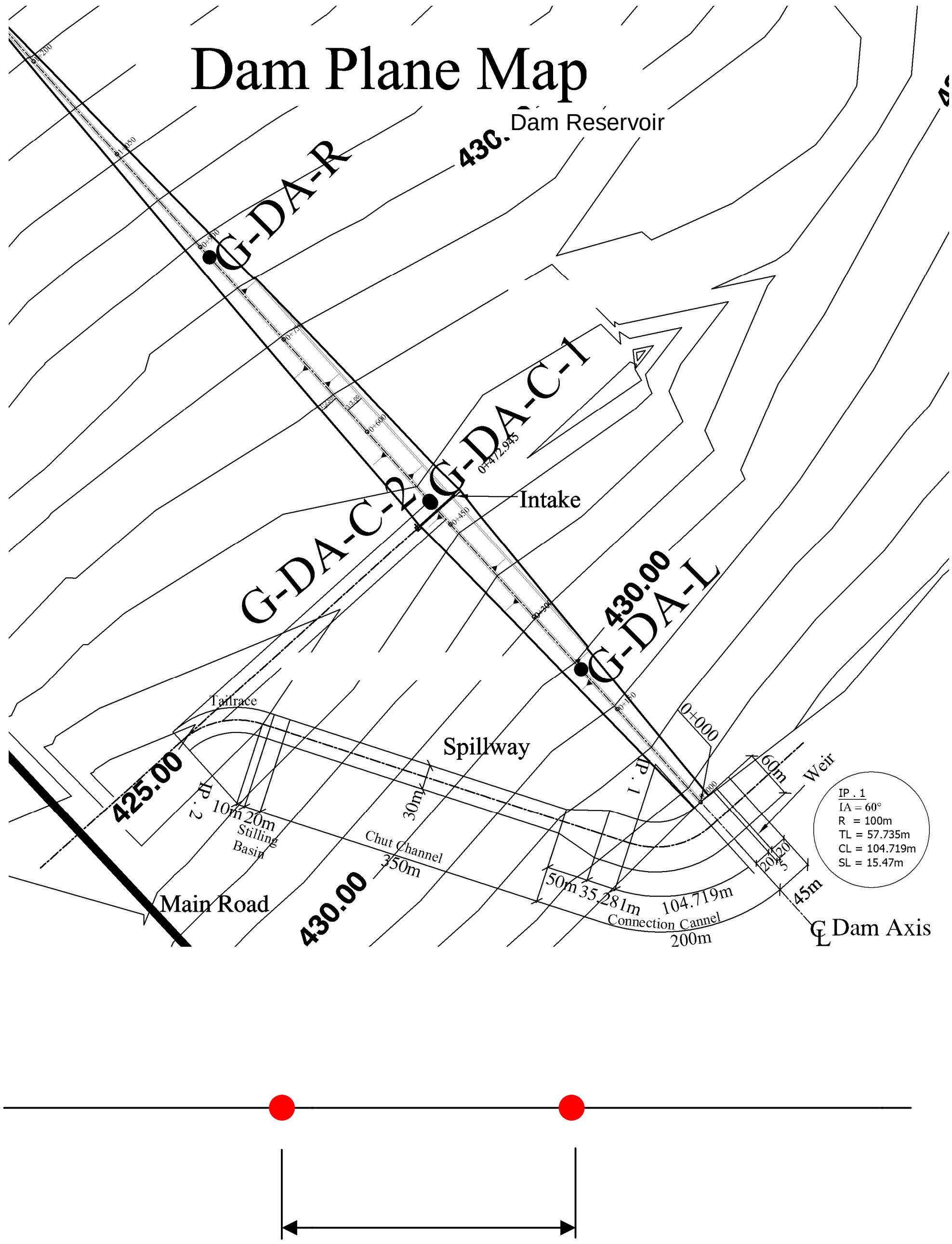


Figure 2.2.3 Location of Borehole

LOG OF BORING

BRIGHT STAR ENGINEERING AND CONSULTING CO. LTD

PROJECT: PROPOSED IRRIGATION DEVELOPMENT MASTER PLAN (IDMP)

CLIENT : M/S: JICA	BORING NO.: G-DA- L	Report No.: ST - 0001500009 Rev.0
LOCATION: WAU - EAST LAND - SOUTH SUDAN	SHEET NO.: 1 of 2	
	GROUND ELEV : 443.4m	
COORDINATES: N: 7° 40' 48" E: 28° 05' 24.0"	GROUND WATER DEPTH (m): 5.57	
DRILLING DATE: 4/21/2015 TO 4/22/2015	TOTAL BORING DEPTH (m):10.0	Orientation: Vertical
BORING METHOD: rotary with mud	BORING DIAMETER: 100mm	
BORING EQUIPMENT: Mobile B31	CASING DIAMETER:	

Depth (m)	R.L. (m)	S.T.	TCR (%)	SCR (%)	RQD (%)	SPT (N)	Legend	Description	U.C.S (kg/cm²)	D.D. (gm/cm³)
0.0–1.0	-1.5					>50		Dense, brown, sandy GRAVEL, GRAVEL (literate.		
2.0	-2.5					8		Medium dense, brownish gray, gravely cayey SILT. (Permeability test) K= 5.42E-05 cm/sec		
3.0	-3.5					>50		Very dense, yellowish brown, silty gravely CLAY, slightly plastic.		
4.0	-4.5					>50		Very dense, yellowish brown, silty gravely CLAY, slightly plastic.		
5.0	-5.5					25		Medium dense, yellowish brown, gravely silty SAND.		
6.0	-6.5							Permeability test K= 1.19E-05 cm/sec		
7.0	-7.5					28		Medium dense, yellowish brown, clayey sandy SILT, low plastic.		
8.0	-8.5							Permeability test K= 6.76E-08 cm/sec		
9.0	-9.5					23		Medium dense, dark brown SILT with some gravel		
10.0								END OF BORING (4.5m)		

1- S.T. : Sampler Type
2- TCR : Total Core Recovery
3- SCR : Solid Core Recovery
4- RQD : Rock Quality Designation
5- SPT : Std. Penetration Test
6- U.C.S. : Unconfined Compressive Strength
7- D.D. : Dry Density
8- R.L. : Reduced Level

Standard Penetration Test; Tricone Bit; Undisturbed Sample; Rock Coring; Water Table; Auger

REMARKS:

Figure 2.2.4 G-DA-R Boring Log

LOG OF BORING

BRIGHT STAR ENGINEERING AND CONSULTING CO. LTD

PROJECT: PROPOSED IRRIGATION DEVELOPMENT MASTER PLAN (IDMP)

CLIENT : M/S: JICA	BORING NO.: G-DA-R	Report No.: ST - 0001500009 Rev.0
LOCATION: WAU - EAST LAND - SOUTH SUDAN	SHEET NO.: 1 of 2	
	GROUND ELEV. 442.9m	
COORDINATES: N: 7° 40' 59" E: 28° 05' 13.0"	GROUND WATER DEPTH (m): 3.60	
DRILLING DATE: 4/20/2015 TO 4/21/2015	TOTAL BORING DEPTH (m):10.0	Orientation: Vertical
BORING METHOD: rotary with mud	BORING DIAMETER: 100mm	
BORING EQUIPMENT: Mobile B31	CASING DIAMETER:	

Depth (m)	R.L. (m)	S.T.	TCR (%)	SCR (%)	RQD (%)	SPT (N)	Legend	Description	U.C.S (kg/cm²)	D.D. (gm/cm³)
0.0–1.0	-1.5					42		Dense, brownish gray, sandy clayey SILT.		
2.0	-2.5					13		Medium dense, brownish gray, gravely cayey SILT.		
3.0	-3.5							Permeability test K= 5.26E-06 cm/sec		
4.0	-4.5					19		Medium dense, brown, silty GRAVEL.		
5.0	-5.5					>50		Very dense, brown, sandy GRAVEL (literate fragment)		
6.0–8.0	-8.5		50	40	34			Weak to very weak, light brown, sandstone (literati). (Permeability test from 7.0 - 7.45m) permeability test 7.0m K= 3.53E-05 cm/sec		
9.0	-9.5					>50		Very dense, brown, sandy GRAVEL (literate fragment)		
10.0								Permeability test K= 4.64E-06 cm/sec		

1- S.T. : Sampler Type
2- TCR : Total Core Recovery
3- SCR : Solid Core Recovery
4- RQD : Rock Quality Designation
5- SPT : Std. Penetration Test
6- U.C.S. : Unconfined Compressive Strength
7- D.D. : Dry Density
8- R.L. : Reduced Level

Standard Penetration Test; Tricone Bit; Undisturbed Sample; Rock Coring; Water Table; Auger

REMARKS:

Figure 2.2.5 G-DA-L Boring Log

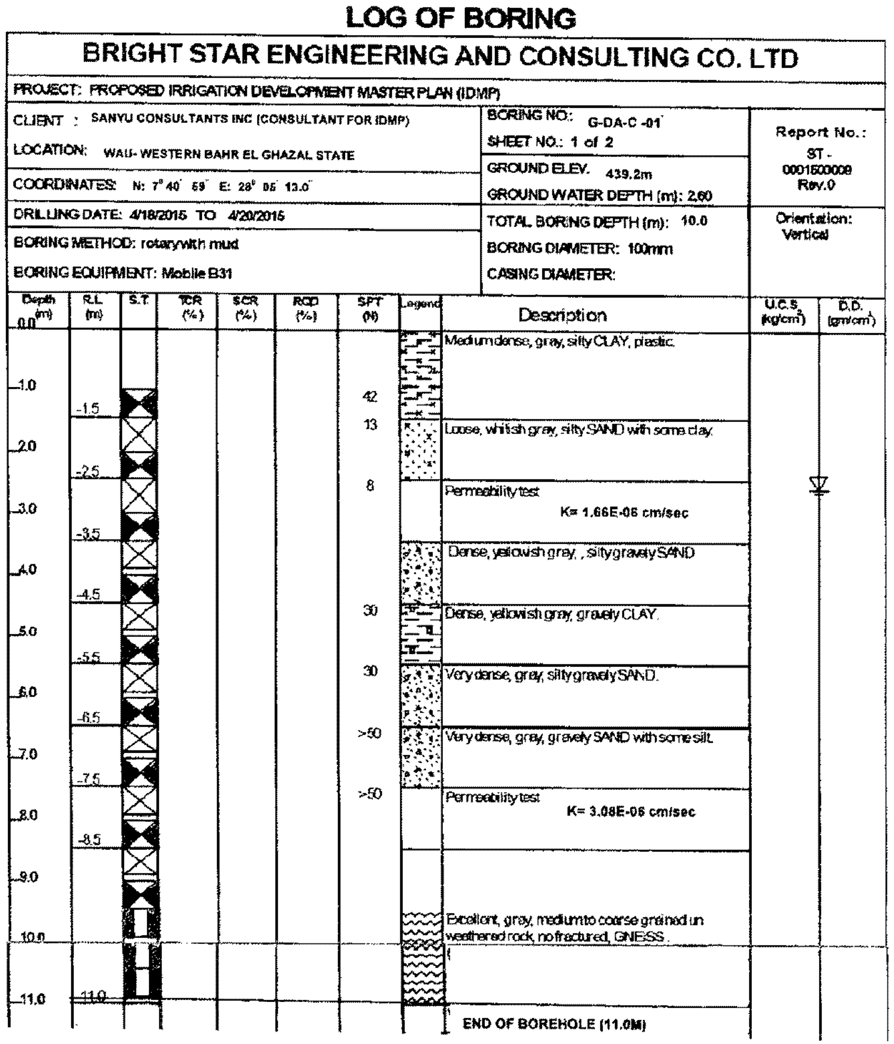


Figure 2.2.6 G-DA-C-01 Boring Log

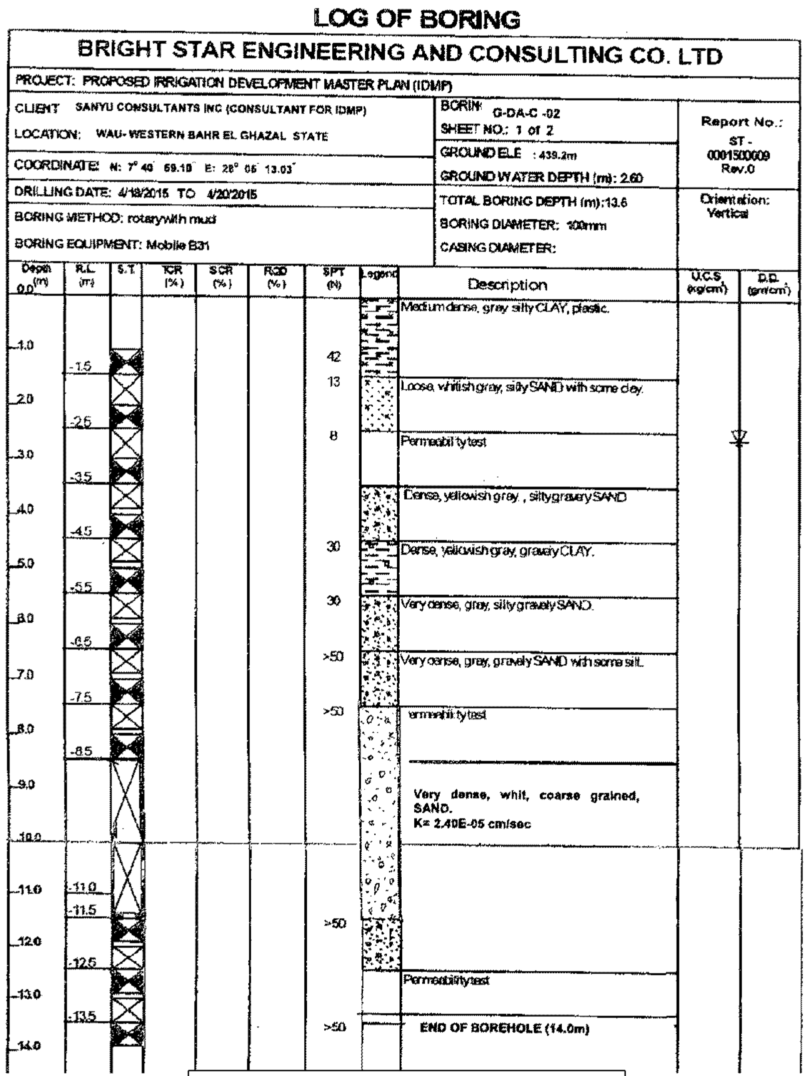
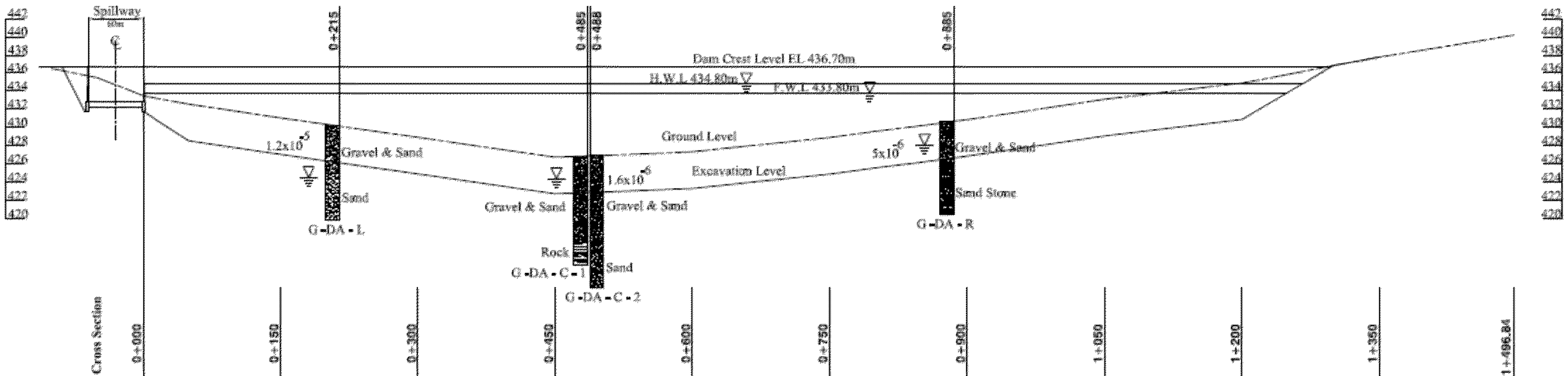


Figure 2.2.7 G-DA-C-02 Boring Log

Figure 2.2.8 Longitudinal Section of Wau Dam



STATION NO.	DAM CREST LEVEL	EXCAVATION LEVEL	GROUND LEVEL
-0+100			436.50
-0+050			435.50
0+000	436.70	431.80	433.50
0+050	436.70	429.58	432.58
0+100	436.70	427.83	431.83
0+150	436.70	427.07	431.07
0+200	436.70	426.37	430.37
0+250	436.70	425.66	429.66
0+300	436.70	424.96	428.96
0+350	436.70	424.23	428.23
0+400	436.70	423.51	427.51
0+450	436.70	422.79	426.79
0+500	436.70	422.96	426.96
0+550	436.70	423.16	427.16
0+600	436.70	423.37	427.37
0+650	436.70	423.89	427.89
0+700	436.70	424.41	428.41
0+750	436.70	424.93	428.93
0+800	436.70	425.61	429.61
0+850	436.70	426.23	430.23
0+900	436.70	426.89	430.89
0+950	436.70	427.62	431.62
1+000	436.70	428.36	432.36
1+050	436.70	429.10	433.10
1+100	436.70	429.71	433.71
1+150	436.70	430.30	434.30
1+200	436.70	430.90	434.90
1+250	436.70	433.85	435.65
1+300	436.70	436.82	436.82
1+350		437.78	437.78
1+400			438.59
1+450			439.40
1+496.84			440.20
1+550			

Vertical Scale:
0 2 4 6 8 10 20

Horizontal Scale:
0 50 100 200

Scale 1:50

2.3 Dam Capacity

2.3.1 Dam Alignment

Wau is a city in northwestern South Sudan, on the western bank of the Jur River, in Wau County, Western Bahr el Ghazal State. It lies approximately 650 km, northwest of Juba, the capital and largest city in that country.

ADMINISTRATIVE MAP OF THE REPUBLIC OF SOUTH SUDAN

Figure 2.3.1 Location Map

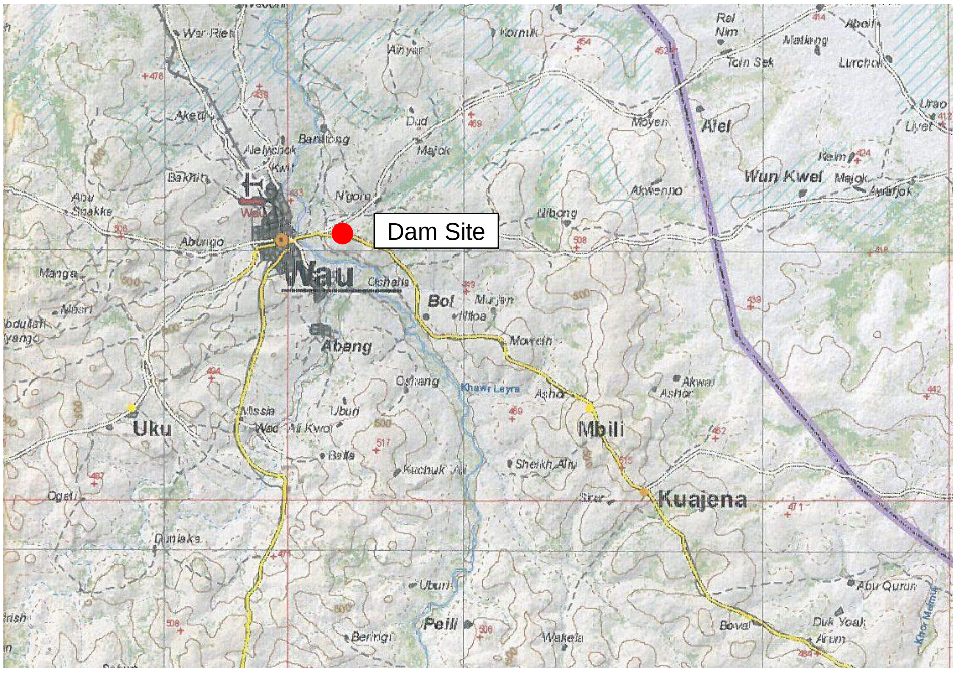


Figure 2.3.2 Location Map at Wau

The dam site is located about 7 km east to the Wau city and the topography of the dam site is the gentle slope of the hill. The dam alignment is laid out on 600 m east of the main road (A43). There is no name of the river at the dam site and it is the tributary of the Swe River. Freshwater marshes spread around the upstream side of the river. The catchment area of the river is 51 km^2 and the reservoir area will be 1.8 km^2.

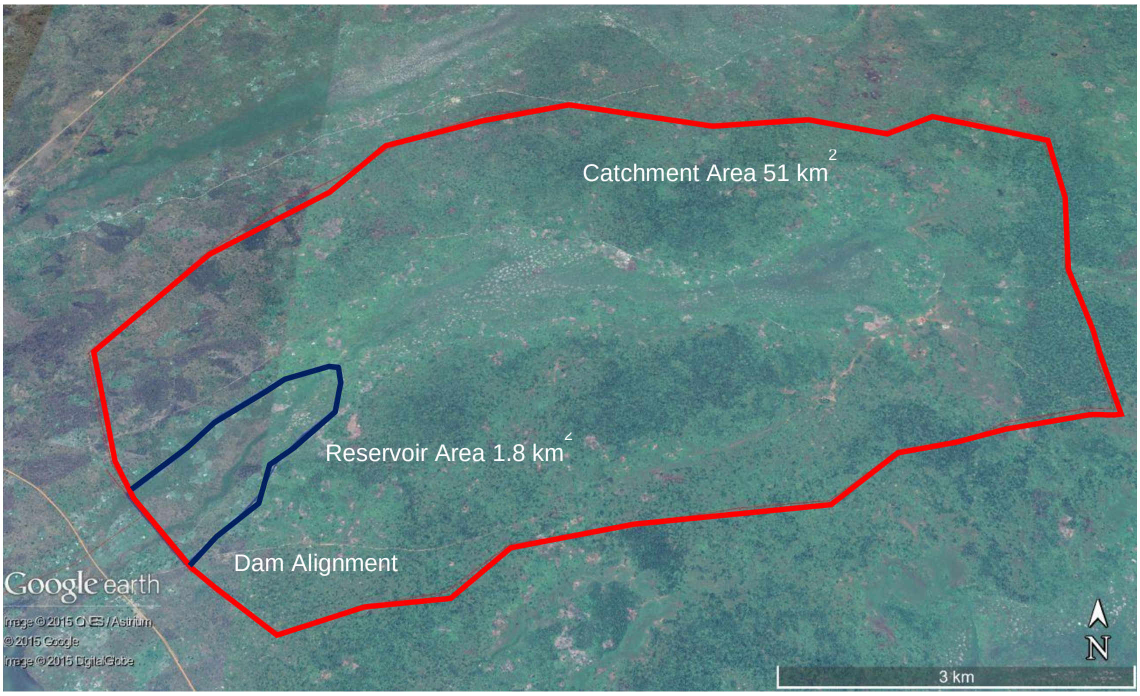


Figure 2.3.3 Dam Location

2.3.2 Dam Capacity

The dam capacity of the Wau is decided by the required water of the irrigation area and the main crops of the irrigation area are planed of paddy field, designed by rice scheme area.

Required Irrigation Water

Irrigation water requirement is obtained through taking into account the characteristics of weather condition and soil moisture on the beneficiary area and consumption of the target crops. Software named "CROPWAT" which was produced by FAO is generally used for this calculation. This software can consider meteorological conditions or actually cultivated or planned cropping pattern. Besides this can apply for the case that actual irrigated area and planned irrigated area is different because this can calculate the water requirement per unit area.

The required irrigation water for the irrigation (500 ha) of dry season is calculated as followed:

Required Irrigation water (500 ha): 4,900,000 m^3 (dry season)
0.53 m^3 / sec (Maximum required water)

Annual Inflow

Average amount of annual specific yield for last 30 years (SY_{30}) is calculated by the following formula.

$$SY_{30} = Q_{30} / A \times 1{,}000$$

SY_{30}: Average annual specific yield for last 30 years (mm/year)

Q_{30}: Average annual river discharge for last 30 years at the exit of the catchment area (MCM/year)

A: Catchment Area (km^2)

Table 2.3.1 Water Requirement of Wau

Water Requirement: Wau

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
			Dry Season					Rainy Season					Dry Season	
(1) ETcrop														
Min Temperature	(°C)	19.0	19.9	22.6	24.0	22.7	22.2	22.4	21.8	21.7	21.6	21.1	19.3	21.5
Max Temperature	(°C)	34.7	36.7	38.2	37.3	34.8	32.7	31.6	31.8	32.5	34.4	35.9	36.0	34.7
Relative Humidity	(%)	31	18	15	37	56	74	72	78	89	64	39	43	51.3
Wind speed	(km/day)	72	69	78	112	104	95	95	69	78	78	78	75	83.5
Sunshine	(hours)	11.7	11.8	12.0	12.3	12.5	12.6	12.5	12.4	12.1	11.9	11.7	11.6	12.1
Radiation	(MJ/m2/day)	24.9	26.5	27.9	28.5	27.9	27.4	27.5	28.1	27.9	26.8	25.1	24.2	26.9
ETo	(mm/day)	4.88	5.09	5.72	6.74	6.21	5.67	5.59	5.60	5.58	5.55	5.25	5.00	CropWar8.0
Crop 1		Jew's mallow					Rice	Rice	Rice			Jew's mallow (leafy veg		
Crop 2		Egg plant (fruit vege.)					Rice	Rice	Rice			Egg plant (fruit vege.)		
Crop coeffient 1	Kc1	1.10	0.53	0.00	0.00	0.55	1.10	1.10	1.10	0.51	0.00	0.93	1.03	
Crop coeffient 2	Kc2	1.04	1.03	0.00	0.00	0.55	1.10	1.10	1.10	0.51	0.00	0.90	0.96	
Etcrop 1 (ET_0 x Kc1)	(mm/day)	5.37	2.70	0.00	0.00	3.42	6.24	6.15	6.16	2.85	0.00	4.88	5.15	
Etcrop 2 (ET_0 x Kc2)	(mm/day)	5.08	5.24	0.00	0.00	3.42	6.24	6.15	6.16	2.85	0.00	4.73	4.80	
(2) Effective Rainfall (Pe)														
Monthly Mean Rainfall	(mm/month)	0.7	3.4	19.4	65.8	127.8	168.1	192.8	212.0	170.7	123.3	13.4	0.3	1,097.7
Dependable Rainfall (80%)	(mm/month)	0.6	2.9	16.8	57.0	110.7	145.6	166.9	183.6	147.8	106.8	11.6	0.3	950.6
Effective Rainfall (ER)	(mm/month)	0.0	0.0	0.0	24.0	65.0	92.0	110.0	123.0	94.0	61.0	0.0	0.0	569.0
Effective Rainfall (ER)	(mm/day)	0.0	0.0	0.0	0.8	2.2	3.1	3.7	4.1	3.1	2.0	0.0	0.0	
(3) Groundwater Contribution (Ge)	(mm/day)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
(4) Stored Soil Water (Wb)	(mm/day)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
(5) Etcrop - (Pe+Ge+Wb)														
Etcrop 1	(mm/day)	5.37	2.70	0.00	0.00	1.22	3.14	2.45	2.06	0.00	0.00	4.88	5.15	
Etcrop 2	(mm/day)	5.08	5.24	0.00	0.00	1.22	3.14	2.45	2.06	0.00	0.00	4.73	4.80	
(6) Total Efficiency														
Conveyance Efficiency	Ec	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	
Field Canal Efficiency	Eb	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	Lining canal
Field Application Efficiency	Ea	0.70	0.70	0.70	0.70	0.32	0.32	0.32	0.32	0.32	0.70	0.70	0.70	
Total Irrigation Efficiency	Ep	0.57	0.57	0.57	0.57	0.26	0.26	0.26	0.26	0.26	0.57	0.57	0.57	
(7) Irrigation Hour	(hour)	24	24	24	24	24	24	24	24	24	24	24	24	
(8) Unit Water Requirement														
Crop 1	(l/s/ha)	1.09	0.55	0.00	0.00	0.54	1.40	1.09	0.92	0.00	0.00	0.99	1.05	
Crop 2	(l/s/ha)	1.03	1.06	0.00	0.00	0.54	1.40	1.09	0.92	0.00	0.00	0.96	0.97	
(9) Command Area														
Crop 1	(ha)	250	250	250	0	250	250	250	250	250	0	250	250	
Crop 2	(ha)	250	250	250	0	250	250	250	250	250	0	250	250	
Total	(ha)	500	500	500	0	500	500	500	500	500	0	500	500	
(10) Water Requirement for Pump														
Crop 1	(m3/s/Crop1)	0.27	0.14	0.00	0.00	0.14	0.35	0.27	0.23	0.00	0.00	0.25	0.26	
Crop 2	(m3/s/Crop2)	0.26	0.27	0.00	0.00	0.14	0.35	0.27	0.23	0.00	0.00	0.24	0.24	
Total	(m3/s/Total)	0.53	0.40	0.00	0.00	0.27	0.70	0.55	0.46	0.00	0.00	0.49	0.51	
(11) Water Requirement for Reservoir	(m3/month/Total)	1,373,760	1,043,280	0	0	699,840	1,814,400	1,412,640	1,192,320	0	0	1,263,600	1,308,960	10,108,800
							Required Pump Capacity (from May to Sep)					0.70 (m3/s)		
							Required Reservoir Capacity (from Nov to Apr)					4,989,600 (m3/year)		

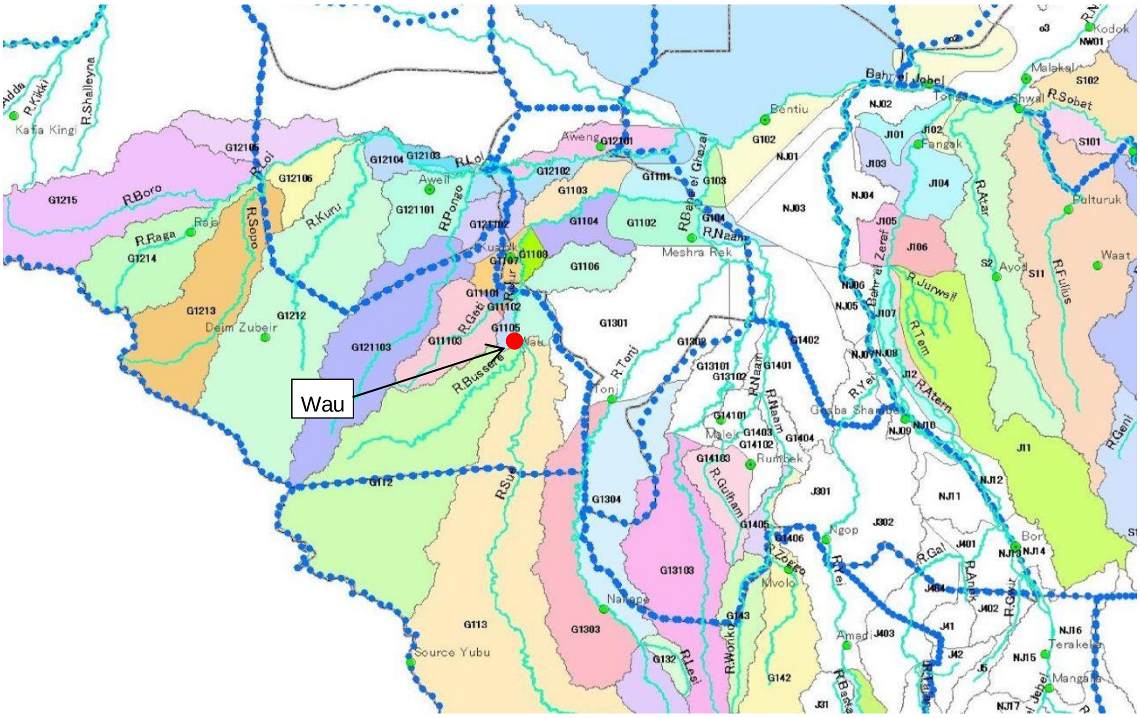


Figure 2.3.4 River Delineation Map

Table 2.3.2 Specific Runoff Yield

SN	CA code	Area (km2)	Runoff Discharge Qm for 30 years (MCM)	Specific Runoff Yield: SY (mm)	Flow Ratio (F)	Discharge Station No.	Remarks
1	G101	70,031	105.75	1.51		-	
2	G102	2,962	4.47	1.51		-	
3	G103	611	0.92	1.51		-	
4	G104	1,022	1.54	1.51		-	
5	G1101	2,214	16.30	7.36		-	
6	G1102	2,450	18.03	7.36		-	
7	G1103	2,104	15.49	7.36		-	
8	G1104	1,981	14.58	7.36		-	
9	G1105	509	53.26	104.6	0.080	40	Wau
10	G1106	3,454	361.33	104.6	0.080	40	
11	G1107	1,314	97.76	74.4		-	
12	G1108	531	39.51	74.4		-	
13	G11101	171	12.70	74.4		-	
14	G11102	79	5.85	74.4		-	
15	G11103	3,750	279.0	74.4	0.021	34	
16	G112	19,391	1,766.54	91.1	0.072	46	Busari River
17	G113	33,381	3,244.66	97.2	0.071	48	Swe River

And then,

$Q_{30} = SY_{30} \times A \times 1,000$

The Wau dam site is situated at G1105 (Wau) and G113 (Swe River) and the values of the Specific Runoff Yield (SY_{30}) are 104.6 and 97.2 respectively. The annual river discharges (Q_{30}) are calculated with the annual specific yield (SY30) as follows:

G1105 (Wau): $Q_{30} = SY_{30} \times A \times 1,000$ (A = 51 km^2: catchment area)

$= 104.6 \times 51 \times 1,000$

$= 5,334,600$ m^3 > 4,900,000 m^3 (Required Irrigation Water)

G113 (Swe River): $Q_{30} = SY_{30} \times A \times 1,000$ (A = 51 km^2: catchment area)

$= 97.2 \times 51 \times 1,000$

$= 4,957,200$ m^3 < 4,900,000 m^3 (Required Irrigation Water)

The annual river discharges (Q_{30}) with the Specific Runoff Yield (SY_{30}) are more than the required irrigation water at the Wau dam site and the storage capacity of the Wau dam is decided as 5.3 MCM (million cubic meter).

2.3.2 Dam Capacity

The dam capacity is decided by the relation curve (H ~ Q curve) between the dam height and the dam quantity at the dam site. The Full Water Level (F.W.L.) is EL. 433.8 m, because the dam capacity is planned as 5,300,000 m^3 (See Figure 2.3.5).

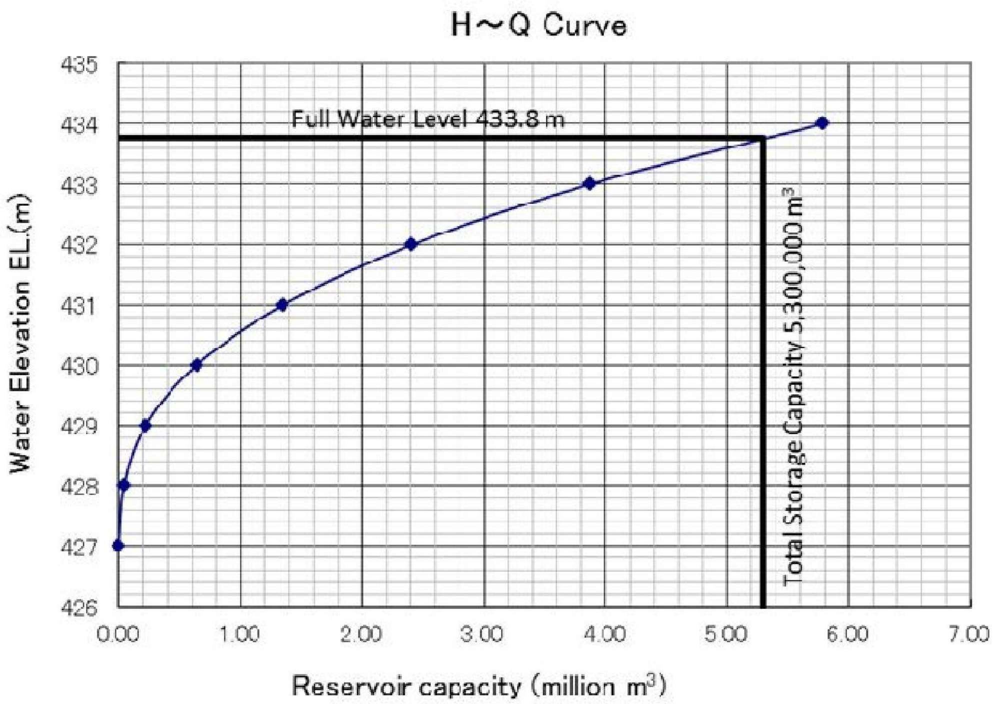


Figure 2.3.5 H ~ Q Curve

The surface area at Full Water Level is 1.8 km^2 based on the H ~ A curve.

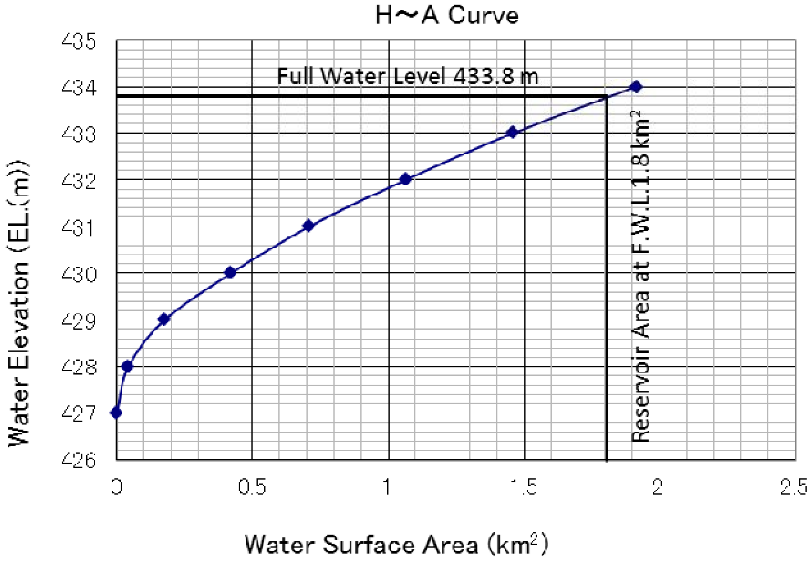


Figure 2.3.6 H ~ A Curve

Table 2.3.3 Reservoir Area and Volume

Water level (m)	Area (m^2)	Area (mean) (m^2)	Depth (m)	Volume (m^3)	Accumulative volume (m^3)
427.0	33,200	0			0
428.0	51,000	42,100	1.0	42,100	42,100
429.0	300,000	175,500	1.0	175,500	217,600
430.0	540,000	420,000	1.0	420,000	637,600
431.0	877,000	708,500	1.0	708,500	1,346,100
432.0	1,250,000	1,063,500	1.0	1,063,500	2,409,600
433.0	1,670,000	1,460,000	1.0	1,460,000	3,869,600
434.0	2,160,000	1,915,000	1.0	1,915,000	5,784,600

2.3.3 Flood Flow Analysis

The design flood discharge should be determined on the basis of hydrometeorology surveys and analyses. The design flood level is defined as the maximum reservoir water level when the design flood discharge occurs.

The design flood discharge is the flood discharge designated for the purpose of securing the safety of the dam and is obtained by adding 20 percent to the maximum values among the followings:

(A) 200 - year flood, which statistically occurs once in 200 years, namely the return period of a flood that could occur every 200 years (herein after referred to as "discharge A")

(B) Maximum experienced flood discharge estimated on the basis of flood records or flood mark survey (herein after referred to as "discharge B")

(C) Maximum flood discharge estimated from hydrological or meteorological records obtained from a nearby watershed with hydro-meteorological characteristics similar to those of the subject river (hereinafter referred to as "discharge C")

Needless to say, the design flood discharge is the maximum flood discharge to be considered for dam design. Therefore the maximum flood, from an engineering view point, expected to occur at the given dam site shall be adopted as the design flood discharge. However, as no set method for determining such exists, the large value among the three discharge described above should be adopted.

(1) 200 - Years Flood

There are two methods to estimate the flood discharge which statistically occurs once in 200 years. One is to estimate directly from the frequently analysis of long term records on flood discharge. The other is to estimate indirectly based on the long term records on rainfall and characteristics of flood discharge in the given watershed.

(a) **Estimation of Peak Flood Discharge by Rational Formula:**

$$Q_p = \frac{1}{3.6} \cdot r_e \cdot A$$

where: Q_p: peak flood discharge (m^3/s)

A: catchment area (km^2)

r_e: average effective rainfall intensive in the catchment within the lag time of flood (mm/hr)

An area approximately less than 40 km^2 is recommended for the application of the above formula, since this formula depends on storm scale. However, in some case this formula may also be applicable to catchments whose areas are 100 to 200 km^2 or more, provided surface conditions and rainfall pattern in the catchment seem nearly uniform. The proper estimation of lag time of flood and average effective rainfall intensity of the catchment for such time are essential in order to apply this formula properly. Accordingly, long term records of short duration rainfall at the dam-site or a representative given point, and stream flow records at or near the dam-site are needed.

(b) **Peak Runoff Coefficient**

Peak runoff coefficient f_p is frequently employed in order to estimate effective rainfall intensity (r_e), which is the factor for determining the effective intensity connected with peak flood discharge based on observed rainfall intensity (r) as follows:

$$r_e = f_p \cdot r$$

r : Rainfall intensity of 200 year return period (mm/hr)

It should be noted that, in principle, values of f_p is greatly affected by surface coverage and topographic conditions, and also antecedent precipitation of the catchment.

Table 2.3.4 Peak Runoff Coefficients showed by Mononobe

Topographical condition	Peak runoff coefficient f_p
Steep mountains	0.75~0.90
Tertiary deposit mountains	0.70~0.80
Undulating surface land and lignose	0.50~0.75
Plain field	0.45~0.60
Irrigated paddy field	0.70~0.80
River in mountains	0.75~0.85
Little river in flat land	0.45~0.75
Large river in flat land more than half at catchment area	0.50~0.75

The catchment area of the Wau pond consists of 'Little river in flat land' and the value of the peak runoff coefficient at Wau is 0.361 on November by the runoff analysis. This is reason why the value of the peak runoff coefficient is adopted as 0.4 (≒0.361) which is maximum values at Wau.

Table 2.3.5 Peak Runoff Coefficients at Wau

1	January	0.097
2	February	0.015
3	March	0.002
4	April	0.001
5	May	0.009
6	Jun	0.028
7	July	0.047
8	August	0.067
9	September	0.119
10	October	0.153
11	November	0.361
12	December	0.212

Rainfall Data

The daily rainfall data cannot be obtained from meteorological analysis and these data are got from Internet site by NOAA NCEP CPC FEWS Africa (National Oceanic and Atmospheric Administration, National Centers for Environmental Prediction, Climate Prediction Center, Famine Early Warning

System, Africa/[1]).

These are the satellite data and the estimated precipitation data whose periods are 30 years from 1983 to 2013. Wau pond is located at 28.087 E (longitude), 7.684N (latitude) and the position of the satellite data is 28.1E, 7.7N. We calculate the return period of the rainfall with the estimated precipitation data of the satellite for 30 years by Iwai method.

The daily rainfall data is shown in Figure 2.3.7.

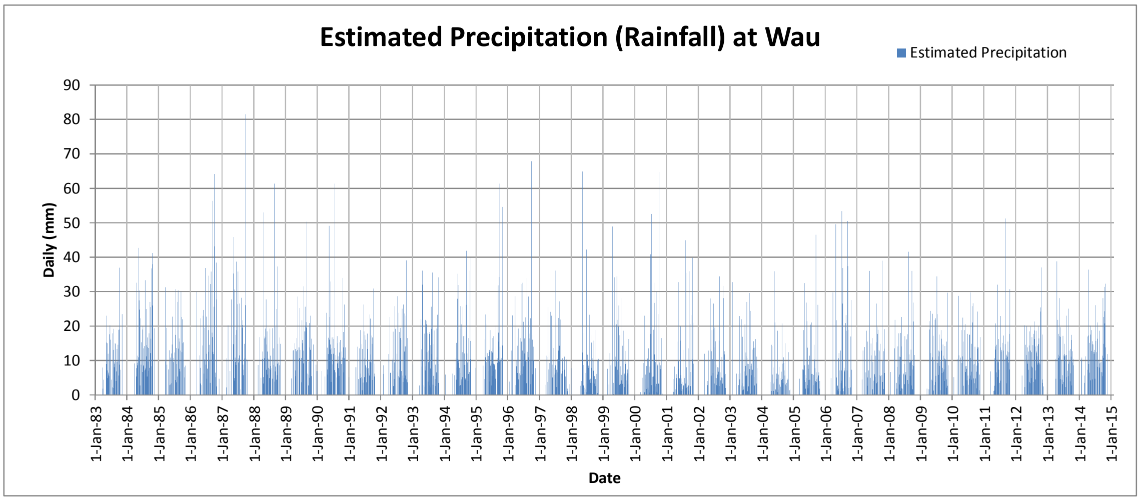


Figure 2.3.7 Rainfall Data

The Iwai method is one of probability of exceedance calculations and return periods are obtained. The statistics method is often used at the probability of the exceedance issues in Japan.

Probability of exceedance: $W(x) = \frac{1}{2} \cdot (1 - F(\xi))$

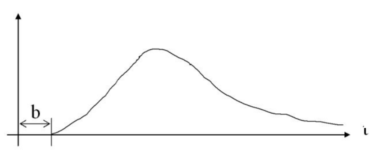
Asymmetrical distribution $f(x) = \exp(-(\alpha \cdot \log \frac{x-b}{x_0 - b})^2)$

Exceedance probability function

$$F(\xi) = \frac{2}{\sqrt{\pi}} \cdot \int_0^{\xi} \exp(-t^2)$$

$$\xi = \alpha \cdot \log \frac{x-b}{x_0 - b}$$

Probability density



Variate

[1] http://iridl.ldeo.columbia.edu/SOURCES/.NOAA/.NCEP/.CPC/.FEWS/.Africa/.DAILY/.ARC2.daily/.est_prcp/?help+dataselection

Table 2.3.6 Maximum Daily Rainfall Values (mm)

No.	Time	Maximum Daily Rainfall (mm)	Time	Maximum Daily Rainfall (mm) Descending Order
1	4-Oct-83	36.9	28-Sep-87	81.5
2	3-Jun-84	53.0	4-Jun-98	70.5
3	15-May-85	40.7	30-Sep-96	67.9
4	3-Oct-86	64.1	5-Oct-00	64.8
5	28-Sep-87	81.5	3-Oct-86	64.1
6	26-Aug-88	61.4	26-Aug-88	61.4
7	1-Sep-89	50.3	1-Oct-95	61.4
8	22-Jul-90	61.4	22-Jul-90	61.4
9	9-Oct-91	30.9	21-Jul-05	58.8
10	20-Oct-92	39.1	22-Jun-07	57.2
11	6-Apr-93	36.9	8-Jul-06	53.4
12	11-Sep-94	41.8	2-Jul-09	53.4
13	1-Oct-95	61.4	3-Jun-84	53.0
14	30-Sep-96	67.9	4-Sep-11	51.3
15	25-Jul-97	44.4	1-Sep-89	50.3
16	4-Jun-98	70.5	18-Apr-99	48.9
17	18-Apr-99	48.9	6-Aug-01	44.9
18	5-Oct-00	64.8	25-Jul-97	44.4
19	6-Aug-01	44.9	30-May-14	41.9
20	2-Sep-02	34.4	11-Sep-94	41.8
21	6-Nov-03	40.7	17-Aug-08	41.6
22	24-May-04	35.9	6-Nov-03	40.7
23	21-Jul-05	58.8	15-May-85	40.7
24	8-Jul-06	53.4	20-Oct-92	39.1
25	22-Jun-07	57.2	18-Apr-13	38.8
26	17-Aug-08	41.6	2-Jul-12	37.5
27	2-Jul-09	53.4	4-Oct-83	36.9
28	7-May-10	30.7	6-Apr-93	36.9
29	4-Sep-11	51.3	24-May-04	35.9
30	2-Jul-12	37.5	2-Sep-02	34.4
31	18-Apr-13	38.8	9-Oct-91	30.9
32	30-May-14	41.9	7-May-10	30.7

The results of the Iwai method are shown as Table 2.3.7.

Table 2.3.7 Results of Iwai Method

Return Period					Return Period Probability (mm)
T Year	ξ	1/a・ξ	Average Y+1/a・ξ	x+b	x
2	0.0000	0.0000	1.5584	36.2	47.9
5	0.5951	0.1192	1.6775	47.6	59.3
10	0.9062	0.1814	1.7398	54.9	66.7
20	1.1630	0.2329	1.7912	61.8	73.6
30	1.2967	0.2596	1.8180	65.8	77.5
50	1.4520	0.2907	1.8491	70.6	82.4
100	1.6450	0.3294	1.8878	77.2	89.0
200	1.8215	0.3647	1.9231	83.8	95.5
500	2.0350	0.4075	1.9658	92.4	104.2
1000	2.1850	0.4375	1.9959	99.1	110.8

According to the Iwai method, the return period rainfall of a flood that could occur every 200 years is calculated at 95.5 mm (≒96 mm)

Rainfall Intension and Period of Flood Concentration

We have no equation of rainfall intension at the Wau dam site and then we employ Mononobe equation for the rainfall intension. The equation is often adopted as the estimation of the rainfall intensity in the arbitrarily rain fall duration or period of flood concentration with only obtained daily precipitations in Japan.

・Rainfall intension

$$r_t = \frac{R_{24}}{24} \cdot \left(\frac{24}{t} \right)^{2/3}$$ ······································ Mononobe equation

r_t : Average effective rainfall intensity in t time (mm/hr)

R_{24} : 24 hours precipitation (mm)

t : Rain fall duration or period of flood concentration (hr)

Period of flood concentration is an index for the travel time required for runoff from the mechanically most further point in a catchment to reach the dam-site, and is evaluated considering topography, surface geology, vegetation, channel arrangement, rainfall pattern etc. At the catchment area of the Wau dam, we cannot get enough data related to the period of the flood concentration, and then we adopt Kadoya-Fukushima equation for the calculation of the period of the flood concentration.

·Period of flood concentration

$$t_p = C \cdot A^{0.22} \cdot r_e^{\ 0.35}$$ ······································ Kadoya·Fukushima equation

t_p : Period of flood concentration (min)

r_e : Average effective rainfall intensity in period of flood concentration t_p (min)

A: Catchment area (51 km^2)

C: Constant number by land use condition in catchment area

The unit of t_p is minute and C constant number is as follows:

Natural hills and mountains: $C = 250 \sim 350 \fallingdotseq 290$

Range land: $C = 190 \sim 210 \fallingdotseq 200$

Estimated Return Period Discharge of Flood

The return period discharge of a flood that could occur every 200 years is calculated with the return period rainfall of 200 years, which is a 24 hours rainfall value of 95.5 mm.

·Rainfall intension

$$r_t = \frac{R_{24}}{24} \cdot \left(\frac{24}{t}\right)^{2/3}$$ ······································ Mononobe equation

r_t : Average effective rainfall intensity in t time (mm/hr) = r_e

R_{24} : 24 hours rainfall (96 mm for 200 year return period)

t : Rain fall duration or period of flood concentration (hr) $t = \frac{t_p}{60}$

$$r_t = \frac{R_{24}}{24} \cdot \left(\frac{24}{t}\right)^{2/3} = \frac{96}{24} \cdot \left(\frac{24}{t}\right)^{2/3} = \frac{96}{24} \cdot \left(\frac{1440}{t_p}\right)^{2/3}$$

·Period of flood concentration

$$t_p = C \cdot A^{0.22} \cdot r_e^{\ -0.35}$$ ······································ Kadoya·Fukushima equation

t_p : Period of flood concentration (min)

r_e : Average effective rainfall intensity in period of flood concentration t_p (min)

A: Catchment area (51 km^2)

C: Constant number by land use condition in catchment area (=200)

$$t_p = 200 \cdot 51^{0.22} \cdot r_e^{-0.35}$$

We get the calculation results of $t_p = 180(\text{min})$, $r_e = 15.8(mm/h)$ from two equations as mention above and estimate the peak flood discharge by rational formula.

$$Q_p = \frac{1}{3.6} \cdot r_e \cdot A$$

where: Q_p: peak flood discharge (m^3/s)

A: catchment area (51 km^2)

r_e: average effective rainfall intensive in the catchment within the lag time of flood (15.8 mm/hr) $r_e = f_p \cdot r = 0.4 \times r$

$$Q_p = \frac{1}{3.6} \times 0.4 \times 15.8 \times 51 = 90.0(m^3/s)$$

The return period discharge of 200 years flood results as $Q_p = 90(m^3/s)$.

In the same procedure of the calculation, each year flood flow of the return period is calculated as follows:

Table 2.3.8 Return Period Discharges

Return period of year	Rainfall Duration (min)	Rainfall Intensive (mm/hr)	Flood Flow Discharge (m^3/s)	24 hr Rainfall (mm)
2	247	6.5	36.6	47.9
5	224	8.5	48.3	59.3
10	212	9.9	56.4	66.7
20	203	11.3	64.1	73.6
30	199	12.1	68.5	77.5
50	193	13.1	74.2	82.4
100	186	14.5	82.1	89.0
200	180	15.8	90.0	95.5
500	173	17.8	100.9	104.2
1000	168	19.3	109.3	110.8

Spillway

The spillway of the Wau dam is located at the right abutment of the dam embankment and the scale of the spillway is as follows:

Type of spillway: Reinforced concrete

200 years flood: 108 m^3/sec (Design Discharge: 90 × 1.2 ≒ 1

2.4 Dam Design

2.4.1 Dam Location

The location of the Wau dam is 8km toward east as follow's survey map. The length of the dam arraignment is 1.5 km and the reservoir area of the dam is 1.8 km^2.

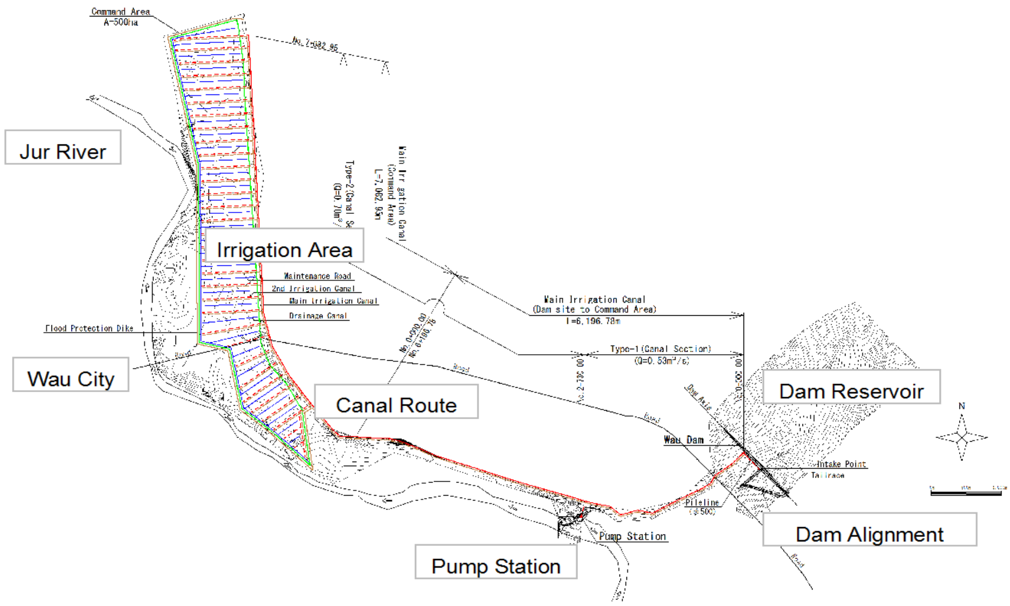


Figure 2.4.1 Location Map of Wau Area

2.4.2 Dam Type

(1) Fill Type

At the decision of dam type, it is important point to figure out the topography and geological conditions. We have mentioned the site conditions of Wau Dam site.

a) The geography of the dam site is gentle slope and hillside.
b) The dam arraignment is long as 1.5 km and the dam height is less than 15 m as the small dam by the H ~ Q curve of the dam.
c) The dam foundation is consist of the gravel and silty sand and is not so hard.
d) There is no quarrel site but it seems that borrow sites are near the site.

According to the site conditions which are the wide and hilly topography and have the soil foundation, the dam type is selected the fill dam and the homogenous type.

(2) Homogeneous Dam

Homogeneous dam is generally difficult to construct the dam which is more than 30 m height, and does not adapt the dam which is the frequently performed operations of rapid draw down at reservoir.

The characteristics of homogeneous dam type are mentioned as follows:

(a) Since the cross section of the dam utilized material of the nearly same quality, the construction is facilitated. However, in case which there is a seepage line at the downstream slope, drains will be required in the dam body.

*Even if the dam type is homogeneous, zone categorization is sometimes undertaken depending on the mechanical characteristics and water permeability characteristics of the materials. Such a dam is referred to a zone type earth fill dam.

(b) Since the length of seepage is longer compared to the zone type, construction is enabled even on foundation ground that has inferior impervious properties.

(c) Pore pressure that is generated during construction is not readily dissipated. For this reason, adjustment of the speed of banking, installation of drains to dissipate the pore pressure, and other methods may be required, depending on the condition of pore pressure generation.

(d) The shear strength is small, and since impervious materials or semi-permeable materials with large deformation characteristics are used, as downstream slopes of the dam body. For this reason, this type of dam is said to be inappropriate for dams with high dams which are the height of 30 m or more.

[Reference] Example of Design of a Homogeneous Type Fill Dam

<Example ①>

The Nagura stratum used as the material for the dam body is extremely weathered terraced gravel. The dam body is categorized the interceptor into the upstream Zone 1, in which the impervious characteristics are emphasized, and Zone 2, in which strength is emphasized, and while the material is the same, zones are deployed with consideration given to construction management. Moreover, in order to prevent the reduction of strength at the surface of Zone 1, and to prevent erosion due to waves, Zone 3 (pulverized stone, etc.) has been deployed at the upstream slope.

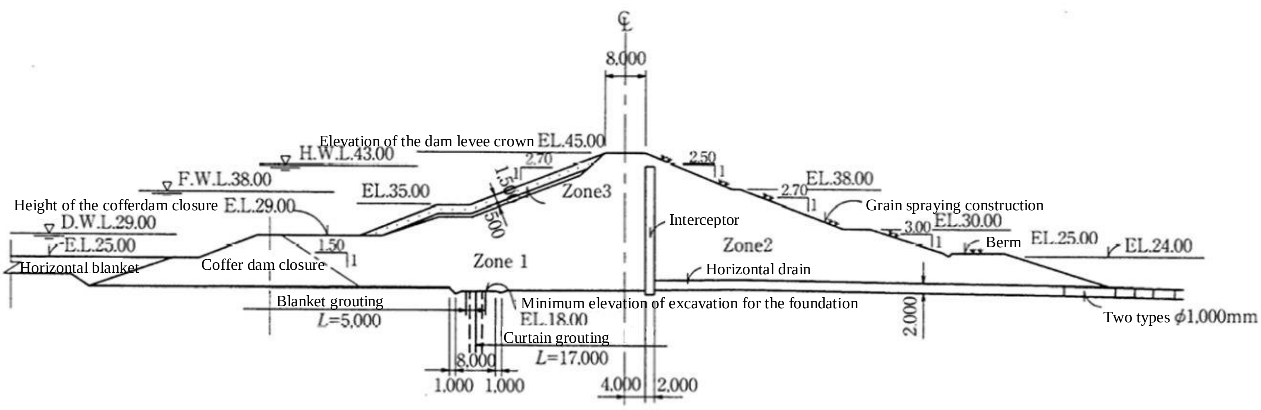


Figure 2.4.2 (Reference 1) Example of a Homogeneous Type Fill Dam

(Maezato Dam, Okinawa General Bureau)

<Example ②>

A mixture of silt clay and sandy tuff (ratio of mixture: 1:3) was used as material for Zone I, with material generated through excavation and material from borrow pits (sandy tuff) were used as the material for Zone II. Since there was concern over liquefaction taking place in the material for Zone II, two rows of horizontal drains were deployed, and the design took into consideration dissipation of the residual pore water pressure and promotion of compaction.

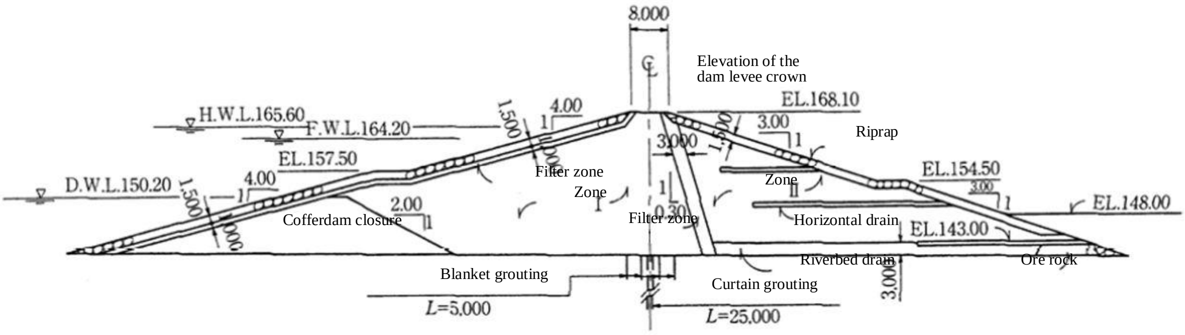


Figure 2.4.3 (Reference 2) Example of a Homogeneous Type Fill Dam

(Sugo Dam, Miyagi Prefecture)

<Example ③>

With consideration given to the effect of regulating infiltration in the sand stratum and connection with the natural ground blanket, a secure impervious zone was deployed on the upstream side.

Moreover, the infiltration in the sand stratum that outcrops from the reservoir is blocked by the downstream fault, and for this reason, a relief well was deployed in order to prevent hydrological failure at the toe drain.

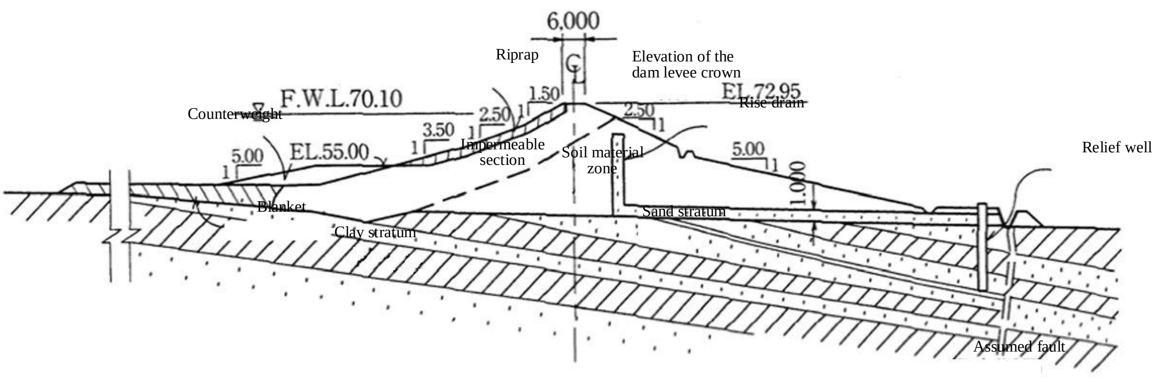


Figure 2.4.3 (Reference 3) Example of a Homogenous Fill Type Dam

(Togo Dam, Former Aichi Utility Water Public Corporation)

<Example ④>

In the impermeable section, Komeno stratum material (gravel mixed with clay) was used. The impermeable section was categorized into Impermeable Section Ⓘ, which emphasized impervious characteristics, and Impermeable Section Ⓘ, which emphasized the aspect of strength. For Impermeable Section Ⓘ, even material on the dry side of the optimum moisture content was used. For Transition Ⓘ of the foundation of the dam body, material from the Komeno stratum material from a high elevation with a high gravel ratio was used, and for Transition Ⓘ, material from the Oizumi stratum (mixture of sand, mudstone and cemented silt) was used.

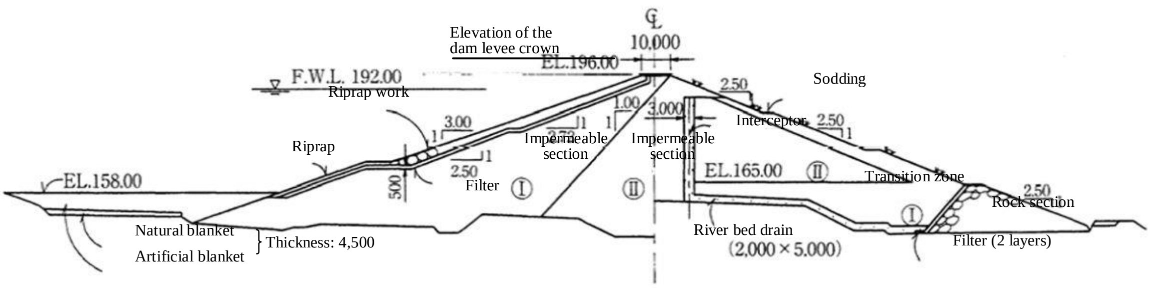


Figure2.4.4 (Reference 4) Example of a Homogeneous Type Fill Dam

(Nakazato Dam, Water Resources Development Public Corporation)

*Reference

Homogeneous Type Fill Dam

In the case of the homogeneous type fill dam, drains are installed to prevent the seepage line from penetrating the downstream side dam slope. Care must be given drain design and construction method to ensure that pressure during construction does not adversely affect dam stability.

General items to be in mind during design for homogeneous type fill dam are as follows:

① In most cases, material for the homogeneous type fill dam consists principally of which the percentage of fine grained impervious and semi-pervious materials in high. As dam height increases, slope gradient must be reduced. Dam volume increases consequently and this type of dam becomes more costly than the zone type. However, where dam height is low, the homogeneous type fill dam is advantageous from a construction standpoint as a single type of material is used.

② Where foundation is impermeable, or where groundwater level is high, the tydraulic mechanism is such that the seepage line for the homogeneous type fill dams will definitely emerge on the downstream slope. To prevent this, as well as to reduce pore pressure during construction, it is general practice to install drain. Drain layout may be considered as follows.

Where dam height is around 15 m, a toe drain or in some cases horizontal drains are installed. Vertical drains are established to the center of the dam body in the case of dam height over 25 m which serve to rapidly lower the seepage line and dissipate pore pressure during construction.

In addition, instability against sliding occurs when pore pressure value during construction is high. As a result, 0.5 ~ 1.0 m thick combined drains are installed within the dam body at intervals of 10 ~ 15 m. This functions to reduce residual pore pressure when water level rapidly drops at the upstream side, as well as to lower the seepage line on the downstream side.

2.4.3 Water Level of Reservoir

Full water level

Full water level is decided by the H ~ Q curve of the dam and its elevation is F.W.L. 433.8 m.

Dead Water Level

Dead water level is decided by H ~ Q curve of the dam and its elevation is L.W.L. 427.0 m.

High Water Level

Estimation of high water level is computed by using 200 years return period flood and it is considered storage effect of the reservoir. The calculation of the flood flow has been made and the results are shown follows:

Flood flow calculation results (200 years return period): Q = 108 m^3/sec

High Water Level ------------------ H. L.W. 434.8 m (Over flow depth = 1.0 m)

2.4.4 Dam Scale

The calculation of dam crest level shall follow ñEngineering Guideline for Small Dams of Irrigation and Drainage by the Ministry of Agriculture, Forestry and Fisheries in Japanò.

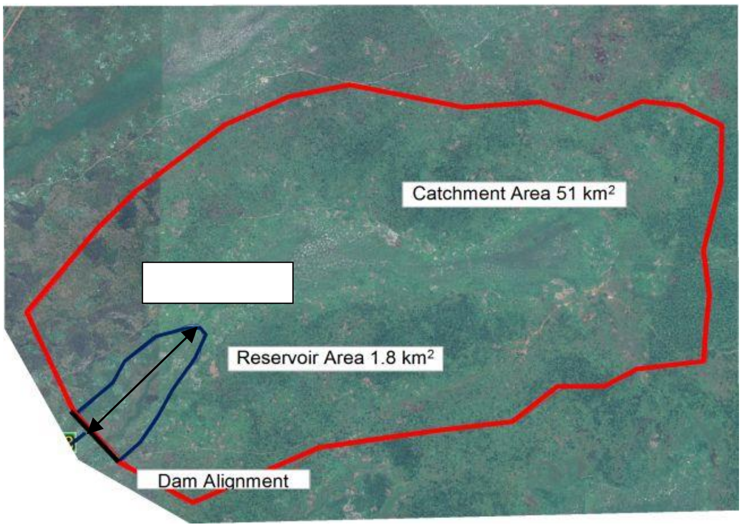
Wave height from reservoir surface by wind shall be determined considering the relationship between the wind velocity and the fetch on design flood level, and also consideration to wave reflection and wave run-up height depending on the structural form of the dam body.

Approximate value of wave run-up height R shall be found as shown in Figure 4.4.1 which shows the relationship between wave run-up height and other factors such as wave height and wave length obtained by S.M.B method, and upstream slope, and slope protection materials by the Saville method.

Freeboard for wind action would be decided by using Wave run-up height by Wilsonôs improved formula in S.M.B method and Saville method.

(1) Fetch

Fetch is the free surface distance to be waved by wind. Fundamentally it will be a distance in a straight line in the direction of maximum wind velocity. However, where available wind direction data in not sufficient, the maximum distance in a roughly straight line from the data shall be regarded as fetch.



F: fetch in km (= 3 km), at H.W.L. 343.8 m

Figure 2.4.5 Dam Reservoir

Wind Velocity

If no data is available on wind velocity according to long-term observation at site, it shall be in principle 30 m/sec, however, 20 m/sec may be acceptable for a dam site where there is no danger of strong wind.

Wind velocity is less than the maximum observed velocity for the following reasons.

(a) Maximum instantaneous wind velocity lacks sufficient blow time for wave inducement.
(b) In many cases coincidence is not seen between wind direction and maximum fetch direction.
(c) Dam sites are mostly located in mountainous area where topographic feature and vegetation break wind velocity.

V: wind velocity 20 m/sec

Roughness of Slope

Smooth surface slope is a relative even surface slope which is made of concrete block or stone pitch. Riprap slope is slope employed on rock fill dams in order to absorb wave force between rock fragments.

Slope of Dam

In the case of riprap, wave run-up height is not affected by slope, but for the case of smooth surface slope, such depends on the steepness of slope and fetch. As a result, no consideration is required for riprap slope in determining wave run-up height, but for the low dam with smooth surface slope, the steeper the slope, the greater the wave run-up height.

Slope of dam (the upstream side): 1:3.0

For the reasons mentioned above, the wave height from reservoir surface is decided as follows:

Wind height: 1.4 m (See Figure 4.4.2)

Freeboard of Dam

The freeboard of the dam shall be decided by using ñEngineering Guideline for Small Dams of Irrigation and Drainage by the Ministry of Agriculture, Forestry and Fisheries in Japanò.

The freeboard equations are as follows:

(a) R≦1.0 m

$$h_2 = 0.05 \cdot H_2 + 1.0(m)$$

(b) R > 1.0 m

$$h_2 = 0.05 \cdot H_2 + R(m)$$

where R: wind height (= 1.4 m)

H_2: H.W.L. ï foundation level = H.W.L. 434.8 m ï 426.0 = 8.8 m

h_2: freeboard of dam

Then the wind height (R) is 1.4 m and the freeboard of the dam is calculated as follow:

$$h_2 = 0.05 \cdot H_2 + R(m) = 0.05 \times 8.8 + 1.4 = 1.84m$$

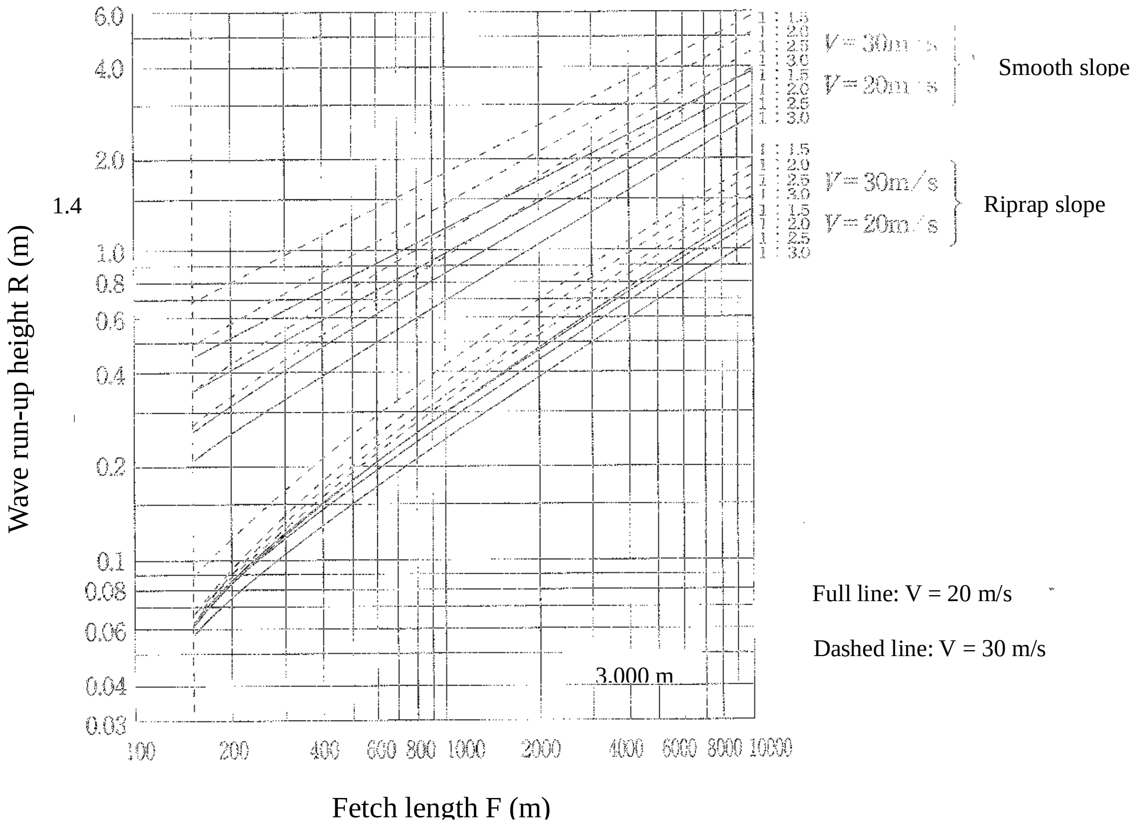


Figure 2.4.6 Wave run-up height by Wilsonȇs improved formula in S.M.B method and Saville method

Dam Crest Level

The dam crest level is added the freeboard height on H.W.L. 434.8 m and then,

Dam Crest Level = H.W.L. 434.8 m + Freeboard 1.84 m = EL. 436.64≒EL. 436.7 m

Dam Height

The dam height is as follows:

Dam Height = Dam Crest EL. 436.7 m ï Dam Foundation EL. 426.0 m = 10.7 m

Dam Crest Width

The width of the dam crest is calculated as follows:

B = 0.2H + 2.0

where, B: Width

H: Dam height (= 10.7 m)

B = 0.2 × 10.7 + 2.0 = 4.14 ≒4.0 m

The typical cross section of the dam is shown at next page, Figure 4.4.3.

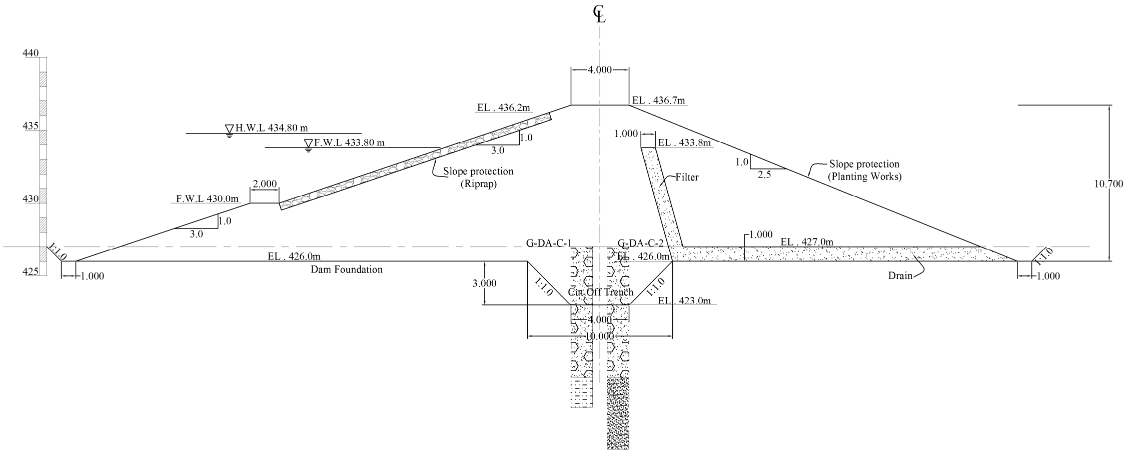


Figure 2.4.7 Typical Cross Section of Dam

2.4.5 Dam Materials

The Wau dam is homogeneous dam type and the dam-body consists of impervious zone and filter and drain zone.

1 Selected Impervious zone (GC or CH, CL)

2 Filter and Drain zone (GW or GP)

(1) Soil Test Results

The soil tests were conducted with bore-hole samples and the test results are as follow's table:

Specific gravity
Grain size analysis
Liquid and Plastic limit

The numbers of the soil tests are as follows:

G-DA-C: 4 samples (1m, 2m (only specific gravity test), 4m, 10m)
G-DA-R: 2 samples (1m, 4m)
G-DA-L: 5 samples (2m, 3m, 5m, 7m, 9m)

DATE	Wau May 2015	1	2	3	4	5	6	7	8	9	10	11
Sample	Borehole	G-DA-C	G-DA-C	G-DA-C	G-DA-C	G-DA-R	G-DA-R	G-DA-L	G-DA-L	G-DA-L	G-DA-L	G-DA-L
	Depth (m)	1.0	2.0	4.0	10.0	1.0	4.0	2.0	3.0	5.0	7.0	9.0
Specific Gravity	Soil ɟs	2.19	1.41	1.87	1.71	2.26	1.45	2.47	1.87	2.36	2.23	1.47
	Maximum Size Dmax mm	5.60	-	12.50	9.50	2.36	12.50	9.50	9.50	12.50	4.75	4.75
	Gravel G(4750εm over) %	3.0	-	4.0	6.0	0.0	3.0	37.0	3.0	13.0	0.0	0.0
	Sand S(75～4750εm) %	36.0	-	42.0	67.0	24.0	36.0	61.0	90.0	40.0	40.0	53.0
	Silt M(5～75εm) %	10.0	-	-	-		10.0		4.0			
	Clay C(1 ~ 5εm) %	51.0	-	54.0	27.0	76.0	51.0	2.0	3.0	47.0	60.0	47.0
	Colloid C(under 5εm) %	-	-	-	-	-	-	-	-	-	-	-
	Fines {F} (less than75εm) %	61.0	-	54.0	9.5	76.0	61.0	2.0	7.0	47.0	60.0	47.0
	D10 mm							1.8	0.18			
	D30 mm							3.0	3.0			
	D60 mm							5.0	5.0			
	Coefficient of uniformity Uc = D60/D10							2.8	27.8			
	Coefficient of curvature Uc' = (D30)^2 / D60・D10							1.0	10.0			
Consistency	Liquid Limit ɤL %	35.0	-	45.0	48.0	42.0	35.0	36.0	0.0	52.0	50.0	62.0
	Plastic Limit ɤp %	20.0	-	24.0	17.0	22.0	13.0	11.0	0.0	28.0	25.0	45.0
	Plasticity Index Ip %	15.0	-	21.0	31.0	20.0	22.0	25.0	0.0	24.0	25.0	17.0
Classification	Soil type	CL	-	CL	SC	CL	CL	SM	SP	SC	CL	ML
	Name	Clay-Low	-	Clay-Low	Sand-Clay	Clay-Low	Clay-Low	Sand-Silt	Sand-Poor	Sand-Clay	Clay-Low	Silt-Low

Table 2.4.1 Soil Test Results

Grain size distribution

The results of the grain size distribution curves are as follows:

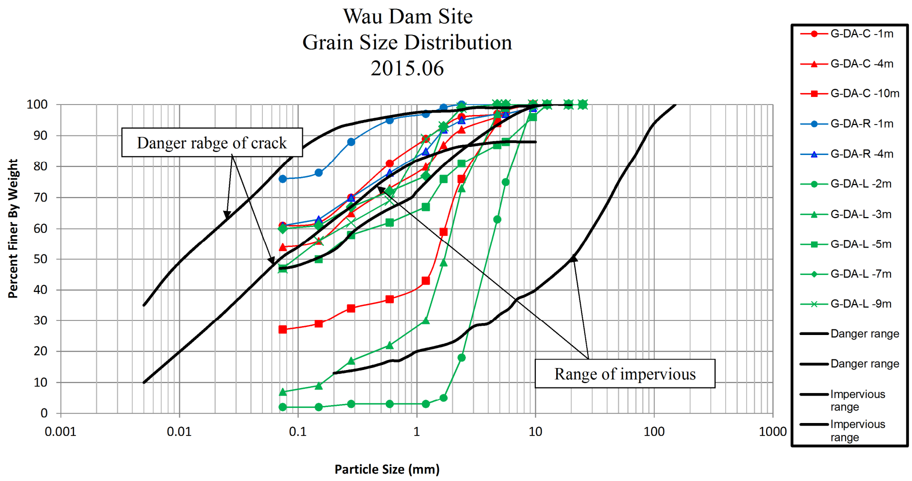


Figure 2.4.8 Grain Size Distribution Curves

According to the grain size distribution, about half numbers of the curves lie down the danger range of crack and remaining half curves are in the range of impervious. In general, the materials of CL or CM are the fine particles and almost their grain size distribution curves lie down in the danger range of crack. The suitability materials of impervious zone are usually GC and SC.

At figure 4.5.2, the range of crack is showed at dry side condition of the embankment case. This chart is made from the investigation results of seventeen dams which USBR (United States Bureau of Reclamation) constructs and then the cracks at the impervious (core) zone happen. It is said that the cracks may be easy to happen at using materials which is a class of inorganic clay that the plastic index is less than 15, low or middle plastic level.

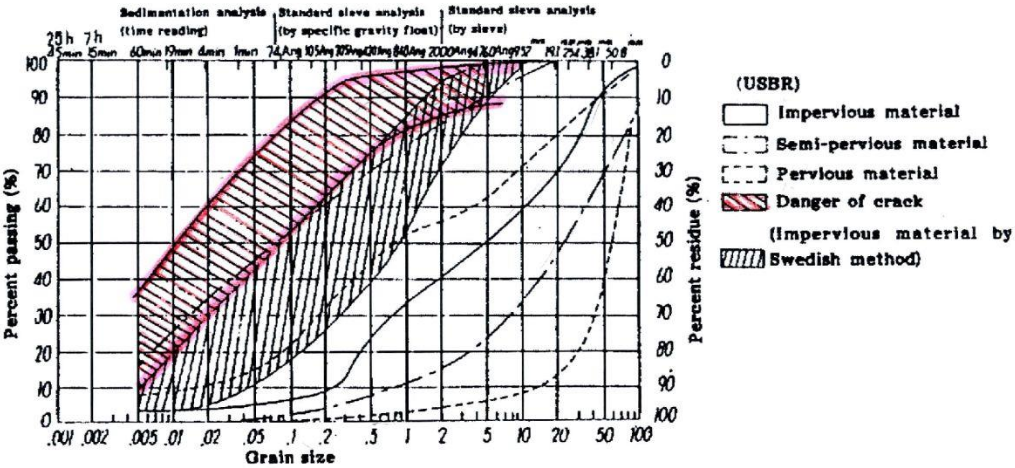


Figure 2.4.9 Grain Sizes for Embankment Materials

óCharacteristics of fill materials and suitability for dam constructionôtable is shown at next page. (The cited reference is Engineering Manual for Irrigation and Drainage Fill Dam by the Japanese Institute of Irrigation and Drainage.)

Table 2.4.2 Characteristics of fill materials and suitability for dam construction

Symbol	Characteristics				Suitability				
	Permeability after compaction	Degree of shearing strength in saturated condition after compaction	Compressibility in saturated condition after compaction	Workability as banking materials	Dam			Foundation	
					Homogeneous dam	Impervious zone	Pervious zone	Seepage flow considered	Seepage flow not considered
GW	Pervious	Excellent	Almost nothing	Excellent	-	-	1	-	1
GP	Highly pervious	Good	Almost nothing	Good	-	-	2	-	3
GM	Semi-pervious ~impervious	Good	Almost nothing	Good	2	4	-	1	4
GC	Impervious	Good-fair	Extremely small	Good	1	1	-	2	6
SW	Pervious	Excellent	Almost nothing	Excellent	-	-	3*	-	2
SP	Pervious	Good	Extremely small	Fair	-	-	4*	-	5
SM	Semi-pervious ~impervious	Good	Small	Fair	4	5	-	3	7
SC	Impervious	Good-fair	Small	Good	3	2	-	4	8
ML	Semi-pervious ~impervious	Fair	Medium	Fair	6	6	-	6	9
CL	Impervious	Fair	Medium	Good-fair	5	3	-	5	10
MH	Semi-pervious ~impervious	Fair-poor	Large	Poor	9	9	-	8	12
CH	Impervious	Poor	Large	Poor	7	7	-	9	13

(Note)* High gravel content.
Under “Suitability”, the larger number shows lesser suitability, namely, the number of “1” is the best suitability.

Table 2.4.3 Standard Classification and properties for soil, gravel and sand

(This table is only rough estimate of each value, soil tests should be performed at design value conclusion.)

Symbol	Standard compaction		Void ratio e_0	Piping resistance	Coefficient of permeability K(cm/sec) Range (average)	Degree of permeability	Shearing strength			Shearing strength	Construction difficulty	No. of actual USBR example	Suitability	Compression (%)		Suitability for foundation	
	Dry density max (t/m3)	Optimum water content (Wopt %)					Cohesion Co (kg/cm2)	Cohesion Csat (kg/cm2)	φ(°)					1.4 kg/cm2	3.5 kg/cm2	Bearing capacity	Seepage prevention
GW	>1.91	<13.3	*	Large	1^{-3}~1^{-1} (2.7^{-2}±1.3^{-2})	Pervious	*	*	>38	Very large	Very easy	-	Suitable (pervious)	<1.4	*	Good	Perfect impervious type
GP	>1.76	<12.4	*	Large -medium	5^{-3}~1^{+1} (6.4^{-2}±3.4^{-2})	Pervious -very pervious	*	*	>36	Large	Very easy	-	Suitable (pervious)	<0.8	*	Good	
GM	>1.83	<14.5	*	Large -medium	1^{-7}~1^{-4} (>3^{-7})	Semi-pervious	*	*	>34	Large	Very easy	4	Suitable (impervious)	<1.2	<0.3	Good	Toe trench not required
GC	>1.84	<14.7	*	Very large	1^{-8}~1^{-5} (>3^{-7})	Impervious	*	*	>31	Large	Very easy	4	Suitable (impervious)	<1.2	<2.4	Good	
SW	1.91±0.08	13.3±2.5	0.37±-	Large -medium	5^{-4}~5^{-2} (*)	Pervious	0.40±0.04	*	38±1	Very large	Very easy	-	Suitable (pervious)	1.4±*	*	Good	Incomplete cut-off wall
SP	1.76±0.03	12.4±1.0	0.50±0.03	Small -very small	5^{-5}~5^{-1} (7.2^{-4})	Pervious -semi pervious	0.23±0.06	*	36±1	Large	Easy -medium	-	Suitable (pervious)	0.8±0.3	*	Good -bad	
SM	1.83±0.02	14.5±0.4	0.48±0.02	Small	1^{-7}~5^{-4} (7.5^{-6}±4.8^{-6})	Semi-pervious -impervious	0.52±0.06	0.20±0.07	34±1	Large	Easy -medium	16	Suitable (impervious)	1.2±0.1	3.0±0.4	Good -bad	
SM~SC	1.91±0.02	12.8±0.5	0.41±0.02	Medium -small	- (8.0^{-7}±6.0^{-7})	-	0.51±0.22	0.15±0.06	33±3	-	-	3	-	1.4±0.3	2.9±1.0	-	-
SC	1.84±0.02	14.7±0.4	0.48±0.01	-	1^{-8}~5^{-5} (3.0^{-7}±2.0^{-7})	Impervious	0.76±0.15	0.11±0.06	31±3	Large -medium	Easy -medium	7	Suitable (impervious)	1.2±0.2	2.4±0.5	Good -bad	Not required
ML	1.65±0.02	19.2±0.7	0.63±0.02	Large	1^{-8}~5^{-5} (5.9^{-7}±2.3^{-7})	Impervious	0.68±0.10	0.09±*	32±2	Large -medium	Medium-very difficult	7	Suitable (impervious)	1.5±0.2	2.6±0.3	Bad	Toe trench
ML~CL	1.75±0.02	16.8±0.7	0.54±0.03	Small -very small	- (1.3^{-7}±0.7^{-7})	-	0.64±0.17	0.22±*	32±3	-	-	-	-	1.0±0.2	2.2±0.0	-	-
CL	1.73±0.02	17.3±0.3	0.56±0.01	-	1^{-8}~1^{-6} (8.0^{-8}±3.0^{-8})	Impervious	0.88±0.10	0.13±*	28±2	Medium	Medium-difficult	10	Suitable (impervious)	1.4±0.2	2.6±0.4	Good -bad	Not required
OL	*	*	*	Large	1^{-8}~1^{-5} (*)	Impervious	*	*	*	Small	Medium-difficult	-	Unsuitable	*	*	Bad	Not required
MH	1.31±0.06	36.3±3.2	1.15±0.12	Medium	1^{-9}~1^{-7} (1.6^{-7}±1.0^{-7})	Very impervious	0.73±0.30	0.20±0.01	25±2	Small	Very difficult	-	Unsuitable	2.0±1.2	3.8±0.8	Bad	Not required
CH	1.50±0.03	25.5±1.2	0.80±0.04	Medium -large	1^{-10}~1^{-8} (5.0^{-5}±5.0^{-8})	Very impervious	1.04±0.34	0.11±0.06	19±5	Small -medium	Very difficult	1	Suitable (impervious)	2.6±1.3	3.9±1.5	Bad	Not required
OH	*	*	*	Very large	- (*)		*	*		-	Compaction impossible	-	Unsuitable	*	*	Bad	Not required
Pt													Unable			Removal of foundation ground	

(Note)

1. This table is prepared on the basis of data from USBR, US Army Civil Engineering Department and Earth and Earth-Rock Dams. Figures stated in the table show an average reliability of 90%.
2. Although not perfectly coincident with the standard for Japanese materials, accuracy is sufficient for preliminary design.
3. Symbol '*' indicates no data.
4. (Co) shows shearing strength at optimum moisture content, and (Csat) shows the same at saturated conditions.
5. Coefficient of permeability (1^{-3}~1^{-1}) indicates (1×10^{-3}~1×10^{-1}).

2.5 Design of Spillway

2.5.1 Specification of Spillway

Spillways are the facilities provided to ensure the safety of dams against floods. Therefore, spillways should be of such structure that outlet capacity of spillway is sufficient to release safety the design flood discharge.

The design flood flow of the spillway (200 years return period) and the standard level of the reservoir are as follows:

Full water level F.W.L. 433.8 m

The design flood flow Qd (= 108 m^3/s by flood flow calculation result)

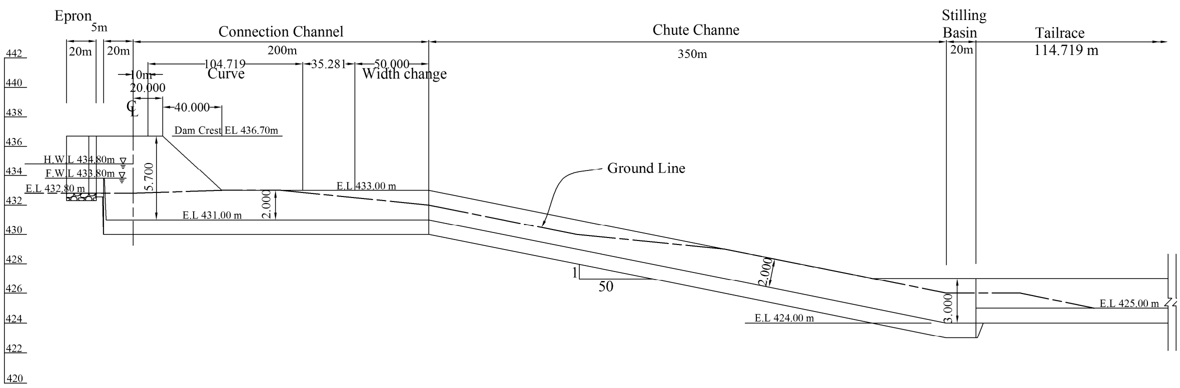


Figure 5.1.1 Longitudinal Section of Spillway

2.5.2 Spillway Hydraulics

Design Condition

Coefficient of Roughness

Spillway hydraulics calculation is used by this coefficient of roughness. Coefficient of roughness on concrete surface (using metal form) is as follow.

Concrete channel: n = 0.015

Spillway Location and Characteristics

The spillway location of the dam site is constructed at the left abutment and the standard type, because the alignment of the main canal is planned at right side of the dam site. If the spillway is planned at the right side, the both alignments are crossed.

Inlet Channel Hydraulics

a) Overflow Depth of Weir

The specifications of the dam and the spillway are the following data:

Dam height ----------------------- 10.7 m

Dam crest elevation-------------- EL. 436.7 m
Spillway width-------------------- 60 m
Full Water Level------------------ F.W.L. 433.8 m
High Water Level-----------------H.W.L. 434.8 m
Overflow depth of weir --------- 1.0 m

Calculation of Overflow Portion

The hydraulic design conditions for the inlet channel are to create low flow velocity, and gradual change of the flow direction and velocity without turbulence by establishing sufficient flow depth.

Hydraulic condition of the inlet channel should be P≧H/5, V≦4.0 m/sec.

Where, H: designõs overflow depth
P: inflow depth under weir crest
V: approach velocity

This hydraulic condition serves to establish a gentle flow with small fluctuation of water surface. Froude number ($F = \frac{q}{\sqrt{g(H+P)^3}}$) may be considered as 0.4 or less.

The overflow weir depth has great influence on discharge coefficient of weir and approach flow condition of channel.

It is determined that the overflow weir depth is $W / H_d > 0.2$, and the approach flow velocity is less than 4.0 m/s.

Overflow weir depth and approach channel elevation are as next page.

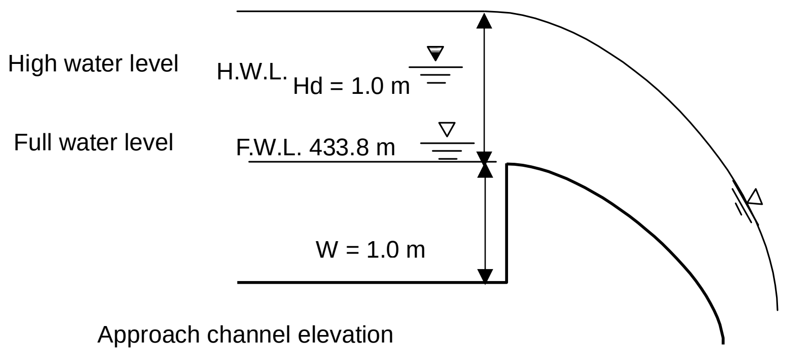


Figure 5.2.1 Cross section of overflow weir

Full water level: EL. 433.8 (m)

High water level: EL. 434.8 (m)

Approach channel elevation: EL. 432.8 (m)

Overflow weir depth: Hd = 1.0 (m)

Overflow weir height: W = 1.0 (m)

W / Hd = 1.0 / 1.0 = 1.0 > 0.2 OK

Flow Discharge Formula and Discharge Coefficient

Flow discharge formula and Discharge coefficient for standard crest is expressed by Iwasakiös Formula as follow.

$$Q = C \cdot L \cdot H^{3/2} \text{ ---------------------------------------(1)}$$

$$Cd = 2.2000 - 0.0416 (Hd / W)^{0.99} \text{ -----------------------(2)}$$

$$C = 1.60 \times \frac{1 + 2a(H / Hd)}{1 + a(H / Hd)} \text{ -----------------------------(3)}$$

Where;

Q : Discharge (= 108 m^3/s)

L : Overflow weir width (= 108 m)

H : Overflow head above crest (m)

Hd : Design overflow head (= 1.0 m)

W : Overflow weir height (= 1.0 m)

a : Constant

C : Discharge coefficient

Cd : Discharge coefficient at H = Hd

Hd / W = 1.0 / 1.0 = 1.0, which is substituted for expression (2)

Cd = 2.200 ï 0.0416 × $1.0^{0.99}$

= 2.1584

H / Hd = 1.00 and C = Cd = 2.15, which are substituted for expression (3) and Constant a is calculated.

a = (1.60 ï Cd) / (Cd ï 3.20)

= (1.60 ï 2.15) / (2.15 ï 3.20)

= 0.523

Then the overflow coefficient (a) for the given overflow head (H) is as follows:

$$C = 1.60 \times \frac{1 + 1.046(H / Hd)}{1 + 0.523(H / Hd)}$$

Discharge coefficient at H = Hd is as below expression:

Cd = 2.14≒2.0 (as the safe side)

Overflow Crest Length (Crest Width)

According to the expression (1), the discharge formula is as follows:

$$Q = C \cdot L \cdot H^{3/2}$$

Where; Q : Discharge (Q = Qd = 108 m^3/s

C : Discharge coefficient at design overflow head (C = Cd = 2.0)

L : Overflow crest width (= 60 m)

H : Overflow head above crest (H = Hd = 1.0 m)

Overflow crest length which can flow down the design discharge with the design overflow head water is as below expression.

$$L = \frac{Q}{C \cdot H^{3/2}} = \frac{108}{2.0 \times 1.0^{3/2}} = 54m < 60m$$

Then the overflow crest length is decided as 60 m to be on the safe side.

Approaching Velocity at Intake Portion

The velocity in the approaching channel is calculated as followsôformula

$$V = \frac{g \cdot L \cdot d^2 - ((g \cdot L \cdot d^2)^2 - 2 \cdot g \cdot d \cdot Qd^2)^{1/2}}{Qd}$$

Where; V: Approaching velocity in front of the weir (m) g: acceleration of gravity (= 9.8 m/sec^2)

d: Water depth in channel (=Hd + W = 1.0 + 1.0 = 2.0 m)

L: Overflow crest width (= 60 m)

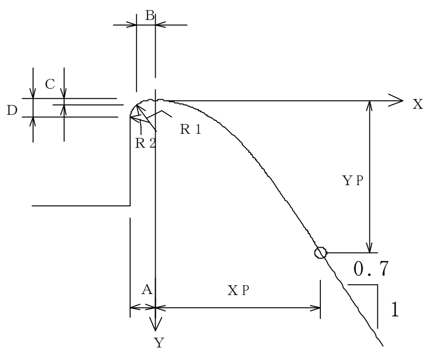
Q: Discharge (Q = Qd = 108 m^3/s)

$$g \cdot L \cdot d^2 = 9.8 \times 60 \times 2^2 = 2352$$

$$V = \frac{2352 - (2352^2 - 2 \times 9.8 \times 2 \times 108^2)^{1/2}}{108} = 0.92 < 4.0 m/\sec$$

Cross Section of Overflow Weir

The cross section of overflow weir is showed as below:



Harroldôs standard overflow weir crest is adopted.

Figure 5.2.2 Cross Section of Crest Weir

Upstream side from crest center

Hd = 4.49 m (design overflow head)

A = 0.282・Hd

B = 0.175・Hd

C = 0.03163・Hd

D = 0.12586・Hd

R1 = 0.5・Hd

R2 = 0.2・Hd

Downstream side from crest center

$$Y = 0.5 \times \frac{X^{1.85}}{Hd^{0.85}}$$

However, the gradient of Haroldôs curve at lower side is 1: 0.7, because the gradient at downstream is adapted for the overflow water.

The calculation results are shown as follows:

Calculation at the lower part of Harrold's curve

Y=0.5*X^1.85/Hd^0.85			xp	yp	Elavation
=	0.5 X^1.85		0	0	433.800
differential of upper expression			0.2	0.025	433.775
Y' =1/	0.7 gradient		0.4	0.092	433.708
Right side=	0.5 *1.85* X^0.85		0.6	0.194	433.606
∴X^0.85=	1.5444		0.8	0.331	433.469
Xp=	1.668	m	1	0.500	433.300
Yp=	1.288	m	1.2	0.700576	433.0994
			1.4	0.931766	432.8682
			1.668	1.288347	432.5117
A=0.282*Hd=	0.282	m			
B=0.175*Hd=	0.175	m			
C=0.03163*Hd=	0.03163	m			
D=0.12586*Hd=	0.12586	m			
R=0.5*Hd=	0.5	m			
R=0.2*Hd=	0.2	m			

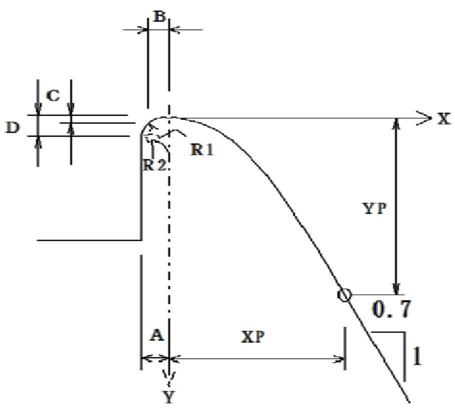


Figure 5.2.3 Calculation Results of Harrold's Curve

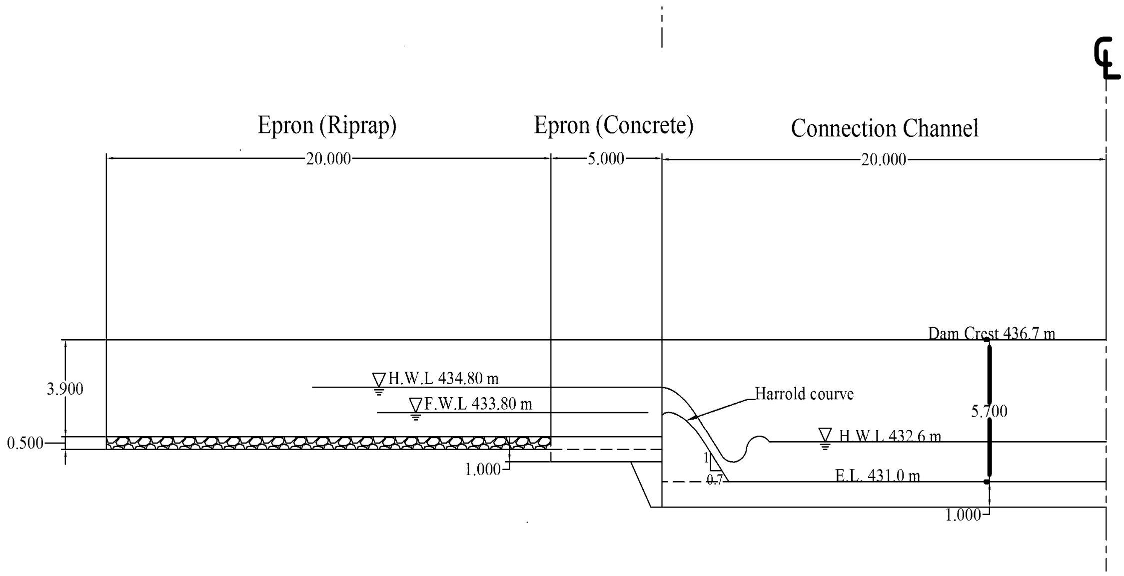


Figure 5.2.4 Longitudinal Section at Weir

Overhead ~ Overflow Discharge

At the overflow crest length L = 60.0 m, the relation between overhead H and overflow discharge Q is calculated. H ~ Q curve is shown as bellows:

Q=CLH^(3/2)
C= 2
L= 60

Elevation	H (m)	Q (m3/s)
433.8	0	0.0
434.0	0.2	10.7
434.2	0.4	30.4
434.4	0.6	55.8
434.6	0.8	85.9
434.8	1	120.0

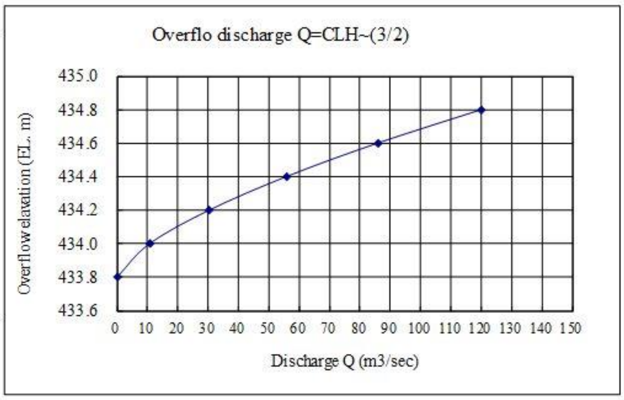


Figure 5.2.5 Overflow Discharge Curve

Connection Channel Hydraulics

The connection channel is made the sudden contraction at both side walls, and then the channel is continued to the connection channel with the rectangular cross section.

Considering the topographic and geological condition or the relation with the location of other structures, the length of the connection channel is made 200 m, and then is continued to the chute portion.

At the end of the connection channel, the overflow weir is set as the control point of the water flow, and is kept the stability of the water flow in the connecting channel. (This weir controls Froude number at the end of the connecting channel and keeps the stability of the water flow in the side channel.)

a) Cross Section of Connection Channel

The channel bottom width is decided as the same the side channel, which is B = 60 m. The longitudinal gradient of the connection channel is necessary to keep gently too enough, because of the prevention of the shock wave occurring or water surface waving, and control of Froude number at the side channel.

The longitudinal gradient is calculated as followsôformula

$$s = \frac{g \cdot n^2 \cdot Fr^2 \cdot (1 + 2d / B)^{4/3}}{d^{1/3}}$$

Where; s: longitudinal gradient of the connection channel

g: acceleration of gravity (= 9.8 m/sec^2)

n: coefficient of roughness (n = 0.015)

Fr: Froude number at the end of side channel

Fr =0.61, as mentioned above

d: Water depth at the beginning of the connection channel (m)

d = 1.62 m

B: Channel bottom width at the beginning of the connection channel (m)

B = 60m

$$s = \frac{9.8 \times 0.015^2 \times 0.29^2 \times (1 + 2 \times 1.62 / 60)^{4/3}}{1.62^{1/3}}$$
$$= 1 / 5904.4$$

Therefore the gradient of the connection channel id decided as level.

Water Surface Calculation of Connection Channel

The control point at the end of the connection channel (the beginning of the chute portion) is made as the start point of the calculation and the water surface calculation for the flood flow is performed to the upstream side from this point.

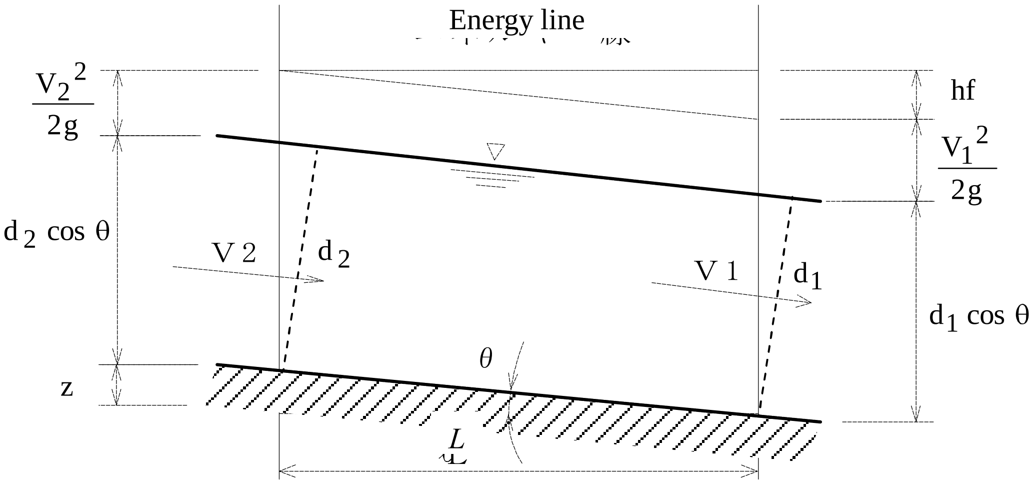


Figure 5.2.6 Conceptual Diagram of Hydraulic Calculation

In the conceptual diagram, Bernoulli's theorem is adopted in both sections.

$$d_1 \cos\theta + \frac{V_1^{\ 2}}{2g} + hf = d_2 \cos\theta + \frac{V_2^{\ 2}}{2g} + z$$

Where,

$$hf = \frac{1}{Cm^{2\cdot} \cdot Rm} \times \left(\frac{V_1 + V_2}{2}\right)^2 \times l = \frac{n^2 \cdot Vm^2 \cdot l}{Rm^{4/3}}$$

$$Cm = \frac{1 \cdot Rm^{1/6}}{n}, Vm = \frac{V_1 + V_2}{2}, Rm = \frac{R_1 + R_2}{2}$$

n: Coefficient of roughness (n = 0.015)

$z = l \cdot \tan\theta$

The water head drop of the calculation interval is as follows:

$$h = d_2 \cos\theta + z - d_1 \cos\theta$$

$$h = \frac{V_1^{\ 2}}{2g} + hf - \frac{V_2^{\ 2}}{2g}$$

The water depth d_2 at the upstream side is done the trial calculation, which satisfies as mentioned above expressions. The calculation results are shown as follows:

The hydraulic critical conditions at the control point are as follows:

$$dc = 0.467 \cdot q^{2/3} = 0.467 \times (108 / 30)^{2/3} = 1.097 \text{ (m)}$$

$$Ac = B \cdot dc = 30 \times 1.097 = 32.91 \text{ (m}^2\text{)}$$

$$Vc = Q / Ac = 108 / 32.91 = 3.18 \text{ (m/sec)}$$

$$hc = Vc^2 / 2g = 3.18^2 / 2g = 0.515 \text{ (m)}$$

Table 5.2.1 Water surface calculation results table of Connection Cannel

Station point	Section NO	Inteval distance ⊿X (m)	Elevation of channel botom Zb (m)	Water height h (m)	Water elevation Zb+h (m)	Discharge Q (m3/s)	Correction coefficient of energy α	Coefficient of roughness n	Water bottom width B1 (m)	Water surface width B2 (m)	Flow velocity v (m/s)	Water flow area A (m2)	Hydraulic mean depth R (m)	Froude number Fr	Velocity head hv (m)	Friction gradient Sf (m)	Fiction head loss hf (m)	Others: Coefficient of loss	Others: head loss (m)	Energy gradient E1 (m)	E1+hf at downstream side E2 (m)	Error E (m)	Judge
0	1	0.000	431.000	1.097	432.097	108.000	1.000	0.015	30.00	30.00	3.282	32.910	1.022	1.00	0.549	0.0024	—	0.000	0.000	432.646	-	-	-
50	1	50.000	431.000	1.589	432.589	108.000	1.000	0.015	60.00	60.00	1.133	95.340	1.509	0.29	0.065	0.0002	0.008	0.000	0.000	432.654	432.655	0.000	OK
100	1	50.000	431.000	1.598	432.598	108.000	1.000	0.015	60.00	60.00	1.126	95.880	1.517	0.28	0.065	0.0002	0.008	0.000	0.000	432.663	432.663	0.000	OK
150	1	50.000	431.000	1.607	432.607	108.000	1.000	0.015	60.00	60.00	1.120	96.420	1.525	0.28	0.064	0.0002	0.008	0.000	0.000	432.671	432.671	0.000	OK
200	1	50.000	431.000	1.616	432.616	108.000	1.000	0.015	60.00	60.00	1.114	96.960	1.533	0.28	0.063	0.0002	0.008	0.000	0.000	432.679	432.679	0.000	OK
	計	200.000																					

Freeboard of Connection Channel

The freeboard at the connection channel of the subcritical flow is calculated as followsôexpression.

$$Fb = 0.07d + hv + (0.05 \sim 0.15)$$

Where;

Fb: channel freeboard (m)

d: Water depth at flood flow (m)

hv: velocity head at flood flow (m)

And the vertical wall height (H) of the channel is calculated as follows:

$$H = d \cdot \cos\theta + Fb$$

According to the mentioned expressions above, the freeboard, the vertical wall height and plan height of the channel wall are calculated as followsôtable.

Table 5.2.2 Calculation Results of Freeboard and Wall Height

Station point	Water height	Velocity head	Freeboard	Calcularion of wall height	Adoption of wall height	Channel bottom elevation	Channel crest elevation	Remark
	h	hv	Fb	Hc	Ha	Eb	Eh	
	(m)	(m)	(m)	(m)	(m)	(m)	(m)	
0	1.097	0.55	0.78	1.87	2	431.000	433	End point at connection portion
50	1.589	0.07	0.33	1.92	2	431.000	433	
100	1.598	0.06	0.33	1.92	2	431.000	433	
150	1.607	0.06	0.33	1.93	2	431.000	433	
200	1.616	0.06	0.33	1.94	2	431.000	433	Beginnig point at connection portion

Chute Channels Hydraulics

a) Longitudinal Section of Chute Channel

The gradient of the chute channel is decided as S = 1: 5.0 with considering the topographical and geological condition and the location of energy dissipater.

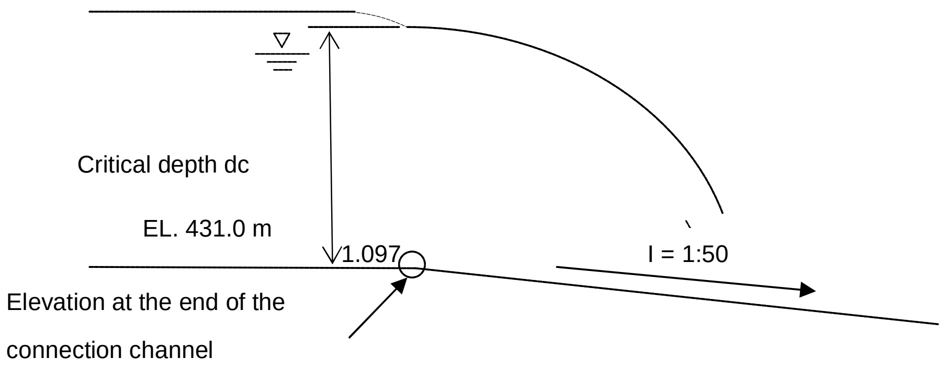


Figure 5.2.7 Weir Cross Section at End of Connection Channel

Cross Section of Chute Channel

The cross section of the chute channel is made the rectangular type. The width of the chute channel is the same of the connection channel B = 30.0 m to consider the joint of the energy dissipater at the downstream side.

Water Surface Calculation of Chute Channel

The water surface calculation is made as the start point which is the end of the connection channel (the control point), and toward the downstream side.

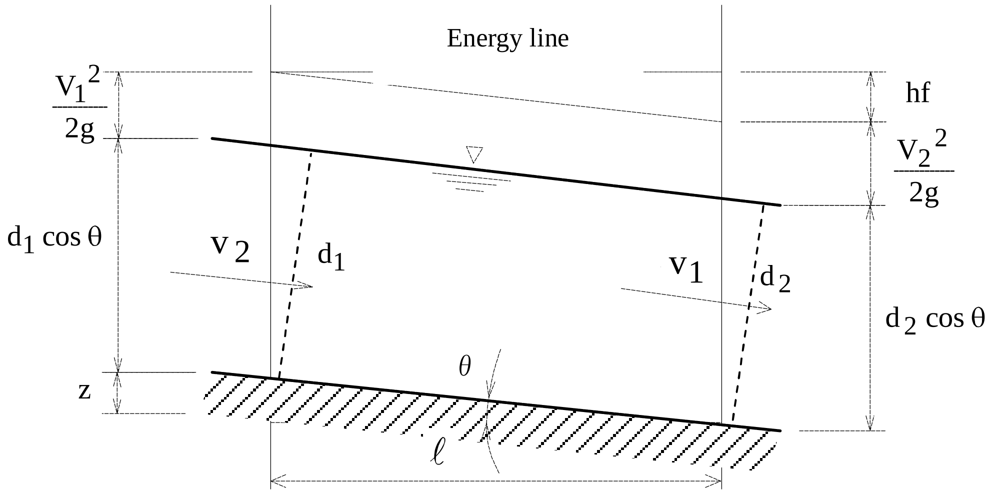


Figure 5.2.8 Conceptual Diagram of Hydraulic Calculation

Above mentioned diagram, Bernoulli's theorem is adopted in both sections.

$$d_1 \cos\theta + \frac{V_1^2}{2g} + z = d_2 \cos\theta + \frac{V_2^2}{2g} + hf$$

$$\mathrm{hf} = \frac{\mathrm{n}^2 \cdot \mathrm{Vm}^2}{\mathrm{Rm}^{4/3}} \times \Delta\ell$$

Vm, Rm: average velocity, average hydraulic mean depth both sections

n: coefficient of roughness

The water head drop of the calculation interval is he as follows:

$$h = d_1 \cos\theta + z - d_2 \cos\theta$$

$$h = \frac{V_2^{\ 2}}{2g} + hf - \frac{V_1^{\ 2}}{2g}$$

The water depth d_2 at the downstream side is done the trial calculation, which satisfies as mentioned above expressions. The calculation results are shown as follows:

Table 5.2.3 Water surface calculation results table of Chute Cannel

Station point	Section NO	Inteval distance ⊿X (m)	Reduce of channel bottom elevation (m)	Elevation of channel botom Zb (m)	Gradient degree θ (°)	Water height h (m)	Water elevation Zb+h/cosθ (m)	Discharge Q (m3/s)	Correction coefficient of energy α	Coefficient of roughness n	Water surface width B (m)	Flow velocity v (m/s)	Water flow area A (m2)	Hydraulic mean depth R (m)	Froude number Fr	Velocity head hv (m)	Fiction head loss hf (m)	Energy gradient E1 (m)	E1+hf at upstream side E2 (m)	Error E (m)	Judge
0	1	0.000	0.000	431.000	0	1.097	432.097	108.000	1.000	0.015	30.00	3.282	32.910	1.022	1.001	0.549	—	432.646	—	-	-
50.000	1	50.000	1.000	430.000	1.146	0.642	430.642	108.000	1.000	0.015	30.00	5.612	19.245	0.615	2.238	1.607	0.397	432.248	432.249	0.000	OK
100.000	1	50.000	1.000	429.000	1.146	0.593	429.593	108.000	1.000	0.015	30.00	6.069	17.796	0.571	2.517	1.879	0.776	431.472	431.472	0.000	OK
150.000	1	50.000	1.000	428.000	1.146	0.578	428.578	108.000	1.000	0.015	30.00	6.229	17.337	0.556	2.618	1.980	0.915	430.558	430.557	0.000	OK
200.000	1	50.000	1.000	427.000	1.146	0.573	427.573	108.000	1.000	0.015	30.00	6.287	17.178	0.552	2.654	2.017	0.969	429.589	429.589	0.000	OK
250.000	1	50.000	1.000	426.000	1.146	0.571	426.571	108.000	1.000	0.015	30.00	6.308	17.121	0.550	2.667	2.030	0.989	428.601	428.600	0.000	OK
300.000	1	50.000	1.000	425.000	1.146	0.570	425.570	108.000	1.000	0.015	30.00	6.316	17.100	0.549	2.672	2.035	0.996	427.605	427.605	0.000	OK
350.000	1	50.000	1.000	424.000	1.146	0.570	424.570	108.000	1.000	0.015	30.00	6.318	17.094	0.549	2.674	2.037	0.999	426.606	426.606	0.000	OK
		350.000																			

Water Surface Calculation at Design Discharge Flow

Design discharge----------------------------Qd = 108.0 (m^3/sec)

Critical depth at control point-------------dc = 1.097 (m)

$$dc = 0.467 \cdot q^{2/3} = 0.467 \times (Qd / B)^{2/3}$$

$$= 0.467 \times (108.0 / 30)^{2/3} = 1.097 \text{ m}$$

Freeboard of Chute Channel

The freeboard at the connection channel of the subcritical flow is calculated as followsôexpression.

$$Fb = 0.6 + 0.037V \cdot d^{1/3}$$

Where;

Fb: channel freeboard (m)

V: velocity

d: water depth at flood flow (m)

The freeboard is calculated with above mentioned expression, and the vertical wall height (H) of the channel is calculated as follows:

$$H = (d + Fb) \times \frac{1}{\cos\theta}$$

θ : gradient degree of channel bottom

The calculation results of Fb and H with the channel wall height at the flood discharge flow are as follows:

Table 5.2.4 Calculation Results of Freeboard and Wall Height

Station point	Water height	Velocity	Freeboard	Calcularion of wall height	Gradient	Gradient	Adoption of wall height	Channel bottom elevation	Channel crest elevation	Remark
	d	V	Fb	Hc	cos ɗ		Ha	Eb	Eh	
	(m)	(m)	(m)	(m)			(m)	(m)	(m)	
0.000	1.097	3.282	0.725	1.82	1.00	1.82	2	431.000	433	End point at connection portion
50.000	0.642	5.612	0.779	1.42	1.00	1.42	2	430.000	432	
50.000	0.593	6.069	0.789	1.38	1.00	1.38	2	429.000	431	
50.000	0.578	6.229	0.792	1.37	1.00	1.37	2	428.000	430	
50.000	0.573	6.287	0.793	1.37	1.00	1.37	2	427.000	429	
50.000	0.571	6.308	0.794	1.36	1.00	1.36	2	426.000	428	
50.000	0.570	6.316	0.794	1.36	1.00	1.36	2	425.000	427	
50.000	0.570	6.318	0.794	1.36	1.00	1.36	2	424.000	426	Beginnig point at connection portion

Hydraulics of Stilling Basin

The purpose of the stilling basin is prevention that the main dam, spillway structure, the downstream side river and other relation structures are collapsed and eroded by the high energy with high velocity flow. It is necessary to return from supercritical flow which flows down the chute channel with high velocity to subcritical flow and to dissipate the high energy of the flood flow.

The stilling basin of fill dam is generally often adopted hydraulic Jump type stilling basin which is said that the method of the energy dissipater is most complete.

The hydraulic jump type stilling basin are designed as the variety types according to the relations between hydraulic Jump water curves and water level and flow curves at downstream river. There are many case that the water level and flow curves at the downstream river is lower side than the hydraulic jump water level and then at this case the adopted types are as follows:

Stilling basin is set at lower portion with excavation.

End-sill dissipater is set at energy dissipater.

Forced hydraulic jump type dissipater is adopted.

a) Energy DissipaterType

The generally types of the energy dissipater are as followsôtable.

Table 5.2.5 Dissipater Type

Type	Dissipater	Select point
Hydraulic Jump type dissipater	It dissipates by using hydraulic jump effect.	If it keeps water height at downstream river, which is higher than hydraulic jump height, this type can be adopted. It is used most commonly.
Impact dissipater	It dissipates by impact and disturbance of flow to baffle wall.	This type is adopted at high head comparatively.
Drop head dissipater	Forced hydraulic type, impact block type, slot grating type etc.	It can be adopted, if the flow is dropped by the free fall from reservoir to diversion channel.

Forced Hydraulic Jump Type

The water depth, water velocity, Froude number and hydraulic jump height are calculated at flood discharge that flows down to dissipater bottom elevation EL. 424.0 m.

Froude Number $Fr = \dfrac{V_1}{(g \cdot d_1)^{1/2}}$

Hydraulic jump height $d_2 = \dfrac{1}{2} \times d_1 \times ((1 + 8 \cdot Fr^2)^{1/2} - 1)$

Where; V_1: Inflow velocity (m/sec)

d_1: Inflow depth (m)

d_2: Water depth at flood flow (m)

g: Acceleration of gravity (m/sec2)

Table 5.2.6 Specification of Inflow to Dissipater

Flood discharge	Dissipate bottom elevation (m)	Inflow depth d_1 (m)	Inflow velocity V_1 (m/sec)	Froude number Fr	Hydraulic jump height d_2 (m)
Design discharge Qd Qd = 108.0 m^3/s	E.L. 424.0	0.57	6.318	2.674	1.889

Energy Dissipater Specification

(i) Dissipater Basin Length

The dissipater basin length is calculated as followsôexpression.

$L = a \cdot d_2$

Where;

L: Length of dissipater basin (m)

a: Coefficient (= 6)

d_2: Hydraulic jump height (m)

d2 = 1.889, a = 6 then,

$\therefore L = 6 \times 1.889 = 11.33$ m < 20 (m)

Therefore the dissipater basin length is decided as L = 20 (m) on the safety side.

(ii) Height of End-sill Dissipater

The height of end-sill dissipater is calculated with Iwasaki formula.

$$\frac{W}{d_1} = \frac{(1+2\cdot Fr^2)\cdot(1+8\cdot Fr^2)^{1/2} - 1 - 5\cdot Fr^2}{1+4\cdot Fr^{1/2} - (1+8\cdot Fr^2)^{1/2}} - (\frac{\sqrt{g}}{C}\cdot Fr)^{2/3}$$

Where;

W: height of end-sill (m)

Fr: Froude number (= 2.674)

d1: water depth before hydraulic jump (= 0.57 m)

v1: water velocity before hydraulic jump (= 6.318 m)

C: discharge coefficient of end-sill (= 2)

g: acceleration of gravity (= 9.8 m/sec^2)

Each values are stimulated and calculated, then W = 0.594 m < 1.0 m

Therefore the height of the end-sill is decided as 1.0 m.

The elevation of the end-sill is as EL. 424.0+1.0 = EL. 425.0 m

(iii) Wall Height of Dissipater Basin

The freeboard of the dissipater basin is calculated as followsôexpression.

$Fb = 0.1 \times (V_1 + d_2)$

Where;

F: freeboard (m)

V_1: inflow velocity (m/sec)

d_2: hydraulic jump height (m)

$Fb = 0.1 \times (6.318 + 1.889)$

$= 0.82$ m

The height of the dissipater basin is as follows:

$H = d_2 + Fb = 1.889 + 0.82 = 2.709\ (m) < 3.0\ m$

The height of the dissipater basin is decided as H = 3.0 (m).

The elevation of the dissipater basin is EL. 424.0＋3.0 = EL. 427.0 (m)

2.6 Design of Intake

The intake facilities consist of intake section, regulating section and tailrace. The most adequate arrangement of facilities should be made in order to facilities timely intake of required volume. The intake portion is an inclined conduit, intake tower or other structure for the intake of water from the reservoir. The regulating portion may be a gate, valves or other facility to regulate intake volume, and is located at the inlet.

2.6.1 General

The intake facility is designed to take safely the maximum intake volume and release safely maximum outlet discharge. The facility at Wau should be designed to secure the condition of intake volume and the outlet discharge as follows:

1) The maximum intake volume is Q = 0.53 m^3/sec and to the main canal.
2) It is unnecessary to warm water of the intake, because the main purpose of the intake facility is to divert water to the main canal.
3) It should be released the emergency outlet discharge for the dam safety. (The water level of the reservoir from F.L.W. to the empty should be reduced for 7 days or 10 days.)
4) The diversion under construction should be used with the existing river.
5) Sedimentation of soil and gravel in the reservoir should be flowed out to the downstream side.

2.6.2 Specification of Intake

The specifications of Wau dam are as follows:

1) Dam crest level	E.L. 436.7 m
2) High water level	H.W.L. 434.8 m
3) Full water level	F.W.L. 433.8 m
4) Low water level	L.W.L. 427.0 m
5) Capacity	5,300,000 m^3
6) Maximum Intake Discharge	0.53 m^3/sec
7) Emergency Release Discharge	7.7 m^3/sec

2.6.3 Type of Intake

The types of intake are mainly divided into inclined conduit and intake tower or drop inlet and the type of waterway are also mainly divided into tunnel and conduit.

The type of the intake at Wau dam is selected as the type of the drop inlet, because the type of the inclined conduit has long pipe line and spindle of gates along dam slope and this type is more costly than the drop inlet.

The type of the waterway is selected as penstock pipe under dam, because the construction cost is lower than the tunnel type.

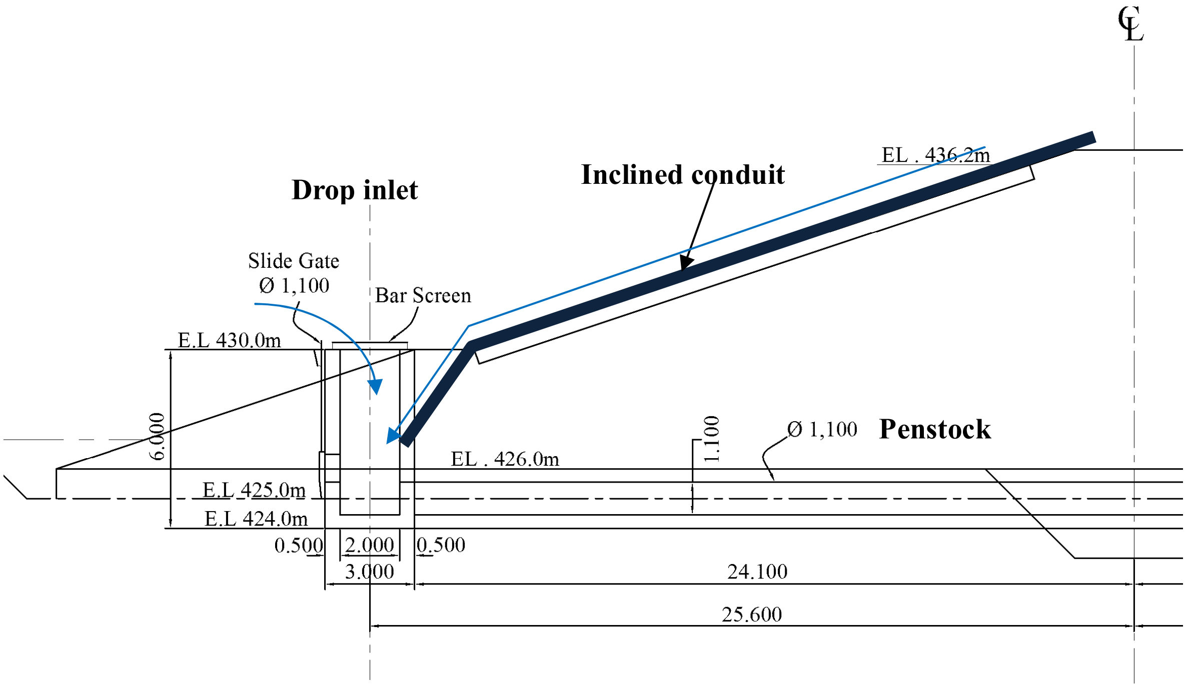


Figure 2.6.1 Type of Intake

2.6.4 Emergency Dicharge

It should be released the emergency outlet discharge for the dam safety. The water level of the reservoir from F.L.W. to the empty should be reduced for 7 days or 10 days. The emergency discharge utilize with the intake facility.

1) Full water level	F.W.L. 433.8 m
2) Low water level	L.W.L. 427.0 m
3) Water Height	6.8 m (= F.W.L. 433.8 m - L.W.L. 427.0 m)
4) Capacity	5,300,000 m^3
5) Outlet level	E.L. 425.0 m

Calculation of Discharge

The discharge from gate installed at end of pen stock pipe line is calculated with the equation as follows:

$$Q = CA\sqrt{2g(H - hf)} \quad \cdots\cdots ①$$

where, Q: discharge (m^3/sec)

C: discharge coefficient

g: acceleration of gravity (= 9.8 m/sec^2)

H: Water head (m)

hf: head of loss (m)

The expression of hf is as follows:

$$
\begin{aligned}
hf &= h_1 + h_2 + h_3 \cdots\cdots + h_n = \sum h_i \\
&= f_1 \frac{v_1^{\,2}}{2g} + f_2 \frac{v_2^{\,2}}{2g} + f_3 \frac{v_3^{\,2}}{2g} + \cdots\cdots + f_n \frac{v_n^{\,2}}{2g} \\
&= f_1 \frac{1}{2g}\left(\frac{Q}{A_1}\right)^2 + f_2 \frac{1}{2g}\left(\frac{Q}{A_2}\right)^2 + f_3 \frac{1}{2g}\left(\frac{Q}{A_3}\right)^2 + \cdots\cdots + f_n \frac{1}{2g}\left(\frac{Q}{A_n}\right)^2 \quad (V^2 = \frac{Q^2}{A^2}) \\
&= \frac{Q^2}{2g}\left(\frac{f_1}{A_1^{\,2}} + \frac{f_2}{A_2^{\,2}} + \frac{f_3}{A_3^{\,2}} + \cdots\cdots + \frac{f_n}{A_n^{\,2}}\right) \\
&= \frac{Q^2}{2g}\sum \frac{f_i}{A_i^{\,2}}
\end{aligned}
$$

where, hi: head of loss at each part (m)

fi: coefficient of loss at each part

vi: velocity at each part (m/sec)

$Q = CA\sqrt{2g(H - hf)} \cdots$ ① is substituted by $hf = \frac{Q^2}{2g}F(F = \sum \frac{f_i}{A_i^{\,2}})$ and

$$Q = CA\sqrt{2g(H - \frac{Q^2}{2g}F)} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{②}$$

The equation ② is developed and

$$Q^2 = \frac{2C^2A^2gH}{1 + C^2A^2F} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{③}$$

$$Q = \sqrt{\frac{2C^2A^2gH}{1 + C^2A^2F}} = CA\sqrt{\frac{2gH}{1 + C^2A^2F}} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{④}$$

Depending the equation ③, the total head of loss at each part is calculated and check the emergency discharge.

Total Head of Loss (Diameter ϕ = 1.1 m)

The total head of loss is calculated with the equation $F = \sum \frac{f_i}{A_i^2}$ as follow's table:

Table 2.6.1 Total Head of Loss (ϕ = 1.1 m)

No	Loss	Position	Ai	Ai^2	fi	fi/Ai^2
①	Screen	Intake screen	4.00	16.00	0.0379	0.002
②	In folw	Intake	0.95	0.90	0.5000	0.554
③	Pipe loss ű1100	Pen stock pipe	0.95	0.90	0.9473	1.049
④	Slide Valve	Valve	0.95	0.90	0.0600	0.066
Total						1.671

The total head of loss $F = \sum \frac{f_i}{A_i^2}$ is 1.671 by the calculation table 6.4.1.

Discharge Volume and Time

The discharge volume and time are calculated by $Q = CA\sqrt{\frac{2gH}{1+C^2A^2F}}$ and F = 1.671 and the other values.

1) Outlet level W.L. 425.0 m
2) Coefficient C C = 1 (open valve)
3) Total head of loss F F = 1.671
4) Penstock Diameter ű=1.1 m, A = 0.95033 m^2

Table 2.6.2 Discharge Volume and Time (ϕ = 1.1 m)

Water level (EL)	Reservoir capacity (m^3)	Capacity (m^3)	Average water level (m)	Average water depth (m)	Discharge Q (m^3/s)	Discharge time (min)	Discharge time (Day, hours, minutes)
433.8	5,300,000						
		1,430,400	**433.400**	8.400	**7.698**	3097.1	2 days 3 hours 37 minutes
433.0	3,869,600						
		1,460,000	**432.500**	7.500	**7.274**	3345.5	2 days 7 hours 45 minutes
432.0	2,409,600						
		1,063,500	**431.500**	6.500	**6.771**	2617.7	1 day 19 hours 37 minutes
431.0	1346100.0						
		708,500	**430.500**	5.500	**6.229**	1895.8	1 day 7 hours 35 minutes
430.0	637600.0						
		420,000	**429.500**	4.500	**5.634**	1242.4	0 day 20 hours 42 minutes
429.0	217600.0						
		175,500	**428.500**	3.500	**4.969**	588.7	0 day 9 hours 48 minutes
428.0	42100.0						
		42,100	**427.500**	2.500	**4.199**	167.1	0 day 2 hour 47 minutes
427.0	0.0						
		5,300,000				12954.2	8 days 23 hours 54 minutes

The discharge volume and time are determined by Table 6.4.2 as the discharge Q_{max} = 7.698 m^3/s ≒ 7.7 m^3/s and the discharge time T≒9 days < 10 days.

Case of Diameter ϕ = 1 m of Pen stock Pipe

The total head of loss is calculated with the equation $F = \sum \frac{f_i}{A_i^2}$ as follow's table:

Table 2.6.3 Total Head of Loss (ϕ = 1.0 m)

No	Loss	Position	Ai	Ai^2	fi	fi/Ai^2
①	Screen	Intake screen	4.00	16.00	0.0379	0.002
②	In folw	Intake	0.79	0.62	0.5000	0.811
③	Pipe loss	Pen stock pipe	0.79	0.62	1.0757	1.744
④	Slide Valve	Valve	0.79	0.62	0.0600	0.097
Total						2.654

The discharge volume and time are calculated by $Q = CA\sqrt{\frac{2gH}{1+C^2A^2F}}$ and F = 1.671 and the other values.

1) Outlet level W.L. 425.0 m
2) Coefficient C C = 1 (open valve)
3) Total head of loss F F = 2.654
4) Penstock Diameter ű=1.0 m, A = 0.7854 m^2

Table 2.6.4 Discharge Volume and Time (ϕ = 1.0 m)

Water level (EL)	Reservoir capacity (m^3)	Capacity (m^3)	Average water level (m)	Average water depth (m)	Discharge Q (m^3/s)	Discharge time (min)	Discharge time (Day, hours, minutes)
433.8	5,300,000						
		1,430,400	**433.400**	8.400	**6.206**	3841.6	2 days 16 hours 01 minutes
433.0	3,869,600						
		1,460,000	**432.500**	7.500	**5.864**	4149.7	2 days 21 hours 09 minutes
432.0	2,409,600						
		1,063,500	**431.500**	6.500	**5.459**	3247.0	2 day 6 hours 06 minutes
431.0	1346100.0						
		708,500	**430.500**	5.500	**5.021**	2351.6	1 day 15 hours 11 minutes
430.0	637600.0						
		420,000	**429.500**	4.500	**4.542**	1541.1	1 day 1 hours 41 minutes
429.0	217600.0						
		175,500	**428.500**	3.500	**4.006**	730.2	0 day 12 hours 10 minutes
428.0	42100.0						
		42,100	**427.500**	2.500	**3.385**	207.3	0 day 3 hour 27 minutes
427.0	0.0						
		5,300,000				16068.5	11 days 3 hours 48 minutes

The discharge volume and time are determined by Table 6.4.2 as the discharge Q_{max} = 6.206 m^3/s ≒ 6.2 m^3/s and the discharge time T≒11days > 10 days.

The penstock pipe (ű=1.0 m) is not available for the emergency discharge of less than 10 days and the diameter of the penstock pipe is determined as ű=1.1 m.

2.6.5 Maximum Intaku Discharge for Irrigation Area

The maximum intake discharge for the irrigation area is calculated as Q_{max} = 0.53 m^3/s and the water level at the beginning point of the main canal is designed as W.L. 427.0 m.

1) Water level at main canal W.L. 427.0 m
2) Reservoir water depth 0.45 m
3) Reservoir water level W.L. 427.45
4) Maximum intake discharge Q = 0.53 m^3/s
5) Pipe diameter Main pipe (penstock) ű=1.1 m, Supply pipe ű=0.7 m

The total head of loss is calculated with the equation $F = \sum \frac{f_i}{A_i^2}$ as follow's table:

Table 2.6.5 Total Head of Loss (ϕ = 1.1m ~ 0.7 m)

No	Loss	Position	Ai	Ai^2	fi	fi/Ai^2
①	Screen	Intake screen	4.00	16.00	0.0379	0.002
②	In folw	Intake	0.95	0.90	0.5000	0.554
③	Pipe loss ű1100	Pen stock pipe	0.95	0.90	0.9473	1.049
④	Pipe loss ű700	Supply pipe	0.38	0.15	2.8845	19.476
⑤	ű1100 - ű700	Distributary	0.95	0.38	3.1922	3.535
⑥	Slide Valve	Valve	0.38	0.15	0.0600	0.405
Total						25.020

The discharge volume and time are calculated by $Q = CA\sqrt{\frac{2gH}{1+C^2A^2F}}$ and F = 25.02 and the other values.

1) Intake level W.L. 427.5
2) Outlet level W.L. 427.0 m
3) Coefficient C C = 1 (open valve)
4) Total head of loss F F = 25.02
5) Pipe Diameter ű=1.1 m ~ 0.7 m, A = 0.38485 m^2
6) Water depth H = 0.5 m

$$Q = CA\sqrt{\frac{2gH}{1+C^2A^2F}} = 1\times 0.38485\times\sqrt{\frac{2\times 9.8\times 0.5}{1+1^2\times 0.38485^2\times 25.02}} = 0.555 \ \ m^3/s > 0.53\ m^3/s$$

The discharge is determined by Table 6.4.2 as the discharge Q_{max} = 0.555 m^3/s > 0.53 m^3/s which is the maximum intake discharge for the irrigation area. The supply pipe (ű= 0.7 m) is available for the irrigation.

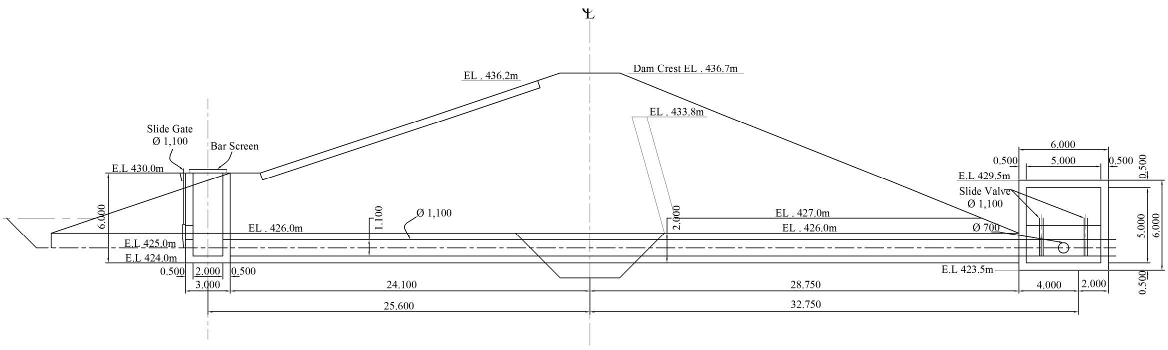


Figure 2.6.32 Longitudinal Section of Intake

CHAPTER 3 PUMP STATION

3.1 Location

The location of pump station should be selected at the safe place, considering the area of the floodplain. The maximum water level of River Jur seems to rise to about WL. 424m at the pump site based on the record of Wau gauge station, which is located about 7km downstream from the pump station.

The pump station shall be built from 50m far from the river at the ground elevation of more than EL. 424m shown in the below figure 3.1.1, and the connection channel shall be planned to conduct the river water to the pump station stably.

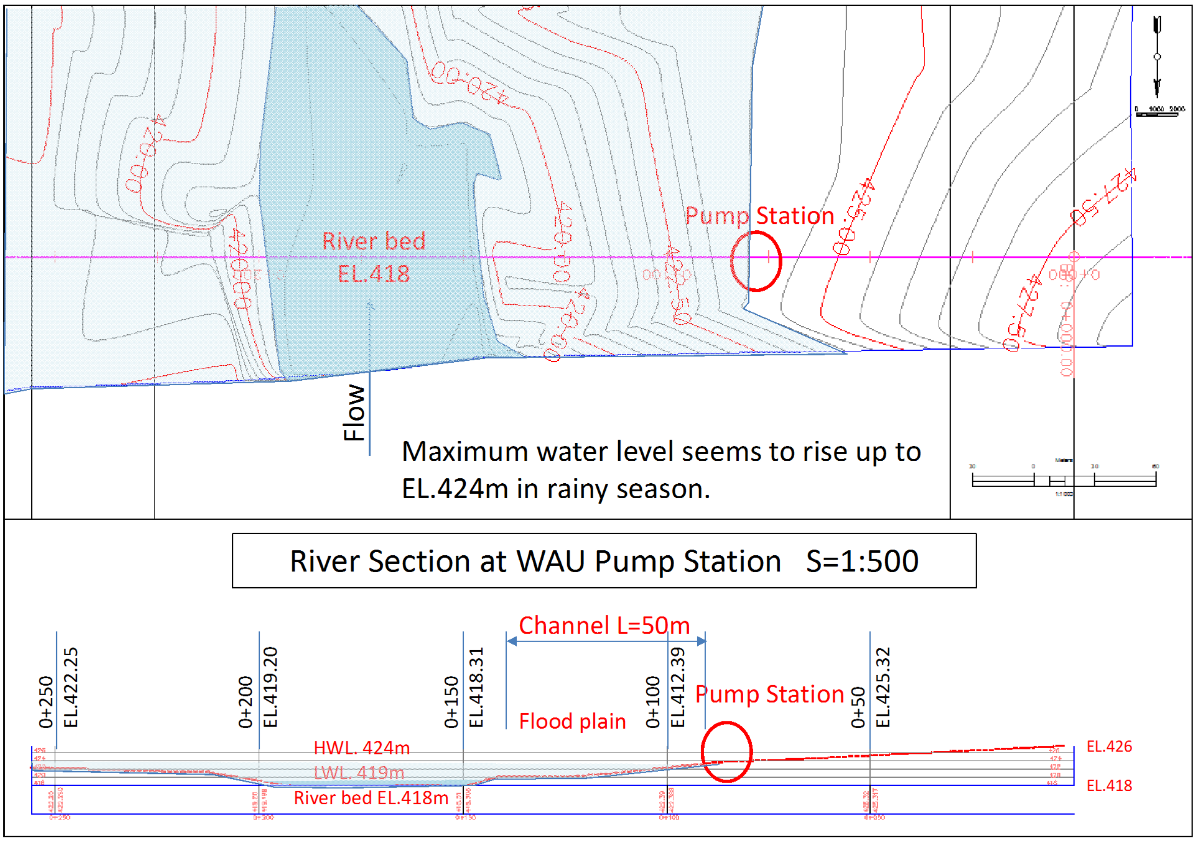


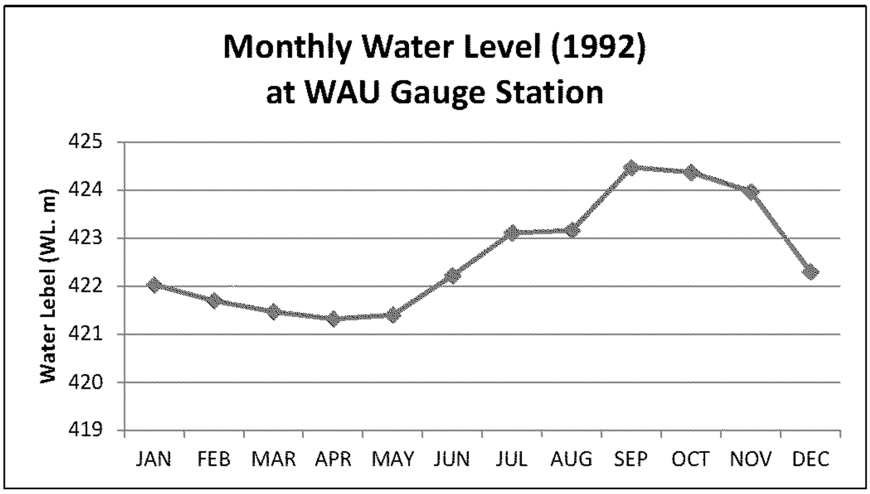
Figure 3.1.1 Location of Pump Station

Water Level of River Jur

The elevation of river bed at the pump station is measured at EL.418.0m in the survey, however, the water level of River Jur was not measured at that time.

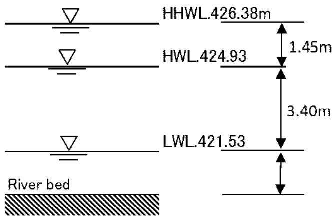
According to the water level record at Wau gauge station which is far from about 7km downstream from the pump station, the water level of River Jurl has been fluctuating throughout the year. Also the range of water level has is observed among 4.85m of water depth as shown in the Figure 3.1.2.

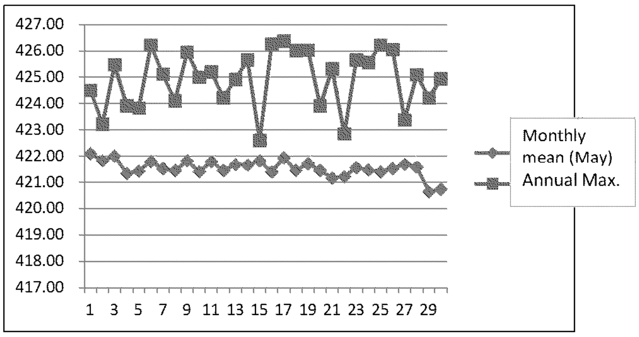
When Lowest Water Level (LWL) would be assumed at WL.419.0m at the pump station, High Water Level (HWL) is determined WL.424.0m in assuming 5.0m of fluctuation.



WAU Gauge Station: (Zero elevation: EL.408.338m)

No.	Year	Monthly Average in May (LWL.m)	Annual Max. (HWL.m)	Remarks
1	1983	422.10	424.50	
2	1984	421.82	423.23	
3	1985	422.01	425.48	
4	1986	421.33	423.92	
5	1987	421.43	423.83	
6	1988	421.79	426.22	4th
7	1989	421.52	425.12	
8	1990	421.44	424.11	
9	1991	421.82	425.96	
10	1992	421.40	425.00	
11	1993	421.79	425.21	
12	1994	421.45	424.23	
13	1995	421.67	424.92	
14	1996	421.66	425.66	
15	1997	421.82	422.60	
16	1998	421.39	426.27	2nd
17	1999	421.94	426.38	1st (HHWL)
18	2000	421.45	426.02	
19	2001	421.71	426.04	
20	2002	421.45	423.92	
21	2003	421.16	425.32	
22	2004	421.21	422.86	
23	2005	421.56	425.66	
24	2006	421.47	425.56	
25	2007	421.39	426.23	3rd
26	2008	*421.53*	426.06	5th
27	2009	421.69	423.38	
28	2010	421.58	425.10	
29	2011	420.64	424.22	
30	2012	420.74	424.94	
	Mean	**421.53**	**424.93**	difference 3.40 m





3

Figure 3.1.2 Flow of River Jur at Wau gauge station

3.2 Connection Channel

The purpose of connection channel is to conduct the river water to the pump station stably.

- The design discharge: Q=0.70m3/s (the same as pump capacity)
- Structure type: Gabion wall (height 0.5 m~ 4.0m), and reinforced concrete (height 4.0m ~ 6.0m)
- Foundation of structure should be studied in the future design stage because of no geological data at the site.

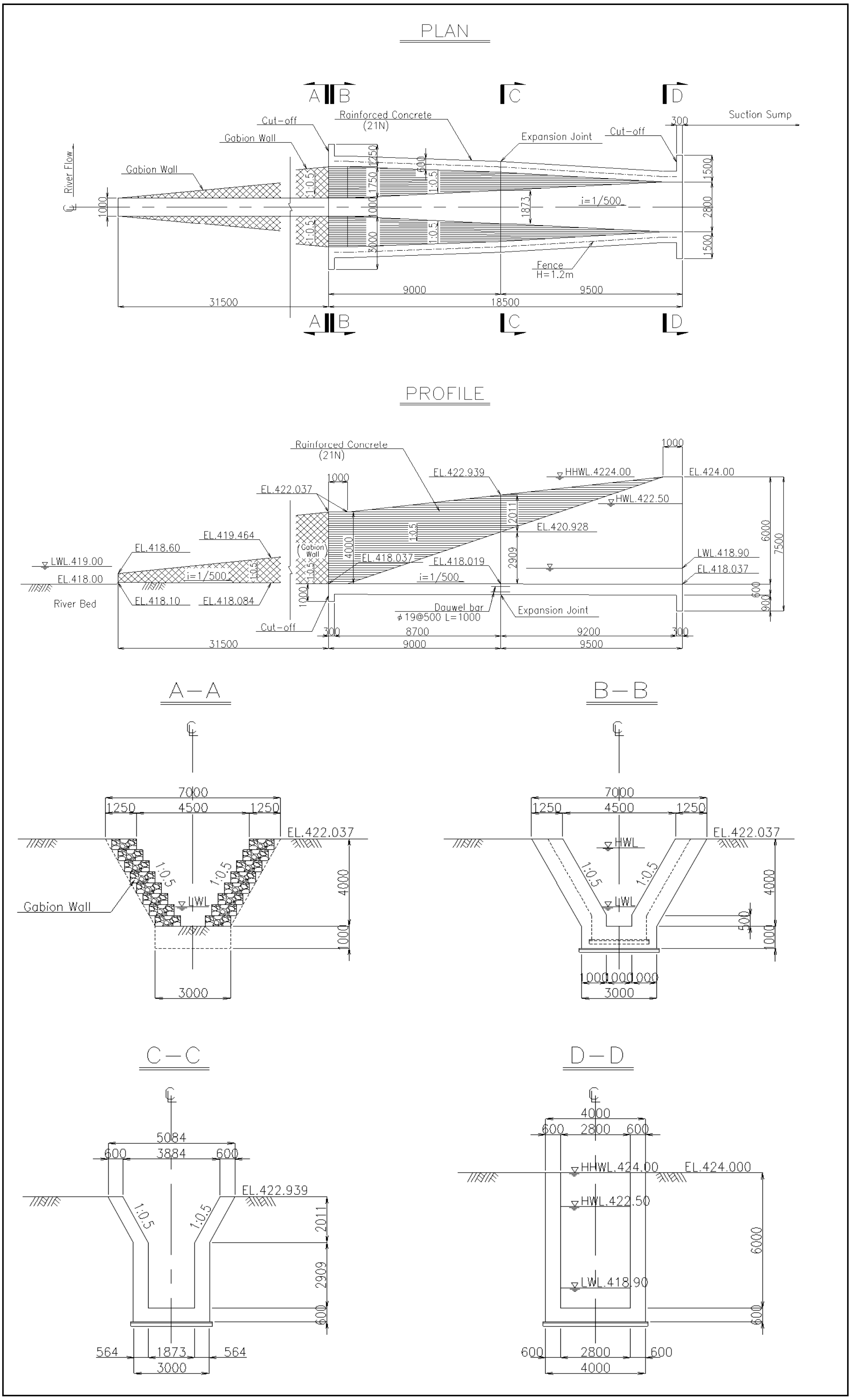


Figure 3.2.1 Connection Channel

3.3 Suction Sump

The shape of suction sump shall be avoid to generate the whirlpoor in the sump. The screen shall be installed to avoid the inflow the float dust and grasses, etc. to the pump inside.

The standard shape of suction sump stipulated in ñDesign Pump Facilities Technical Document (Japan)ò is shown in the Table 3.3.1 and Figure 3.3.1.

Table 3.3.1 Dimension of Suction sump

Discharge Q (m3/min)	ϕ D(mm)	W	E	F	G	R
1.8＜Q≦3.0	150	900	≧500	250	250	450
3.0＜Q≦5.0	200	900	≧500	250	300	600
5.0＜Q≦8.0	250	900	≧500	250	350	750
8.0＜Q≦12.0	300	900	≧600	300	400	900
12.0＜Q≦18.0	350	1050	≧700	350	450	1050
18.0＜Q≦23.0	400	1250	≧750	400	500	1200
23.0＜Q≦28.0	450	1350	≧850	450	550	1350
28.0＜Q≦36.0	500	1500	≧900	500	600	1500
36.0＜Q≦50.0	600	1800	≧1100	600	700	1800
50.0＜Q≦70.0	700	2100	≧1300	700	800	2100
70.0＜Q≦90.0	800	2400	≧1400	800	900	2400
90.0＜Q≦115	900	2700	≧1600	900	1000	2700

Note: in case of $\theta \leqq 30^\circ$ ・・・ L=3D, and $30^\circ < \theta \leqq 45^\circ$ ・・・ 4.5D

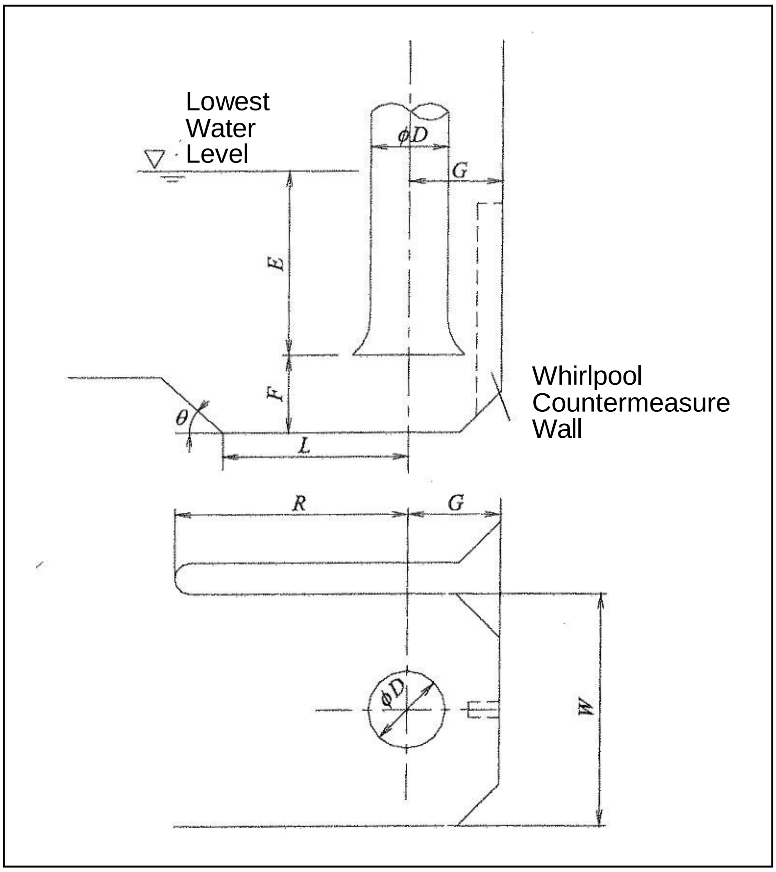


Figure 3.3.1 Shape of Suction Sump

Based on the considerations as above, the plan and section of suction sump are planned as shown in the Figure 3.3.2.

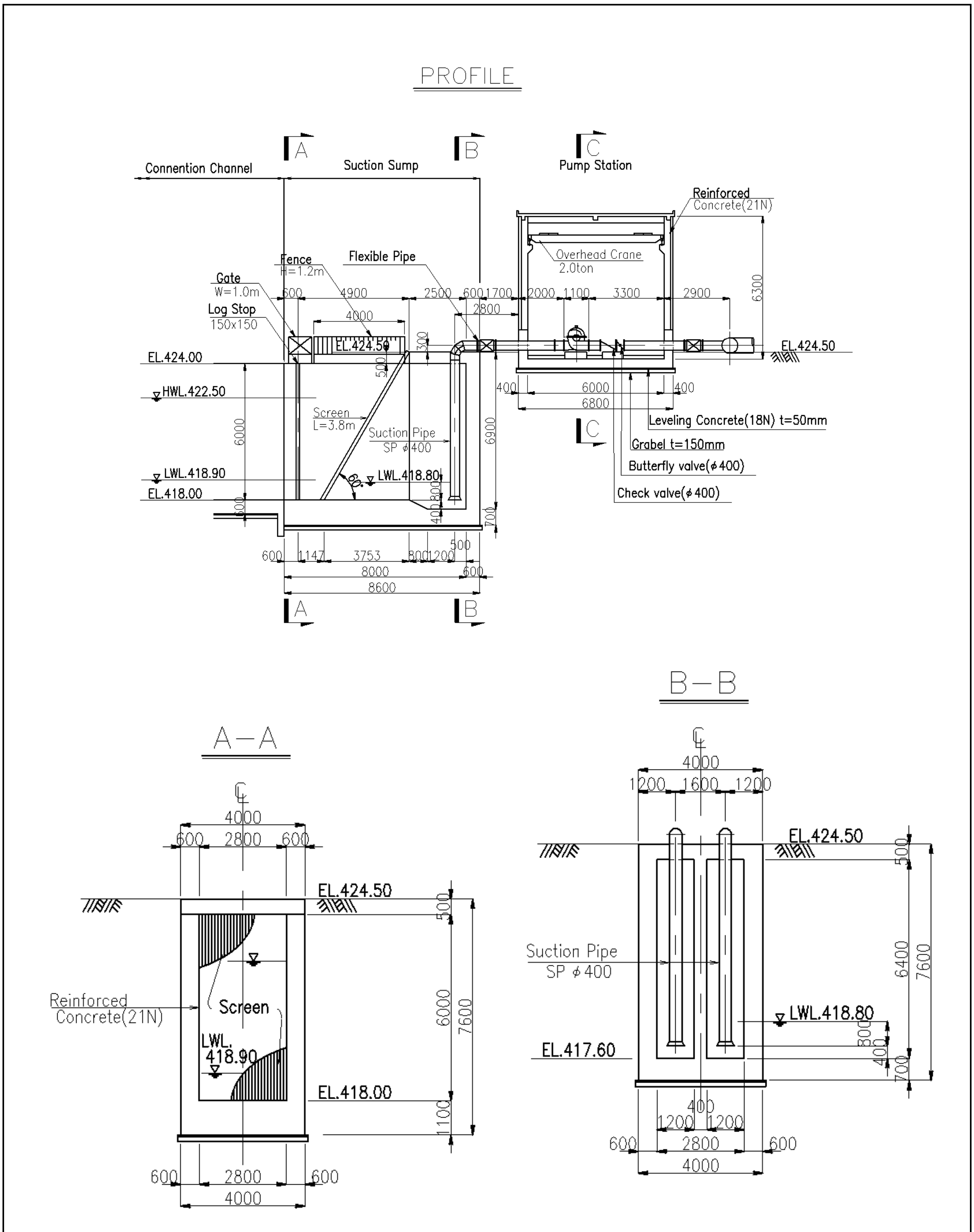


Figure 3.3.2 Suction Sump

3.4 Pump Facilities

(1) Pump type and number of pump

For the pump type, the horizontal centrifugal and double suction is adopted as it is commonly used with high suction efficiency.

The unit capacity (Discharge) of per pump varies depending on the planned number of pumps to be equipped for a scheme. In order to operate the pumps effectively and to minimize the running cost in conformity with the fluctuating supply demands, a combination of pumps with different capacities can be considered possible, however, it is judged to be more advantageous to apply a certain number of pumps with the same capacity taking into such viewpoints as 1) reducing of pump procurement cost, 2) possible equalization in running pumps and 3) need for harmonious collaboration of pump operation with the pump equipment.

As for the discharge control by pumps, the most simple, common and effective manner by the numbers of pumps run shall be employed. The manner has been practiced for a considerable period with which much fluctuating monthly water demands can be managed by adjusting the operation hours of pumps in addition to the control on the number of units run. In this case, the more the number of pumps, with higher efficiency the pumps can be operated to meet the fluctuating demands. However, this is not always the effective case due to the larger requirement of land space for the station and further causing more complicated piping works leading to higher construction cost as well as land acquisition cost.

Therefore the two (2) same capacity pumps are planned to provide at the site.

$$0.35m^3/s \text{ (unit capacity)} \times 2 \text{ set} = 0.70\ m^3/s$$

Table 3.4.1 Water Requirement

Month	May	Jun.	Jul.	Aug	Average
Water Requirement（m^3/s）	0.35	0.70	0.55	0.46	0.50

(2) Total head of pump

1) Designed water level for pump (Suction and discharge)

The suction water level for pump is determined based on the water levels of River Jur. The pump operation is planned to begin from May in accordance with farming plan, and the planned suction water level shall be fixed based on the record of water level in May at the site. On the other hand, the planned pump discharge level is to be fixed with the high water level in the irrigation canal which is obtained from the site survey result.

2) Actual head

The actual head is given as the difference between the discharge water level and the suction water level and calculated as in the followings.

Calculation of actual head

Ha = DWL ï LWL

Where,	Ha	:	Actual head (m)
	DWL	:	Discharge water level (m)
	LWL	:	Suction water level (m)

3) Calculation of total head

The total head is obtained by adding various losses in pipes to the actual head and calculated by using the following formula.

Calculation formula for the total head

$$H = Ha + H1 = (DWL-LWL) + hf + fn \cdot V^2/2g$$

Where,	H	:	Total head (m)
	Ha	:	Actual head
	H1	:	Total head loss (m)
	DWL	:	Discharge water level (m)
	LWL	:	Suction water level (m)
	hf	:	Friction head loss of pipes (m)
	fn	:	Coefficient of various friction loss
	V	:	Velocity (m/s)
	G	:	Gravity acceleration (m/s^2) = 9.8 (m/s^2)

Friction Loss Calculation of the pipe aligned in the pump station by Darcy・Weisbach

$$h_f = \lambda \cdot (L/D) \cdot V^2/2g \quad \cdots\cdots\cdots\cdots \text{ Darcy・Weisbach formula}$$

λ	:	Coefficient of friction；normal steal pipe $\lambda = \{0.0144+9.5/(1000 \cdot \sqrt{V})\} \cdot 1.5$
L	:	Length of pipe (suction & discharge) (m)
D	:	Pipe Diameter corresponding to Pipe Length L (m)

Friction Loss Calculation of the pipe aligned at outside of the pump station by Hazen・Williams

$$h_f = 10.666 \cdot \{Q^{1.85}/(C^{1.85} \cdot D^{4.87})\} \cdot L \quad \cdots\cdots \text{ Hazen・Williams}$$

Q	:	discharge (m^3/s)
C	:	Velocity Coefficient; Steal Pipe (No Coating） C=100

D : Diameter (m)

L : Length of Pipeline (m)

The calculation results of pipe losses around the pump and the total head are as shown in the Table 3.4.2.

Table 3.4.2 Pipe losses and total head of each station

Site		Unit	Wau	Remarks
Pump capacity		(m^3/s)	0.70	provided with 2 pumps
1.Actual head (ha)	Design Intake water level	LWL(m)	418.8	
	Design outlet water level	DWL(m)	426.2	
	Actual head (ha)	(m)	7.40	
2.Friction head loss				
(1)Suction pipe	Q	(m^3/s)	0.350	per pump
	Pipe		Steel	
	Diameter(D)	(mm)	400	
	Length(L)	(m)	12.0	
	Flow coefficient(C)		100	
	Water velocity(V)	(m/s)	2.79	$V=Q/(\pi/4\cdot(D/1000)^2)$, $0.3m/s \leqq V \leqq 2.0m/s$
	Friction head loss(fs)	(m)	0.32	$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(2)Delivery pipe1	Q	(m^3/s)	0.350	
	Pipe		Steel	
	Diameter(D)	(mm)	400	
	Length(L)	(m)	7.0	
	Flow coefficient(C)		100	
	Water velocity(V)	(m/s)	2.79	
	Friction head loss(fs)	(m)	0.19	$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(3)Delivery pipe2	Q	(m^3/s)	0.700	
	Pipe		Steel	
	Diameter(D)	(mm)	700	
	Length(L)	(m)	150.0	
	Flow coefficient(C)		100	
	Water velocity(V)	(m/s)	1.82	$V=Q/(\pi/4\cdot(D/1000)^2)$, $0.3m/s \leqq V \leqq 2.0m/s$
	Friction head loss(fs)	(m)	0.94	$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(4)Total Friction loss		(m)	1.45	
3.Partial head loss				
(1)Check valve		(Nos.)	2	
	Diameter(D)	(mm)	400	
	Water velocity(V)	(m/s)	2.79	
	Coefficient of valve loss(fcv)		0.96	
	Check valve loss(hcv)	(m)	0.76	$hcv=fcv\cdot V^2/2g$
(2)Sluice valve		(Nos.)	2	
	Diameter(D)	(mm)	400	
	Water velocity(V)	(m/s)	2.79	
	Coefficient of valve loss(fsv)		0.44	
	Sluice valve loss(hsv)	(m)	0.35	$hsv=fsv\cdot V^2/2g$
(3)90°elbow		(Nos.)	2	
	Diameter(D)	(mm)	400	
	Water velocity(V)	(m/s)	2.79	
	Coefficient of elbow loss(fbe)		1.10	
	90°elbow loss(hbe)	(m)	0.87	$hbe=fbe\cdot V^2/2g$
(4)T Interflow		(Nos.)	1	
	Diameter(D)	(mm)	700	
	Water velocity(V)	(m/s)	1.82	
	Coefficient of elbow loss(f13)		0.65	
	T Interflow loss(hbe)	(m)	0.11	$h13=f13\cdot V^2/2g$
(5)Remnant head	Diameter(D)	(mm)	700	
	Water velocity(V)	(m/s)	1.82	
	Coefficient of head loss(fo)		1.00	
	Remnant velocity head(Lo)	(m)	0.17	
(6)Total parcial los	Particial head loss(Lp)	(m)	2.26	
4.Head loss(hf)	Total	(m)	3.71	
5.Total head(H)	H=ha+hf	(m)	11.11	
6.Design total head(H)		(m)	12.00	

(3) Rating point of pumps

The rating point for planning of pump is to be fixed in a way that the designed discharge will flow by the maximum pump lift within the actual pump operation range.

Table 3.4.3 Rating point of pumps

Planned Discharge of Pump（m^3/s/unit）	0.35
Planned Total head(m)	12.0

(4) Number of revolution, installation position and design point of pumps

For the pump facilities, in future design stage, the examination is necessary to design the facilities to be operated safely against the possible cavitation in any range of pump running through analyzing varieties of pump installation positions, number of revolutions and design points in all cases.

(5) Pump shaft power and planned diesel engine output

No electricity is in the pump station site. Therefore the diesel engine is adapted for the pump operation. The pump shaft power required can be calculated with the following formula.

Formula for Pump Shaft Power

L = 0.163・Q・H・ɔ/(ɖ/100)

L: Pump shaft power (kW)
Q: Discharge (m^3/min)
H: Total head (m)
ɔ : Unit weight of water; 1.0 (kgf/l)
ɖ : Pump efficiency (%); 82.5 % at design point for centrifugal pump

The planned diesel engine output is estimated with the following, where, the power transfer efficiency and allowance are added on the basic pump shaft power.

Formula for diesel engine output

P = L・(1+A)/ɖt

P : Planned diesel engine output (kW)
L : Pump shaft power (kW)
A : Allowance (0.15 for the case of diesel engine)
ɖt : Transfer efficiency (Fixed at 1.0 as direct jointing is applied)

As the power source, diesel engine shall be adopted and standard type is planned in view of the compatibility. As the results, the diesel engine capacities of 57kw are determined.

L= 0.163×21×12×1.0 / (82.5/100) = 49.8 kw P= 49.8×(1+0.15) / 1.0 = 57 kw

Table 3.4.4 Pump Efficiency of Centrifugal Pump

Discharge (m^3/min)	Diameter (mm)	Specific Speed (Ns)			
		160	250	400	630
4.0	200	0.710	0.720	0.720	0.710
6.3	250	0.740	0.750	0.750	0.740
10	300	0.770	0.780	0.780	0.770
16	350	0.810	0.820	0.820	0.810
20	400	0.815	0.825	0.825	0.815
25	450	0.825	0.835	0.835	0.825
32	500	0.830	0.840	0.840	0.830
40	600	0.840	0.850	0.850	0.840
63	700	0.850	0.860	0.860	0.850
85	800	0.855	0.865	0.865	0.855
100	900	0.860	0.870	0.870	0.860
130	1000	0.860	0.870	0.870	0.860
160	1200	0.865	0.875	0.875	0.865

Source: Design Pump Facilities Technical Document (Japan)

(6) Valves around the pump

1) Sluice valve on discharge side

At the time of starting the pump operation, there will be an inhalation of air as caused by the vacuum pump running. To shut this air, a valve is necessary to be provided.

2) Check valve

In case of main pump shut-down in a condition the discharge valve opened due to the sudden power cut etc, a check valve is needed on the discharge side to stop the pumped water to counter to pump equipment.

3.5 Pump Building

(1) Style of building

Pump station building is constructed for the purpose to protect the equipment and O&M works from winds and rains, and the structure and layout shall be of percolation-proof from outer and inner basin as well as rain water.

The style of the building is in general to be determined in consideration of the kinds and types of pump and in connection with the suction sump. Water level fluctuation in the River Jur is quite large and therefore the building is planned as a single-floor type located at the basement at the level where pump operation can be made without cavitation even with the low water level.

(2) Pump room

The plan of pump room shall be decided mainly by the alignment of pumps. The alignment shall be considered in a way that pumps may satisfy various hydraulic conditions required by pump operation and also attention be paid on daily operation, inspection and maintenance to be made easily and safely.

Under the subject project, double-suction pump is to be introduced and the linear alignment shall be adopted where hydraulic condition is the best without having eccentric flow. In the case of linear alignment, the length of building becomes a little longer in the right angle direction to the pumped water flow, however, there is no problem in the required land lot for construction. The length between beams of the pump room shall be calculated by adding the suction and discharge pipe lengths on the dimensions of the space between each of flange face, assuming that such major equipment as pumps, valves, engines and etc be hanged vertically by the overhead crane. While the length of building (Right angle direction to the flow) shall be determined so that the required space for effective O&M works could be secured around the pumps and motors under the concept of safety first. Further, the height of the building may be determined taking into consideration the height of hanging required for installation as well as O&M works for the equipment in the pump room.

Based on the considerations as above, the plan and section of pump station building are planned as shown in the Figure 3.5.1 and Figure 3.5.2.

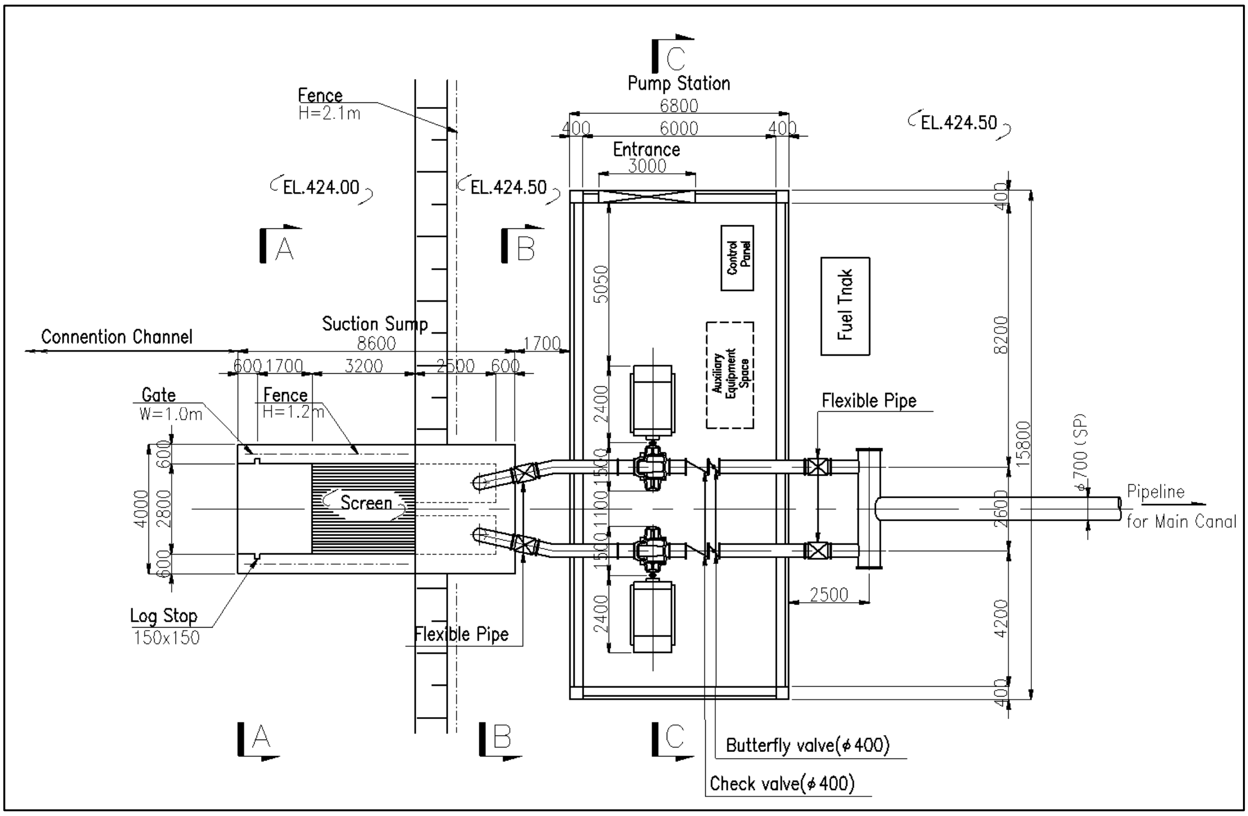


Figure 3.5.1 Plan and Section of Pump Station Building (Plan)

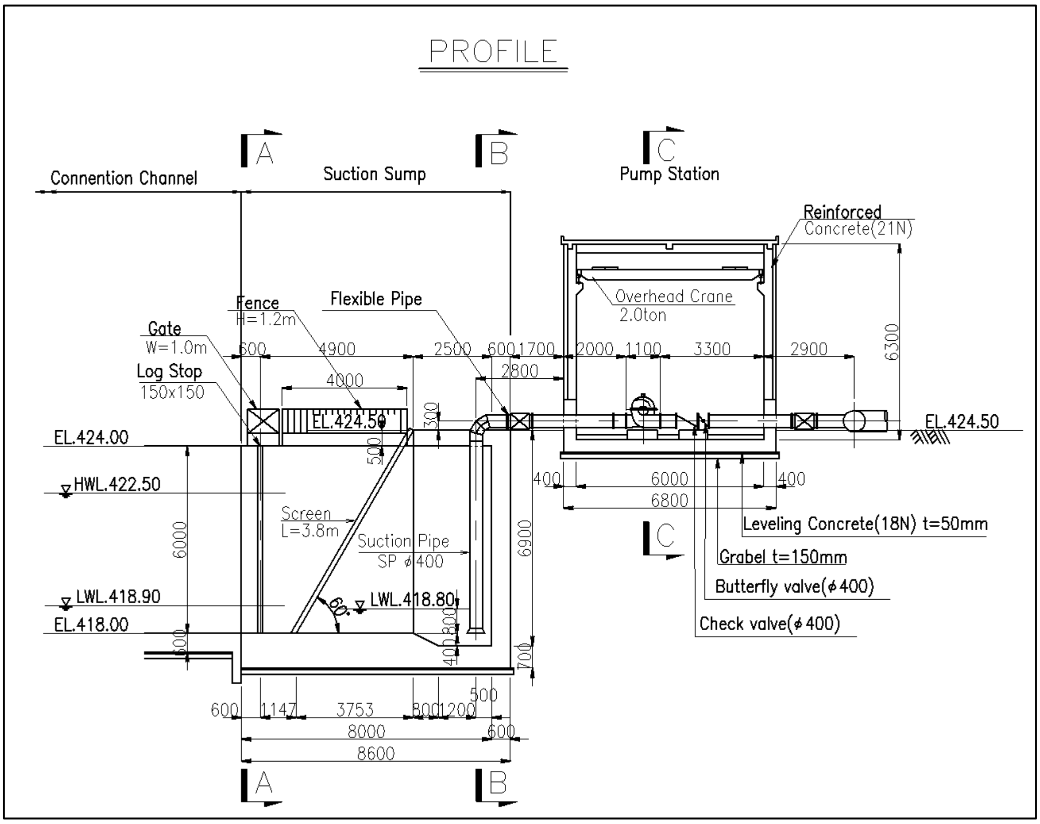


Figure 3.5.2 Plan and Section of Pump Station Building (Profile)

(3) Structure of building

The structure type of pump station building shall be of reinforced concrete which is superior in the characteristics of fire-proofing, durability and anti-wind, though concrete blocks shall be used for the wall body on ground.

(4) Foundation work

As the types of foundation works for pump station building, there are spread foundation, pile foundation and caisson foundation and the selection shall be made on considerations on the ground condition, characteristics of the upper structure, construction period as well as the economic aspect. Generally, the spread foundation is adopted for the case of about 2 m depth to the bearing stratum and the pile foundation for the depth longer than 5 m.

For reference, the log of boring nearest the pump station is shown in the Figure 9.2, which is located far about 150m from the pump station. The general condition seems to be adequate as the spread foundation for structures. However, after conducting the additional geological investigation at the just pump station in the future design stage, the allowable bearing capacity shall be examined to judge whether the spread foundation type can be adopted or not.

LOG OF BORING

BRIGHT STAR ENGINEERING AND CONSULTING CO. LTD

PROJECT: PROPOSED IRRIGATION DEVELOPMENT MASTER PLAN (IDMP)

CLIENT : SANYU CONSULTANTS INC (CONSULTANT FOR IDMP) LOCATION: WAU- WESTERN BAHR EL GHAZAL STATE	BORING NO.: G-C-R - 01 SHEET NO.: 1 of 1	Report No.: ST - 0001500009 Rev.0
COORDINATES: N: 7° 41' 04" E: 28° 02' 46.0"	GROUND ELEV. : 435.6 m GROUND WATER DEPTH (m): 1.70	
DRILLING DATE: 4/22/2015 TO 4/22/2015	TOTAL BORING DEPTH (m): 5.0	Orientation: Vertical
BORING METHOD: Rotary with mud BORING EQUIPMENT: Mobile B31	BORING DIAMETER: 100mm CASING DIAMETER:	

Depth (m)	R.L. (m)	S.T.	TCR (%)	SCR (%)	RQD (%)	SPT (N)	Legend	Description	U.C.S (kg/cm²)	D.D. (gm/cm³)
0.0 – 1.5	-1.5					22		Medium Dense, brownish gray, sandy silty CLAY.		
1.5 – 2.5	-2.5					24		Medium, reddish gray, sandy CLAY.		
2.5 – 3.5	-3.5					32		Dense, gray, silty sandy GRAVEL.		
3.5 – 4.5	-4.5					>50		Very dense, dark gray, silty SAND.		
4.5 – 5.0	-5.0					>50		Very dense, gray, silty SAND, residual soil of highly to completely weathered rock		
								END OF BORING (5.0m)		

Depth scale: 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0

1- S.T. : Sampler Type
2- TCR : Total Core Recovery
3- SCR : Solid Core Recovery
4- RQD : Rock Quality Designation
5- SPT : Std. Penetration Test
6- U.C.S. : Unconfined Compressive Strength
7- D.D. : Dry Density
8- R.L. : Reduced Level

Legend: Standard Penetration Test; Tricone Bit; Undisturbed Sample; Rock Coring; Water Table; Auger

REMARKS:

Figure 3.5.3 Log of Boring (the distance is about 150m from pump station)

3.6 Pipeline

3.6.1 Typical Section

The irrigation water lifted by the pump is carried to the discharge chamber, which is located at the intermediate point of the irrigation canal, through the pipeline of 700mm diameter. The pipe diameter is to be so determined that the flow velocity inside pipe would be in the range of 1.5-2.5 m/s in general considering such factors as protection of turbulent flow and sedimentation as well as economy.

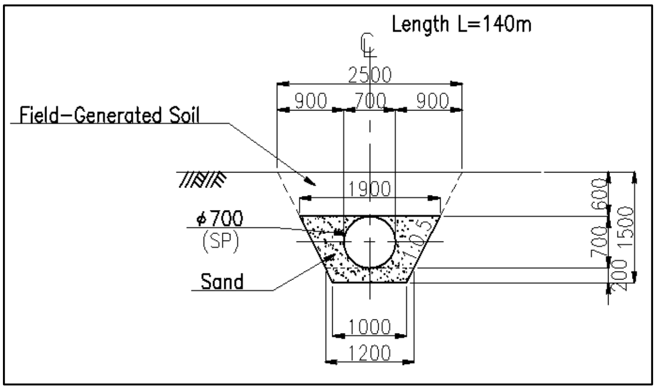


Figure 3.6.1 Typical Section of Pipeline

3.6.2 Discharge Chamber

The discharge chamber is to dissipate the flow from discharge pipe, change the flow direction and divert the flow to the downstream canal so that the pressure fluctuation accompanying the sudden change of flow quantity as caused by the start and stop of pump operation can be absorbed in the chamber as the change of water level in the chamber.

In the discharge chamber, tractive force will occur due to the disturbance of flow and the high velocity. Therefore, the structure shall be of firm reinforced concrete type.

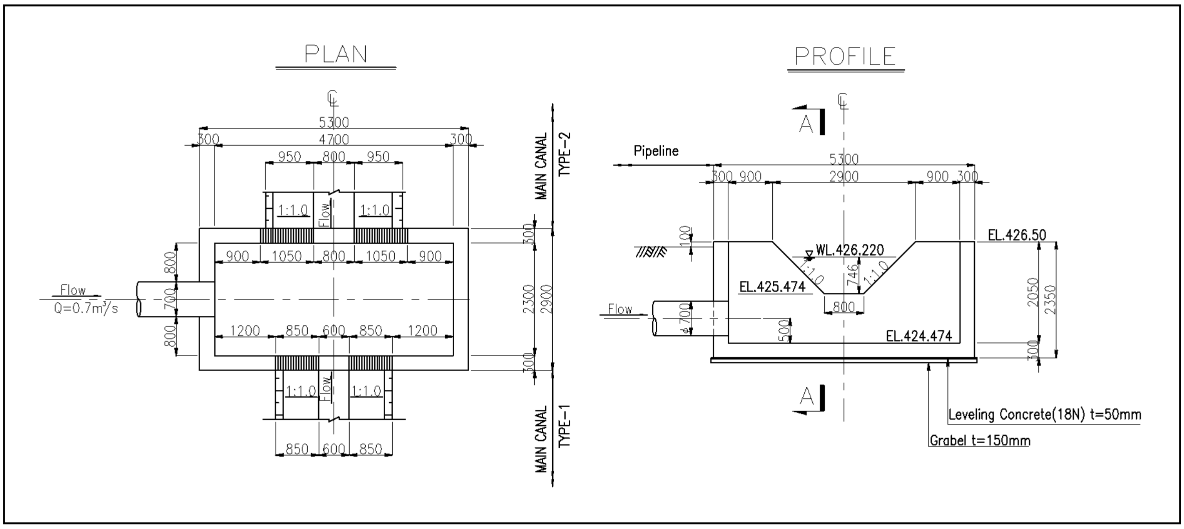


Figure 3.6.2 Discharge Chamber

CHAPTER 4 IRRIGATION CANAL AND DRAINAGE

4.1 Design Discharge of Canal and Drainage

4.1.1 Irrigation Canal

Unit water requirement was estimated at 1.400 l/s/ha, depending on the calculation of the water requirement.

Design discharge is estimated by the method that the unit water requirement multiples the subject area.

$Q = q \times A$

Where, Q: Design irrigation discharge (m^3/s)

q: Unit water requirement (0.0014 m^3/s)

A: Subject area (ha)

Design discharge

- Main irrigation canal Type-1: Q= 0.53m^3/s, depending on the dam outflow
- Main irrigation canal Type-2: Q= 0.70m^3/s, depending on the pump capacity
- Secondary Canal: Q= 0.022m3/s (=0.0014×averagely 16ha)

4.1.2 Drainage

Unit area drainage discharge was estimated at 0.0095m3/s/ha, depending on the calculation of the flood flow analysis.

Return period T=5 year: outflow Q= 48.3m^3/s (catchment area A= 51km^2)

Unit area drainage discharge: q= Q/A = 0.95 $m^3/s/km^2$ = 0.0095$m^3/s/ha$

Design discharge is estimated by the method that the unit water requirement multiples the subject area.

$Q = q \times A$

Where, Q: Design drainage discharge (m^3/s)

q: Unit area drainage discharge (0.0095 $m^3/s/ha$)

A: Subject area (ha)

- Main Drainage canal : Q= 0.0095× 500ha = 4.75m^3/s
- Drainage: Q= 0.15m^3/s (=0.0095×averagely 16ha)

4.2 Main Irrigation Canal

Main canal shall be constructed to conduct the irrigation water from Wau Dam at the east side of command area. Main canal is mainly separated two section. Upper section lies between the Wau Dam and command area (L=6.2km), and lower section go through the command area (L=7.1km). And the pipeline, which conduct the irrigation water from pump station located at the bank of Nile River, shall connect to main canal at the 2.7km lower from Wau Dam.

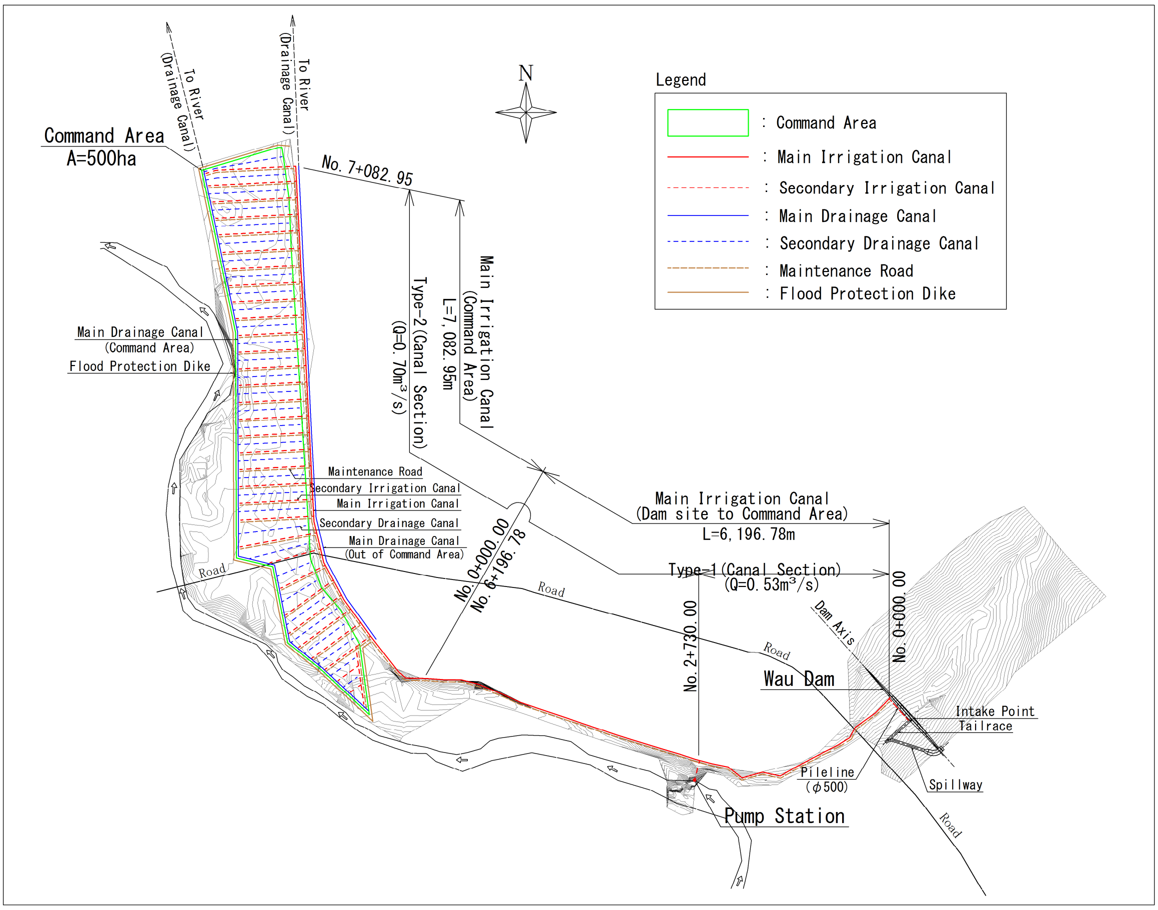


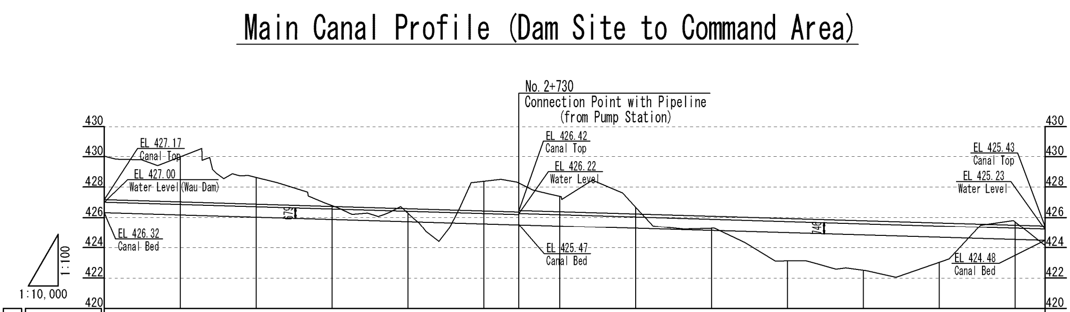
Figure 4.2.1 Location Map

The station number, length, and design discharge of each section is shown in the Table 4.2.1

Table 4.2.1 Main canal

Type	Station	Length (m)	Design Discharge (m3/s)
Main Canal (Dam Site to Command Area)		6.197	
Type-1	No. 0+0.00 ~ No. 2+730.00	2730	0.53
Type-2	No. 2+730.00 ~ No. 6+196.78	3,467	0.70
Main Canal (Command Area)		7,083	
Type-2	No. 0+0.00 ~ No. 7+082.95	7,083	0.70

Canal profile and canal section are shown in the Figure 4.2.2, Figure 4.2.3 and Figure 4.2.4.

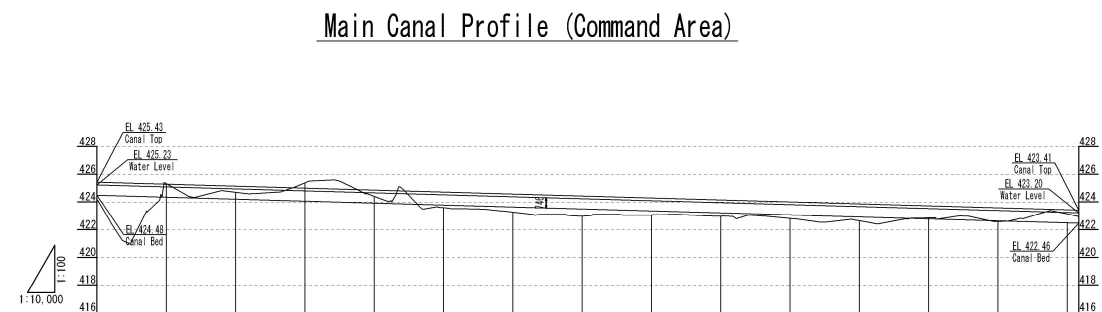


GRADIENT: i=1:3,500 (0.0286%)

DESIGN DISCHARGE: Q = 0.53m²/s (Type-1) (0+000 to 2+730); Q = 0.70m²/s (Type-2) (2+730 to 6+199)

STATION NO.	WATER LEVEL (PLAN)	TOP ELEVATION (PLAN)	BED LEVEL (PLAN)	GROUND LEVEL (PRESENT)
0+000	427.00	427.17	426.32	430.03
0+500	426.86	427.03	426.18	429.99
1+000	426.71	426.89	426.04	428.63
1+500	426.57	426.74	425.89	426.78
2+000	426.43	426.60	425.75	426.27
2+500	426.29	426.46	425.61	428.38
2+730	426.22	426.39	425.54	428.16
3+000	426.14	426.35	425.40	427.39
3+500	426.00	426.20	425.25	426.68
4+000	425.86	426.06	425.11	425.57
4+500	425.72	425.92	424.97	423.13
5+000	425.57	425.78	424.83	422.51
5+500	425.43	425.63	424.68	423.03
6+000	425.29	425.49	424.54	425.70
6+199	425.23	425.43	424.48	424.14

Figure 4.2.2 Main Canal Profile (Dam Site to Command Area)



GRADIENT: i=1:3,500 (0.0286%)

DESIGN DISCHARGE: Q = 0.70m²/s (Type-2)

STATION NO.	WATER LEVEL (PLAN)	TOP ELEVATION (PLAN)	BED LEVEL (PLAN)	GROUND LEVEL (PRESENT)
0+000	425.23	425.43	424.48	423.95
0+500	425.09	425.29	424.34	425.33
1+000	424.94	425.14	424.20	424.70
1+500	424.80	425.00	424.05	425.39
2+000	424.66	424.86	423.91	424.41
2+500	424.51	424.72	423.77	423.59
3+000	424.37	424.58	423.63	423.25
3+500	424.23	424.43	423.48	422.99
4+000	424.09	424.29	423.34	423.05
4+500	423.94	424.15	423.20	422.97
5+000	423.80	424.00	423.05	422.81
5+500	423.66	423.86	422.91	422.65
6+000	423.51	423.72	422.77	422.85
6+500	423.37	423.57	422.62	422.58
7+000	423.23	423.43	422.48	423.12
7+083	423.20	423.41	422.46	422.94

Figure 4.2.3 Main Canal Profile (Command Area)

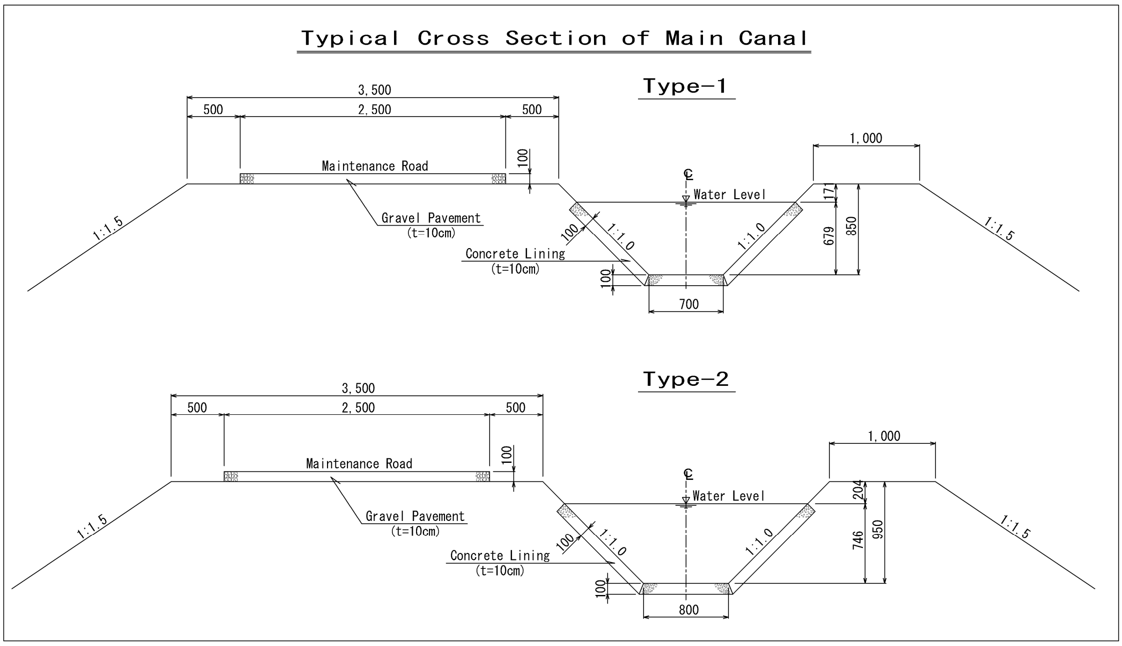


Figure 4.2.4 Typical Cross section of Main Canal

4.3 Secondary Canal and Drainage in command area

Secondary canal and drainage are planned in the command area for the distribution of irrigation water to the farms and the evacuation of surplus water including rainfall from the farms. The total length of secondary canal and drainage in command area is almost 49km and 23km respectively.

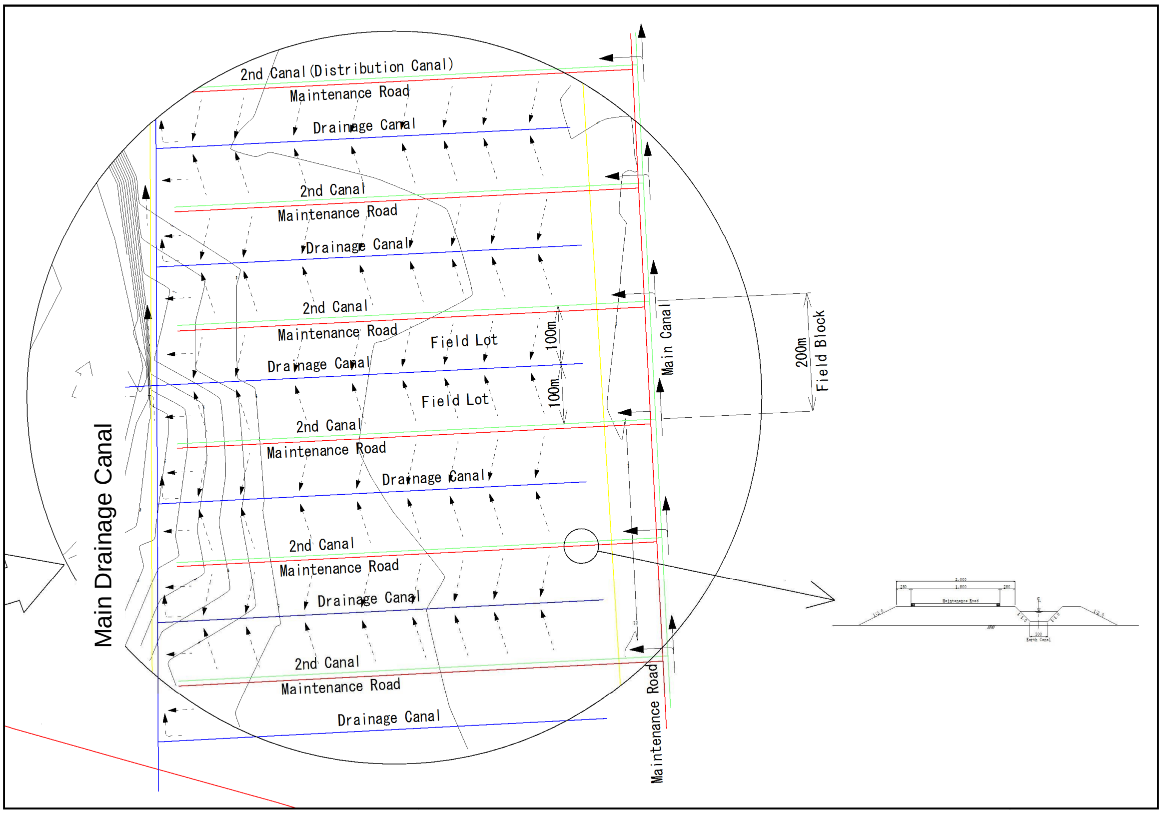


Figure 4.3.1 Secondary Canal and Drainage in Command Area

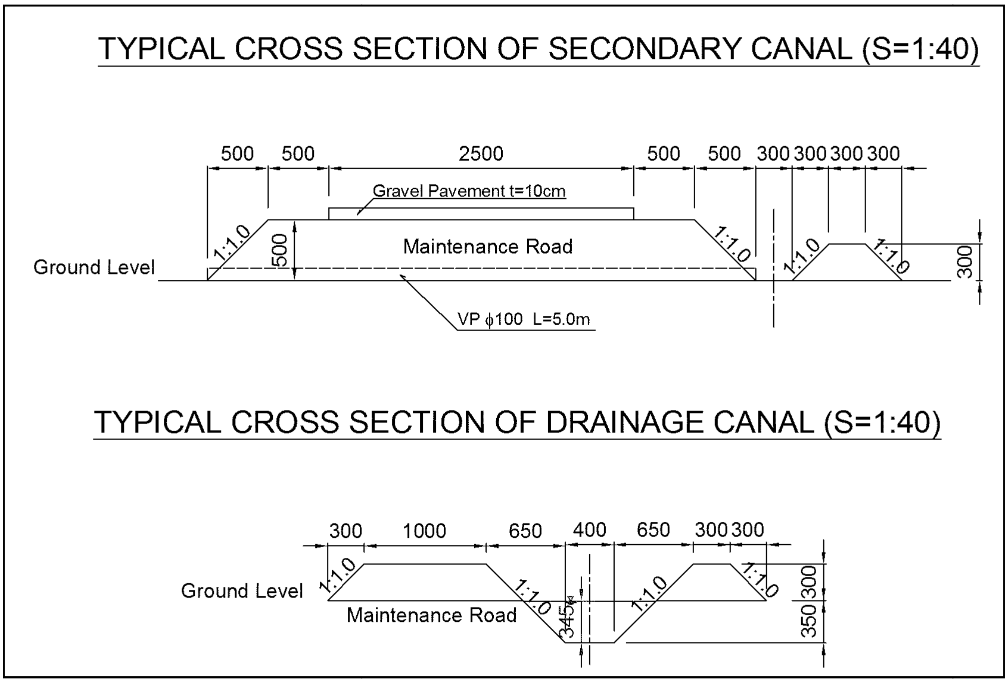


Figure 4.3.2 Typical Cross section of Secondary Canal & Drainage

4.4 Main Drainage Canal

Main drainage canal, which has a function for gathering the drainage from command area, is located at the west side of command area and along the flood protection dike.

On the other hand, another main drainage canal is required to protect the command area against the outflow from the catchment area out of command area. However, the range and size of catchment area is unclear as well as the current flow direction at the site. The study of main drainage canal for catchment area is required in the future design stage.

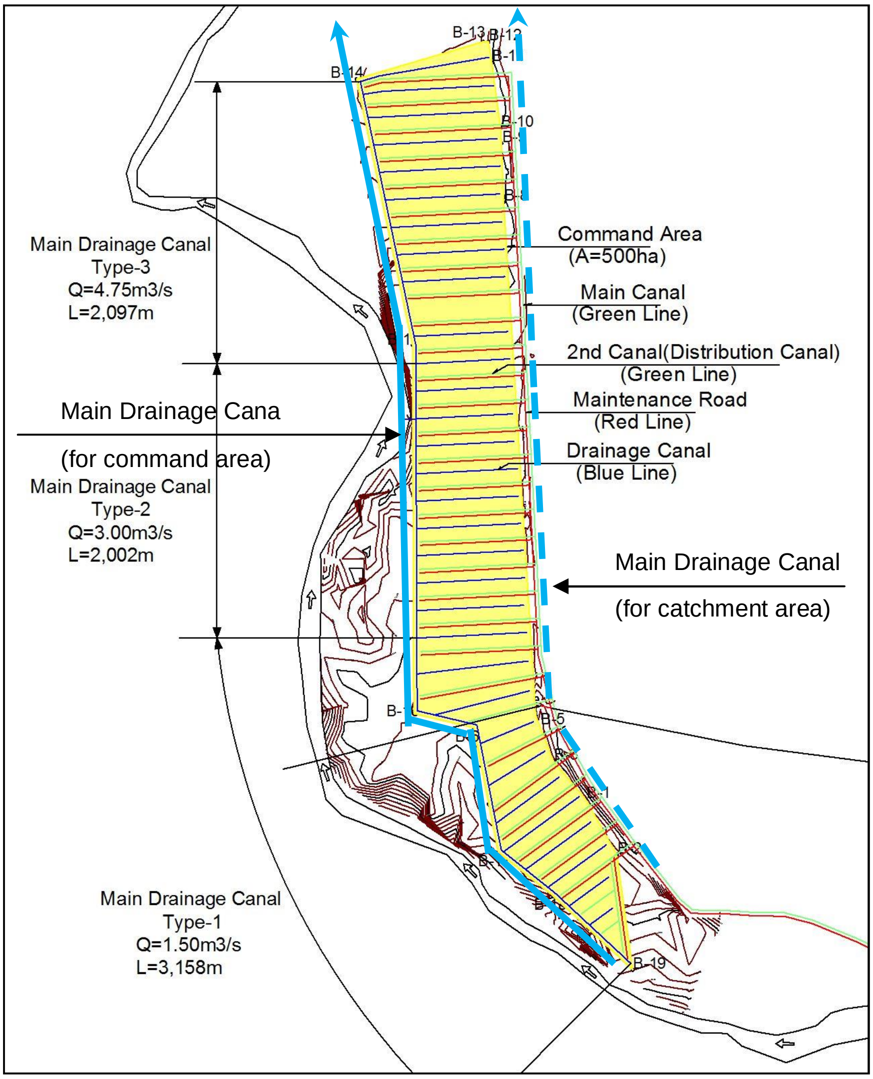


Figure 4.4.1 Main Drainage Canal

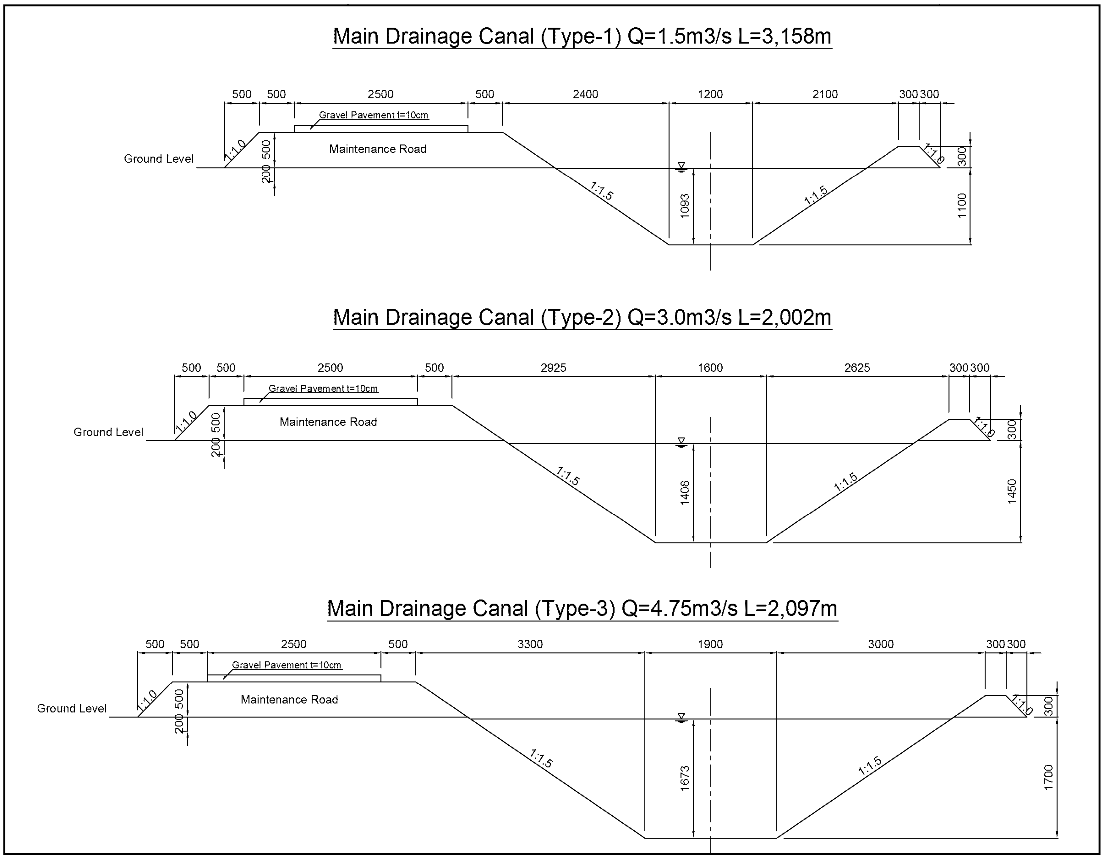


Figure 4.4.2 Typical Cross section of Main Drainage Canal

4.5 Examination Method of Canal Capacity

Main irrigation canal is designed as the concrete lining canal. Secondary canal and Drainage are designed as the earth canal. The required function of canal is to convey the irrigation water properly with the required water level and water volume supplied to the farms. The size of the cross section is planned by the volume of the required water with Manning formula as follows.

$Q = A \cdot V$

where, Q : Discharge (m^3/sec)

A : Flow Area (m^2)

V : Average flow velocity (m/sec);

Manningôs formula : $V = 1/n \cdot R^{2/3} \cdot I^{1/2}$

n : Roughness coefficient, for concrete lining canal : n = 0.015,

and for earth canal : n = 0.025

R : Hydraulic radius (m)

I : Hydraulic gradient

Therefore the examination method for the canal capacity will apply the followings.

・Firstly, calculate the required water volume for the each irrigation block at schemes

・Secondly, examine the required size of the cross section to discharge for the above water volume

As for the detail method in examining the size of the existing canal section and required size of the canal section, it will be carried out as follows.

1. The canal bed slope, bank slope and bed width are estimated for each section of canal, then the target cross section is selected from each irrigation blocks
2. The clearance of the water level is decided by referring Japanese Design book in which the following calculation formula is shown, and the clearance should be higher than the calculated figure.

   Fb＝0.05d＋β・hv＋hw

   Fb : clearance (m)

   d : depth of the design discharge

   hv : velocity head (m)

   β : conversion coefficient from velocity head to static head. (generally it is 0.5～1.0)

   hw : clearance for the waving of water surface. (generally it is 0.10～0.15cm)

Table 4.5.1 Calculation of Irrigation Canal Section

Items		Main Irrigation Canal		Secondary Canal
		Type-1	Type-2	
Design discharge	Q (m^3/s)	0.53	0.70	0.022
Width of canal bed	B (m)	0.70	0.80	0.30
Water depth	d (m)	0.679	0.746	0.14
Bank slope	1:N	1.0	1.0	1.0
Cross-sectional area of flow	A (m)	0.936	1.153	0.062
Wetted perimeter	P (m)	2.621	2.910	0.696
Hydraulic mean depth	R (m)	0.357	0.396	0.089
Coefficient of roughness	n	0.015	0.015	0.025
Canal bed slope	I (%)	0.0286	0.0286	0.20
Mean velocity	V (m/s)	0.567	0.608	0.357
Velocity head	hv (m)	0.016	0.019	0.006
Free board	Fb (m)	0.171	0.204	0.16
Height of canal	H	0.85	0.95	0.30

Table 4.5.2 Calculation of Drainage Section

Items		Drainage	Main Drainage		
			Type-1	Type-2	Type-3
Design discharge	Q (m^3/s)	0.15	1.50	3.00	4.75
Width of canal bed	B (m)	0.40	1.20	1.60	1.90
Water depth	d (m)	0.345	1.093	1.408	1.673
Bank slope	1:N	1.0	1.5	1.5	1.5
Cross-sectional area of flow	A (m)	0.257	3.104	5.226	7.377
Wetted perimeter	P (m)	1.376	5.141	6.677	7.932
Hydraulic mean depth	R (m)	0.187	0.604	0.783	0.930
Coefficient of roughness	n	0.025	0.025	0.025	0.025
Canal bed slope	I (%)	0.20	0.0286	0.0286	0.0286
Mean velocity	V (m/s)	0.585	0.483	0.574	0.644
Velocity head	hv (m)	0.017	0.012	0.017	0.021
Free board	Fb (m)	0.255	0.207	0.192	0.227
Height of canal	H	0.60	1.30	1.60	1.90

Typical canal section are shown in the Figure 4.2.4, Figure4.3.2 and Figure 4.4.2 respectively.

4.6 Relative structures

In general the relative structures such as diversion gate, water measurement facilities and road crossing are required in the canal system if necessary. They shall be designed considered the canal system and farm shape in the future design stage.

4.7 Recommendation

The route of main canal in the survey work is located in the undulating land, particularly the section between the dam and the upstream of command area. As shown the canal profiles of Figure 4.2.2, the route of main canal in the survey work numerous earthworks for embankment are required in whole section. It is recommended that these routes shall be reviewed to reduce the amount of earthworks, save the construction cost and shorten the constriction term in the future design stage.

Reference: Minimum / Maximum allowable velocity

For reference, the Canal works guideline published in Japan shows the minimum / maximum allowable velocity as follows. It is recommended to take a caution for them in canal design.

6.1.1 Minimum allowable velocity

(1) Object discharge
Object discharges in studying the minimum allowable velocity are as shown in Table 6.1.1.

Table 6.1.1 Object discharges in studying the minimum allowable velocity

Type of canal	Object discharges
Irrigation canal	Most frequent discharge (the discharge which occurs most times in the pentad mean discharge unit through out the water conveyance period of the canal)
Drainage canal	Discharge to study the low water revetment, etc. (1-year or 2-year probability discharge)

(2) Minimum allowable velocity
It is appropriate that the minimum allowable velocity would not be below the velocity under the object discharge flow condition. However, when the velocity is below the minimum allowable velocity out of necessity, the structure and management system that are capable of maintaining drainage function of the canal shall be provided.
Also, the minimum allowable flow velocities shall follow values provided in Table 6.1.2.

Table 6.1.2 Minimum allowable flow velocities

Condition of canal	Minimum allowable velocity
Canal where concerns regarding deposition of floating sediment do exist.	0.45 - 0.90 m/s
Canal where concerns regarding overgrowth of water weed do exist.	0.70 m/s

Note: The minimum allowable velocity shall be determined by the grain size of floating sediment.

6.1.2 Maximum allowable velocity

(1) Object discharge

Object discharges in studying the maximum allowable velocity are as shown in Table 6.1.3.

Table 6.1.3 Object discharges in studying the maximum allowable velocity

Type of canal	Object discharges
Irrigation canal	Planned maximum flow discharge
Drainage canal	Discharge to study the low water revetment, etc. (1-year or 2-year probability discharge) 185-day water discharge or firm drainage discharge during irrigation season

(2) Maximum allowable velocity

The maximum allowable velocity involves uncertainties because it significantly varies depending on the material constituting the canal. Therefore, judgments have to be exercised based on experiences and other case examples. Based on materials and thickness of the members of the canal and the inside surface of the canal structure, those values shown in Table 6.1.4 are considered as approximate limiting values.

Table 6.1.4 Maximum allowable velocity

Type of material	Velocity (m/s)	Classification	Velocity (m/s)
Sandy soil	0.45	Thick concrete (approximately 18 cm)	3.00
Sandy loam	0.60	Thin concrete (approximately 10 cm)	1.50
Loam	0.70	Asphalt	1.00
Clayey loam	0.90	Block cavity wall (buttress pier less than 30 cm)	1.50
Clay	1.00	Block cavity wall (buttress pier 30 cm or larger)	2.00
Sandy clay	1.20	Block mortar masonry	2.50
Soft rock	2.00	Reinforced concrete pipe	3.00
Semi-hard rock	2.50	Steel pipe, ductile cast iron pipe	5.00
		Petrochemical products group (polyvinyl chloride pipe, reinforced plastic composite tube)	5.00
Hard rock	3.00	Reinforced concrete secondary product canal (excluding fence culvert)	3.00

Notes:
1. The maximum allowable velocity is a value determined mainly by structural durability of the material of the canal structure against scour and wear. Specifically when a velocity close to the maximum allowable velocity value is used, it is necessary to study the hydraulic stability (especially regarding waves, water level rise at the cross section transition point, air entrapment in pipes, etc.).
2. Maximum allowable velocities for structures such as wasteways/spillways that are part of the canal and convey temporary flows shall be equal to or less than 1.5 times of values listed in the table above.
3. In cases of drainage canals, the value equal to or less than 1.5 times of values in this table shall be applied to discharges (1-year or 2-year probability discharge) to study the low water revetment. However, such value shall not also exceed values in this table at the time of 185-day water discharge or firm drainage discharge during irrigation season. Additionally, this table is not applicable to cases where appropriate erosion protections such as bed protection, etc., are provided for the subject facility in areas such as chutes, steep slope drainage canals, etc., or where structural members are reinforced by means such as increasing concrete thickness or reinforcing bars, or where the drainage canal is as large as a river. In such cases, the maximum allowable flow velocities shall be determined by referring to the structure and topography/geology of the subject canal as well as similar case examples.
4. The maximum allowable flow velocities for cast-in-place concrete structures whose member thickness is 13 cm or larger shall be 3.0 m/s or less.
   Also, values of thick concrete or thin concrete in the above table may be applied to the maximum allowable flow velocities for plain concrete structures and for thickness between 10 cm and 18 cm, the value may be determined by proportional distribution.
5. In case of increasing the covering thickness, according to the standard of the U.S. Reclamation Bureau, regarding structures where the velocity exceeds 3.0.m/s, the structural durability can be secured by adding 1.5 cm of covering thickness to the value shown in Table 7.8.34, and by increasing the thickness by 1.5 cm every time the velocity is increased by 3.0 m/s.

CHAPTER 5 FLOOD PROTECTION DIKE

5.1 Flood Protection Dike

Flood protection dike shall be constructed around command area to protect the farmland from flood of Nile River. Height of dikeôs crest shall be decided by considering flood water level which was confirmed at site investigation conducted by RSS-TT. The gradient of River Jur is supposed one to fifty thousand (1/5,000) by topographic survey, and the gradient of dike shall be same as River Jur.

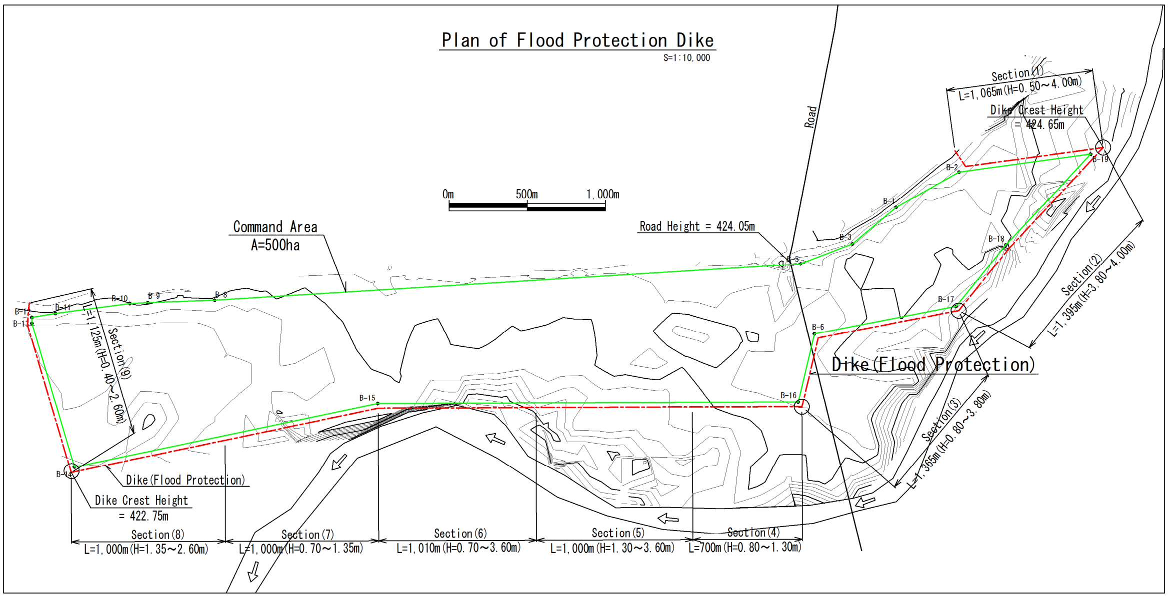


Figure 5.1.1 Location Map

Total length of flood protection dike is 9.66 km, and dike is divided into 9 section. Height of dike is calculated in each section as shown in the Table 5.1.1.

Table 5.1.1 Plan of Flood Protection Dike

	Distance	Crest Height		Crest Width	Height			Dike Width		
		Upper	Lower		Minimum	Minimum	Average	Minimum	Minimum	Average
Section(1)	1,065 m	424.65 m	424.65 m	2.00 m	0.50 m	4.00 m	2.25 m	3.75 m	16.00 m	9.88 m
Section(2)	1,395 m	424.65 m	424.30 m	2.00 m	3.80 m	4.00 m	3.90 m	15.30 m	16.00 m	15.65 m
Section(3)	1,365 m	424.30 m	423.95 m	2.00 m	0.80 m	3.80 m	2.30 m	4.80 m	15.30 m	10.05 m
Section(4)	700 m	423.95 m	423.77 m	2.00 m	0.80 m	1.30 m	1.05 m	4.80 m	6.55 m	5.68 m
Section(5)	1,000 m	423.77 m	423.52 m	2.00 m	1.30 m	3.60 m	2.45 m	6.55 m	14.60 m	10.58 m
Section(6)	1,010 m	423.52 m	423.26 m	2.00 m	0.70 m	3.60 m	2.15 m	4.45 m	14.60 m	9.53 m
Section(7)	1,000 m	423.26 m	423.01 m	2.00 m	0.70 m	1.35 m	1.03 m	4.45 m	6.73 m	5.61 m
Section(8)	1,000 m	423.01 m	422.75 m	2.00 m	1.35 m	2.60 m	1.98 m	6.73 m	11.10 m	8.93 m
Section(9)	1,125 m	422.75 m	422.75 m	2.00 m	0.40 m	2.60 m	1.50 m	3.40 m	11.10 m	7.25 m
Total	**9,660 m**									

Dike sections are shown in the Figure 5.1.2.

5.2 Recommendation

It is recommended that the flood water level should be observed continuously, and the height of dike crest should be re-examined.

Figure 5.1.2 Cross section of Dike