PART 2 IRRIGATION SCHEME DEVELOPMENT PLAN

CHAPTER 5 INSTITUTIONAL SET-UP OF THE IRRIGATION SCHEME

5.1 Demarcation of Stakeholders' Roles

MEDIWR takes primal responsibility to develop the Jebel Lado Irrigation Scheme, including feasibility study, design works, implementation, O&M of the main structures, and monitoring and evaluation of the project.

Key directorates of MEDIWR in development of the Jebel Lado Irrigation Scheme are six which, includes; Directorate of Irrigation and Drainage (DID), Directorate of Planning and Programmes (DPP), Directorate of Water Resources Management (DWRM), Directorate of Power Engineering and Grid (DPEG), and Directorate of Hydrology and Survey (DHS). Their main functions in development of the Jebel Lado Irrigation Scheme are summarized in Table 5.1.1.

Table 5.1.1 Key Directorate of MEDIWR in National Irrigation Development Programme

Organization	Stakeholders	Key Functions in Irrigation Development
MEDIWR	Directorate of Irrigation and Drainage (DID)	Construction and operation of irrigation scheme; including pump station, canals, farm lots and flood control structures.
	Directorate of Planning and Programmes (DPP)	Coordinate staff training including State government staff; Coordinate planning process; Monitoring and Evaluation of the project implementation, Harmonize budgeting procedure for effective budget execution.
	Directorate of Water Resources Management (DWRM)	Establishment of institutional framework; Integrated Water Resources Management approach; Pollution prevention and mitigation.
	Directorate of Power Engineering and Grid (DPEG)	Construction, rehabilitation, maintenance and operation of power plant and grid.
	Directorate of Hydrology and Survey (DHS)	Resource assessment, feasibility studies, information management and research; Establishment of centralized hydromet and water use/abstraction information management system; Accumulation of long time historical Hydromet and water use/abstraction data/information using hydromet equipment installed at the Jebel Lado Irrigation Scheme.

Source: Main Functions of directorates MWRI
Strategic Plan 2012-2017, MWRI
Programme Profile of IDMP, National Irrigation Development Programme (NISDP)

In addition to MEDIWR, MAFCRD, MLFI, MOE, MWLCT and etc. are also important stakeholders in development of the Jebel Lado Irrigation Scheme. At the planning stage, MAFCRD is required to develop water demand plan for crops related the project. MAFCRD also takes responsibility for on-farm level irrigation management, including allocation of farm plot to farmers, preparation of cropping calendar, estimation of water demand, extension of irrigation farming, and O&M of irrigation facility at on-farm level.

MWLCT also plays important role for conservation of wild life in and around the project site, while MOE is a primal ministry for environmental protection including watershed conservation. Table 5.1.2 shows stakeholders and their key functions in Jebel Lado Irrigation Scheme development.

Table 5.1.2 Stakeholders Involved in National Irrigation Development Programme

Organization	Stakeholders	Key Functions in Irrigation Development
FSC	Food Security Council	Create a national food security policy to ensure adequate food availability throughout South Sudan.
MAFCRD	Directorate of Agriculture Production and Extension Services (DAPES)	Promote development and adaptation of appropriate technology for irrigation farming; Establish and manage an effective agricultural extension service; Human resource

Organization	Stakeholders	Key Functions in Irrigation Development
		training in the field.
	Directorate of Cooperatives (DC)	Provide guidance to establish cooperatives and issuance of the registration certificate if necessary.
	Directorate of Rural Development (DRD)	Provide technical assistance and training to State governments and other local governments to build their capacity to assume their responsibilities for irrigated agriculture.
	Directorate of Planning (DP)	Formulate registration, policies, standards and plans for irrigated agriculture development.
	Directorate of Special Projects and Donors Coordination (DAPDC)	On-farm level irrigation management, including allocation of farm plot to farmers, preparation of cropping calendar, estimation of crop water requirement, and instruction to farmers for O&M of irrigation facility at on-farm level.
MOE	Ministry of Environment	Conduct EIA of irrigation projects; Environmental protection including watershed conservation; Advice and support States and local governments in their responsibilities for environmental protection.
MWLCT	Directorate of Wild Life Conservation	Develop water demand plan for wildlife and other conservation purposes if any.
MLHPP	Ministry of Lands, Housing and Physical Planning	Surveying and mapping of the project area and safe keeping maps ad documents; Establish and oversee the operation of the land registry.
LC	Land Commission	Establish and oversee the operation of the Land Registry.
NBS	National Bureau of Statistics	Provide socio-economic data/information for irrigation development plan and M&E.
MTRB	Ministry of Transport, Roads and Bridges	Construction of Farm-To-Market road to improve market accessibility of irrigation command areas; Permit common use of road/bridge for irrigation scheme development and Hydromet equipment installation.
MGCSW	Ministry of Gender, Child and Social welfare	Promote income generating activities of vulnerable groups; Plan and implement repatriation, relief, resettlement and reintegration of internally displaced persons and refugees.
MFEP	Ministry of Finance and Economic Planning	Budgetary arrangement for irrigation development; Supporting donor buying process for irrigation development.
MTII	Ministry of Trade, Industry and Investment	Promotion of Public Private Partnership and private sector investment in future.
MLFI	Directorate of Animal Production and Range Management (DAPRM)	Coordinate participation of livestock keepers in irrigation planning; Develop water demand plan for dipping and watering facilities for livestock if necessary.
	Directorate of Livestock and Fisheries Research Development (DLFRD)	Provision of research results to mitigate conflict between farmers and pastoralists so as to sustain irrigation water use among stakeholders.
	Directorate of Extension and pastoralists Development (DEPD)	Coordinate participation of pastoralists in irrigation planning; Develop water demand plan for pastoralistsôwatering points if necessary.
	Directorate of Fisheries and Aquaculture Development (DFAD)	Coordinate participation of fisher folks and aquaculture business entity in irrigation planning if any; Develop water demand plan for fisheries and aquaculture related facilities if any
	Directorate of Investments Planning and Statistics (DIPS)	Collection and provision of necessary data/information for irrigation development plan and M&E.
WRMA	Water Resources Management Authority (WRMA)	【After the Water Bill being enacted】 Regulate the management; Development and use of water resources; Issue regulation on water resources allocation and the issuance of permits; Issue permits for inter-basin water transfer; Provide guidelines to BWB on the pricing strategy for charges to be levied under the Water Bill; Ensure collection, analysis and dissemination of data and information on water resources, etc.
BWB	Basin Water Boards (BWB)	【After the Water Bill being enacted】 Protecting water resources and increasing water availability, Receiving permit applications for water abstraction, for water use and recharge,

Organization	Stakeholders	Key Functions in Irrigation Development
		determining, issuing and varying water permits and enforce the conditions of those permits; Receiving permit applications for the construction of works, and determining, issuing and enforcing the conditions of those permits; Enforcing regulations; Coordinate and facilitate the formation and activities of WUAs; Setting the level of charges to be levied under this Act in accordance with the pricing strategy and guidelines issued by the WRMA; Collecting water permit and water use charges; etc.
IB	Irrigation Boards (IB)	【After the Water Bill being enacted】Protecting water resources and increasing irrigation water availability, Receiving permit applications for irrigation water users, for water use and recharge, determining, issuing and varying water permits and enforce the conditions of those permits; Receiving permit applications for the construction of irrigation and drainage facilities, and determining, issuing and enforcing the conditions of those permits; Enforcing regulations; Coordinate and facilitate the formation and activities of WUAs; Setting the level of charges to be levied under this Act in accordance with the pricing strategy and guidelines issued by the WRMA; Collecting irrigation fee for O&M of irrigation facilities; etc.
C/WC	Catchments/Watersheds Committees	【After the Water Bill being enacted】To formulate catchment or sub-catchment integrated water resources management plans; To resolve water resources conflicts in the catchment or sub-catchment; To perform other functions delegated by the BWB.
WUA	Water Users Association (WUA)	Manage, distribute and conserve water from a source/facility used jointly by the members of the WUA; Resolve conflicts between members of the association; Collect water user fees on behalf of the BWB; Represent the special interests and values arising from water used for both public and private purposes.
SDWS	State Directorate of Water and Sanitation (SDWS)	Coordination between central government, counties and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
SDALFF (SLMALFF)	State Directorate of Agriculture, Livestock, Fisheries and Forestry (SDALFF)	Coordination between central government, counties and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
SDC/RD (SLMC/RD)	State Directorate of Cooperatives, Rural/Community Development	Coordination between central government, counties and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
SDLS (SLMLS)	State Directorate of Land and Survey	Coordination between central government, counties and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
CDWS (LG)	County Department of Water and Sanitation (CDWS)	Coordination between central government, state and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
CDALFF (LG)	County Department of Agriculture, Livestock, Fisheries and Forestry (CDALFF)	Coordination between central government, state and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
CDC/RD	County Department of Cooperatives, Community/Rural Development	Coordination between central government, state and communities concerned to formulate irrigation development plan, implementation and O&M of the project; participation in M&E of the project.
At Community	Farmers/Pastoralists Union, Cooperatives Society,	Participation in irrigation development planning, implementation and O&M of the project; participation in M&E of the project.

Organization	Stakeholders	Key Functions in Irrigation Development
Level	Fishing Folks, Civil Society	

Source: Main Functions of directorates MWRI
Roles, Functions and Responsibilities of the National Ministries, Ministry of Cabinet Affairs, November 4th, 2013.
Programme Profile of IDMP, National Irrigation Development Programme (NISDP)

5.2 Category of Irrigation Scheme

The Jebel Lado Irrigation Scheme will be developed under the National Irrigation Scheme Development Programme (NISDP). The NISDP is owned by the national government with large/medium scale command area and irrigation facilities and is developed by the national government. Definition of the NISDP is summarized in Table 5.2.1

Table 5.2.1 Categorization of Irrigation Scheme Development

Programme	Definition	Capital Investment (funding source)	Implementation (Construction)	Owner	O&M /a	Responsible Organization of Land Allocation	Technical Assistance	Supervision of Scheme Management
National Irrigation Scheme Development Programme (NISDP)	- Large (more than 500 ha <) - Land property belongs to National	National/ Private Sector (Bank)/ International Development Bank/ DPs (grant)	National	National	National (Scheme Management Office)/ WUA	National/ Community	National/ DPs/ NGOs	National

Note: a/ Operation and maintenance of irrigation scheme could transfer to local government in the long-term, depending on their capability.

5.3 Division of Roles within the Irrigation Scheme

MEDIWR takes primal responsibility to develop the Jebel Lado Irrigation Scheme, from planning, designing, implementation, and O&M. The line ministries of the MEDIWR at state government and local government also play key roles in irrigation development planning in terms of coordination among grassroots level stakeholders, and M&E of the irrigation programmes/projects.

Community participation in planning, implementation, operation and maintenance of on-farm level irrigation scheme is a key for successful implementation of the irrigation development. In some cases, land belongs to communities, and the government cannot start any irrigation development procedures without permission and participation of communities. Table 5.3.1 shows role and responsibility for implementation of the Jebel Lado Irrigation Scheme development project.

Table 5.3.1 Roles and Responsibilities of Programmes/Projects Implementation

Type of programme/project	Responsibilities				
	National Government/DPs	State Government/DPs	County or LG	Community	Private Sector
National programme/project (Nationally planned and nationally implemented)	・Planning ・Financing ・Implementation ・M&E	・Coordination ・M&E	・Coordination ・M&E	・Contribution ・Coordination ・M&E	

5.4 Private Sector Involvement

In irrigation development, there are several types of private sector involvement including participatory irrigation management (PIM), irrigation management transfer (IMT), and public private partnership (PPP). In the Republic of South Sudan, the irrigation development under the current government has just started through the IDMP, and establishment of organizational structure and capacity development of the government officials has just started at the national level. Technical and administrative capacity development at state, county and community level will be conducted afterward.

When we consider current constraints on irrigation development including sophisticated land holding system, capacity of the government in terms of financial and human resources, introduction of PIM must be necessary to promote the irrigation development to nationwide. In this regard, community participation in irrigation development from planning stage till operation and maintenance of irrigation facilities at least on-farm level is required. Following table shows range of institutional arrangement of PIM. Among them, the shared management is suitable for the Jebel Lado Irrigation Scheme.

Table 5.4.1 Range of Institutional Arrangements for PIM

Activity	Full Agency Control	Agency O&M (User Input)	**Shared Management**	WUA Owned (Agency Regulation)	Full WUA Control	Irrigation Management Company Board
Regulation	Agency	Agency	**MEDIWR**	Agency	WUA	Agency
Ownership of Structure & Assets	Agency	Agency	**MEDIWR**	WUA	WUA	Private Company
O&M Responsibility	Agency	Agency	**Scheme Management Office/ WUA**	WUA	WUA	Private Company
Collection of Water Charges	Agency	Agency	**Scheme Management Office/ WUA**	WUA	WUA	Private Company
Unit of Representation	Agency	WUA	**WUA**	WUA	WUA	Company & User Committee

Source: Arranged by the IDMP-TT based on "Participatory Irrigation Management", J. Raymond Peter, Executive Director, International Network on Participatory Irrigation Management, Washington DC (INWEPF/SY/2004(06))

CHAPTER 6 AGRICULTURAL PLANNING

6.1 Basic Concept of Agricultural Planning for Priority Projects

The priority project areas will be the model of irrigated agriculture in RSS in future after IDMP actually start working. Therefore, the farming plans of priority project areas should have form that can contribute to the strategic plan specified in the governmental policies related to agricultural sector. In addition, it would be necessary to examine the agricultural potential of each project areas from various aspects, such as natural condition, marketing, and beneficiariesôcapacity and their technical potential.

Government plans to be considered

Agricultural Sector Policy Framework (2012-2017) with its setting vision of ñFood security for all the people of the Republic of South Sudan, enjoying improved quality of life and environmentò. Food insecurity is the most critical issue for South Sudanese and sustainable irrigation infrastructure and flood management system is expected to improve agricultural productivity and food security enhancement. This document also addressed some key issues as the mission of MAFCRD for instance acceleration of food and agricultural production through commercial smallholder and large scale agriculture, using mechanized and irrigation technology.

In addition, the comprehensive national development plan initiated just after the independence, namely ñSouth Sudan Development plan (SSDP) 2011-2013ò prioritizes the agricultural sector for economic development. In fact, main means of livelihoods of South Sudanese are agriculture and animal husbandry. To achieve basic improvement of peopleôs livelihoods, commercial agriculture should be promoted for future economic growth.

To make farming systems of priority project areas to follow the above strategic plan of the government, followings should be incorporated into the farming plans.

- ✓ Mechanized and intensive farming system
- ✓ To grow staple crops for subsistence giving priority to the crops with high water requirement
- ✓ To grow commercial crops for cash generation

With setting the above as basic concept of farming plan for priority project areas, crops to be cultivated for each area are examined considering the specific conditions, such as natural condition, marketing, and beneficiariesôcapacity and their technical potential.

6.2 Agricultural Planning (Cropping Pattern)

The command area in Jebel lado is owned by two (2) communities namely; Nyuwa and Piete community. Therefore, beneficiaries the farmers in future irrigation scheme are expected to be mainly the members from those two (2) communities. Hence, their potentiality and intension should be taken into consideration for the plan.

According to the socio economic survey, the farmers in the communities seem to prioritize staple crop cultivation because those cultivations are necessary for them. In fact, self-supply amount per person of maize and sorghum was the lowest among three (3) sites even though those cropsô household use percentages in Jebel lado were tend to be higher than other two (2) sites. Hence, it would be required to incorporate staple crop into future farming plan.

On the other hand, the farmers in Jebel Lado are producing vegetables mainly for sale achieving fairly good yield with low production cost, resulting in relatively high profitability. The farmers in Jebel

Lado seem to have a technical potential for commercial farming. Commercial crop like banana can be tried for cash generation under irrigated farming. Also leafy vegetable which is daily consumed is high potential crop for cash generation because leafy vegetables are not much imported from neighbouring countries because of its perishability and Jebel Lado is located near from Juba, where is high demand of daily consumed vegetables. Leafy vegetable cultivation in urban area is one of the typical success stories in farming.

In addition, in socio-economic survey, the farmersôrequest for crop that they are willing to cultivate in future irrigation scheme was asked to the interviewees. Table 6.2.1 shows the percentage of the number of answers obtained in the interview. The major requests were sorghum, maize, and ground nuts. Among the vegetables, Tomato was the most popular, followed by Okra and Jewôs mallow. However, considering that soil in the command area generally is with high clay contents, Ground nut cannot be selected as representative crop for farming plan because heavy high clay soil can damage ground nuts at harvesting.

Table 6.2.1 Requested Crops

Crop	Percentage of Answers
Maze	10.5
Sorghum	11.4
Cassava	5.3
Common Beans	7.0
Ground nut	10.5
Cowpea	6.1
Sesame	8.8
Sugarcane	7.0
Vegetables	33.3
Tomato	(9.6)
Okra	(8.8)
Jew's mallow	(8.8)
Amaranthus	(1.8)
Egg plant	(4.4)
Other vegetables	-

Source: IDMP TT (Socio-economic survey, 2015)
* The questionnaire allowed multiple answers to the interviewee
* Parenthesized numbers show the breakdown of vegetables.

Like Ground nut above, natural conditions which can be obstacle in cultivation should be considered, such as high temperature in dry season, soil alkalinity in the command area in Jebel Lado. From these aspects, crops with high heat temperature tolerance and crops whose preferable soil pH is relatively high have been selected.

Taking into consideration the above reasons, Maize, Tomato, Jewôs mallow and banana have been selected for farming plan for irrigation scheme in Jebel Lado.

Figure 6.2.1 shows the planned cropping pattern with project for Jebel Lado.

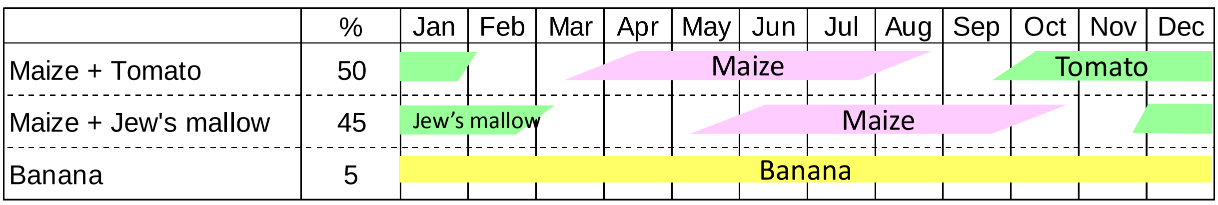


Figure 6.2.1 Planned Cropping Pattern

CHAPTER 7 IRRIGATION AND DRAINAGE PLAN

7.1 Parameters Affecting Crop Water Requirement

7.1.1 Climate and Weather Parameters

(1) Meteorological stations

The nearest meteorological stations for the priority project site are shown as below (Figure 7.1.1). These meteorological stations have the data, such as rainfall, temperature, relative humidity, and wind speed and so on. Though the sunshine hour data cannot be found at the meteorological stations, it should be estimated by ñFAO Irrigation and Drainage Paper No.24ò.

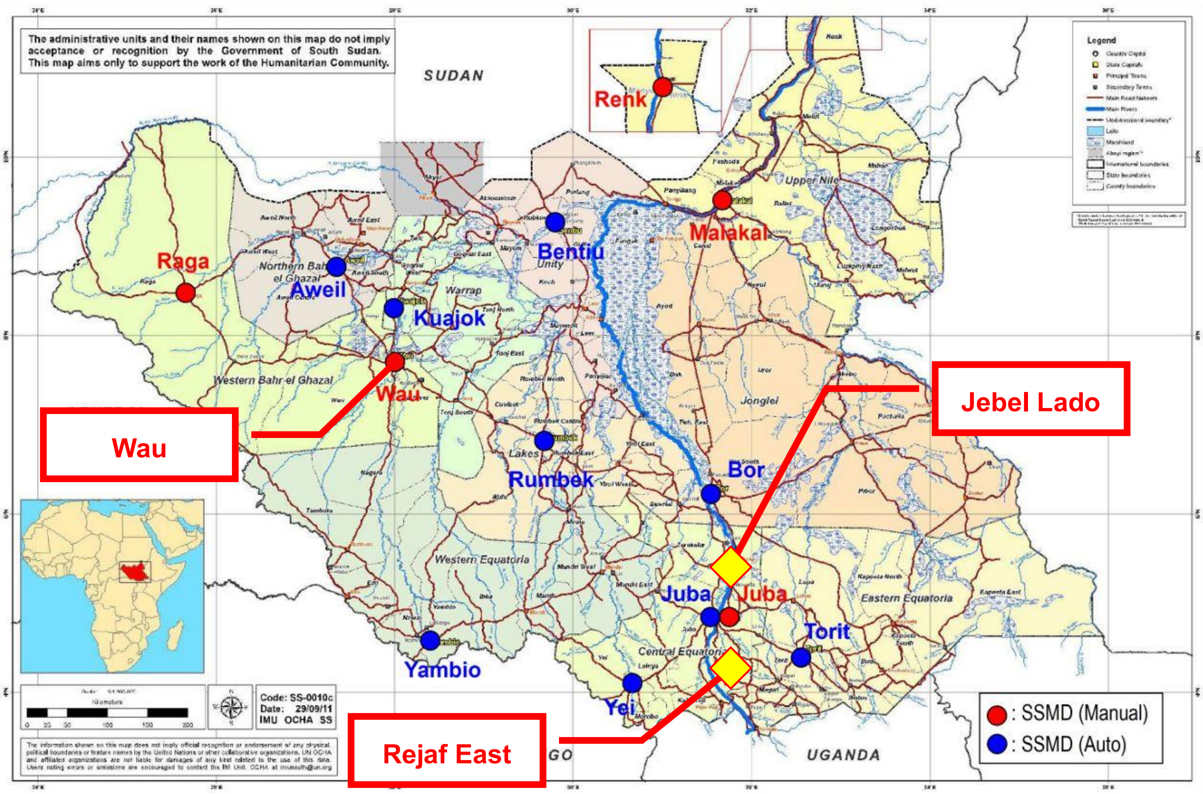


Figure 7.1.1 Meteorological Stations in South Sudan

Table 7.1.1 Meteorological Stations for Necessary Climate Data

Priority Project Site	Climate Data	Meteorological Station	Remarks
Wau	Temperarure, Rainfall	Wau	
	Relative Humidity, Wind Speed	Kauajok	No data in Wau meteorological station
Jebel Lado	Temperarure, Rainfall Relative Humidity Wind Speed	Juba	The nearest meteorological station
Rejaf East	Temperarure, Rainfall Relative Humidity Wind Speed	Juba	The nearest meteorological station

(2) Rainfall

The priority project areaôs climate belongs to the equatorial subtropical type. According to the data, it is categorized that Apr-Oct period is ñRainy seasonò, and Nov-Mar period is ñDry seasonò. And it is characterized by mean annual rainfall of about 1,000 mm distributed in one rainy seasons, high temperatures and whereby consequently high evaporation. The mean monthly rainfall for Juba station is given in Table 7.1.2 and Figure 7.1.2.

Table 7.1.2 Mean Monthly Rainfall at Juba

Meteorological Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Juba (mm)	4.4	11.8	41.7	105.6	149.2	116.1	134.0	136.7	112.4	112.6	42.9	9.8	977

Source: Meteorological Station Data (1901-2012 complied from several sources)

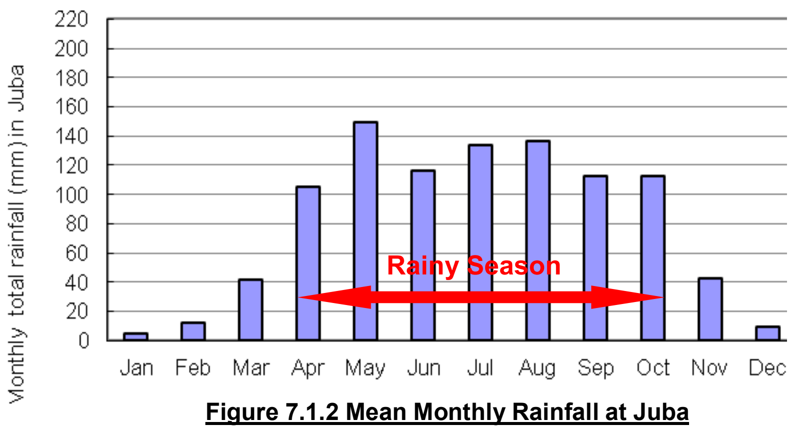


Figure 7.1.2 Mean Monthly Rainfall at Juba

(3) Temperature

The temperature in Wau and Juba area does not vary much throughout year. The hottest temperature appears in Feb -Mar, which corresponds to the end of the dry season. In both area, the mean monthly maximum temperature varies between 30 ^{o}C and 38 ^{o}C while the minimum temperature varies between 19 ^{o}C and 24 ^{o}C (see Table 7.1.3, Figure 7.1.3).

Table 7.1.3 Monthly Mean Max and Min Temperature at Juba

Item	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Average
Min Temp (^{o}C)	20.0	22.5	24.4	23.7	22.7	21.8	21.1	21.3	21.1	21.4	20.9	20.6	21.8
Max Temp (^{o}C)	36.6	38.1	37.2	35.6	33.1	31.3	31.2	30.6	32.7	33.3	34.0	35.2	34.1

Source: Meteorological Station Data (2009-2012 provided by SSMD)

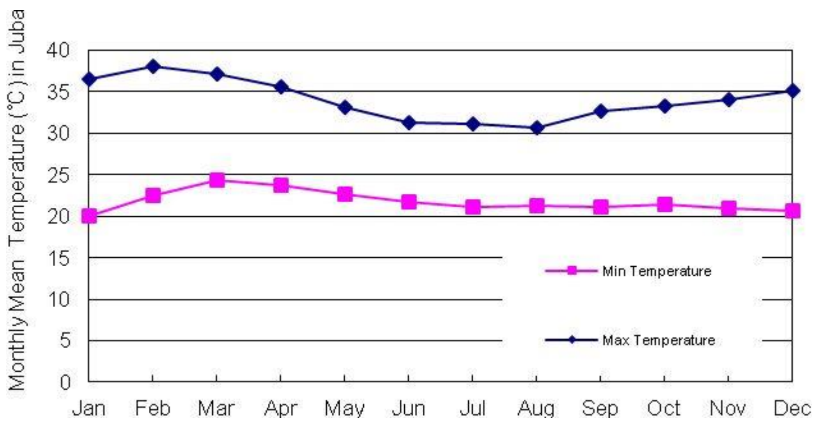


Figure 7.1.3 Mean Monthly Max and Min Temperature at Juba

(4) Sunshine hours

Average sunshine hour is given in Table 7.1.4 estimated by FAO Irrigation and Drainage Paper No.24. By using the table by FAO, the shine hour can be estimate on a pro-rata basis of the latitude. It can be said that throughout the year, the isolation in priority area is long and strong, and the annual average keeps about 12 hours per day.

Table 7.1.4 Average Sunshine Hours Estimated by FAO Irrigation and Drainage Paper No.24

North Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Average
50°	8.5	10.1	11.8	13.8	15.4	16.3	15.9	14.5	12.7	10.8	9.1	8.1	
40°	9.6	10.7	11.9	13.3	14.4	15.0	14.7	13.7	12.5	11.2	10.0	9.3	
30°	10.4	11.1	12.0	12.9	13.6	14.0	13.9	13.2	12.4	11.5	10.6	10.2	
20°	11.0	11.5	12.0	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9	
10°	11.6	11.8	12.0	12.3	12.6	12.7	12.6	12.4	12.1	11.8	11.6	11.5	
7.7° (Wau)	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
5.1° (Jebel Lado)	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8	12.1
5°	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8	
0°	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	

Source : FAO Irrigation and Drainage Paper No.24

(5) Relative humidity

The yearly mean relative humidity is calculated at 63% for Juba. At Juba, it has 35 % in February, and has 82% in August. The monthly relative humidity data is given in Table 7.1.5 and in Figure 7.1.4, and as shown it is characterized by equatorial subtropical type.

Table 7.1.5 Monthly Mean Relative Humidity at Juba

Meteorological Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Average
Juba (%)	37	35	42	62	73	78	78	82	79	76	63	52	63

Source : Meteorological Station Data (2009-2012 provided by SSMD)

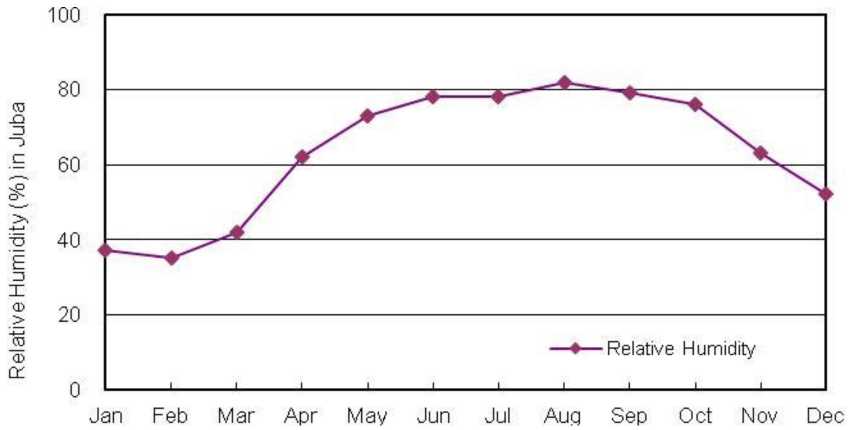


Figure 7.1.4 Mean Monthly Relative Humidity at Juba

(6) Wind speed

Mean annual velocity exceeds 70 km/s and even reaches as high as 4 m/s speed during the dry months (refer to Table 7.1.6 and Figure 7.1.5). The wind conditions are determined mainly by the breeze effect from the Indian Ocean. Night winds originate in gales which start blowing during the previous afternoon on the Somalian Coast.

Table 7.1.6 Monthly Mean Wind Speed at Juba

Meteorological Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Average
Juba (km/day)	112	130	156	216	156	104	95	95	95	112	112	121	125.3

Source : Meteorological Station Data (2009-2012 provided by SSMD)

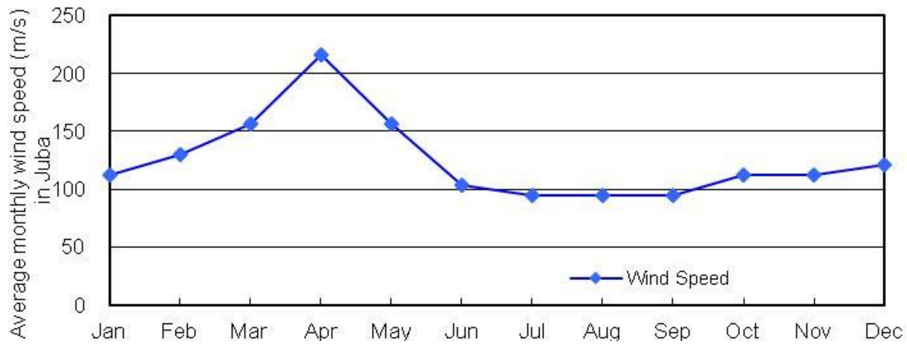


Figure 7.1.5 Mean Monthly Wind Speed at Juba

(7) Summary of the necessary climate and weather data

Priority Project : Jebel Lado, Rejaf East
Station : Juba
Altitude :462m, Latitude : 5° 4'N, Longitude : 31° 40'E

Table 7.1.7 Summary of the Climate Data at Juba

Item	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average Total
Min Temp (°C)	20.0	22.5	24.4	23.7	22.7	21.8	21.1	21.3	21.1	21.4	20.9	20.6	21.8
Max Temp (°C)	36.6	38.1	37.2	35.6	33.1	31.3	31.2	30.6	32.7	33.3	34.0	35.2	34.1
Humidity (%)	37	35	42	62	73	78	78	82	79	76	63	52	63
Wind (km/day)	112	130	156	216	156	104	95	95	95	112	112	121	125.3
Sunshine (hours)	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8	12.1
Rainfall (mm)	4.4	11.8	41.7	105.6	149.2	116.1	134.0	136.7	112.4	112.6	42.9	9.8	977

7.1.2 Cropping Pattern Plan in the Farmlands

The crop type, variety and development stage should be considered when assessing the evapotranspiration from crops grown in large, well-managed fields. Differences in resistance to transpiration, crop height, crop roughness, reflection, ground cover and crop rooting characteristics result in different ET levels in different types of crops under identical environmental conditions. The Cropping Pattern Plan in Jebel Lado Farmlands is shown in Table 7.1.8.

Table 7.1.8 Cropping Plan

Project site	Rainy season	Dry season
Jebel Lado	Maize, banana	Vegetable (Okra/Egg plant/tomato/cucumber/ Jew's mallow), banana

7.1.3 Crop Coefficient Factor

Most of the effects of the various weather conditions are incorporated into the ETo estimate. Therefore, as ETo represents an index of climatic demand, Kc varies predominately with the specific crop characteristics and only to a limited extent with climate. This enables the transfer of standard values for Kc between locations and between climates.

7.2 Estimation of Crop Water Requirement

To estimate the crop water requirements, guidelines were developed and published by FAO ñFAO Irrigation and Drainage Paper No. 24, Crop Water Requirementsò.

7.2.1 Reference Evapo-transpiration (ETo)

(1) Estimation methods

The evapo-transpiration rate from a reference surface, not short of water, is called the reference crop evapo-transpiration or reference evapo-transpiration and is denoted as ETo. The reference surface is a hypothetical grass reference crop with specific characteristics. The only factors affecting ETo are climatic parameters. Consequently, ETo is a climatic parameter and can be computed from weather data. ETo expresses the evaporating power of the atmosphere at a specific location and time of the year and does not consider the crop characteristics and soil factors.

Although several methods exist to determine ETo such as 1) Blaney-Criddle, 2) Radiation, 3) Modified Penman and 4) Pan evaporation methods as shown in Table 7.1.9. The modified Penman method was considered to offer the best results with minimum possible error in relation to a living grass reference crop. It was expected that the pan method would give acceptable estimate, depending on the location of the pan. The radiation method was suggested for areas where available climatic data include measured air temperature and sunshine, cloudiness or radiation, but not measured wind speed and air humidity. Finally, the publication proposed the use of the Blaney-Criddle method for areas where available climatic data cover air temperature data only.

Table 7.2.1 Water Requirement Estimation Methods by FAO

Estimation Methods	Feature	Necessary data	Remarks	Adoption
1) Blaney-Criddle	The most simplest method	Temperature		
2) Radiation	Simple method	Temperature, Sunshine		
3) Modified Penman (Penman-Montieth)	Suggested method by FAO	Temperature, Humidity, Wind, Sunshine	Calculated by CROPWAT 8.0	The Project Team adopted this method.
4) Pan evaporation	Actual Measurement method	Evaporation		

The FAO Penman-Monteith method is recommended as the sole standard method. It is a method with strong likelihood of correctly predicting ETo in a wide range of locations and climates and has provision for application in data-short situations. Therefore the project team adopted this Penman-Montieth method as the estimation method of the water requirement.

(2) Monthly values of reference (potential) evapo-transpiration (ETo)

Monthly values of potential/reference evapo-transpiration (ETo) can be estimated using Penman-Monteith method. Data used in estimating the potential/reference evapo-transpiration using Penman-Monteith method are the mean monthly values of temperature, relative humidity, ratio of actual sunshine duration to the maximum possible one, and wind speed. Together with the climate data recorded at Juba meteorological station and employed in estimating the ETo, the monthly ETo values are given in Table 7.2.2, which range from 5 mm to about 7 mm per day:

Table 7.2.2 Evapo-transpiration (ETo) in Jebel Lado and Rejaf East Estimated by Penman-Monteith

Particulars	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Min Temperature (°C)	20.0	22.5	24.4	23.7	22.7	21.8	21.1	21.3	21.1	21.4	20.9	20.6
Max Temperature (°C)	36.6	38.1	37.2	35.6	33.1	31.3	31.2	30.6	32.7	33.3	34.0	35.2
Relative Humidity (%)	37	35	42	62	73	78	78	82	79	76	63	52
Wind speed (km/day)	112	130	156	216	156	104	95	95	95	112	112	121
Sunshine (hours)	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8
Radiation (MJ/m2/day)	25.7	27.1	28.1	28.2	27.3	26.7	26.8	27.8	28.0	27.3	26.0	25.2
ETo (mm/day)	5.84	6.64	7.20	7.08	5.96	5.35	5.30	5.39	5.66	5.67	5.56	5.66

Source: JICA Team based on meteorological data recorded at Juba station.

7.2.2 Crop Coefficient (Kc)

The crop coefficient is depended on the crop development stages. The crop coefficient curve is shown (Kc curve) to Figure 7.2.1. The crop coefficient (Kc) estimated as follows Table 7.2.3, which varies from the initial stage to the peak stage. Estimation of crop coefficient (Kc) refers to the recommended figures in the ñCrop Water Requirements No.24 FAO Irrigation and Drainage paperò.

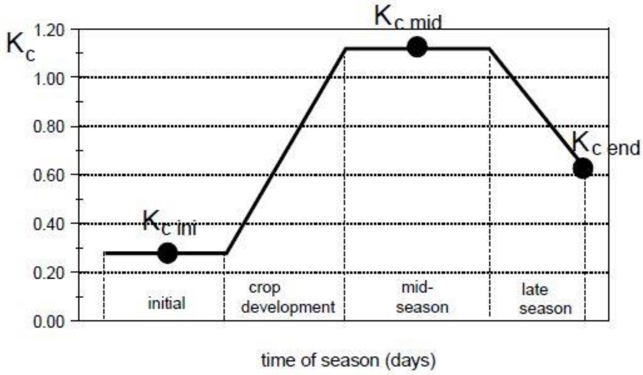


Figure 7.2.1 Crop Coefficient Curve

Table 7.2.3 Crop Coefficient by Each Crop

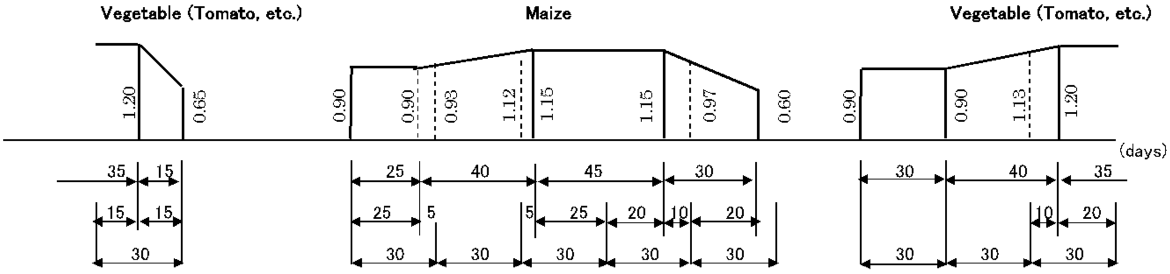
Crop	Kc ini	Kc mid	Kc end
Rice	1.10	1.10	0.95
Maize	0.90	1.15	0.60
Egg plant	0.90	1.05	0.85
Tomato	0.90	1.20	0.65
Jewõs mallow*	0.90	1.10	1.10

Note: Kc of Jewõs mallow is applied Kc of celory.

Jebel Lado: Crop Coefficient (Kc)

Crop-1

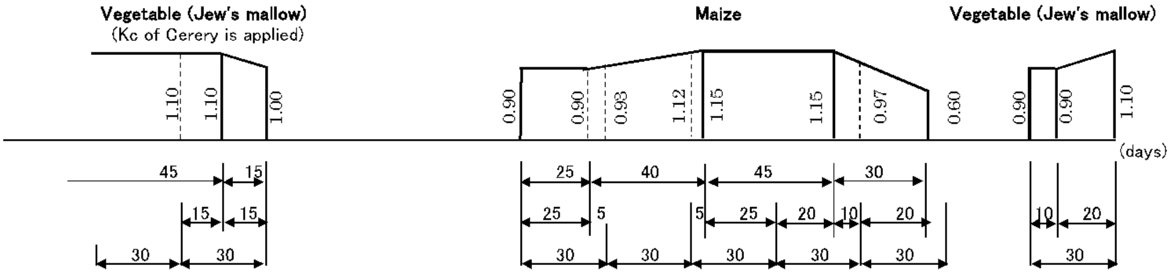
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Kc	1.06	0.00	0.00	0.90	1.03	1.15	1.12	0.52	0.00	0.90	1.02	1.19



Jan Kc= (1.20*15+ (1.20+0.65)/2*15)/30 = 1.06
Feb Kc= 0
Mar Kc= 0
Apr Kc= (0.90*25+ (0.90+0.93)/2*5)/30 = 0.90
May Kc= (0.93+1.12)/2*30/30 = 1.03
Jun Kc= ((1.12+1.15)/2*5 + 1.15*25)/30 = 1.15
Jul Kc= (1.15*20+ (1.15+0.97)/2*10)/30 = 1.12
Aug Kc= (0.97+0.60)/2*20/30 = 0.52
Sep Kc= 0
Oct Kc= 0.90
Nov Kc= (0.90+1.13)/2*30/30 = 1.02
Dec Kc= ((1.13+1.20)/2*10 + 1.20*20)/30 = 1.19

Crop-2

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Kc	1.10	1.08	0.00	0.00	0.00	0.90	1.03	1.15	1.12	0.52	0.00	0.97



Jan Kc= 1.10
Feb Kc= (1.10*15+ (1.10+1.00)/2*15)/30 = 1.08
Mar Kc= 0
Apr Kc= 0
May Kc= 0
Jun Kc= (0.90*25+ (0.90+0.93)/2*5)/30 = 0.90
Jul Kc= (0.93+1.12)/2*30/30 = 1.03
Aug Kc= ((1.12+1.15)/2*5 + 1.15*25)/30 = 1.15
Sep Kc= (1.15*20+ (1.15+0.97)/2*10)/30 = 1.12
Oct Kc= (0.97+0.60)/2*20/30 = 0.52
Nov Kc= 0
Dec Kc= (0.90*10+ (0.90+1.10)/2*20)/30 = 0.97

Crop-3 **Banana** : P.66, FAO No.24

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Kc	0.45	0.50	0.60	0.70	0.85	1.00	1.10	1.10	0.90	0.80	0.80	0.95

Figure 7.2.2 Crop Coefficient

7.2.3 Crop Evapo-transpiration under standard conditions (ETc)

The crop evapotranspiration under standard conditions, denoted as ETc, is the evapotranspiration from disease-free, well- fertilized crops, grown in large fields, under optimum soil water conditions, and achieving full production under the given climatic conditions. Crop evapotranspiration can be calculated from climatic data and by integrating directly the crop resistance, albedo and air resistance factors in the Penman-Monteith approach. As there is still a considerable lack of information for different crops, the Penman-Monteith method is used for the estimation of the standard reference crop to determine its evapotranspiration, ETc = Kc ETo.

7.3 Estimation of Irrigation Water Requirements

7.3.1 Calculation of Consumptive Irrigation Requirements (CIR)

The consumptive irrigation requirement is the quantity of water actually required by the plant.

CIR = Consumptive use ï effective rainfall
CIR = ETc ï Eff. rainfall

(1) Effective rainfall (dependable rainfall)

Effective rainfall should be estimate by ñDependable Rainfall (Pd)ò. The ñDependable Rainfall (Probability=80%)ò is used for the design of irrigation system capacity. The ñDependable Rainfall (80%)ò is corresponding to 80% probability of exceedance and representing a dry year.

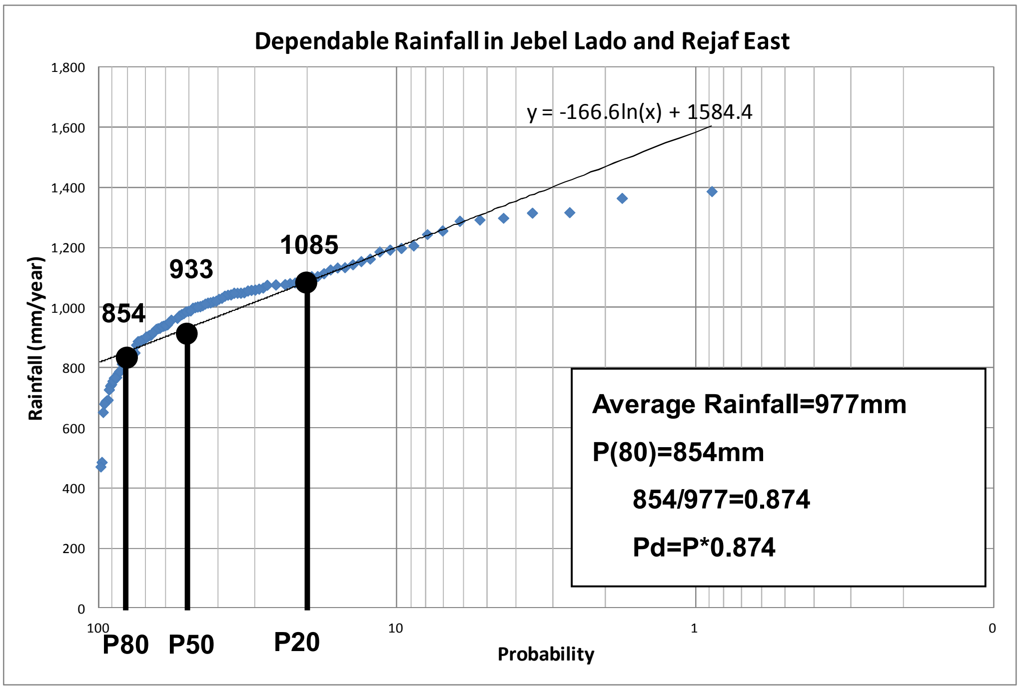


Figure 7.3.1 Dependable Rainfall at Juba

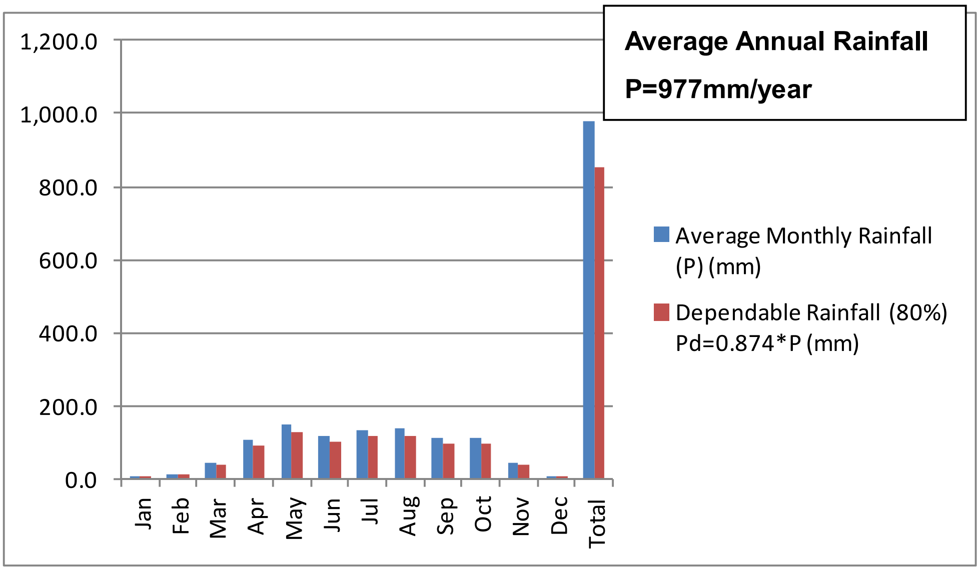


Figure 7.2.4 Effective Rainfall at Juba

(2) Estimation of the effective rainfall

Effective rainfall should be estimate by the formula suggested by FAO.

FAO Method (Suggested Method), Pd: Dependable Rainfall (Probability=80%)

$$Pe=0.6*Pd-10\ (Pd \leqq 70mm/month)$$

$$Pe=0.8*Pd-24\ (Pd > 70mm/month)$$

The estimated effective rainfall for Jebel Lado scheme is shown in Table 7.3.3.

7.3.2 Calculation of Net Irrigation Requirements (NIR)

The net irrigation requirement (NIR) is equal to consumptive irrigation requirement (CIR) plus the water required for other purpose, such as leaching of alkaline or salty soils.

NIR = CIR + Le

Where Le is the water required for leaching and other purposes.

Note: when leaching is not required; the net irrigation requirement is equal to consumptive irrigation requirements.

The calculated NIR (ETcrop1, ETcrop2 and ETcrop3) for Jebel Lado scheme is shown in Table 7.3.3.

7.3.3 Calculation of Field Irrigation Requirements (FIR)

The field irrigation requirement (FIR) is the amount of water required to be applied to the field. It is equal to the net irrigation requirements plus the amount of applied water lost as surface runoff, evaporation and deep percolation.

FIR = NIR + Water application losses

FIR = NIR/Ea

Where Ea is field application efficiency

7.3.4 Calculation of Gross Irrigation Requirements (GIR)

The gross irrigation requirement is the quantity of water required at the head of the canal; is greater than the field irrigation requirements because there are always some transit (conveyance) losses.

GIR = FIR + Conveyance losses

GIR = FIR/Ec

7.3.5 Calculation of Irrigation Water Requirement

Irrigation is required when rainfall is insufficient to compensate for the water lost by evapotranspiration. The primary objective of irrigation is to apply water at the right period and in the right amount. By calculating the soil water balance of the root zone on a daily basis, the timing and the depth of future irrigations can be planned.

The daily water balance, expressed in terms of depletion at the end of the day is:

Dr, i = Dr, i-1 - (P - RO)i - Ii - CRi + ETc, i + DPi

Where:

Dr, i: root zone depletion at the end of day i [mm],

Dr, i-1: water content in the root zone at the end of the previous day, i-1 [mm],

Pi: precipitation on day i [mm],

ROi: runoff from the soil surface on day i [mm],

Ii: net irrigation depth on day i that infiltrates the soil [mm],

Cri: capillary rise from the groundwater table on day i [mm],

ETc, i: crop evapotranspiration on day i [mm],

Dpi: water loss out of the root zone by deep percolation on day i [mm].

During this Pre-feasibility study for Jebel Lado scheme most of the soil water balance parameters were negligent but they must be considered during feasibility study stage.

Therefore the daily water balance is expressed as follow:

0 = 0 - (P - 0)i - Ii - 0 + ETc, i + 0

Ii = ETc,i ï Pi or

NIR = ETc ï Eff. rainfall

There is no leaching required; the CIR equal to NIR

The Scheme/Farm irrigation requirement is equal to net irrigation requirements plus field application losses, filed canal losses and conveyance losses.

Scheme/farm irrigation water requirement = Net Irrigation requirement/Ep

Where Ep is overall Irrigation Efficiency

Ep= Ec.Eb.Ea

Where Ec is Conveyance efficiency, Eb is field canal efficiency and Ea field application efficiency.

(1) Overall irrigation efficiency

Overall irrigation efficiency, so-called project irrigation efficacy, is composed of 1) conveyance efficiency (Ec), 2) field canal efficiency (Eb) or distribution efficiency, and 3) field application efficiency (Ea). The project irrigation efficiency is estimated by multiplying these 3 efficiencies. Table 7.2.3 presents the efficiencies applied in the target project with reference to the recommended efficiencies in the óFAO Irrigation and Drainage Paper No.24, Crop Water Requirementsô, as 0.90 for the conveyance efficiency, 0.90 for the field canal efficiency, 0.60 for the field application efficiency in Jebel Lado and Rejef East scheme, whereby the project irrigation efficiency comes to 0.49 for Jebel Lado and Rejef East. Furrow irrigation methods are adopted in the farmlands because of the gentle terrain and soil feature of loam

Table 7.3.1 Irrigation Efficiencies for Jebel Lado and Rejaf East

Efficiency	E	Remarks
Conveyance Efficiency (Ec)	0.90	Continuous supply
Field Cancel Efficiency (Eb)	0.90	Blocks larger than 20 ha
Field Application Efficiency (Ea)	0.60	Referred to the case of furrow irrigation
Project Irrigation Efficiency	0.49	Overall irrigation efficiency

Source: JICA Project Team based on Crop water requirements No.24 FAO irrigation and drainage paper

Irrigation Project Efficiency or Overall Irrigation Efficiency = Ec.Eb.Ea

The overall efficiency for Jebel Lado Scheme = 0.9*0.9*0.6 = 0.49

7.3.6 Calculation of Scheme/Farm Water Requirements

q = NIR/Ep

Where q is The Scheme irrigation water requirements, NIR is Net irrigation water requirements and Ep is overall irrigation efficiency.

NIR (ETcrop1, ETcrop2 and ETcrop3) is expressed in average mm/day, in mm/month and in l/s/ha.

Therefore q = NIR (mm/day)/Ep = NIR ((mm*ha)/(24 hr*ha))/Ep

= NIR ((mm*(10000 m^2))/((24*60*60 s)*ha))/Ep

= NIR (((10^{-3} m)*(10000 m^2))/((86400 s)*ha))/Ep

= NIR ((10^{-3} *10^4 m^3)/((86400 s) *ha))/Ep

= NIR (10 m^3)/(86400 s)/ha/Ep

= NIR (10*(1000 l)/(86400 s)/ha/Ep

= NIR (10000 l)/(86400 s)/ha/EP

= NIR ((10000/86400) l/s/ha)/Ep

= NIR ((1/8.64) l/s/ha)/EP = NIR (0.1157 l/s/ha)/0.49

C.F = 1/8.64 = 0.1157

Hence q = NIR ((C.F) l/s/ha)/0.49

Where C.F is Conservation factor from mm/day to l/s/ha

(2) Calculated water requirement

The calculation of Jebel Lado scheme irrigation water requirement is shown in Table 7.3.2 and Table 7.3.3.

Table 7.3.2 Jebel Lado Scheme Irrigation Water Requirements

1. Site	Jebel Lado
2. Command Area	1,330 ha
3. Water Source	River
3. Irrigation Facility	Pump
4. Irrigation Water Requirement	Pump 1.92 m^3/s q= 1.444 l/s/ha

Table 7.3.3 Calculation of Irrigation Water Requirements per Month for Jebel Lado Scheme

Water Requirement: Jebel Lado

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
		Dry Season			Rainy Season							Dry Season		
(1) ETcrop														
Min Temperature	(°C)	20.0	22.5	24.4	23.7	22.7	21.8	21.1	21.3	21.1	21.4	20.9	20.6	21.8
Max Temperature	(°C)	36.6	38.1	37.2	35.6	33.1	31.3	31.2	30.6	32.7	33.3	34.0	35.2	34.1
Relative Humidity	(%)	37	35	42	62	73	78	78	82	79	76	63	52	63.1
Wind speed	(km/day)	112	130	156	216	156	104	95	95	95	112	112	121	125.3
Sunshine	(hours)	11.8	11.9	12.0	12.2	12.3	12.4	12.3	12.3	12.1	12.0	11.9	11.8	12.1
Radiation	(MJ/m2/day)	25.7	27.1	28.1	28.2	27.3	26.7	26.8	27.8	28.0	27.3	26.0	25.2	27.0
ETo	(mm/day)	5.84	6.64	7.20	7.08	5.96	5.35	5.30	5.39	5.66	5.67	5.56	5.66	CropWar8.0
Crop 1		Vegetable			Maize	Maize	Maize	Maize	Maize		Vegetable	Vegetable	Vegetable	
Crop 2		Vegetable	Vegetable				Maize	Maize	Maize	Maize	Maize		Vegetable	
Crop 3		Banana	Banana	Banana	Banana	Banana	Banana	Banana	Banana	Banana	Banana	Banana	Banana	
Crop coeffient 1	Kc1	1.06	0.00	0.00	0.90	1.03	1.15	1.12	0.52	0.00	0.90	1.02	1.19	
Crop coeffient 2	Kc2	1.10	1.08	0.00	0.00	0.00	0.90	1.03	1.15	1.12	0.52	0.00	0.97	
Crop coeffient 3	Kc3	0.45	0.50	0.60	0.70	0.85	1.00	1.10	1.10	0.90	0.80	0.80	0.95	
Etcrop 1 (ET_0 x Kc1)	(mm/day)	6.19	0.00	0.00	6.37	6.14	6.15	5.94	2.80	0.00	5.10	5.67	6.74	
Etcrop 2 (ET_0 x Kc2)	(mm/day)	6.42	7.17	0.00	0.00	0.00	4.82	5.46	6.20	6.34	2.95	0.00	5.49	
Etcrop 3 (ET_0 x Kc3)	(mm/day)	2.63	3.32	4.32	4.96	5.07	5.35	5.83	5.93	5.09	4.54	4.45	5.38	
(2) Effective Rainfall (Pe)														
Monthly Mean Rainfall	(mm/month)	4.4	11.8	41.7	105.6	149.2	116.1	134.0	136.7	112.4	112.6	42.9	9.8	977.2
Dependable Rainfall (80%)	(mm/month)	3.8	10.3	36.5	92.3	130.5	101.5	117.2	119.5	98.3	98.5	37.5	8.6	854.5
Effective Rainfall (ER)	(mm/month)	0.0	0.0	12.0	50.0	80.0	57.0	70.0	72.0	55.0	55.0	13.0	0.0	464.0
Effective Rainfall (ER)	(mm/day)	0.0	0.0	0.4	1.7	2.7	1.9	2.3	2.4	1.8	1.8	0.4	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)	6.19	0.00	0.00	4.67	3.44	4.25	3.64	0.40	0.00	3.30	5.27	6.74	
Etcrop 2	(mm/day)	6.42	7.17	0.00	0.00	0.00	2.92	3.16	3.80	4.54	1.15	0.00	5.49	
Etcrop 3	(mm/day)	2.63	3.32	3.92	3.26	2.37	3.45	3.53	3.53	3.29	2.74	4.05	5.38	
(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
Field Application Efficiency	Ea	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	Furrow
Total Irrigation Efficiency	Ep	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	
(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)	1.46	0.00	0.00	1.10	0.81	1.00	0.86	0.09	0.00	0.78	1.24	1.59	
Crop 2	(l/s/ha)	1.52	1.69	0.00	0.00	0.00	0.69	0.75	0.90	1.07	0.27	0.00	1.30	
Crop 3	(l/s/ha)	0.62	0.78	0.93	0.77	0.56	0.81	0.83	0.83	0.78	0.65	0.96	1.27	
(9) Command Area														
Crop 1	(ha)	660	660	660	660	660	660	660	660	660	660	660	660	
Crop 2	(ha)	600	600	600	600	600	600	600	600	600	600	600	600	
Crop 3	(ha)	70	70	70	70	70	70	70	70	70	70	70	70	
Total	(ha)	1,330	1,330	1,330	1,330	1,330	1,330	1,330	1,330	1,330	1,330	1,330	1,330	
(10) Water Requirement for Pump														
Crop 1	(m3/s/Crop1)	0.96	0.00	0.00	0.73	0.53	0.66	0.57	0.06	0.00	0.51	0.82	1.05	
Crop 2	(m3/s/Crop2)	0.91	1.01	0.00	0.00	0.00	0.41	0.45	0.54	0.64	0.16	0.00	0.78	
Crop 3	(m3/s/Crop3)	0.04	0.05	0.07	0.05	0.04	0.06	0.06	0.06	0.05	0.05	0.07	0.09	
Total	(m3/s/Total)	1.92	1.07	0.07	0.78	0.57	1.13	1.08	0.66	0.70	0.72	0.89	1.92	
(11) Water Requirement for Dam	(m3/month/Total)	4,974,048	2,769,811	168,739	2,021,501	1,487,290	2,930,774	2,788,214	1,704,240	1,805,587	1,872,202	2,295,475	4,972,234	29,790,115

Required Pump Discharge	1.92	(m3/s)
Annual Water Requirement	29,790,115	(m3/year)

CHAPTER 8 FACILITY PLAN AND DESIGN

8.1 General

8.1.1 Outline of Main Facilities

Main facilities planed in Jebel Lado area are as follows,

- Command area: A=1330ha (Northern Block 560ha, Southern Block 770ha)
- Pump station: 1 place
- Northern Main Irrigation Canal: L=6.4km
- Southern Main Irrigation Canal: L=7.0km
- Irrigation and Drainage Facilities iin command area: 1 L.S
  Secondary canal, Tertiary canal, Feeder canal, Drainage, Road, Road crossing, Distribution gate, Water measurement facilities, etc.

Pump facility are operated through the year for farming, withdrawing from Bahr el Jebel. For reference, there is a candidate site for dam but the site is very far of 25km from the subject command area and the construction cost of the dam and the distribution canal requires numerous expenses. Therefore it is judged that the irrigation method operated by the dam is dismissed due to its economic efficiency.

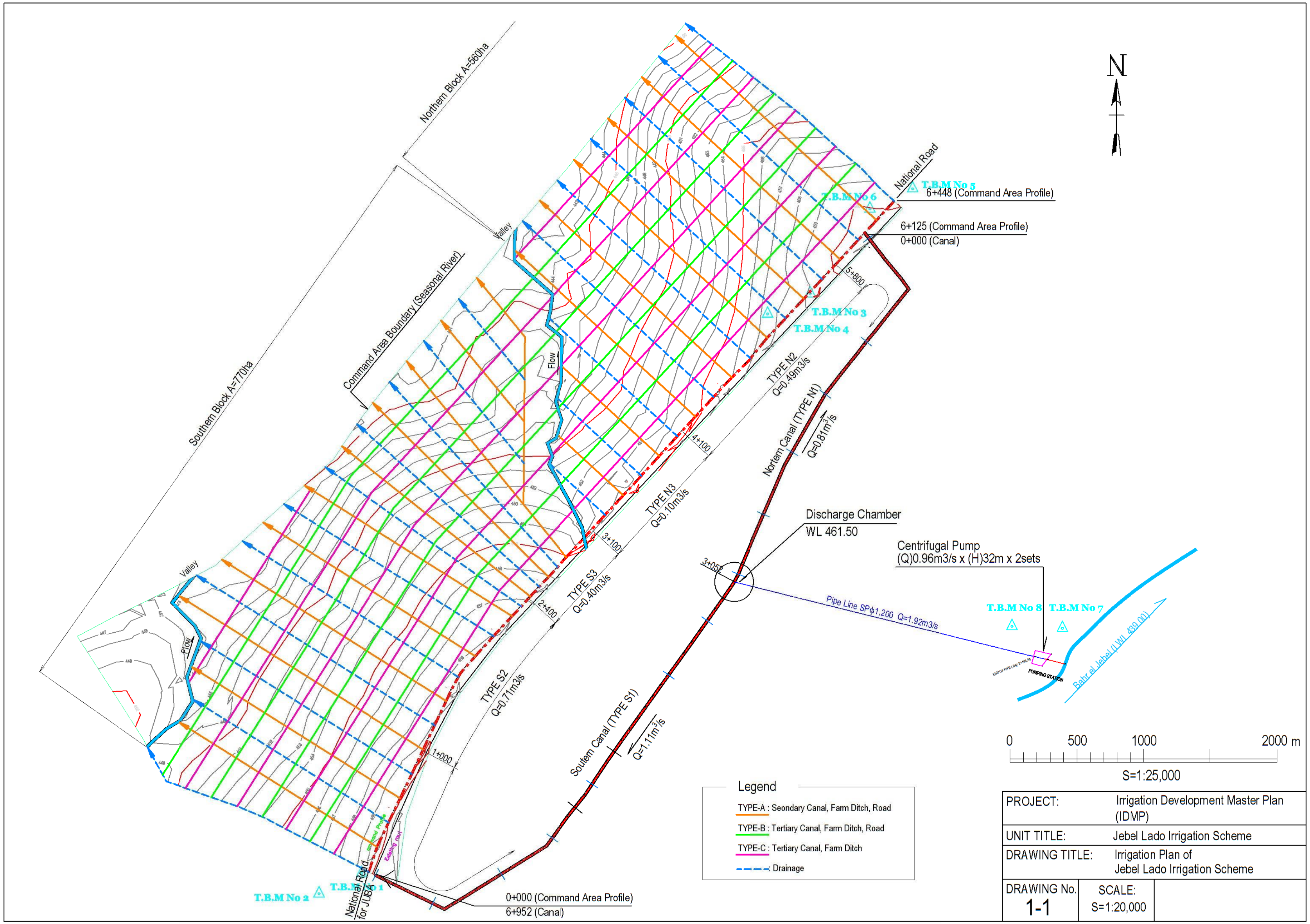
8.1.2 Command Area

The command area is located 3.5km from Bahr el Jebel. Bushes, trees and grasses dominate in the site. The terrain is almost flat and the land gradient shows around 0.9% toward the northwest from the southeast.

The command area is divided into two (2) blocks of north and south by the valley located in the middle of the command area. The drainage from the command area flows down to the seasonal river directly.

Pump station site is located beside Bahr el Jebel. The land is almost bare and some trees are shown. In the pipe line and canal line, there are community road among some small communities, bushes and trees etc. along the line.

Figure 8.1.1 Location Map



8.2 Pump Station

(1) Location

The pump station is planned at the left bank of Bahr el Jebel. The location of pump station should be selected, considering the few silting in front of an intake. The outside of curved course of river is generally suitable for an intake point because of comparatively few silting.

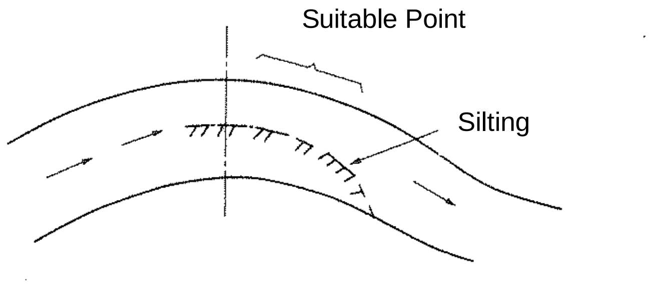


Figure 8.2.1 Flow of Curved River

Certainly the location of pump station was decided at a suitable point as shown in the figure 8.2.2

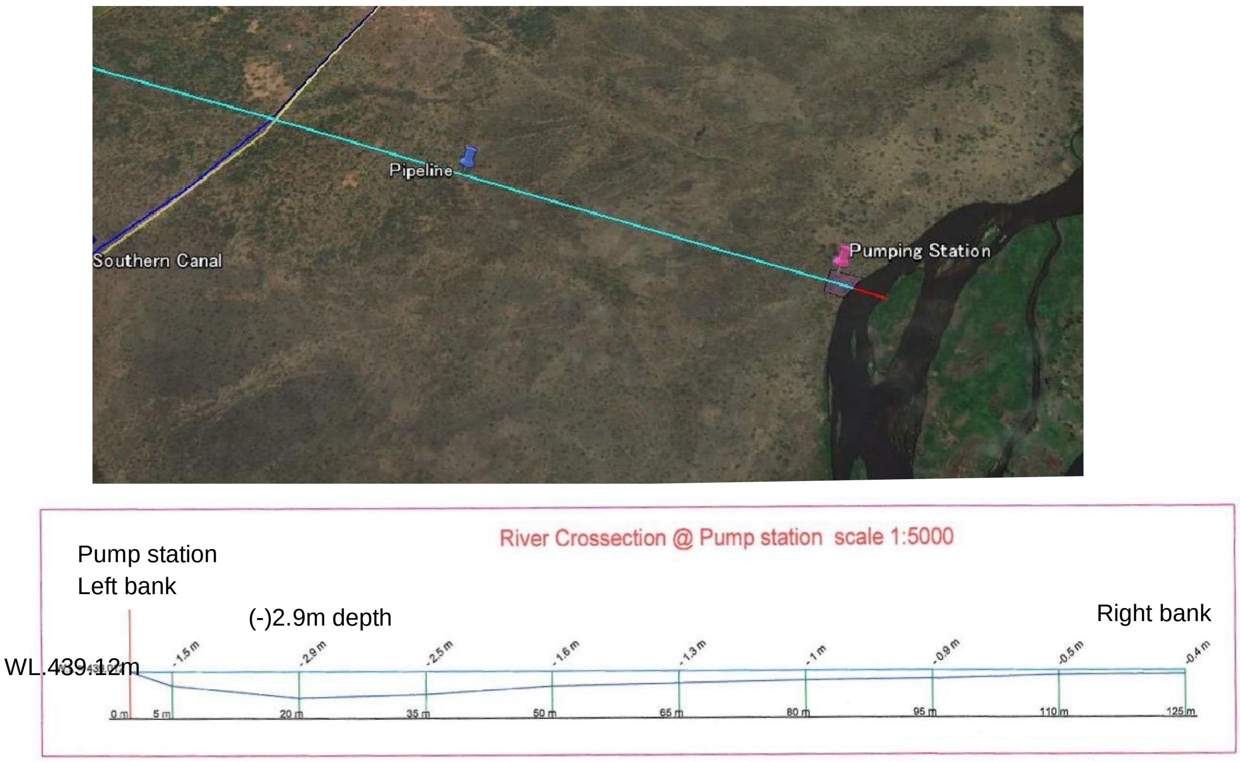


Figure 8.2.2 Location and River Cross Section at Pump Station

(2) 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.

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

$$0.96\text{m}^3/\text{s (unit capacity)} \times 2 \text{ set} = 1.92 \text{ m}^3/\text{s}$$

Table 8.2.1 Water Requirement (m^3/s)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Average
1.92	1.07	0.07	0.78	0.57	1.13	1.08	0.66	0.70	0.05	0.89	1.92	0.90

According to the ñPump Design Guideline in Japanò, the pump diameter is determined 700mm based on the pump capacity of 0.96m^3/s.

2) Total head of pump

The actual head is given as the difference between the discharge water level and the suction water level. The total head is obtained by adding various losses in pipes to the actual head.

Table 8.2.2 Total Head of Pump

Items	Unit	Dimension
Pump Capacity per Unit	Q (m^3/s)	0.96
Design outlet Water level	DWL (m)	461.50
Design Intake Water Level	LWL (m)	439.00
Actual head	Ha (m)	22.50
Total Head Loss	(m)	8.62
Total Head	(m)	31.12
Design Total Head	H (m)	32.00

3) 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 the capacities of 402kw depending on the calculation.

(3) 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.

Based on the considerations of the space of installation for the pumps, engines, valves and auxiliary equipment and the required space for effective O&M works, the plan and section of pump station building are planned as shown in the Figure 8.2.3 and Figure 8.2.4.

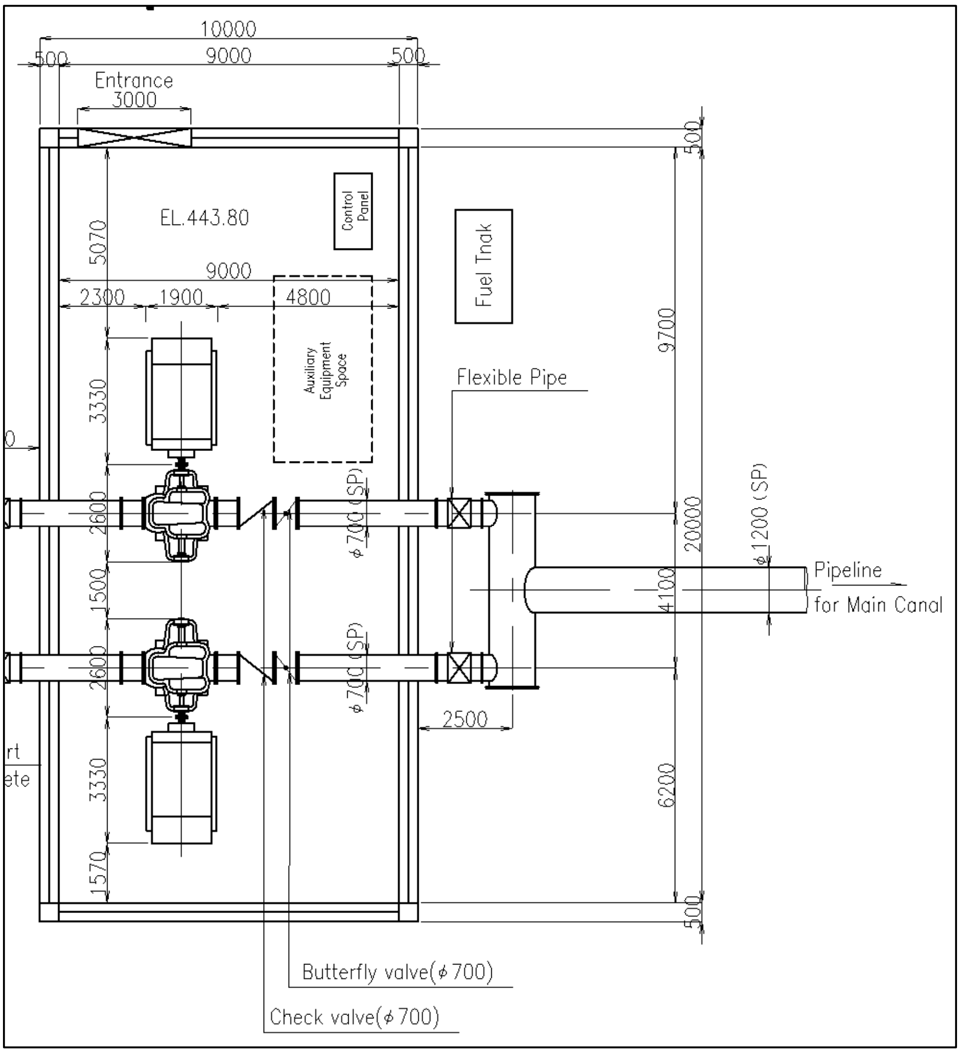


Figure 8.2.3 Plan and Section of Pump Station Building (Plan)

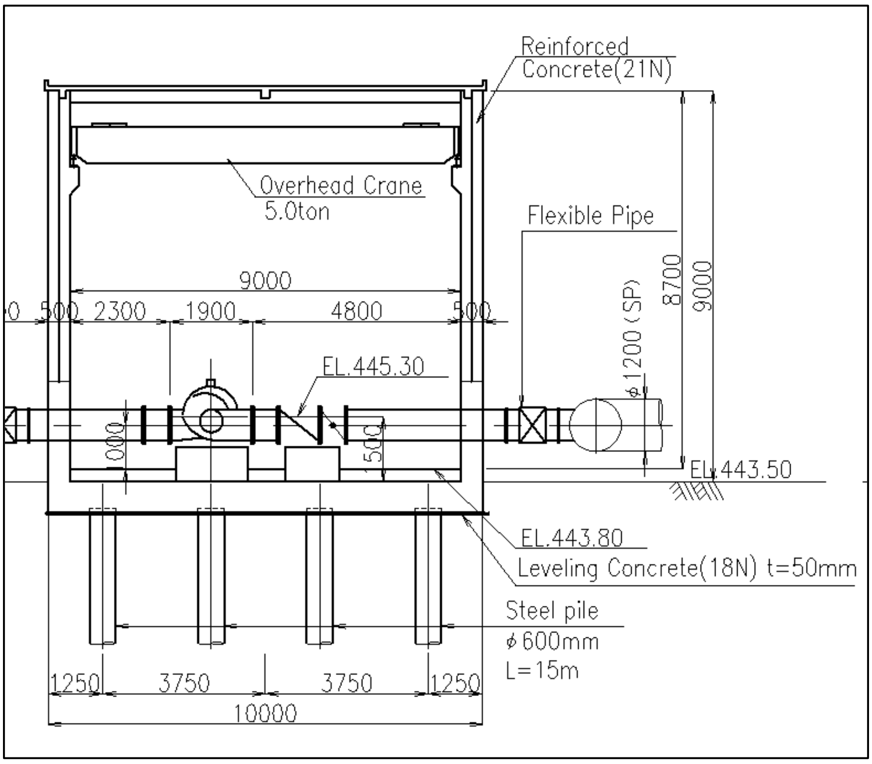


Figure 8.2.4 Plan and Section of Pump Station Building (Profile)

2) 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.

3) Foundation work

According to the log of boring at the pump station, the geological conditions are featured from loose to medium dense and it is not suitable for the spread foundation. It is recommended to adopt the pile foundation in future design stage through the additional geological investigation for clarifying the very dense layer.

(4) Riverbank protection

1) Installation range

Due to the possible erosion by river flow, however, the suction pipe embedded underground might be exposed and the safety of the pipe could be endangered with the trashes/drifts clinched. This requires the riverbank protection works for attaining sustainable operation of the pump station. The extent of the riverbank protection shall cover 20 m each of both upstream and downstream directions from the centre of suction pipe.

2) Structure type

The structure of mortar masonry retaining wall by using natural stones shall be adopted for the protection works, considering the gentle slope of 1:2.0, safety against the effect of water flow, availability of required materials, other viewpoints including landscape evaluation, economy and easiness in construction etc.

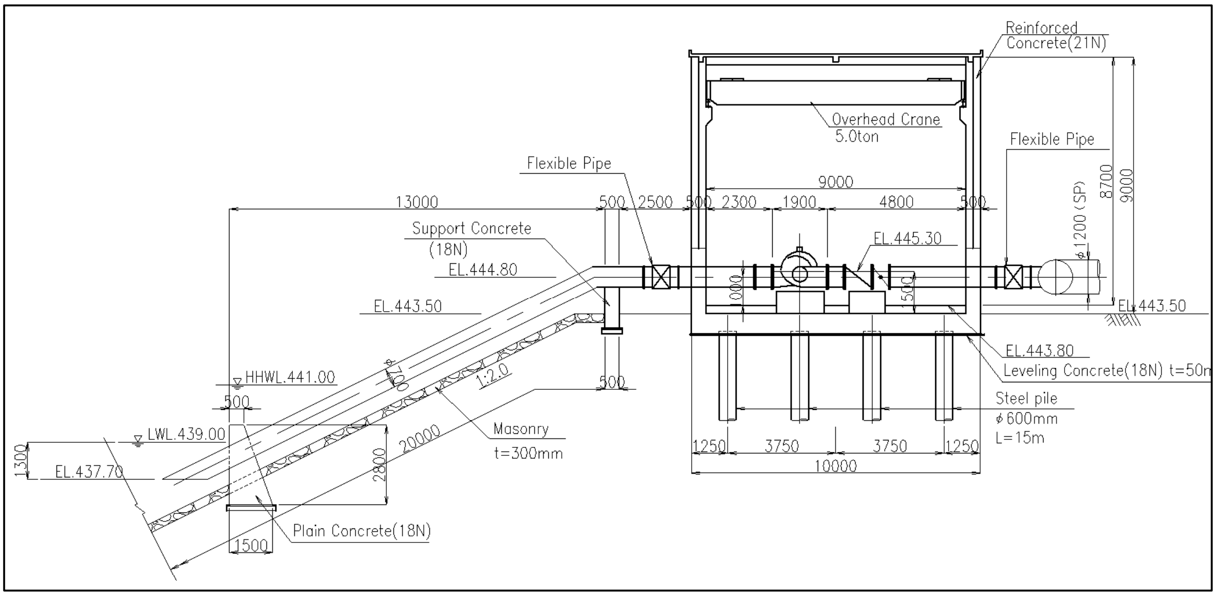


Figure 8.2.5 Plan and Section of Pump Station Building (Profile)

(5) Pipeline

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 1200mm 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.

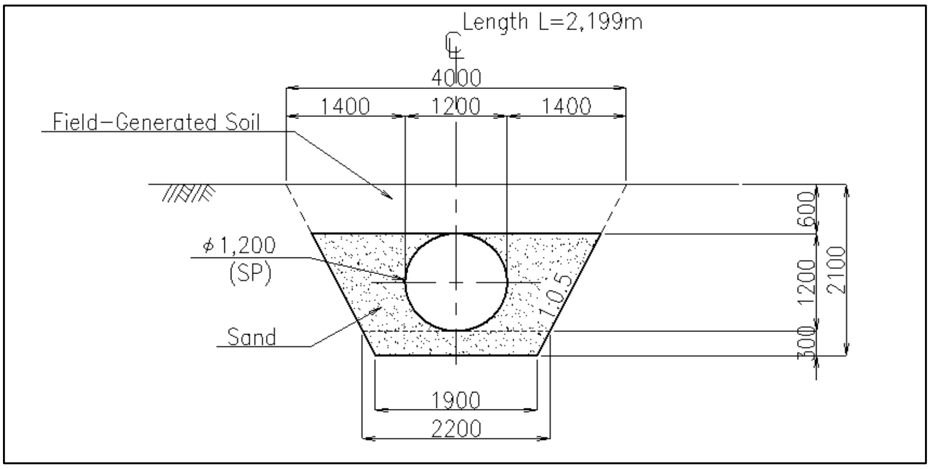


Figure 8.2.6 Typical Section of Pipeline

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.

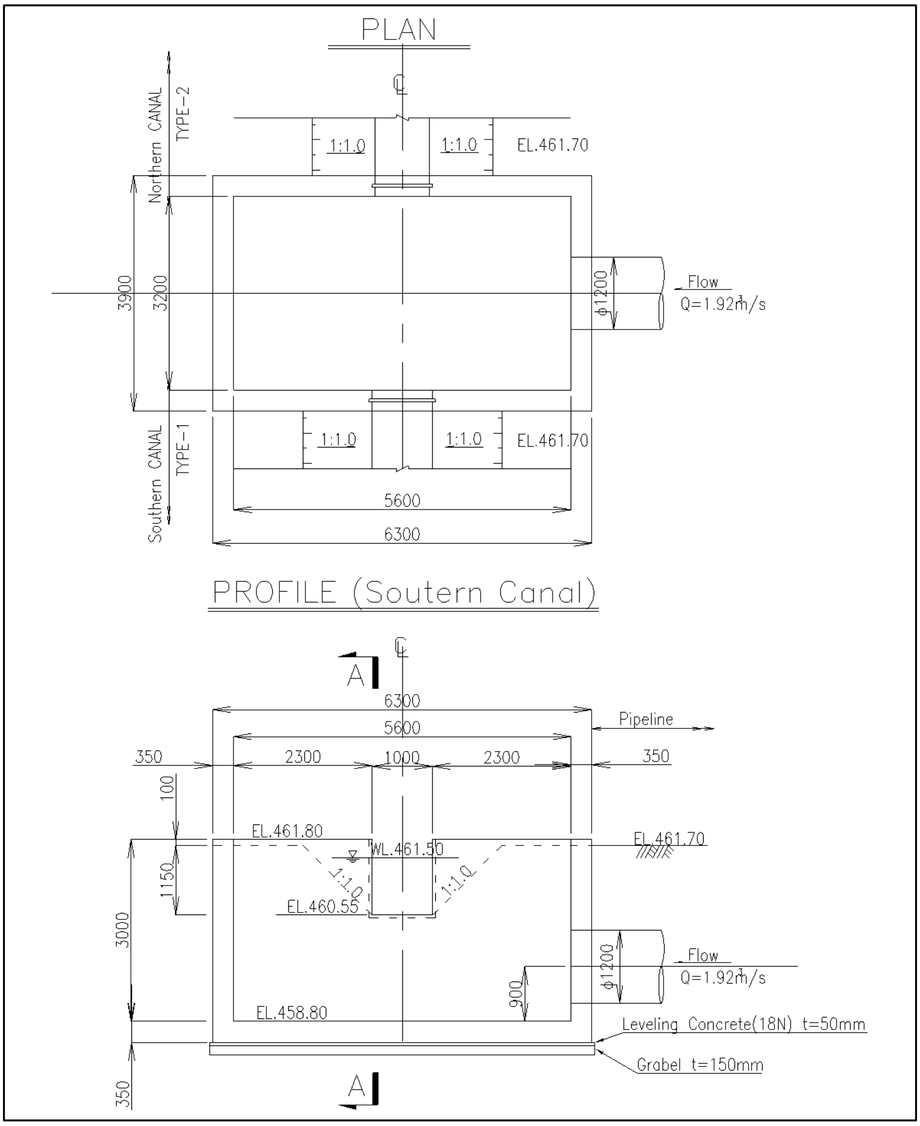


Figure 8.2.7 Discharge Chamber

8.3 Main Canal

(1) Location

Main canal shall be planned to conduct the irrigation water from the discharge chamber at the end of pipeline to the command area. Also main canal is planned in the east side of command area which is featured as the high land comparatively. The main canal is separated to two (2) routes, one is the northern canal which is the length of 6.4km, and another is the southern canal which is the length of 7.0km.

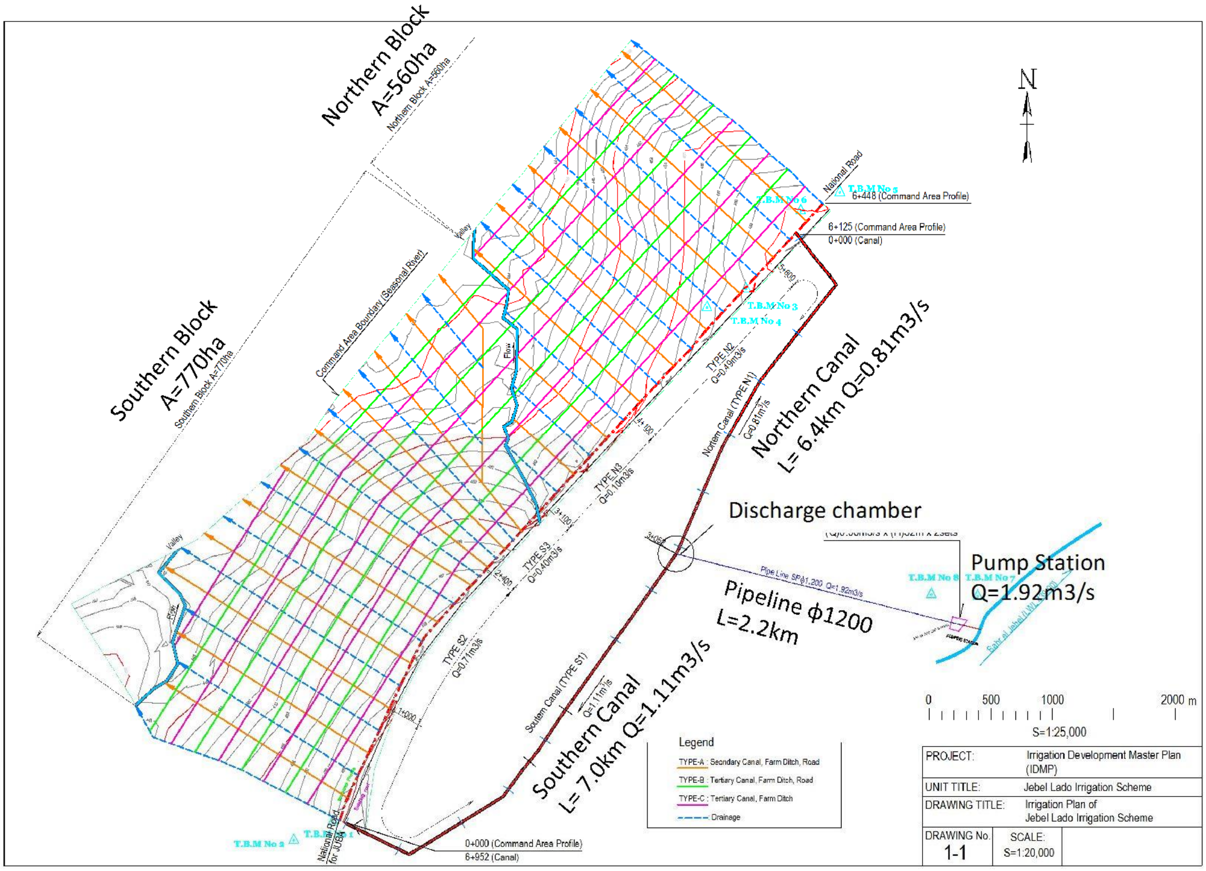


Figure 8.3.1 Location Map

The station number, length, and design discharge of each section is shown in the Table 8.3.1

Table 8.3.1 Main Canal

Type	Station	Length (m)	Design Discharge (m3/s)
Northern Canal		6.400	
N1	3+052 + 0.000, and 6+125 to 5+800	3,052 325	0.81
N2	5+800 to 4+100	1,700	0.49
N3	4+100 to 3+100, and 6+125 to 6+448	1,000 323	0.10
Southern Canal		7,000	
S1	3+052 to 6+952, and 0+000 to 1+000	3,900 1,000	1.11
S2	1+000 to 2+400	1,400	0.71
S3	2+400 to 3+100	700	0.40

(2) Examination method of canal capacity

Main canal is designed of the plain concrete lining, considering hydraulic characteristics, conveyance efficiency, durability, and maintenance. The size of the cross section is planned by the volume of the required water with Manning formula.

Table 8.3.2 Calculation of Main Canal Section

Items		N1	N2	N3	S1	S2	S3
Design discharge	Q (m^3/s)	0.81	0.49	0.10	1.11	0.71	0.40
Width of canal bed	B (m)	0.90	0.50	0.30	1.00	0.60	0.40
Water depth	d (m)	0.843	0.401	0.280	0.948	0.531	0.347
Bank slope	1:N	1.0	1.0	1.0	1.0	1.0	1.0
Cross-sectional area of flow	A (m)	1.469	0.361	0.162	1.847	0.601	0.259
Wetted perimeter	P (m)	3.284	1.634	1.092	3.681	2.102	1.381
Hydraulic mean depth	R (m)	0.447	0.221	0.148	0.502	0.286	0.187
Coefficient of roughness	n	0.015	0.015	0.015	0.015	0.015	0.015
Canal bed slope	I (%)	0.02	0.315	0.11	0.02	0.167	0.50
Mean velocity	V (m/s)	0.551	1.361	0.620	0.595	1.181	1.544
Velocity head	hv (m)	0.016	0.094	0.020	0.018	0.071	0.122
Free board	Fb (m)	0.207	0.249	0.170	0.202	0.219	0.253
Height of canal	H	1.05	0.65	0.45	1.15	0.75	0.60

Canal profile and canal section are shown in the Figure 8.3.2, Figure 8.3.3 and Figure 8.3.4 respectively.

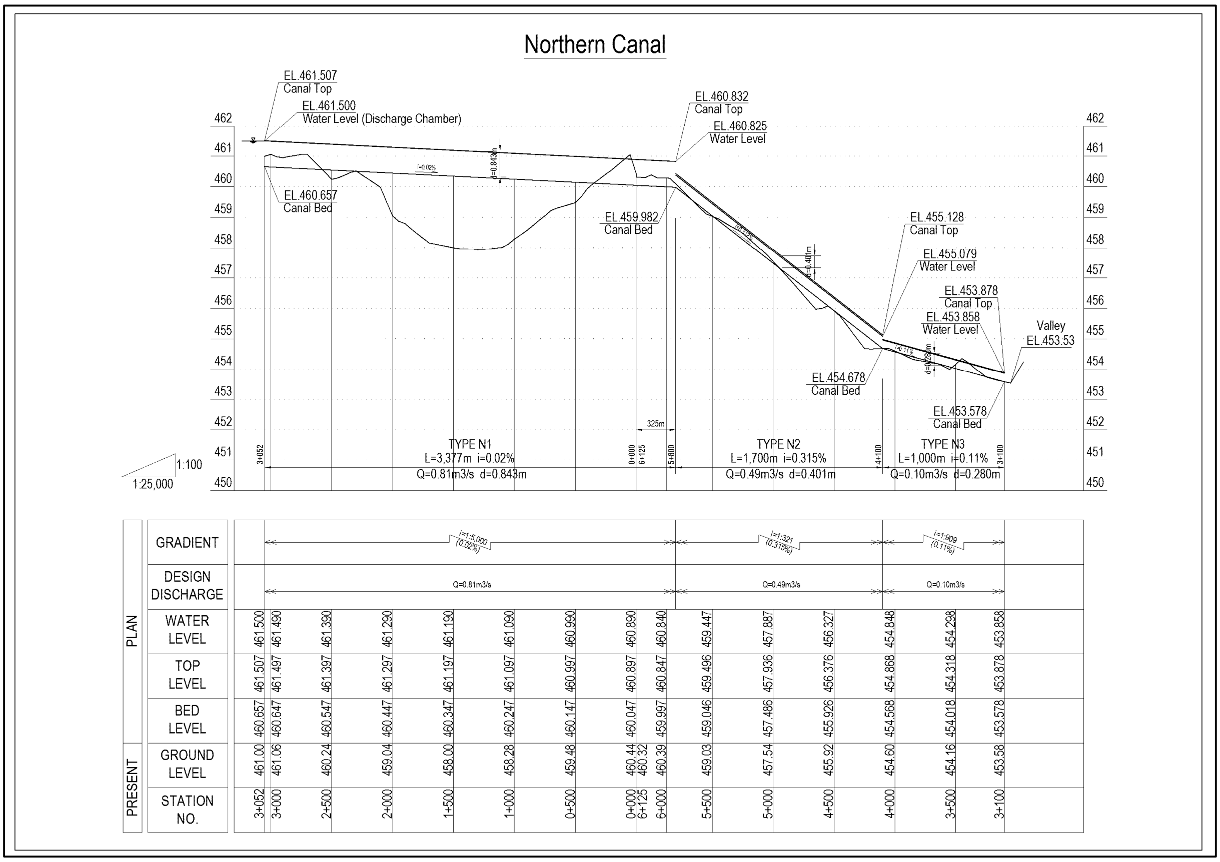


Figure 8.3.2 Northern Canal Profile

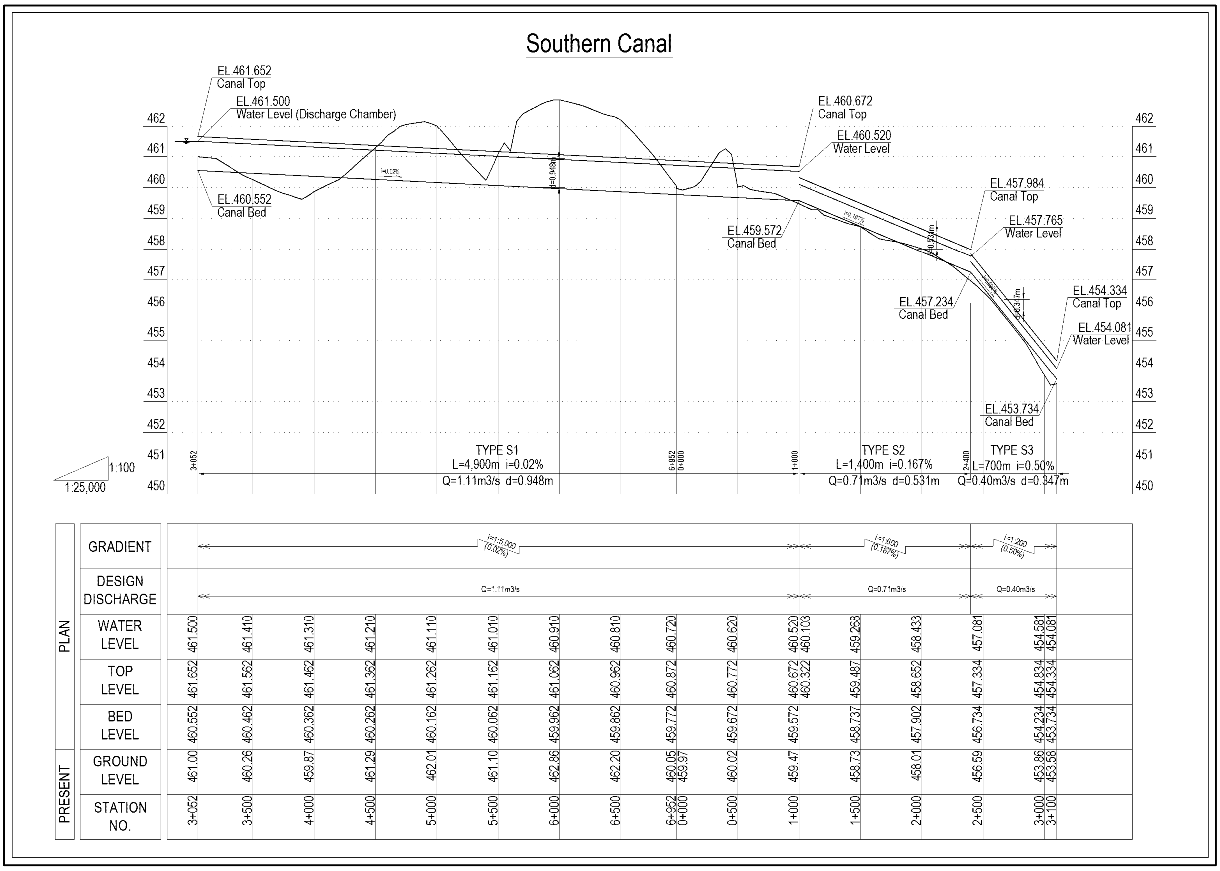


Figure 8.3.3 Southern Canal Profile

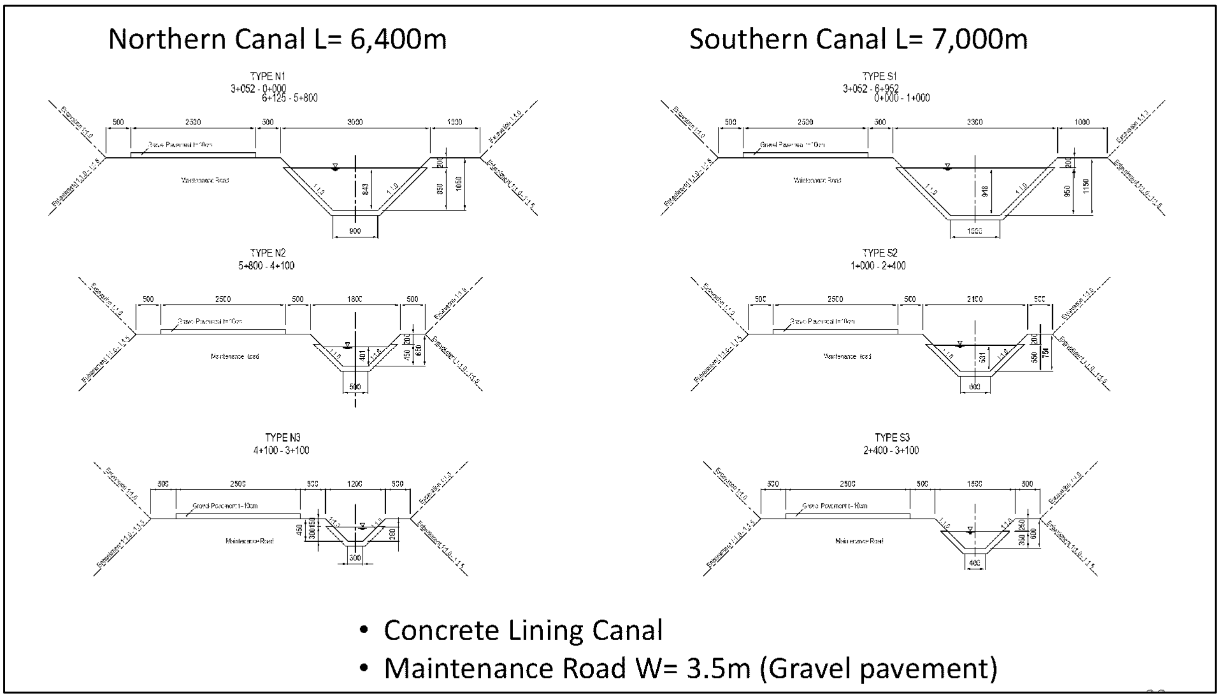


Figure 8.3.4 Typical Cross Section of Main Canal

(3) Recommendation

As shown the canal profiles of Figure 8.3.2 and Figure 8.3.3, the route of northern canal in the survey work is located in the low land and the numerous earthworks for embankment are required in this section. On the other hand, the route of southern canal in the survey work is located in the high land and the numerous earthworks for excavation are required in this 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.

8.4 Irrigation and Drainage System in Farmlands

(1) Outline of command area

As for the command area shown in the Figure 8.4.1 and Figure 8.4.2, the major facilities such as the secondary canal, drainage and road are generally arranged from highland toward lowland. The tertiary canal is planned to branch off from the secondary canal, also the feeder canal is planned to branch off from the tertiary canal for the distributing the irrigation water to the furrows. The canal is made of the earth because of a small size.

The drainage is allocated between the both secondary canals. The surplus water from tertiary canal and farm flows down to the drainage. The drainage is planned to be the earth canal.

The road used for farming and maintenance for facilities shall be planned along the secondary canal and tertiary canal. The road crossing is placed at the crossing point between the canal and road.

The length of furrow of 100m is assumed because the soil classification is mainly loam at the site.

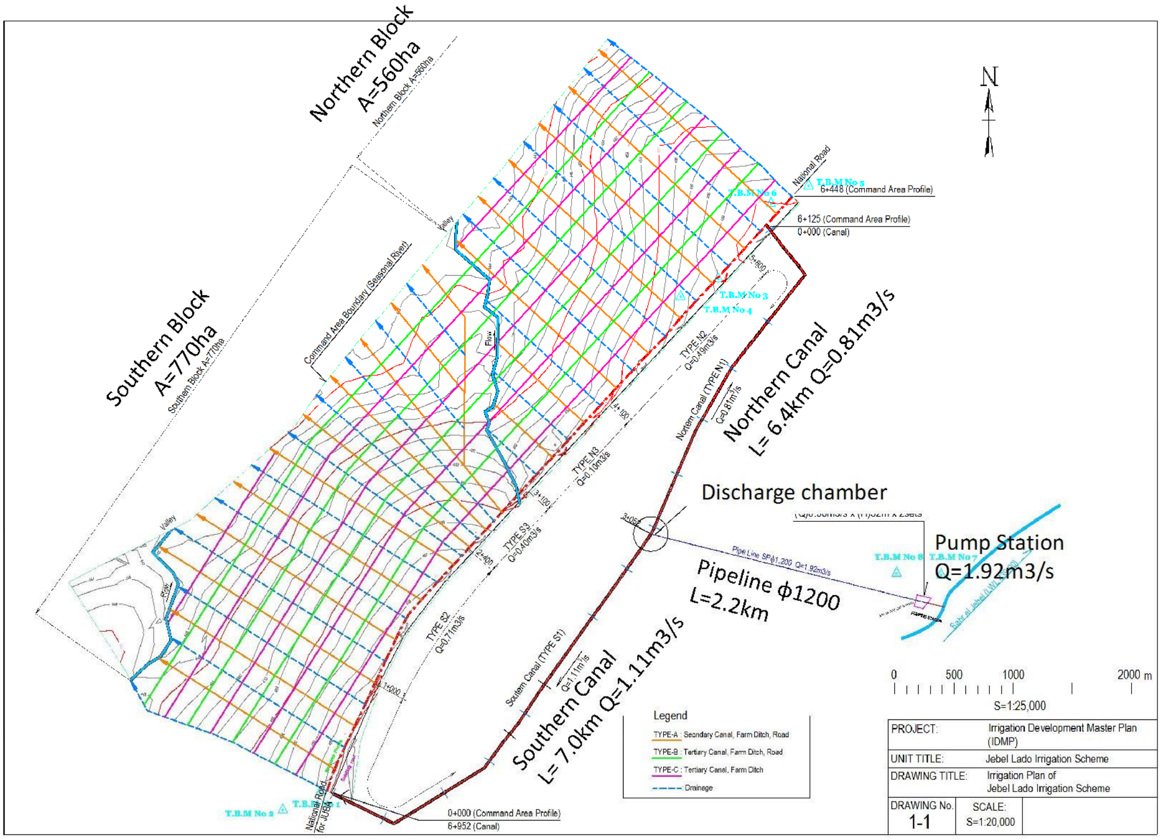


Figure 8.4.1 Location Map

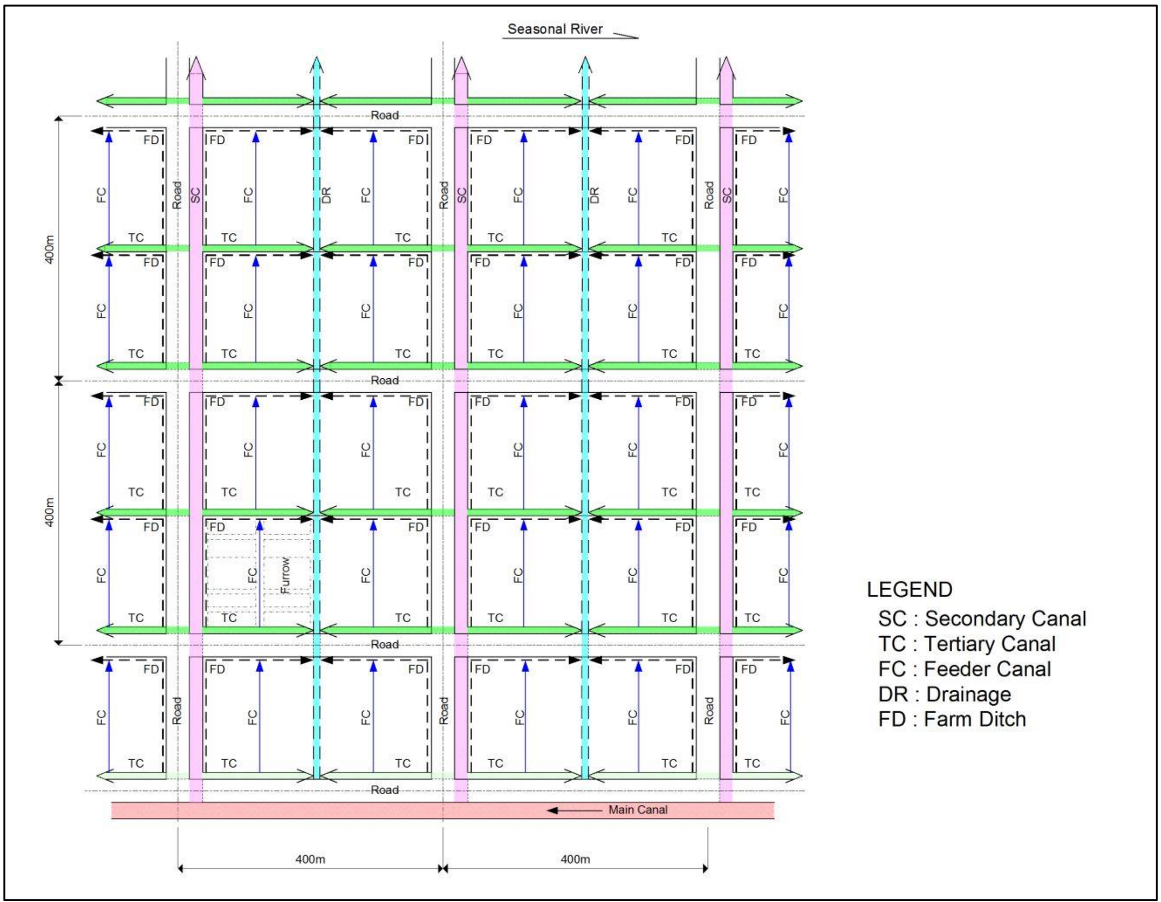


Figure 8.4.2 Layout of Irrigation and Drainage Facilities in Command Area

(2) Design discharge of canal and drainage

1) Irrigation canal

Unit water requirement was estimated at 1.44 l/s/ha, depending on the calculation of the water requirement.

- Secondary canal: Q= 0.12m^3/s (=0.00144×averagely 80ha)
- Tertiary canal and Feeder canal: Q= 0.023m^3/s (=0.00144×16ha)

2) Drainage

Unit area drainage discharge was estimated at 0.045m3/s/ha, depending on the calculation of runoff discharge analysis.

- Drainage : Q= 3.78m^3/s (=0.045× averagely 84ha)
- Farm ditch: Q= 0.090m^3/s (=0.045×averagely 2ha)

(3) Examination method of canal capacity

All of canals in the command area are designed of the earth canal, considering the economical reason. The size of the cross section is planned by the volume of the required water with Manning formula.

Table 8.4.1 Calculation of Irrigation Canal Section and Farm Ditch

Items		Secondary canal	Tertiary canal	Feeder canal	Farm ditch
Design discharge	Q (m^3/s)	0.12	0.023	0.023	0.090
Width of canal bed	B (m)	0.30	0.30	0.30	0.30
Water depth	d (m)	0.270	0.174	0.092	0.182
Bank slope	1:N	1.0	1.0	1.0	1.0
Cross-sectional area of flow	A (m)	0.154	0.082	0.036	0.088
Wetted perimeter	P (m)	1.064	0.792	0.560	0.815
Hydraulic mean depth	R (m)	0.145	0.104	0.064	0.108
Coefficient of roughness	n	0.025	0.025	0.025	0.025
Canal bed slope	I (%)	0.50	0.10	1.0	1.0
Mean velocity	V (m/s)	0.780	0.279	0.642	0.907
Velocity head	hv (m)	0.031	0.004	0.021	0.042
Free board	Fb (m)	0.180	0.126	0.203	0.168
Height of canal	H (m)	0.45	0.30	0.30	0.35

Table 8.4.2 Calculation of Drainage Section

Items / Type		SD-1	SD-2	SD-3	SD-4	SD-5
Design discharge	Q (m^3/s)	3.78	2.88	2.16	1.44	0.72
Width of canal bed	B (m)	1.30	1.10	1.00	0.80	0.60
Water depth	d (m)	1.233	1.074	0.931	0.786	0.585
Bank slope	1:N	1.5	1.5	1.5	1.5	1.5
Cross-sectional area of flow	A (m)	3.883	2.912	2.231	1.555	0.864
Wetted perimeter	P (m)	5.746	4.972	4.357	3.634	2.709
Hydraulic mean depth	R (m)	0.676	0.586	0.512	0.428	0.319
Coefficient of roughness	n	0.025	0.025	0.025	0.025	0.025
Canal bed slope	I (%)	0.100	0.127	0.143	0.167	0.200
Mean velocity	V (m/s)	0.974	0.990	0.968	0.927	0.835
Velocity head	hv (m)	0.048	0.050	0.048	0.044	0.036
Free board	Fb (m)	0.217	0.226	0.219	0.214	0.165
Height of canal	H (m)	1.45	1.30	1.15	1.00	0.75

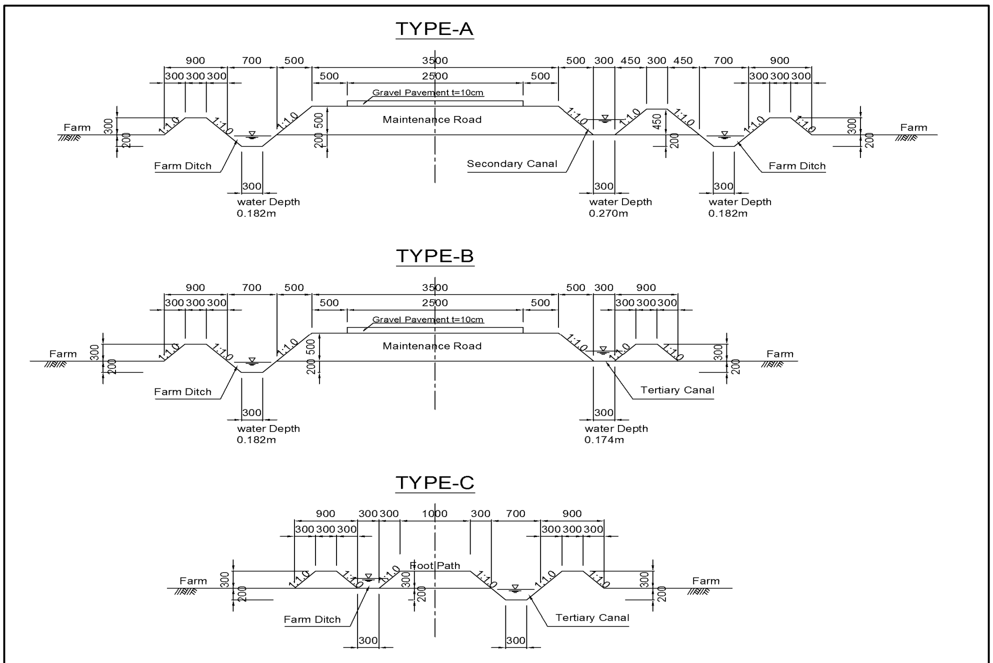


Figure 8.4.3 Typical Cross Section of Irrigation and Drainage Facilities in Command Area

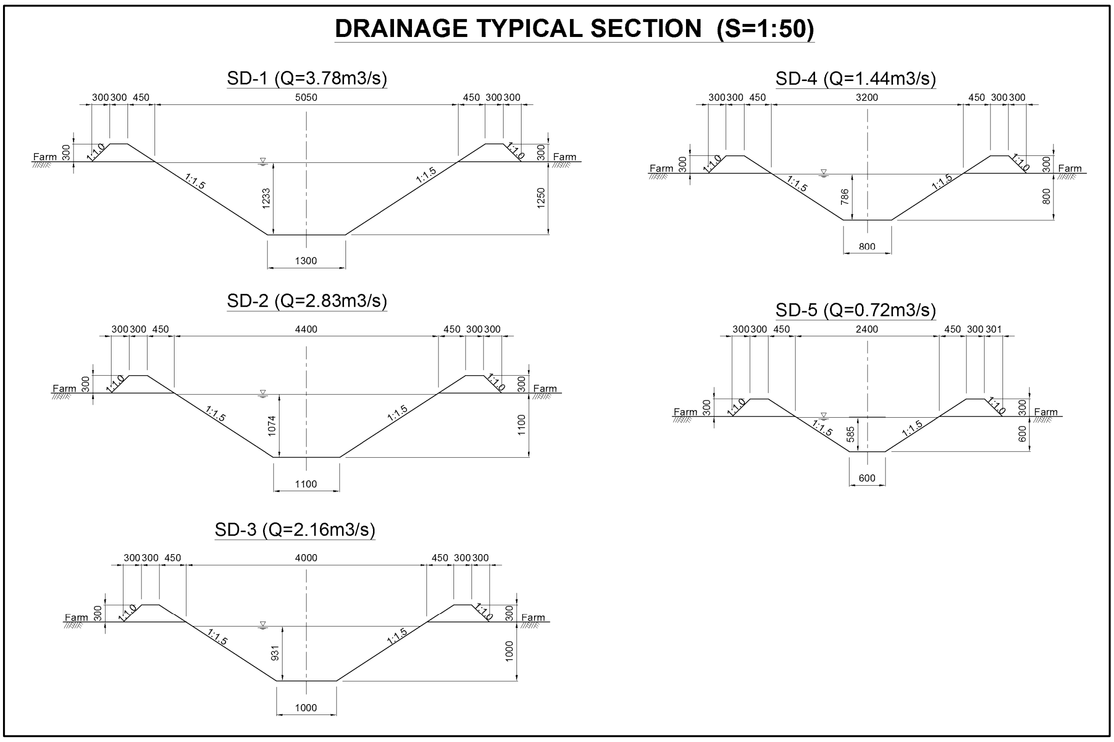


Figure 8.4.4 Typical Cross Section of Drainage in Command Area

(4) Relative structures

In general the relative structures such as diversion gate, drop, water measurement facilities, cross culvert and siphon etc. are required in the canal system if necessary. They shall be designed considered the canal system and the terrain around canal in the future design stage.

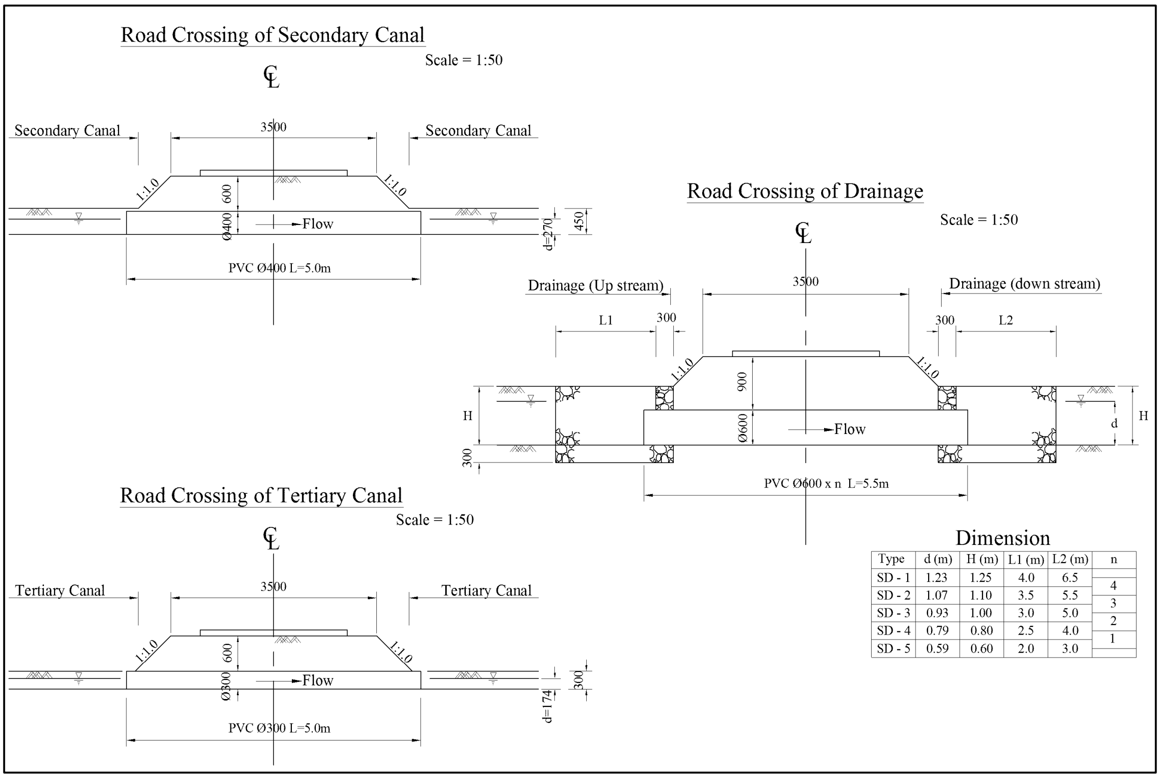


Figure 8.4.5 Typical Cross Section of Road Crossing in Command Area

(5) Recommendation

1) Design of upland irrigation and intake rate

In the future design stage, more investigation is required to carry out the design of upland. For example, the intake rate is very important factor to make a plan of irrigation system. The intake rate is the rate for irrigation water or rainwater infiltration into soil under the specific conditions, and generally measurement in term of mm/hr. As an index of water permeability in unsaturated soil, it is an important factor to be considered in deciding the irrigation method and the appropriate irrigation intensity for upland irrigation.

The intake rate is measured either by the cylinder intake rate or by the furrow intake rate, depending on the purpose of the measurement. For furrow irrigation, the intake rate is measured by the furrow intake rate.

2) Drainage in command area

As shown in the table 8.4.2 of calculation of drainage section, the canal bed slope is determined gentler than the existing ground gradient of 0.9% so that the velocity of canal does not exceed the maximum allowable velocity of 1.0m/s by clay. In this case, many drop structures are required on the drainage line at the site. It is recommended to study the canal type more or compare with the concrete lining canal from the viewpoint of economical efficiency. Moreover, it is recommended to review the unit area drainage discharge whether it could be reduce or not.

CHAPTER 9 OPERATION AND MAINTENANCE PLAN

9.1 Establishment of Scheme Management Office

Irrigation method in Jebel Lado and Rejaf East is pump irrigation. Pump irrigation scheme also required a senior pump engineer to maintain its operational function properly. Under the senior pump engineer, several support specialists are required in accordance with scale of the scheme. For example, assignment of electric engineer and service technician are necessary. Following table shows ideal management structure of pump irrigation scheme.

Table 9.1.1 Management Structure of Jebel Lado Irrigation Scheme

Department	Functions and Responsibilities	Required Staff	Proposed No.
1. Admin.	• Overall management of the scheme • Coordination among stakeholders	Manager (Irrigation/Dam Eng.)	1
		Deputy Manager (Electromechanical Eng.)	1
	• Marketing • Procurement • Assets tracking • Keeping books of accounts for scheme operations • Irrigation fee collection • Administration of salaries, wages and other disbursements	Senior Accountant	1
		Cooperative Officer	1
		Asst. Accountant	1
		Asst. Cooperative Officer	1
		Tariff Collector	2
		Messenger/Guard/Driver	6
2. Irrigation/Dam O&M	• Annual planning and monitoring of dam/ pump operations, water distribution, etc. • Maintenance of dam/pump facilities, distribution network, etc. • Hydromet data recording, monitoring and reporting	Senior Irri./Dam Eng. (Dams, Pumps, Canals, etc)	1
		Electromechanical Eng.	1
		Planning and Budgeting Officer	1
		Asst. Irrigation/Dam Eng.	1
		Asst. Planning/Budgeting Officer	1
	• Opening, closure and maintenance of water control and distribution gates • Supervision of canals maintenance • Safeguarding of supplies and the facilities	Irrigation Technician	2
		Pump operator	2
		Irrigation Water Controller (Gate Keeper)	2
		Facilities' Guards	4
3. Farm Level O&M	• Seed multiplication, observation trials for new rice varieties • Annual planning and monitoring of cropping plan and water requirement • Extension of irrigated agriculture • On-farm water management planning and supervision	Senior Agronomist	1
		Agronomist	1
		Agricultural Engineer	1
		Asst. Agricultural Engineer	1
	• Provision of outreach services to farmers • On-farm water management among farmers • Supervision of distribution and field canals maintenance	Extension Worker	2
		Tractor Operator	1
		Asst. Tractor Operator	1
4. Processing O&M	• Collection, drying, milling of rice • Storing rice with proper pesticide control	Rice mill operator	0
		Asst. Rice mill operator	0
Total			37

For operation and maintenance purpose, following equipment and machineries are needed.

Table 9.1.2 Ideal Equipment and Machineries at Scheme Management Office

Function	Equipment and Machineries
1. Administration	· PC for accounting and financial management purpose
2. Irrigation Engineering	· PC for planning and data management purpose · Amphibious excavator, Backhoe, Bulldozer, Dump track
3. Agricultural Extension	· PC for planning and data management purpose · Motorbike for extension purpose (4)
4. Farm Operation	· Tractor (7), Harvester (5)
5. O&M	· Working machines (Lathe Machine, Welding Machine, Power Drill, Power Saw, Generator, Portable Generator, etc.)

MEDIWR takes an initiative to organize the Irrigation Scheme Management Office. However, the Scheme Management Office cannot be managed by officials from MEDIWR alone, and collaboration with relevant stakeholders especially MAFCRD and Central Equatoria state government are inevitable. At the time of design work (detail design stage of the irrigation development planning), it is recommended to establish the management office through intensive discussion on function of the management office, demarcation of responsibility, staff allocation, and budget allocation. Also, it is important to discuss the demarcation with WUA. Ideal demarcation among stakeholders is as follows;

Table 9.1.3 Ideal Demarcations among Stakeholders

Stakeholders	Demarcation
1. National Government	· Taking initiative to establish SMO (MEDIWR) · Based on the report from SMO, taking necessary measure to repair or rehabilitate the irrigation system (MEDIWR) · Assign relevant officials to SMO (MEDIWR, MAFCRD)
2. Central Equatoria State Government, Juba County, Payam Office	· Assign relevant officials to SMO · Supervising and support SMO's activities · Coordination among and mobilization of communities
3. Jebel Laso Irrigation Scheme Management Office (SMO)	· Coordinate and facilitate the formation and activities of WUA · O&M of main irrigation facilities (pump station, main and secondary canal, intake gate until on-farm) · Provision of tractor service · Provision of seeds and other inputs if any · Collection of irrigation service fee and tractor service fee
4. WUA	· On-farm level operation and maintenance · Payment of irrigation service fee · Selling of products

9.2 Operation Plan

(1) Water distribution plan

Operation plan includes basic operation plan at feasibility planning stage, and annual operation plan after implementation of the project. Objective of the basic operation plan is to establish basic method of operation, such as selection of water distribution method and order of the water distribution among upstream/downstream or large-/small-scale farmers. Typical water distribution methods are summarized in Table 9.2.1 Responsible organizations at this stage are the scheme management officials from MEDIWR and MAFCRD, and collaboration between both organizations and communities is necessary.

Table 9.2.1 Typical Water Distribution Method in Open Canal Scheme

Method	Description
Flow Sharing (Proportional Delivery) Method	Every farm receives an equal share of the canal discharge. The structure that is suitable for this method of water distribution is the proportional division box. The flow over each weir is proportional to the width of the crest, provided that these crests have the same height and shape. This method does not need any action by farmers or operators for regulating the flow of irritation water to the farms.
Time Sharing (Rotation) Method	Every farm receives the full canal discharge. The distribution of an irrigation delivery to one farm must be chosen in a way that both meets the irrigation water needs of the crops and is convenient to the farmers. With this method, there is no need for a flow division structure. It may be convenient to have structures with allow either closure or passage of the full canal flow. The method does require action from operators or farmers to direct the canal flow to the farm that is schedule to receive irrigation water.

Source: Irrigation Scheme Operation and Maintenance, Irrigation Water Management Training Manual No.10, FAO1996.

(2) Annual operation plan

The annual operation plan includes preparation of cropping calendar, estimation of expected water demand and supply, and irrigation facility operation planning. After irrigation system being constructed, MAFCRD takes responsibility on developing annual clopping calendar, which in turn utilized in estimation of crop water requirement or water demand. Then, water distribution plan is developed by MEDIWR, based on water distribution method, irrigation water availability, and management capacity of gate operator. Basic process of the water management is as follows;

1) The scheme management officials from MAFCRD, in collaboration with farmers, develop cropping calendar and crop water requirement. Then the scheme management officials estimate seasonal water demand of command area
2) Based on the request from the water users, the scheme management officials from MEDIWR decide water volume at intake facility and develop pump operation plan
3) Based on the above plan, in-charge of water control makes schedule of water distribution including gate operation plan
4) The above water distribution plan should be informed to all over the operators at main and branch as well as terminal canals thoroughly.

According to FAOô s guideline for irrigation development, the planning of irrigation schedules should take into consideration the following issues[1].

- Irrigation schedules must be simple, in particular in irrigation schemes where many farmers are involved. It will often be necessary to discuss with the farmers the various alternatives and come to an agreement which best satisfies all parties involved. Important to guarantee is that in these discussions all groups of farmers, small and large, head-end and tail-end, women and men, are properly represented.

- On-demand water delivery ensures the farmers an adequate and timely water supply, in cases where water is not a limiting factor. On-demand rotation is often convenient for them in terms of flexibly planning their work. A disadvantage might be that influential irrigators can better defend their interests than vulnerable or female irrigators, whose ódemand may not be heardô. Especially during peak periods such as land preparation or transplanting, less influential farmers, notably women farmers, could have problems to secure their water turn.

1 SEAGA Sector Guideline, FAO, 1998

- A scheduled water delivery or rotation system has the advantage that it guarantees a regular supply of water to each plot, although timing might be less convenient and quantity not always adequate, especially in the tailend of the scheme. If possible a design that plans for night irrigation should be avoided, as especially for women it might not be socially acceptable or dangerous to go out at night for their irrigation turn. During planning meetings with the farmers these issues need to be discussed, and a decision reached on what type of water delivery suits everyone best.
- In a scheduled rotation system it is crucial for all groups of farmers to have access to information regarding the timing of their water turn. Women may have less access to this information than men. Not having access to the right information results in sometimes losing all, or part, of their water share.

Table 9.2.2 shows a typical operation activities and their responsible organization;

Table 9.2.2 Typical Operation Activities and Responsible Organizations

Planning	Activity	Details	Timing	Responsible Organization
Basic Operation Planning (before construction)	Establishment of basic method of operation	Whether to adopt Flow Sharing (proportional delivery) Method or Time Sharing (rotation) Method. How to coordinate the intention of large-scale farmers and small-scale farmers, upstream farmers and downstream farmers.	at the F/S stage, design work stage, at the start of every season or every two seasons	MEDIWR/ MAFCRD
Annual Irrigation Planning (after construction)	Preparation of cropping calendar	Develop cropping calendar by season (dry and rainy season), per month, taking into consideration of pattern of planting (gradual increase in planting season and gradual decrease in harvesting season)	at the start of every season or every two seasons	Scheme Management Office (MAFCRD)
	Estimation of expected water demand and supply	Estimation of crop water requirement, based on cropping calendar. Water demand is estimated by considering effective rainfall, runoff, evaporation, transpiration, percolation, and conveyance loss.	at the start of every season or every two seasons	Scheme Management Office (MAFCRD)
	Irrigation scheduling and facility operation planning	Water distribution plan (including pump operation plans) is developed based on water distribution method, irrigation water availability, and management capacity of gate operator.	at the start of every season or every two seasons	Scheme Management Office (MEDIWR)

9.3 Maintenance Plan

(1) Maintenance method

Division of role in maintenance work is a key for successful and sustainable operation of irrigation system. Maintenance plan have to be developed based on clear commitment of all stakeholders, in addition to financial and human resources, and technical capacity of them. At the time of maintenance planning, technical and financial capabilities of stakeholders have to be discussed. In this regard, it is necessary to identify required maintenance works of each irrigation facilities.

For pump stations, followings are necessary operation and maintenance activities.

- Specification sheets, operation & maintenance manuals, spare parts list, operation records and so on should always be available for the daily inspection and maintenance. To prolong the equipment life, the operation records should be described in accordance with the checking items (suction pressure, discharge pressure, current, voltage, operation hour, vibration, noise, etc).

- Spare parts, packing, oil and grease should be kept.
- Inspection shall be made before operation for related facilities as well as pump equipment in order to maintain fitness and stability among equipment, intake and discharge pipes, discharge reservoir, canal, etc.

Following table shows major structure of the Jebel Lado Irrigation Scheme. The required maintenance works vary from structure to structure as follows.

Table 9.3.1 Typical Maintenance Activities of Irrigation Facilities

Irrigation Facilities	Maintenance Activities
Pump Station	Inspection of deterioration of bearing grease and bearing surfaces, Changing and/or addition of bearing grease, Checking of vibration and noise, Changing of packing, Disassemble inspection, checking of tightness of bolts and nuts, checking of abnormal parts and inside valves, checking of accessories, cleaning
Irrigation Network (lined canals = main canal)	Removal of silt and solid deposition, Repair of damaged joints, slabs and lining concrete with cracks, Weed control at joints and on surface of slabs
Irrigation Network (unlined canals = in-field distribution canal)	Removal of silt, Cutting and removal of earth weeds and waterweeds on wetted parts of canal slopes, and floating waterweeds, Plugging small holes and replacement of porous soils to prevent seepage, Rebuilding of eroded banks
Head gates, check dates and other structures	Removal of silt and obstructions, Lubrication (oiling and greasing) of gates, Anticorrosion treatment (painting) of mechanical elements
Drainage Network	Removal of silt and solid deposition, Weed control in the canal section, Repair and shaping of canal section
Farm Road	Refilling of holes on road surface, Grading road surface, Repair of road shoulders eroded, De-silting and repair of side ditches and culverts, Provision of additional pavement materials for paved roads

(2) Maintenance activities and responsible organizations

Maintenance works consist of routine maintenance, periodical maintenance and emergency maintenance works. The routine maintenance is a day-to-day maintenance work including cleaning silt at flow measuring devices, removal of floating debris, minor repair of canal and structures and greasing or oiling of gates of facilities. WUA should actively participate in this activity at least for on-farm level structure.

Periodical maintenance is works to be done at a certain interval, after harvest season or before planting season for example. Basically, WUA bear a responsibility for on-farm level maintenance, whereas the Jebel Lado Irrigation Scheme Management Office are obligated to main facilities such as intake facilities, main and second canals, and gate structures. Emergency maintenance is an emergency works at the time of natural disasters which causes damages on irrigation structures. This type of maintenance requires large investment for long term and/or large scale of replacement, and main responsible organization should be the National Government (MEDIWR) except on-farm level structures.

Following table shows ideal demarcation of each stakeholder in maintenance works.

Table 9.3.2 Typical Maintenance Activities and Responsible Organizations

Maintenance Level	Description	Activities	Responsible Organization
Routine Maintenance	Day-to-day maintenance work.	- removal of earth weeds and waterweeds - cleaning silt at flow measuring devices - removal of floating debris - minor repair of canal and structures - greasing or oiling of bearing, gates, and other metal structures	- On-farm: WUA/Community - Main facilities: Scheme Management Office
Periodical Maintenance	Works to be done at a certain interval.	- strengthening of banks and structures - Removal solid deposition & silt - grass cutting of embankment & canal banks - repair of damaged structures /a - repair of damaged equipment /b - painting of structures - checking of tightness of bolts, nuts, inside valves, & accessories at pump station	- On-farm: WUA/Community - Main facilities: Scheme Management Office
Emergency Maintenance /a	Emergency work	- repair of damaged structure caused by unforeseen disasters, including floods, heavy rainfall, earthquake, theft, etc.	- Main facilities: Scheme Management Office/ County/ State/National - On-farm: WUA/Community

Note: a/ Diagnosis of damaged structures (e.g. gate) is outsourced to engineering firms.
b/ Maintenance of equipment (pump, electric supply, etc.) are outsourced to suppliers and manufacturer.

9.4 Financial Management of Irrigation Scheme

(1) Cost recovery through irrigation service Fee

Whether an irrigation system is operated and maintained by a government agency or private organization, it always requires budget to undertake O&M activities. It needs budget for; 1) the services rendered by people in the delivery and distribution of irrigation water, 2) the normal maintenance of irrigation facilities and structures, and 3) the periodic and emergency repair of irrigation facilities and structures. Therefore, generating budget for these O&M activities is one of major function of the Scheme Management Office.

It is an important issue that, to which extent, the irrigation service fee (ISF) should cover costs of irrigation management, so called cost recovery principle. The costs to be discussed in the ISF estimation of the Jebel Lado Irrigation Scheme are shown in table below.

Table 9.4.1 Annual O&M Cost

Cost Items	Amount (SSP/year)
Annual Operation and Maintenance Cost	
Personnel Expenses	626,472
Pump Operation	4,106,400
Equipment and Machineries (fuel, lubricant, etc.)	99,000
Normal Maintenance Cost of Irrigation Facilities	90,800
Depreciation Cost /a	
Project Facilities	4,197,600
Equipment and Machineries	728,000
Total Costs	9,848,272

Note: a/ Straight line method is adopted to estimate depreciation cost.

Even though cost recovery is a basic principle of ISF introduction, it is recommended to start at a lower level upon its introduction. The main focus at this stage is to let farmers develop the healthy habit of paying ISF regularly for the supply service of irrigation water, and enjoy timely and sufficient volume of water for crop production. Thereafter, the consumers, upon recognizing that irrigation water is indispensable in their farming, will be more open to a higher ISF level and the next round of increases can be made to meet the cost recovery requirement.

Therefore, it is recommended to take step-wise targets for financial management of the Jebel Lado Irrigation Scheme to materialize sustainable operation and management of the scheme.

- Short-term target is to make farmers familiarize irrigation farming and develop the healthy habit of paying ISF regularly for the irrigation water supply
- Mid-term target is to materialize cost recovery of annual O&M costs including personnel expenses, pump operation fee, equipment and machinery operation costs, and normal maintenance cost of irrigation facilities
- Long-term target is to accumulate earning retention for periodic and emergency repair of irrigation facilities and structures

(2) Affordability to pay (ATP)

The level of the ISF is a sensitive issue in managing an irrigation scheme. If the level of ISF is too low, it would be impossible to mobilize adequate fund for regular operation and maintenance of the scheme, which in turn result in poor service delivery of the scheme. In contrast, if the ISF level is too high for farmers, price of products will increase due to high production cost, and farmers may lose incentive to participate in management of the irrigation scheme.

Therefore, it is quite important to set up a reasonable level of ISF to ensure management of the irrigation scheme. To identify the reasonable level of ISF, the planner sometime conducts interview survey to farmers for grasping their willingness-to-pay (WTP) and affordability-to-pay (ATP). Usually, WTP is estimated based on the socio-economic survey, and the survey was conducted in the course of IDMP formulation. However, since most farmers had no idea for systematic provision of irrigation water, it was difficult to obtain proper reply to estimate WTP. Therefore, in this ISF estimation, ATP was figured out to obtain proper level of ISF.

In water sector, ATP is usually estimated at 3 to 5% of disposable income. By following the precedent, the lowest figure of 3% was applied in this analysis, and ATP was estimated based on net income of planned crops in the Jebel Lado Irrigation Scheme. Following table shows estimated ATP of the scheme.

Table 9.4.2 Annual O&M Cost

Planned Crops /a	Net Income /b (SSP/ha)	Affordability Rate (%)	ATP (SSP/ha)	ATP (SSP/feddan)
Maize	5,222	3 %	160	70
Vegetables	89,235	3 %	2,680	1,130
Banana	35,117	3 %	1,050	440

Note: a/ Vegetable is represented by Tomato.
b/ Family labour is excluded from net income of planned crops.

(3) Pricing method for the ISF

There are two (2) major practical pricing methods, namely area-based pricing and volumetric pricing. The area-based pricing is a fixed charge based on the area irrigated or supposed to be irrigated. They are often calculated by dividing the total area irrigated into the O&M costs of providing irrigation water, which basically follows the average cost pricing principle. While the volumetric pricing method is estimated and charged in accordance with amount of water delivered.

Further, the volumetric pricing method can be divided into two (2) methods, including block pricing and two-parts pricing. The block pricing involves varying the water price when water use for a set time period exceeds a set volume. If high water charges are a concern, an increasing block charge can be used. Whereas the two-part pricing is a combination of volumetric pricing and a fixed admission charge. The volumetric part can be based on marginal cost, which encourages less water use, while the fixed part can be used to make up any deficits and ensure a certain revenue flow regardless of how much water is available and delivered.

In this analysis, the area-based pricing method is adopted. The O&M costs composed of fixed parts and variable parts. The former is depreciation costs which are constant during economic life of the equipments, machineries and facilities, whereas the latter is changeable in accordance of irrigation scheme management. Followings are assumption of the ISF estimation.

- Depreciation cost of project facilities are excluded from the fixed charge estimation since investment cost of the project facilities are too heavy for farmers to shoulder, and can be regarded as the national governmentôs property.
- On the other hand, equipment and machineries, including tractors and its attachments, can be regarded as properties of the irrigation management office since their economic life are relatively short, and should be reinvested by the users.
- As for the variable part, in this analysis, it includes personnel expenses, pump operation fee, equipment and machinery operation costs, and normal maintenance cost of irrigation facilities. This part was divided by proportion of water consumption volume of each crop, and then divided by planted area of each crop, so that ISF rate of each crop can be obtained.
- Minimum farm lot size is set as 1 acre.

Based on the above assumptions, following formulas are applied to obtain ISF of the Jebel Lado Irrigation Scheme.

Fixed Charge (Member Fee) = Dem ÷ Nl

Where: Dem = Depreciation cost of equipment and machineries

Nl = Number of farming lot

$$\text{Variable Charge } (ISF_{C1}) = O\&M \times \frac{VC1}{\Sigma VC1{\sim}3} \div A_{C1}$$

Where: ISF_{C1} = ISF of Crop1

O&M = Annual O&M costs

$VC_{1\sim3}$ = Total volume of water consumption of crops

A_{C1} = Cropped area of Crop1

Based on the above formula, fixed charge as a member fee, and variable charge as an ISF were estimated. Then, on one hand, the estimated ISF was adjusted by ATP to obtain payable and practical level of ISF. However, since ATP of maize is too low, ISF of maize is further adjusted to provide incentive to shift cash crop production. On the other hand, member fee is not adjusted by ATP, but can be paid by in kind. Following table shows proposed ISF and membersôfee in the Jebel Lado Irrigation Scheme.

Table 9.4.3 Proposed ISF and Members' Fee

Crop	ISF			Members Fee	
	Estimated ISF (SSP/ha)	ATP (SSP/ha)	Adjusted ISF (SSP/ha)	Members' Fee (SSP/ha)	In Kind (=Labour in Days)
Maize	1,667	160	160	548	14 days
Vegetables	2,143	2,680	2,143		
Banana	4,286	1,050	1,050		

(4) Collection method for the ISF

There are two key steps in cost recovery; the first is to design a pricing mechanism that covers the appropriate costs, and the second is to achieve high collection rates through effective water management. Collecting ISF from farmers is crucial in many developing countries since most farmers are poor. Followings are ideal method for collecting ISF and membersôfee.

- Farmers have to inform their cropping plan of the season, before starting the crop season. WUA will compile each farmerôs plan and submit to the Scheme Management Office. Then the Office will issue ISF bill to each farmers through WUA. SMS billing system through mobile phone is more effective since most people nowadays use mobile phone.
- ISF and membersô fee is collected after harvesting crops when farmers can obtain cash income from their farm products. Payment methods include cash, bank transmission, check, and in kind. Farmer should pay at the Scheme Management Office after harvest of the season.
- Membersô fee can be paid by in kind and is estimated at SSP216/acre in the Jebel Lado Irrigation Scheme, which could be converted to 11 days of labour work. ISF can also be paid by in kind, but it is recommended to collect ISF in cash since it is equal to or less than the ATP.
- Penalty clause must be clearly stated in statute, and properly be executed.
- Introduction of an incentive measure to ISF collectors is effective. Each collector should have own jurisdiction and those who mark the highest ISF collection rate of the year will be commented by managers of the Scheme.
- Privatization of billing and ISF collection (PPP) is also effective. Traditional chief or local authority would be involved with a certain incentives.

(5) Cash flow analysis to set management target

To see the balance of revenue and expenditure and assure the sustainability of the irrigation scheme management, cash flow statement of the scheme management office is effective. The cash flow statements show the movement of the scheme management officeôs revenue and expenditure during a certain period. Cash inflow comes from daily operation of the scheme management office, including the collected ISF and other revenue such as membersô fee and penalty fee, whereas cash outflow includes regular operation and maintenance expenditures. Cash flow analysis will help the scheme management office to set relevant ISF to cover O&M costs of irrigation management, and help the office foresee potential deficit which would be the subsidy from the national or state government.

In the short-term, it could be happed that the revenue of the scheme management office cannot cover all O&M costs and the office heavily depends on subsidy from the national government, since farmers are still poor and cannot pay higher ISF. However, in the mid-term, it is better to increase ISF rates in accordance with growing farmerøs income so that the revenue can cover normal O&M expenditure of the scheme. In the long-term, it is important to accumulate the earning retention for periodic and emergency repair of irrigation facilities and structures.

To see the degree of cost recovery based on the proposed ISF rate, three targets were set up in the cash flow analysis as follows;

- Target 1: Cost recovery of the annual O&M cost, which includes personnel expenses, pump operation fee, equipment and machinery operation costs, and normal maintenance cost of irrigation facilities.
- Target 2: Cost recovery of the annual O&M cost and a part of depreciation cost (equipment and machineries cost)
- Target 3: Cost recovery of the annual O&M cost and the total depreciation cost, including equipment and machineries cost, and project facilities such as pump station, canals, and on-farm structures.

Then, before starting the cash flow analysis, followings assumptions were established.

- Revenue includes ISF, membersô fee, tractor service fee, whereas expenditure includes annual O&M cost and depreciation of equipment, machineries and the project facilities.
- Price escalation is taken into consideration in the cash flow analysis. By taking linear regression of consumer index for four years (2011-2015), price escalation rate of 1.67%/annum for general consumption goods and 3.34% for fuel and electricity is estimated.
- ISF collection rate is lower at the beginning of irrigation service provision, but will increase after 5 years, and 10 years on the ground of incentive measures to the collectors and penalty measures to the farmers. As a default setting, ISF collection rate is set as 60% in the short-term, 70% in the mid-term and 80% in the long-term.
- Cropping area will change in short-term, mid-term, and long-term. According to the socio-economic survey conducted by the IDMP-TT at the project site, most farmers want to plant cereal crops for food security reason. However, it can be reasonably assumed that as farmer experiences irrigated agriculture more, they recognizes potential of irrigation farming and tend to increase cash crop production more.
- ISF is estimated based on the ATP of planted crops. In the short-term, minimum rate of 3% is applied in due consideration of farmersô financial capacity. However, as farmers become more familiar with irrigation farming and obtain more income from the farming, the ATP will be increase. In the mid-term and the long-term, the ATP of 5% is adopted.

Based on the above assumptions, cash flow analysis was conducted. The major findings of the cash flow analysis are as follows, and the results are shown in Appendix 4.

- Among three targets, only Target 1 could show positive result in the mid-term and the long-tem operation period, and other two targets were far from the cost recovery. It means the cost recovery of the annual O&M is achievable, whereas the cost recovery of depreciation costs is quite difficult in this scheme.

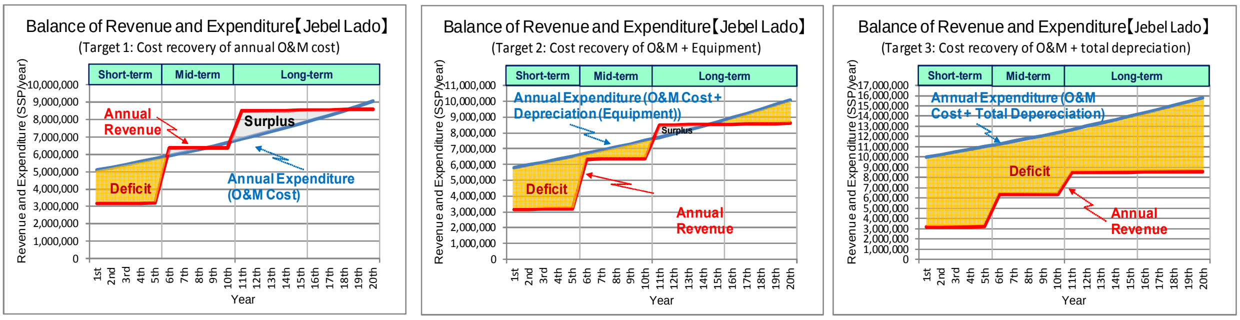


Figure 9.4.1 Balance of Revenue and Expenditure

- As for the Target 1, in the short-term, in other words, during the first 5 years, the balance of annual O&M cost and revenue is ñminusò. The deficit must be compensated by the national government as a subsidy. However, in the mid-term and the long-term, the balance will become ñplusò, meaning the Scheme Management Office can start accumulation of the earning retention to cover a part of depreciation costs after 6th year of its operation.
- As for the Target 1, the balance of revenue and expenditure cannot be ñplusò during the short-term period. To overcome this situation, there are two possible ways for the scheme management, including increase in ISF rate, or increase in ISF collection rate. Among the alternatives, increase in ISF is not better solution since farmers are still poor at the beginning of irrigation water provision. Rather, making efforts to increase ISF collection rate is realistic. However, even if ISF collection rate becomes 100%, the balance at the short-term period is still ñminusò due to mainly high project cost, O&M costs, and low revenues.
- Result of the cash flow analysis indicated that the Scheme Management Office can achieve the target 1, and can manage at least annual O&M cost under the proposed ISF level. Also, the Scheme Management Office can obtain a surplus from the 6th year, which can be the internal revenue fund for covering a part of depreciation costs or unexpected invents.
- However, the office cannot manage depreciation costs in full including amortization of equipment, machineries, and project structures, since the initial investment costs is quite high. Therefore, government support as a subsidy to cover the depreciation costs is necessary for reinvestment of the Jebel Lado Irrigation Scheme.

(6) Recommendation

The cost recovery principle must be adapted in the irrigation scheme management, and its revenue should be their services including provision of irrigation water supply, tractor service, and others. However, as result of cash flow analysis indicated, it is not easy to recover depreciation cost. Therefore, it is recommended that financial target will be cost recovery of annual operation costs of the scheme. The annual O&M cost includes personnel expenditure, pump operation cost, equipment and machinery operation cost, and regular maintenance cost of the scheme.

For this purpose, proposed ISF level is considered reasonable and proper. The proposed ISF estimated in the analysis is set in low level at the first 5 years in due consideration of farmersôfinancial capacity. However, it should be increased from 6th year when farmer beneficiaries become familiar with irrigation farming, and will be ready enough for paying higher ISF. Also, form the mid-term operation, it is necessary to advice farmers to shift more capital intensive farming, from current cereal crop production to more profitable crops through applying suitable farm inputs including high quality seeds, fertilizers and pesticides.

On the other hand, to materialize sustainable financial management of the scheme, administrative efforts and engineering efforts are necessary. If farmers could satisfied to the irrigation service from the Scheme Management Office, farmers will show their satisfaction through continuous payment of the ISF, which result in increase in revenue of the scheme. Therefore, the Office should provide demand oriented or used friendly services including input service, farming technology extension, post harvest services, and off-farm training for example, in addition to the regular supply of irrigation water.

If the financial target of the scheme is set to recover annual operation and maintenance costs, the Scheme Management Office can acquire surplus from 6th year. Accumulated amount of the surplus will be 12,279 thousand SSP by the end of the long-term period, after 20 years from the project completion. It is recommended that the surplus will be retained in the account of the scheme so that the scheme can reinvest a part of equipment and machinery costs needed, or can address unexpected event in the future.

CHAPTER 10 COST ESTIMATE

The project costs are estimated at USD. The unit price is set up on the basis of the actual construction orders done by MEDIWR.

10.1 Conditions for Cost Estimate

Table 10.1.1 presents the conditions for cost estimate.

Table 10.1.1 Conditions for Estimate

Items	Contents and Conditions
a) Direct Construction cost	Labour, materials, machinery, etc. and including pump and relative facilities, etc
b) Indirection construction cost	45% of the above a), as overhead cost
c) Administration	4% of the above a)
d) Consultant Fee	5% of the above a)
e) Physical Contingency	5% of the above a)

10.2 Project Cost

Project cost of Jebel Lado is shown in Table 10.2.1.

Out of the total project cost, 44% for main irrigation canal and facilities in farmlands direct construction occupy large part of the total cost.

Table 10.2.1 Project Cost

No.	Work Description	Unit	Quantity	Price (million US$)	Rate (%)
1.	Direct Construction Cost				
1-1	Pump Station	L.S.	1	2.0	5.9
1-2	Pipeline	L.S.	1	3.4	10.0
1-3	Main Irrigation Canal	L.S.	1	8.0	23.5
1-4	Facilities in Farmlands	L.S.	1	7.2	21.2
	Sub-total (A)			20.5	60.6
2.	Overhead (B=A*45%)	L.S.	1	9.2	27.1
	C=A+B	L.S.	1	29.7	87.7
3.	Administration (D=C*4%)	L.S.	1	1.2	3.5
4.	Consultant Fee (E=C*5%)	L.S.	1	1.5	4.4
5.	Physical Contingency (F=C*5%)	L.S.	1	1.5	4.4
	Total			34.0	100.0
	Command Area A=1330ha			25,600 US$/ha	

CHAPTER 11 IMPLEMENTATION PLAN

11.1 Conditions of Construction

(1) Rainfall

Rainy season in Jebel Lado seems to be from April to October in general. The earthworks are strongly influenced by rainfall. Therefore, the construction at the site might be intermitted in the vicinity of July.

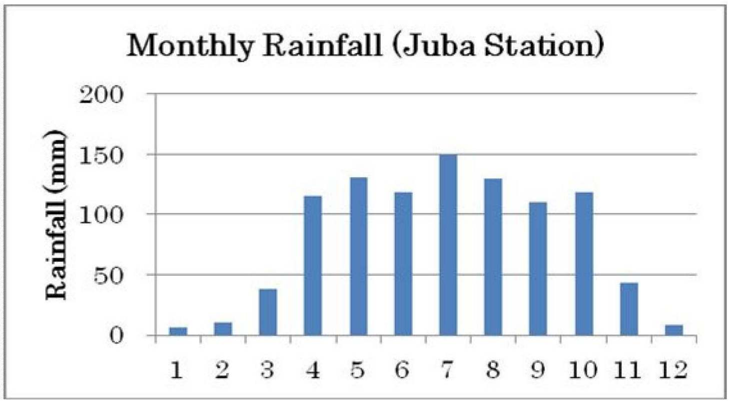


Figure 11.1.1 Monthly Rainfall (Juba Station)

(2) Land acquisition and Collaboration with relative agencies

Land acquisition shall be finished by the beginning of construction. MEDIWR shall prepare the budget for the land acquisition and proceed the procedure. Moreover MEDIWR shall proceed to collaborate with the relative agencies.

11.2 Implementation Schedule

It is proposed that all of the construction works should be achieved by 3rd year, considering their high priority and the earlier effective benefit.

Table 11.2.1 Implementation Schedule

Work Description	Project Cost	Quantity	Year		
			1st	2nd	3rd
Pump Station	3.3	Investigation, Detail Design, Procurement: Pump etc. Construction			
Pipeline	5.7	Investigation, Detail Design, Procurement: Steel Pipe SPϕ1200, L=2.2km			
Main Irrigation Canal	13.1	Investigation, Detail Design, Main Canal L=13.4km			
Facilities in Farmlands	11.9	Investigation, Detail Design, Canal & Drainage A=1330ha			
total	34.0	(million US$)			

CHAPTER 12 ENVIRONMENTAL AND SOCIAL CONSIDERATIONS

12.1 Purposes

The irrigation development master plan (IDMP) has selected three priority projects in Wau, Jebel Lado and Rejaf East. Those projects are expected to contribute to agricultural improvement in the RSS, while it is also important to avoid and/or mitigate any environmental and social impacts.

A guideline of environmental and social considerations for irrigation development (ESCID Guideline) has been developed in formulating the irrigation master plan. An IEE study was preliminarily taken for one of the priority projects in Jebel Lado by using the ESCID Guideline.

The purposes of the IEE study are:

- To figure out current environmental and social aspects in the project site;
- To preliminarily assess the impacts likely affected by the priority projects;
- To indicate scope of works of an environmental impact assessment in the further process of feasibility study, e.g.

12.2 Methods

(1) Process of Environmental and Social Considerations

According to the draft ESCID Guideline, the IEE is taken through the following main process.

1. Screening process: to identify whether or not further environmental and social considerations are necessary
2. Preliminary Survey: to find key environmental aspects
3. Scoping: to indicate highlighted impacts and the impact levels, and also to address the study method for a further study

(2) Methods for the Preliminary Survey

The preliminary survey was taken in the manner of hearing with local communities, government organization (county government, ministry, e.g.), visual observation, etc. The following table shows summary of the methods.

Table 12.2.1 Summary of Preliminary Survey Methods

Survey Methods	Target Items
Data collection	Protected wildlife,
Interview with Local communities State / County government Ministry of Wildlife Conservation and Tourism, Wildlife officials	 Community profile, local economy, wildlife, flood records Current plan, program, project, etc., flood records, wildlife Wildlife
Visual observation	Landuse, wildlife, local economy, water use, etc.
Topographic and geographic survey (conducted under the IDMP)	Topographic and geographic condition

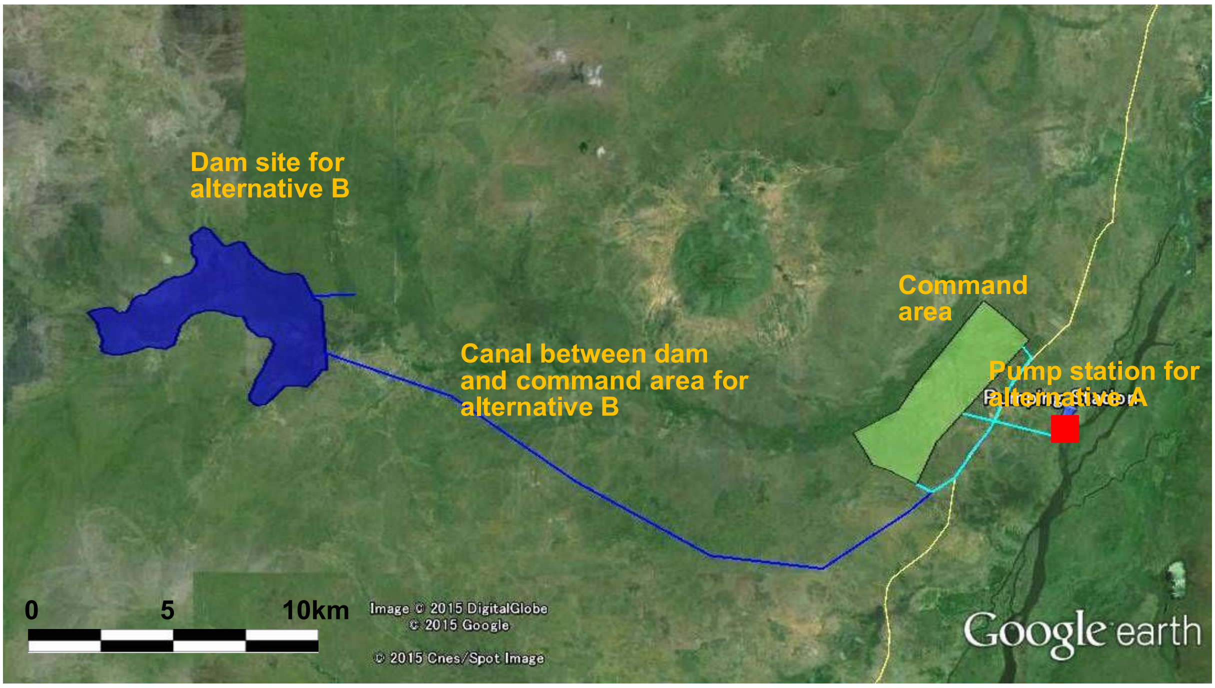
12.3 Evaluation of Alternatives

(1) Description of the alternatives

The three project designs listed in the following table were evaluated.

Table 12.3.1 Summary of Project Alternatives' Descriptions

	Alternative A	Alternative B	Zero option
Description	Pump irrigation	Dam irrigation	No project



Command area Total area Crop pattern	 1,330 ha Maize, vegetable / Banana	 Same as on the left	-
Dam site Reservoir area Reserve capacity Outlet discharge Operation time	-	 12.2 km^2 36,500,000 m^3 1.92 m^3/s Whole year	-
Pump station Number of pump Power/power source Operation time	 2 Diesel Whole year	-	-
Canal / pipeline Pipeline Length / diameter Volume Irrigation canal Main Length Volume Drainage canal Length	 2.2 km / 1,200 mm 1.92 m^3/s 13.4 km 1.92 m^2/s Total of approx. 36 km	 - - 37.0 km 1.92 m^2/s Total of approx. 36 km	-

Source: IDMP-TT

Evaluation was judged trough scoring method on each following evaluation item.

Table 12.3.2 Evaluation Methods (Evaluation Items)

Score	Evaluation Items
Natural Environment	Pollution (Air pollution, Water pollution, Waste, Soil/Sediment contamination, Noise and vibration, Odour, Global warming)
	Biodiversity (Protected areas, Ecosystem)
	Nature, disasters (Hydrology, Topography and geology, Subsidence / Erosion, Landscape)
Social Environment	Land occupies resettlement (Resettlement. Landuse)
	Social conflict (Vulnerable groups, Water use / Rights)
	Living condition (Living and livelihood, Local economy, Historical / Cultural heritage, Social infrastructure / Services, Infectious diseases)
Economy, development	Economy, development
	Consistency

(2) Results of comparison

The summary of score is shown in the table below (details are given in Appendix 5):

Table 12.3.3 Summary of Scoring and Ranking

Evaluation Items	Alternative A	Alternative B	Zero Option
Natural Environment	2.7	2.3	3.0
Social Environment	2.7	2.0	3.0
Economy, development	4.0	4.0	2.5
Total Score	**9.4**	**8.3**	**8.5**
Rank	**1**	**3**	**2**

Source: IDMP-TT

1) Zero option

It is, of course, not expected to generate any environmental and social impacts by zero option. On the other hand, food security and economic improvement are urgent challenges in the RSS, especially agricultural / irrigation development can have high potential on these matters. ñNo project implementationò, hence, is not suggestible. On the other hand, the priority projects are formulated based on the irrigation master plan, therefore it can be consistent with the RSS policies and directions.

2) Alternative A

Components of the alternative A are pump station, canal and pipeline, and command area. The proposed command area occupies large land, approximately 1,300 ha. Major impacts will be caused by land occupation. Though some small scale of activities such as hunting, cattle grazing are operated in the command area, most of the land seems to be bare land with low production. It is, hence, not expected to generate crucial impacts on resettlement, disturbance of local businesses / activities.

On the other hand, the project can increase agricultural land, and encourage agricultural production.

3) Alternative B

Addition to the command area, alternative B requires large scale of land occupation for dam site construction. The area counted for approximately 12.2 km^2. And there was information that the area was under the RSS army control. Yet land use of this area, location of communities and their houses, etc. have not been investigated, it is possible that certain significant impacts are generated by this land occupation. Also the dam site is located more than 20 km far from the command area. Canal route may occupy and split wide area, it will obstruct communityôs living condition and transportation as well as wildlife migration.

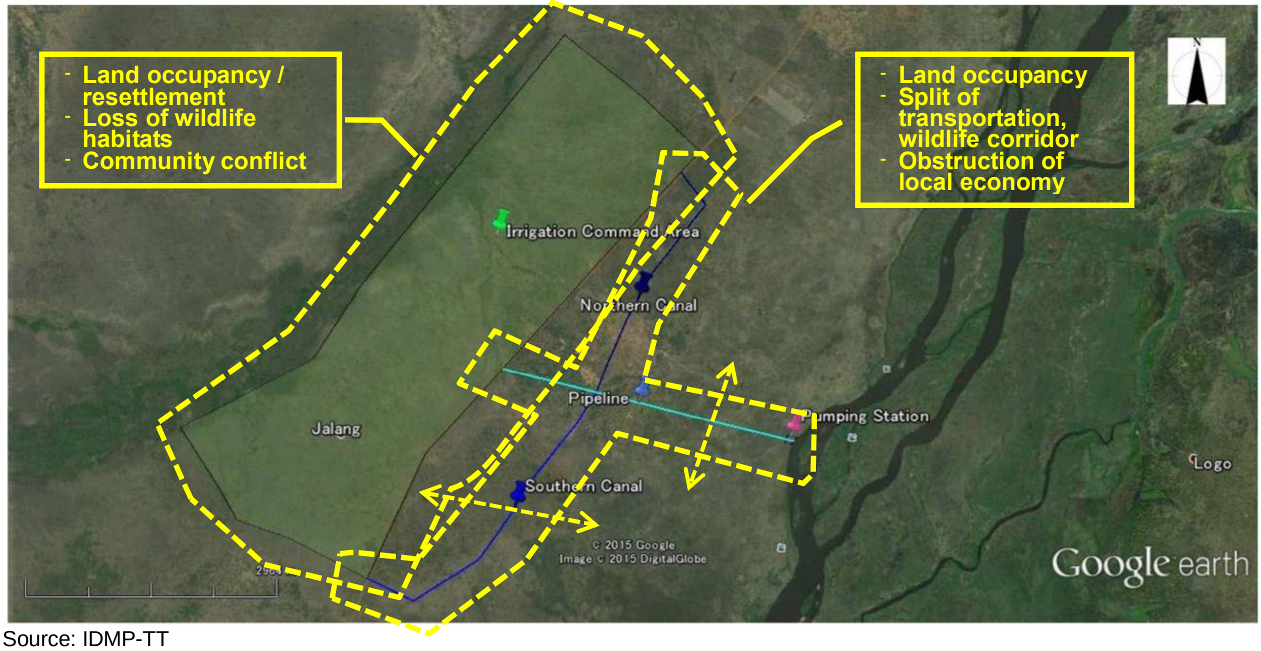
4) Results

Based on the above evaluation, alternative A is most suggestible. One of possible considerable impacts will be generated by large scale of land occupation. The proposed command area is covered with savanna, bare land, bush, e. g. Farming as well as cattle grazing, hunting are not so active according to the communities. Therefore those impacts can be mitigated by proper management and compensation. Benefit, while, on increasing agricultural production can raise economic improvement in local and national scale.

Alternative B also can contribute to economic improvement, however large scale of land occupation may cause significant obstruction and damage on ecosystem and social issues.

12.4 Current Environmental and Social Aspects

Overview of possible adverse impacts is illustrated in the following map:

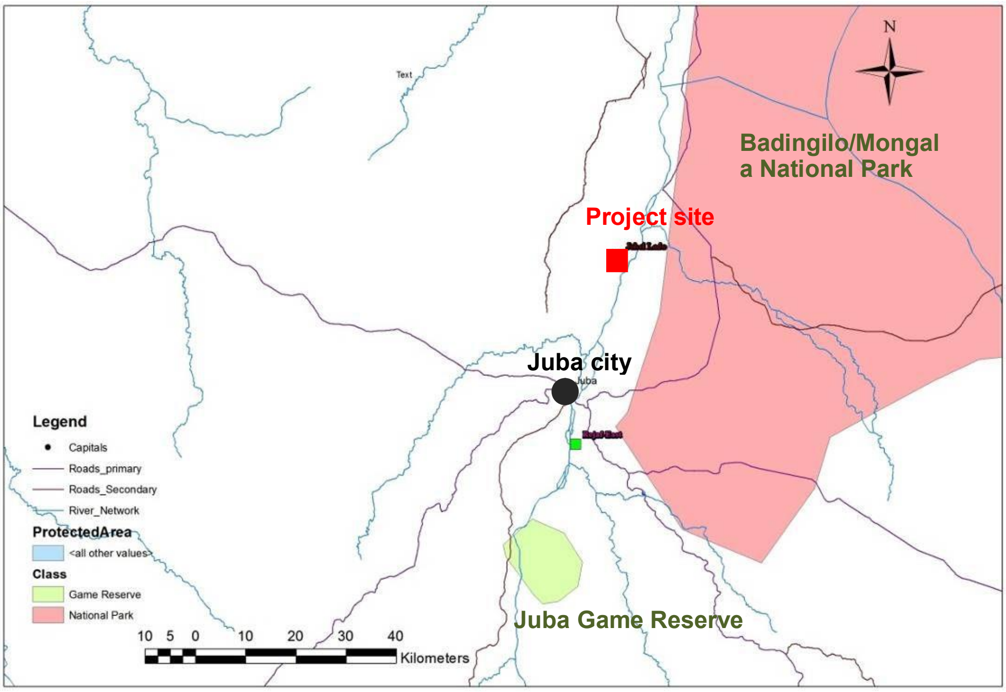


Source: IDMP-TT

Figure 12.4.1 Overview of Possible Impacts

(1) Natural environmental aspects

The project site, as shown in Figure 12.4.2, is located near Badingilo / Mongala National Park. The national park is located in the right river side of the White Nile River, while the project site is in the left river side. Therefore ecosystems of both sides may show different feature. Yet detail study about wildlife habitats, feeding sites, migration corridors, etc. has not been taken, critical areas for wildlife conservation have not been identified in the project site. The site mostly features savannah or bare land covered with bush, dotted forest. Land production is relatively low.



Source: Ministry of Environment

Figure 12.4.2 Location of Designated Areas for Wildlife Conservation

The animals shown in Figure 12.4.3 are popular in savannah and river bank in South Sudan. According to the community people, previously lion was also observed near the project site, however recently no lion has been reported.

			
Crocodile*	Sparted Hyena*	Bird nest**	Cactus**

Source: IDMP-TT
*: Taken pictures at the Wau Animal Zoo
**: Pictures taken in the project site

Figure 12.4.3 Typical Wildlife

Pump station will be installed beside Bahr el Jebel in the White Nile River. The river water flow is estimated approximately 31,400 MCM/sec, amount of pumped water is quite small comparing with the river flow. Therefore Impact on hydrological condition is not significant.

The command area is located 3.5 km far from Bahr el Jebel, the terrain is almost flat.

The project site is between the White Nile River and a seasonable stream. Altitude of intermediate between these two water ways is relatively high.

(2) Social and environmental aspects

The project site is located close to Juba city, approximately 25 km, high demand on food supply is expected due to its large population.

The project site is tenured under the two communities, namely Nyuwa and Peiti, population accounts for approximately 2,000 and 800 respectively. These two communities are belonging to the same tribal group (Bari). Dominant livelihood of those communities is farming, and fishery in the White Nile River.

Community has experienced simple irrigated agriculture using buckets near the White Nile River.

It, therefore, is expected for the community to easily adopt a new irrigation system introduced by the project.

Area beside the left side of the river has been used as community farmlands, and community people have resided in this area (see Figure 12.4.4). On the other hand, the proposed command area has rarely been used. The community showed objection to use this area for the priority project, and then they suggested shifting a command area to beside the seasonal stream.

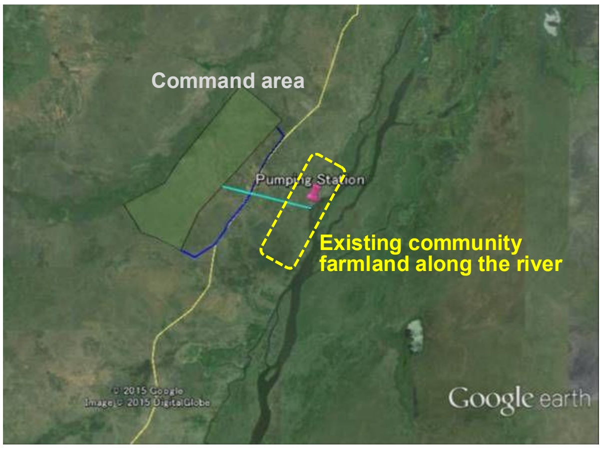


Figure 12.4.4 Location of the Existing Farm Land

Floods are one of the most considerable issues according to the community, but the proposed command area has not been suffered from floods. No historical / cultural heritage was confirmed. No grave yards were observed in the command area.

School

Community house

Source: IDMP-TT

Figure 12.4.5 Facilities in the Project Site

12.5 Evaluation of the Impact

(1) Overall evaluation

According to the preliminary survey, major impacts can be described as follows:

(1) Obstruction of ecosystem, wildlife (negative impact);
(2) Living and livelihood, local economy (positive impact) and
(3) Local conflict (negative impact).

The most considerable impacts are caused by land possession. Although the projects site is not located adjacent Bandingilo / Mongala National Park, it is possible that kinds of designated wildlife under the RSS and international rules are living in the project site. The proposed command area may lead loss of wildlife habitats, feeding and nurturing area, and canals and pipeline routes can disturb their migration.

On the other hand, obstruction of community area may not be significant because the command area is rarely used by the communities, possible affected area may be along pipeline and canal routes.

Demand on construction materials, tools / equipment, job opportunity may give benefits to communities in terms of improvement of living condition and livelihood. Also improvement of agricultural production can contribute to local and national economy.

Mostly farmers can benefit from the project, while fishery activity, hunting, cattle grazing can be affected. Therefore fair allocation of benefit and proper compensation must be considered.

(2) Results of Scoping

Results of scoping are summarised in Table 12.5.1.

Table 12.5.1 Results of Scoping

Environmental Items		Pre-construction	Construction	Operation	Summary of Impact
Pollution	Air Pollution	D	-C	-C	Construction works and operation of pump may generate exhaust gas, it can be controlled by moderate measures.
	Water Pollution	D	-C	-B	Construction works may generate turbid water, e.g., but it can be controlled by moderate measures. Storage of oil, hazardous waste must be properly managed. Use of pesticide and fertilizers need proper rules.
	Waste	-C	-C	-C	Construction waste will be considerable.
	Soil/Sediment Contamination	D	D	-C	Though polluted water can contaminate soil / sediment, it can be controlled by moderate measures.
	Noise and Vibration	-C	-C	-C	Construction works and operation of pump may generate noise, however its scale may not be significant, and it can be controlled by moderate measures.
	Odour	D	D	D	No certain odour is anticipated.
Natural Environment	Protected Areas	D	D	D	There are no protected areas adjacent the project site.
	Ecosystem	-C	-B	+C	No proper studies have been conducted, therefore level of impacts are not identified.
	Hydrology	-C	-C	D	Most possible impact to change hydrological feature may result in floods in / around the command area.
	Topography and Geology	D	D	D	No certain impacts are not anticipated.
	Subsidence / Erosion	D	-C	D	Risk of erosion during construction phase can be controlled by proper and moderate measures.
	Global Warming	D	D	D	No impact on global warming is anticipated.
	Landscape	D	D	D	There is no activity to affect landscape.
Social Environm	Resettlement	-C	-C	D	Scale of resettlement may not be significant, however further study is needed.
	Living and Livelihood	-C	-C	+B	Land occupancies in command area may affect communityõs living

Environmental Items		Pre-construction	Construction	Operation	Summary of Impact
					condition, livelihood as well. While recruitment and job opportunity are one of the most expected benefits.
	Local Economy	D	+B	+A	Construction works require provision of material, tools /equipment, man power, etc. Agricultural production can raise local economy.
	Historical / Cultural Heritage	D	D	D	There is no historical / cultural heritage observed.
	Land Use	-C	-C	-B	Though impacts by change of land use are relatively small, the situation can lead social conflict among the communities.
	Vulnerable Groups	D	D	-C	Possible adverse impacts are child labour, unfair allocation of benefit, etc.
	Local Conflict	-C	-C	-B	Farmers are mostly given benefits, while fishery, hunting, cattle grazing must be limited.
	Water Use / Right	D	-C	-C	Use of water newly provided by irrigation shall need a rule to mitigate conflict.
	Social Infrastructure / Services	-C	-C	D	Local grave yards are possibly scattered in the project site.
	Infectious Diseases	D	D	D	Mosquito bleeding in water area (canals, etc.) can be very limited.

+/-A: Significant positive/negative impact is expected.
+/-B: Positive/negative impact is expected to some extent.
+/-C: Extent of positive/negative impact is unknown (Examination is needed. Impacts may become clear as study progresses.)
D : No impact is expected.

12.6 Conclusions and Recommendations

(1) Conclusions

Conclusions are:

- The most significant impacts are related to land possession. The command area occupies large land. According to the communities, the proposed area seems to be low production, and few community houses, facilities, etc. have been existed. But further study is needed.
- Land possession can also affect ecosystem even though the project site is not located adjacent Bandingili / Mongala National Park. Recently no important habitats of wildlife, especially endangered / rare / threatened species have been recorded, however possibility of living of those animals could not be denied.
- On the other hand it is expected to improve communityös livelihood, local economy through raise of agricultural production.
- Pollutions related to air, water, noise, etc. can be controlled by moderate measures.
- The project is expected to effectively contribute to improvement of agricultural production.

(2) Recommendations

Recommendations are:

- Yet less significant risk on resettlement is expected, further study in order to identify land use, location of community houses and other facilities is required.
- Ecosystem in/around the project site has been hardly studied. Therefore appropriate scientific survey is recommended.
- Though workshops were conducted under the IDMP, public consultation with the communities is useful to know their opinions, concerns etc., in order to take consensus building among them, and to formulate adequate compensation plan.
- Most of the group who benefits from the project are farmers; on the other hand the people who are engaged in fishery, hunting, cattle grazing could be less benefited. Adequate compensation must be given in order to avoid social conflict. In addition benefits from the project must be fairly allocated among the communities.
- Further certain environmental assessment will be required in a feasibility study, e.g. The following survey methods are recommendable:

Table 12.6.1 Recommended Survey Methods for Further Study

Survey Items	Possible Methods	Points to be surveyed
Air pollution	- Check of quality of construction equipment and pump in terms of prevention from exhaust gas - Site survey on location of possible sensitive zones against air pollution such as residential area, school zone, etc.	- Possible affected area especially sensitive zone - Selection of environmentally friendly equipment with proper maintenance
Water pollution	- Measure of current water quality - Examine of possible pollution sources by the project	- Possible water pollution source and affected area - Farming plan in terms of use of chemicals
Waste	- Investigation of possible disposal site for construction waste - Estimation approximate waste volume	- Location of possible disposal site - Types of waste - Procedure / rules of storage and disposal of waste
Soil / sediment contamination	- Examine of possible water pollution sources by the project	- Same as ñwater pollutionò
Noise and vibration	- Check of quality of construction equipment and pump in terms of prevention from noise / vibration - Site survey on possible sensitive zones against noise / vibration such as residential area, school zone, etc.	- Possible affected area especially sensitive zone - Selection of environmentally friendly equipment with proper maintenance - Pump operation schedule
Ecosystem	- Interview with local communities - Direct observation on wildlife habitats, migration, etc. - Trap survey	- Wild life corridor - Wildlife habitats - Forest, plantation, e.g.
Hydrology	- Historical records of floods - Site reconnaissance on water body condition	- Condition of water body in rainy season - Historical records of floods
Subsidence / erosion	- Historical records of subsidence / erosion	- Location of possible erosion site

Survey Items	Possible Methods	Points to be surveyed
Resettlement	- Survey on land use, land status, land ownership, etc. - Estimation of land and asset price - Public consultation for consensus building	- Number and location of houses / facilities likely to be relocated - Agreement on the project - Resettlement plan
Living and livelihood	- Investigation of community living condition and livelihood - Interview with communities	- Possible job opportunities by the project in both construction and operation phases
Local economy	- Investigation of local economic profile - Investigation of future plans, developments, investments	- Possible materials, equipment for the project - Possibility of procurement in local
Land use	- Survey on land use, land status, land ownership - Investigation of land use plan - Public consultation	- Land map describing houses, facilities, land use, etc. - Existing and/or further land use plan
Local conflict	- Investigation of job profile, income level and sources - Public consultation	- Community profile, job profile - Consensus building among communities - Compensation plan
Water use / right	- Investigation of water use / right - Public consultation	- Status of water use, legal status on water right - Consensus building among communities
Social infrastructures / services	- Site survey on location of social infrastructures - Interview with local communities, etc.	- Location of infrastructure - Location of grave yards - Existing and/or further infrastructure development plan

CHAPTER 13 PROJECT EVALUATION

13.1 Outline of the Project Area

The irrigation development project in Jebel Lado located in CE will serve water to total of 1,330 ha of target field. The project will develop the present unused land into large farming fields with irrigation. The project will furnish the infrastructures to introduce an irrigated agriculture, leading to increase farming income for the farmer beneficiaries. Following are the outline of the site.

Outline of Project (Jebel Lado):

Location:	Just north of Juba City
Project area:	1,330 ha
Land Holding:	Ave. 1.8 ha/household (7.0 members/household)

13.2 Farming Plan

(1) Cropping pattern

Around the site in Jebel Lado, major crops are maize and sorghum, and vegetables are grown as cash crops. The crops to be grown in the project area will be represented by maize, tomato, jewsôs mallow and Banana. In the project, the cropping intensity is expected to increase by the improvement of farming conditions, compared to the current cropping intensity. Considering the situation of existing areas, where the irrigation development project is implemented, the cropping intensity in the project is assumed to be 195 %.

(2) Irrigation System

Irrigation water is taken from the Nile by a suction pump and transported through a pipeline, whose operation of the pump takes high cost. The pumped water is sent to a high place and will be delivered to three directions until each field by gravity.

13.3 Basic Assumptions for Economic Analysis

Upon conducting the economic analysis, following assumptions are set:

Financial prices of farming commodities are based on the results of Agriculture and Socioeconomic Survey in May 2015.

Financial prices are converted into economic prices using Standard Conversion Factor (SCF) of 0.90 and Labour Conversion Factor (LCF) of 0.45 (0.5 × SCF). Transfer payments are eliminated in converting economic price. Next table shows the summary of financial and economic prices.

Foreign exchange rate of 1 US$ = 2.95 SSP is applied, which is the current official exchange rate.

Cash flow analysis was conducted with 30 years since there is no significant replacement cost which will influence the economic efficiency and present value of cash flow. Values after 30 years will become very low as the influence in calculation is considered very little.

Table 13.3.1 Financial and Economic Price of Agricultural Produces/Inputs

No.	Description	Unit	Financial Price (SSP)	Conversion Factor	Economic Price (SSP)	Remarks
A.	**Agriculture Product**					
	- Maize	kg	3.90	-	3.20	Estimated by import parity price
	- Tomato	kg	5.20	-	4.09	- do -
	- Jew's mallow	kg	3.50	0.90	3.15	
	- Banana	kg	2.70	-	2.26	Estimated by import parity price
B.	**Farm Input**					
1	**Seed**					
	- Maize	kg	14.00	0.90	12.60	
	- Tomato	kg	500.00	0.90	450.00	
	- Jew's mallow	kg	200.00	0.90	180.00	
	- Banana sucker	kg	1.00	0.90	0.90	
2	**Fertilizer**					
	- DAP	kg	12.25	0.90	11.03	
	- Urea	kg	11.75	0.90	10.58	
	- CAN	kg	12.00	0.90	10.80	
	- NPK	kg	12.25	0.90	11.03	
	- KCL	kg	12.06	0.90	10.85	
	- Foliar (liquid)	lit	70.00	0.90	63.00	
3	**Agro Chemical**					
	- Pesticdes (insecticide)	lit	85.00	0.90	76.50	
	- Fungicide	lit	107.00	0.90	96.30	
4	**Labor**					
	- Family Labor	m*d	70.0	0.45	31.5	
	- Hired Labor	m*d	70.0	0.45	31.5	
5	**Equipment**					
	- Tractor rental	ha	476.19	0.90	428.57	
	- Sprayer	ha	100.00	0.90	90.00	
	- Threshing and milling	kg	1.21	0.90	1.09	
	- Transportation	time	22.20	0.90	19.98	
6	**Others**					
	- Sack / Box	piece	7.50	0.90	6.75	

Source: Agriculture and Socioeconomic Survey, 2015

13.4 Project Cost

(1) Project cost at financial price

The project cost of Jebel Lado at financial price is estimated 34 million US$ or 25 thousand US$/ha. Next table summarizes the project cost at financial price.

Table 13.4.1 Summary of Project Cost at Financial Price

Item	US$/ha	ha	Total (,000US$)
1. Direct Construction Cost	15,418	1,330	20,506
2. Indirect Construction Cost	6,938		9,228
Sub-total	22,356		29,734
3. Administration (4%)	894		1,189
3. Consultant Fee (5%)	1,118		1,487
4. Physical Contingency (5%)	1,118		1,487
Total	25,486		33,897

Source: JICA Project Team

(2) Project cost at economic price

Project cost at financial price was categorized into foreign currency portion (F/C), local currency portion (L/C) and transfer payments such as taxes. Local currency portion was further divided into skilled labour, unskilled labour, and others. Relevant conversion factors (CF) were applied for respective categories of cost to estimate the project cost at economic price. The project cost at economic price was, then, estimated at 30 million US$ or 22 thousand US$ per ha. Next table shows the estimation of the project cost at economic price.

Table 13.4.2 Estimation of Project Cost at Economic Price

Item	Financial Cost (,000US$) ①	CF	F/C 1.00 ②	L/C Skilled Labor 0.90 ③	L/C Unskilled Labor 0.45 ④	L/C Others (SCF) 0.90 ⑤	Tax 0.00 ⑥	Conversion Factor ⑦ Sum(②〜⑥)	Economic Cost (,000US$) ⑧=①*⑦
Direct Construction Cost		%	60.0	10.0	20.0	10.0	0.0		
	20,506	CF×%	0.600	0.090	0.090	0.090	0.000	0.870	17,840
Indirect Construction Cost		%	60.0	10.0	20.0	10.0	0.0		
	9,228	CF×%	0.600	0.090	0.090	0.090	0.000	0.870	8,028
Administration		%	60.0	10.0	20.0	10.0	0.0		
	1,189	CF×%	0.600	0.090	0.090	0.090	0.000	0.870	1,034
Consultant Fee		%	60.0	35.0	5.0	0.0	0.0		
	1,487	CF×%	0.600	0.315	0.023	0.000	0.000	0.938	1,394
Physical Contingency		%	60.0	10.0	20.0	10.0	0.0		
	1,487	CF×%	0.600	0.090	0.090	0.090	0.000	0.870	1,294
Total	33,897								**29,590**

Source: IDMP-TT

13.5 Project Benefits

(1) Category of benefits

Though benefits with the project of the site Wau is the yield by new cultivation, the expected benefits compared with existing farming in surrounding area of the site will be as follows:

- Increase of crop yield by irrigation
- Increase of cropping intensity
- Reduction of farming cost by increasing farming efficiency

(2) Project benefits at financial price

Based on the estimations of net benefit (gross output ï production cost including family labour value), the net incremental benefits were calculated. Next table is the summary of the net incremental benefit. Total net incremental benefit was estimated at 7,928 thousand US$ or 5,961 US$/ha.

Table 13.5.1 Summary of Net Incremental Benefit at Financial Price

Net Bennefit: Gross output － Production cost including family labor value

Crop	Area (ha) ①	Gross output (000US$) ②	Production cost (000US$) ③	Net Benefit (000US$) ④=②－③	Net Benefit (US$/ha) ⑤=④/①
Maize	1,264	5,013	5,373	-360	-285
Tomato	665	12,894	5,207	7,687	11,559
Jew's mallow	599	3,219	3,201	18	31
Banana	67	1,141	558	583	8,701
Total	2,595	22,267	14,339	**7,928**	**5,961**

Source: IDMP-TT

(3) Project benefits at economic price

Project benefits at financial price were converted into the ones at economic prices, using conversion factors and import party prices as it has been mentioned. For economic analysis, incremental benefit (count family labour as cost) will be also considered since economic analysis stands on the viewpoint of the national economy to examine the efficiency of resources use in the country.

Next table shows the summary of farm benefit. Total incremental benefit was estimated at 9,406 thousand US$ or 7,072 US$/ha.

Table 13.5.2 Summary of Economic Incremental Benefits

Net Bennefit: Gross output － Production cost including family labor value

Crop	Area (ha) ①	Gross output (000US$) ②	Production cost (000US$) ③	Net Benefit (000US$) ④=②－③	Net Benefit (US$/ha) ⑤=④/①
Maize	1,264	4,113	3,278	836	661
Tomato	665	10,142	2,951	7,191	10,814
Jew's mallow	599	2,897	2,097	800	1,336
Banana	67	955	376	579	8,638
Total	2,595	18,107	8,702	**9,406**	**7,072**

Source: IDMP-TT

13.6 Project Evaluation

(1) Cash flows of cost and benefit

Following is the proposed cash flow of investment (project cost) and the benefits accruing from the investment:

<u>Investment (Project Cost):</u>

Construction: Construction including survey, examination, etc. will be implemented in the first and second year.

O & M: Annual Operation and Maintenance (O&M) cost excluding the fuel of the pump is assumed 5 % of the total construction cost. The fuel cost of the pumping is estimated at 1,402 thousand US$/year.

Replacement: The introduced suction pump has to be replaced in the 21st year after 20 years of service life. Other irrigation facilities have durability of more than 30 years.

Benefit:

Crop production: Benefit will start fully realizing three years after implementation of planned farming, namely from the fourth year of cultivation. It is assumed that 60 %, 70 %, 80 % and 90 % of the full benefit will be achieved in the first, the second, the third and the fourth year respectively.

(2) Financial analysis

With the costs and benefits at financial price, here we apply financial internal rate of return (FIRR), financial net present value (FNPV) and cost-benefit ratio (B/C) for examining the efficiency of the investment. To estimate FNPV and B/C, discount rate of 8.83 % was applied, which is average of short-term lending interest rates of commercial banks in January - March 2015.

Family labour in this analysis is counted as cost for we stand on the viewpoint of private enterprise (farm household as a firm), all the inputs should be counted as cost; namely, net incremental benefit will be applied for the analysis.

The FIRR, FNPV and B/C are calculated at 10.9 %, 6,795 thousand US$ and 1.12 respectively. The FIRR is over the interest rate of 8.83 %, the FNPV is over zero and the B/C is over 1.00. Therefore, it can be said the project is financially viable.

(3) Economic evaluation

With the economic costs and benefits estimated above, the Economic Internal Rate of Return (EIRR) is calculated. Cash flow is same with the one of the financial analysis. The EIRR was calculated at 16.8 %. Opportunity cost of capital in RSS is considered around 7.5 %, therefore, it can be said that the project is economically feasible. Economic net present value (ENPV) discounted at the rate of 7.5 % was calculated at 33,782 thousand US$. The B/C discounted at 7.5 % was 1.61.

CHAPTER 14 CONCLUSION AND RECOMMENDATIONS

Project evaluation shows that the project is economically feasible and also financially viable. In terms of the selection of crops, the most profitable crop among the planned crops is tomato. It seems that crops whose fruit contains a lot of water have the advantage of profitability, considering watermelon also showed high profit in the irrigation plan in Wau. Banana is also considered to be a good introducing crop.

Beneficiary farmers in Jebel Lado can bring their produce easily to market in Juba: however, there is hard competition with imports from neighbouring countries. Leafy vegetables like Jew's mallow may have an advantage in suburban areas of Juba since they are easy to be damaged and not suitable for transportation though they are not so high profit.

APPENDIX - 1

FACILITY PLAN AND DESIGN

CHAPTER 1 GENERAL

1.1 Outline of Main Facilities

Main facilities planed in Jebel Lado area are as follows,

- Command area: A=1,330ha (Northern Block 560ha, Southern Block 770ha)
- Pump station: 1 place
- Northern Main Irrigation Canal: L=6.4km
- Southern Main Irrigation Canal: L=7.0km
- Irrigation and Drainage Facilities iin command area: 1 L.S

  Secondary canal, Tertiary canal, Feeder canal, Drainage, Road, Road crossing,

  Distribution gate, Water measurement facilities, etc.

Pump facility are operated through the year for farming, withdrawing from Bahr el Jebel.. For reference, there is a candidate site for dam but the site is very far of 25km from the subject command area and the construction cost of the dam and the distribution canal requires numerous expenses. Therefore it is judged that the irrigation method operated by the dam is dismissed due to its economic efficiency.

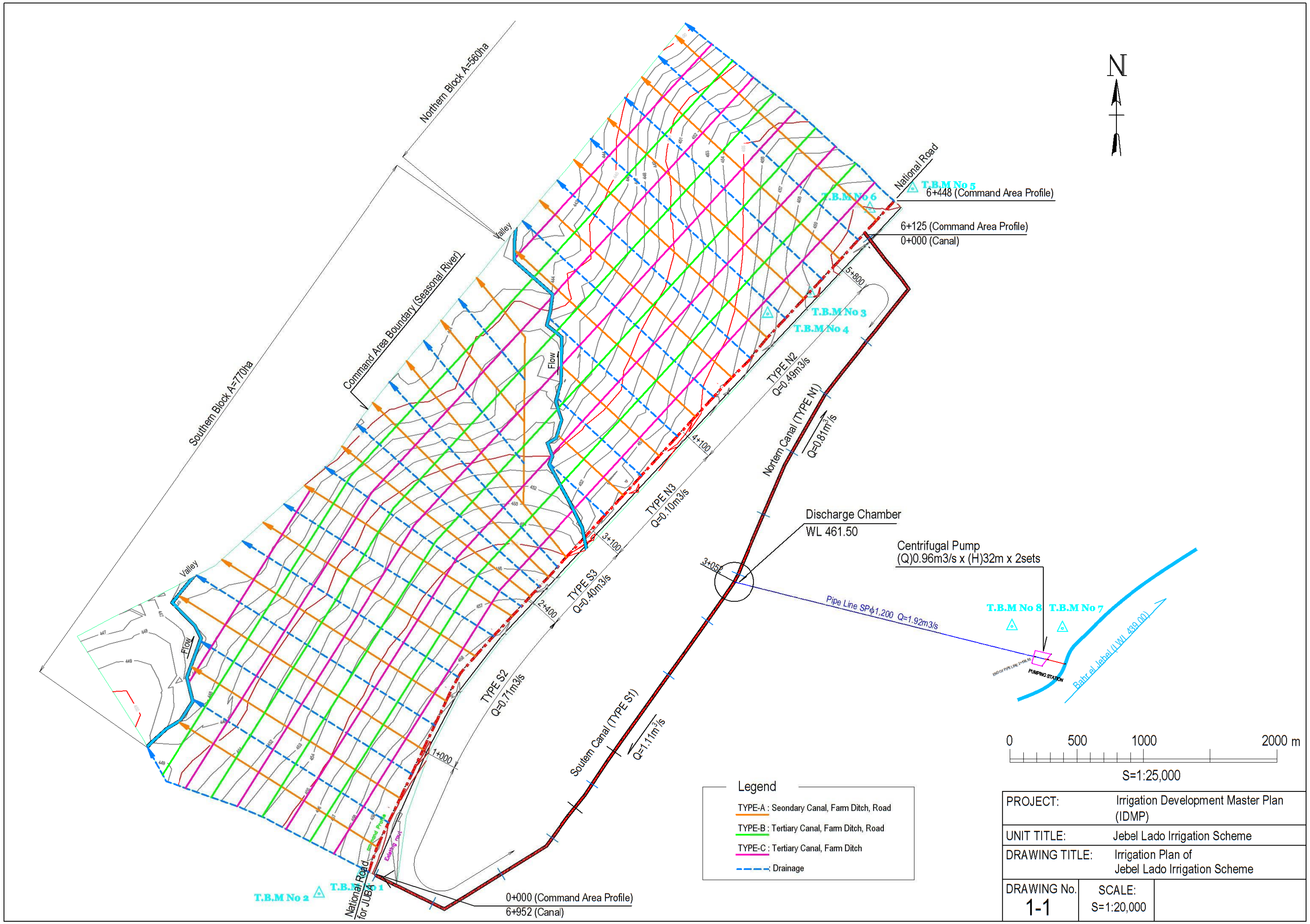
1.2 Command Area

The command area is located 3.5km from Bahr el Jebel. Bushes, trees and grasses dominate in the site. The terrain is almost flat and the land gradient shows around 0.9% toward the northwest from the southeast.

The command area is divided into two (2) blocks of north and south by the valley located in the middle of the command area. The drainage from the command area flows down to the seasonal river directly.

Pump station site is located beside Bahr el Jebel. The land is almost bare and some trees are shown. In the pipe line and canal line, there are community road among some small communities, bushes and trees etc. along the line.

Figure 1.1.1 Location of Map



CHAPTER 2 PUMP STATION

2.1 Location

The pump station is planned at the left bank of Bahr el Jebel. The location of pump station should be selected, considering the few silting in front of an intake. The outside of curved course of river is generally suitable for an intake point because of comparatively few silting.

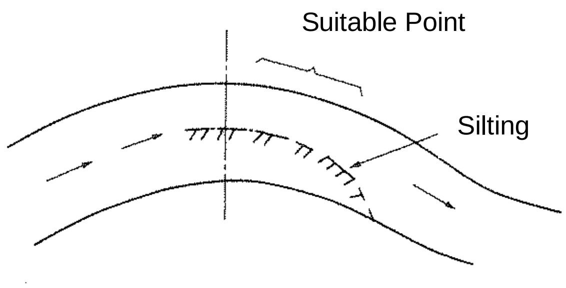


Figure 2.1.1 Flow of Curved River

Certainly the location of pump station was decided at a suitable point as shown in the figure 2.1.2

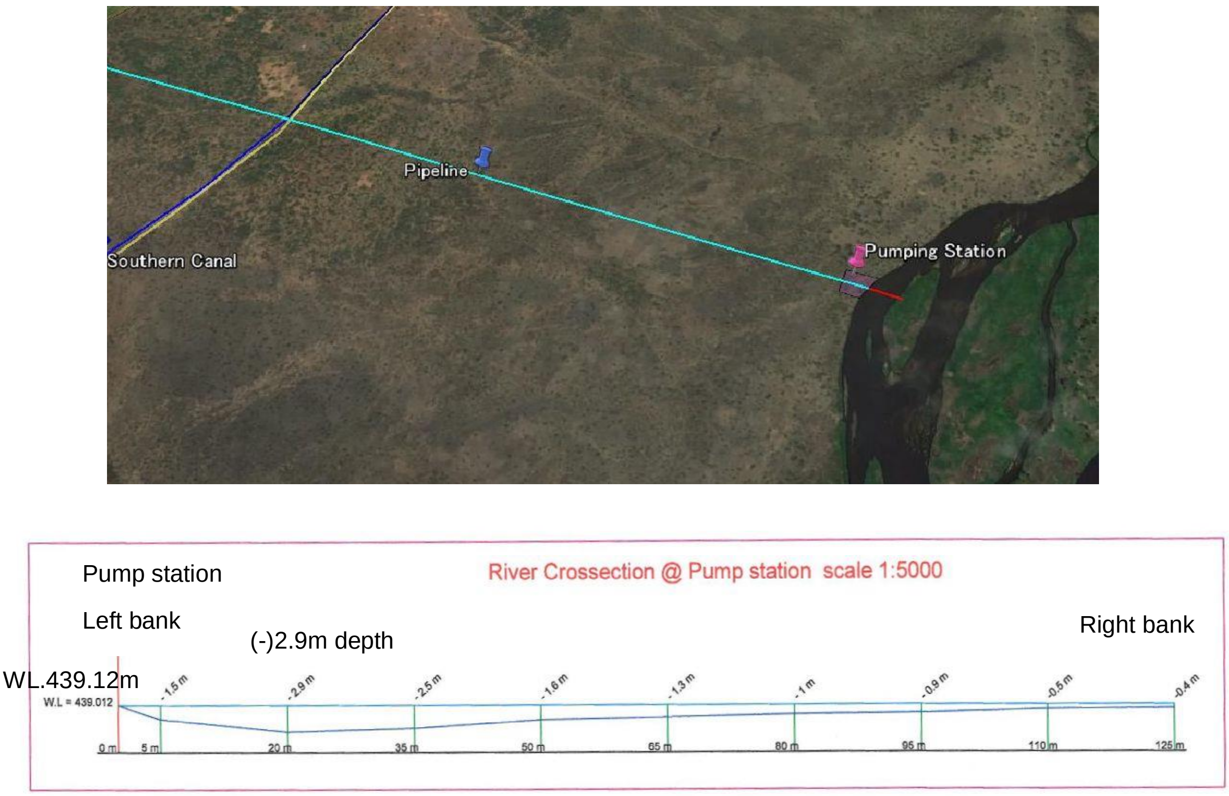


Figure 2.1.2 Location and River Cross Section at Pump Station

The water level measured in the survey work was WL.439.0m at the pump station as of the beginning of May. According to the water level record at Mongalla gauge station which is comparatively near to the pump station, the range of water level fluctuation is observed among 1.87m water depth as shown in the Figure 2.1.3.

Therefore High Water Level (HWL) at the pump station is determined WL.441.0m in assuming 2.0m of fluctuation.

Mongalla Gage Station:

No.	Year	Depth Annual Min. (LWL.m)	Depth Annual Max. (HWL.m)	Remarks
1	2008	12.66	14.26	1st (HHWL)
2	2009	12.33	13.04	
3	2010	12.16	13.95	
4	2011	12.39	14.09	
	Mean	**12.39**	**13.84**	**difference 1.45 m**

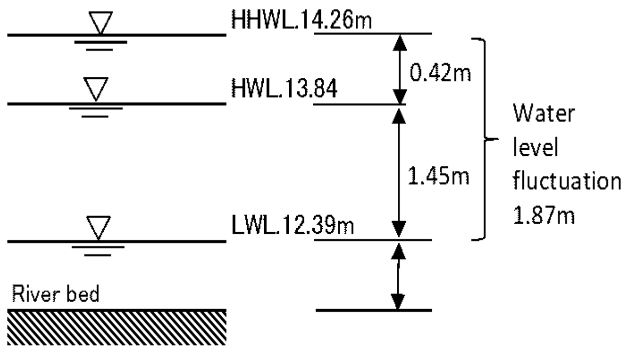


Figure 2.1.3 Mongaal Gauge Station

2.2 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.

$0.96 m^3/s$ (unit capacity) × 2 set = $1.92\ m^3/s$

Table 2.2.1 Water Requirement (m^3/s)

Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Average
0.78	0.57	1.13	1.08	0.66	0.70	0.05	0.89	1.92	1.92	1.07	0.07	0.90

According to the ñDesign Pump Facilities Technical Document (Japan)ò, the pump diameter is determined 700mm based on the pump capacity of $0.96 m^3/s$.

(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 Bahr el Jebel. The pump operation is planned throughout the year in accordance with farming plan, and the planned suction water level shall be fixed based on the record of lowest water level 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)

<u>Friction Loss Calculation of the pipe aligned in the pump station by Darcy・Weisbach</u>

$h_f = \lambda \cdot (L/D) \cdot V^2/2g$ ・・・・・・・・・・・・ 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)

<u>Friction Loss Calculation of the pipe aligned at outside of the pump station by Hazen・Williams</u>

$h_f = 10.666 \cdot \{Q^{1.85}/(C^{1.85} \cdot D^{4.87})\} \cdot L$ ・・・・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 2.2.2.

Table 2.2.2 Pipe losses and total head of each station

Site		Unit	Jebel Lado	Rejaf East	Remarks
Pump capacity		(m^3/s)	1.92		provided with 2 pumps
1.Actual head (ha)	Design Intake water level	DWL(m)	439.0		
	Design outlet water level	LWL(m)	461.5		
	Actual head (ha)	(m)	22.50		
2.Friction head loss					
(1)Suction pipe	Q	(m^3/s)	0.960		per pump
	Pipe		Steel		
	Diameter(D)	(mm)	700		
	Length(L)	(m)	30.0		
	Flow coefficient(C)		100		
	Water velocity(V)	(m/s)	2.49		$V=Q/(\pi/4\cdot(D/1000)^2)$, $0.3m/s \leqq V \leqq 2.0m/s$
	Friction head loss(fs)	(m)	0.34		$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(2)Delivery pipe1	Q	(m^3/s)	0.960		
	Pipe		Steel		
	Diameter(D)	(mm)	700		
	Length(L)	(m)	10.0		
	Flow coefficient(C)		100		
	Water velocity(V)	(m/s)	2.49		
	Friction head loss(fs)	(m)	0.11		$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(3)Delivery pipe2	Q	(m^3/s)	1.920		
	Pipe		Steel		
	Diameter(D)	(mm)	1200		
	Length(L)	(m)	2200.0		
	Flow coefficient(C)		100		
	Water velocity(V)	(m/s)	1.70		$V=Q/(\pi/4\cdot(D/1000)^2)$, $0.3m/s \leqq V \leqq 2.0m/s$
	Friction head loss(fs)	(m)	6.44		$hf=10.67\cdot(Q^{1.85}\cdot C^{-1.85}\cdot D^{-4.87})\cdot L$
(4)Total Friction loss		(m)	6.89		
3.Partial head loss					
(1)Check valve		(Nos.)	2		
	Diameter(D)	(mm)	700		
	Water velocity(V)	(m/s)	2.49		
	Coefficient of valve loss(fcv)		0.90		
	Check valve loss(hcv)	(m)	0.57		$hcv=fcv\cdot V^2/2g$
(2)Sluice valve		(Nos.)	2		
	Diameter(D)	(mm)	700		
	Water velocity(V)	(m/s)	2.49		
	Coefficient of valve loss(fsv)		0.33		
	Sluice valve loss(hsv)	(m)	0.21		$hsv=fsv\cdot V^2/2g$
(3)90°elbow		(Nos.)	2		
	Diameter(D)	(mm)	700		
	Water velocity(V)	(m/s)	2.49		
	Coefficient of elbow loss(fbe)		1.10		
	90°elbow loss(hbe)	(m)	0.70		$hbe=fbe\cdot V^2/2g$
(4)T Interflow		(Nos.)	1		
	Diameter(D)	(mm)	1200		
	Water velocity(V)	(m/s)	1.70		
	Coefficient of elbow loss(f13)		0.65		
	T Interflow loss(hbe)	(m)	0.10		$h13=f13\cdot V^2/2g$
(5)Remnant head	Diameter(D)	(mm)	1200		
	Water velocity(V)	(m/s)	1.70		
	Coefficient of head loss(fo)		1.00		
	Remnant velocity head(Lo)	(m)	0.15		
(6)Total parcial los	Particial head loss(Lp)	(m)	1.73		
4.Head loss(hf)	Total	(m)	8.62		
5.Total head(H)	H=ha+hf	(m)	31.12		
6.Design total head(H)		(m)	32.00		

(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 2.2.3 Rating point of pumps

Planned Discharge of Pump （m^3/s/unit）	0.96
Planned Total head(m)	32.0

(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 (%); 86 % 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 402kw are determined.

L= 0.163×57.6×32×1.0 / (86/100) = 349.4 kw　　P=349.4×(1+0.15) / 1.0 = 402 kw

Table 2.2.4 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.

2.3 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, enginew 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 2.3.1.

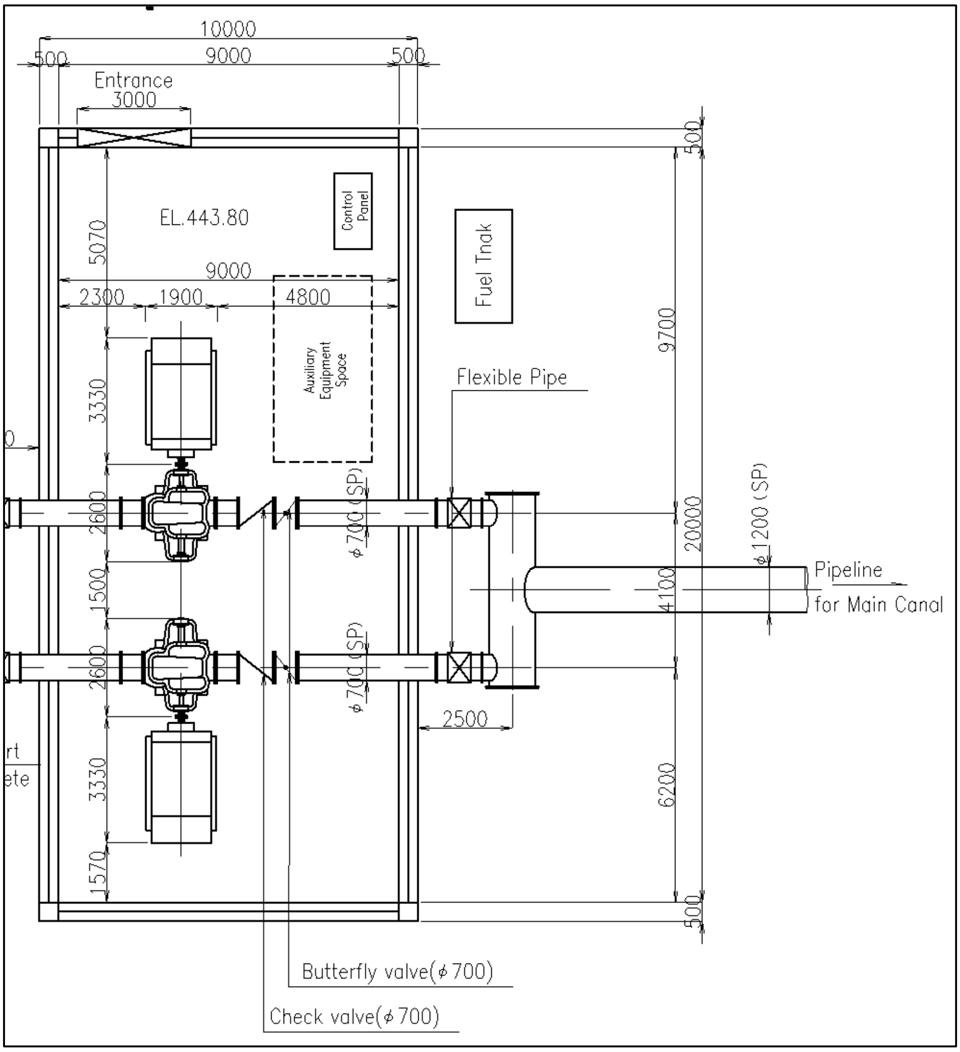


Figure 2.3.1 Plan and Section of Pump Station Building (Plan)

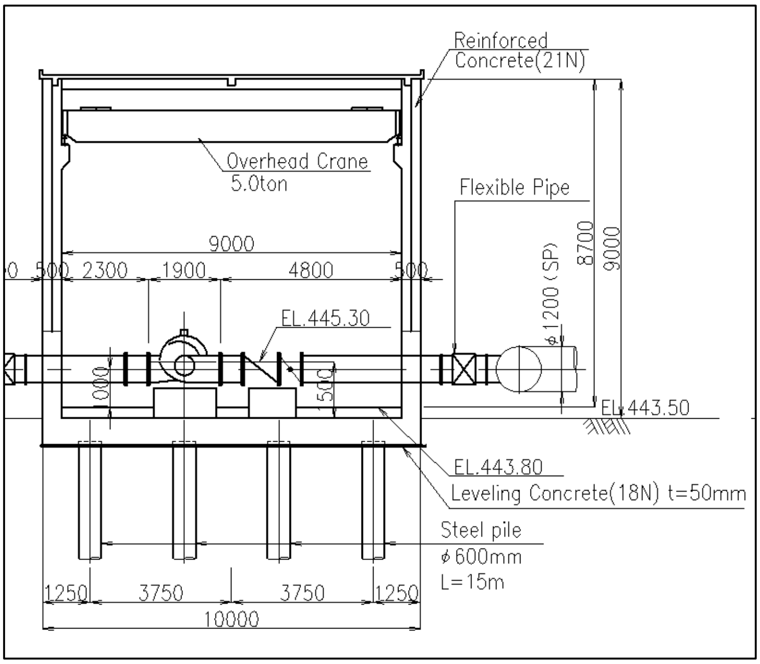


Figure 2.3.2 Plan and Section of Pump Station Building (Profile)

(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 at pump station is shown in the Figure 2.3.3. According to the log of boring, the geological condition are shown from loose to medium dense and it is not suitable for the spread foundation. It is recommended to adopt the pile foundation in future design stage through the additional geological investigation for clarifying the very dense layer. Also the groundwater seems to fluctuate seasonally from 4m to 7m of depth, according to the site report in the drilling work.

QUALITY IS OUR FOUNDATION

SMAT TECHNICAL SERVICES LIMITED

TEST BORING LOG SHEET

Project Name:	IRRIGATION DEVELOPMENT MASTER PLAN(IDMP)	Date:	24th-29th April-2015
Client:	SUE FOR CONSTRUCTION AND WATER RESOURCES ENGINEERING COMPANY LIMITED	Logged by:	E.R
Site Location:	JEBEL LADO N-5-02-54.77, E-31-41-27.78 Elevation - 462m	Drilling Method:	Rotary Core Drilling
Drilling Equipment:	Model XY-1B	Boring Dia. (mm)	120

SPT: Sampler with 63.5kg hammer falling 750mm — Bore hole No. PB-01

Depth (m)	Water level	USCS	GRAPHICS	Soil Description	Sample Type	Natural Moisture (%)	SPT - N (blows) N_{60}	Allowable Bearing Capacity	Remarks
0.0 - 1.50		SP		SAND: Medium dense Dark brown Silty sands	D	12.9	16	95	
1.5 - 2.5		SC		SAND: Medium dense Light brown silty clayey sands	D	19.0	13	82	
2.5-3.5		SC		SAND: Very loose Yellowish brown silty clayey sands	D	21.8	3	29	
3.5-4.5		SC		SAND: Very loose Yellowish brown clayey silty medium sands	D	17.9	3	31	
4.5-5.5		SC		SAND: Very loose grey Silty medium sands	D	18.8	3	31	
5.5-6.5		SC		SAND: Medium Dense yellowish brownish gravelly silty clayey sands	D	14.8	14	103	Water table was observed
6.5-7.5		SP		SAND: Medium Dense Yellowish brown Silty sands	D	16.4	11	86	
7.5-8.5		SC		SAND: Loose Greyish brown Silty clayey sands	D	14.7	8	69	
8.5-9.5		SC		SAND: Loose Yellowish brownish Clayey silty sands	D	16.3	7	62	
9.5-10.5		SP		SAND: Loose Greyish Brown gravelly clayey silty sands	D	19.8	5	51	

*(0.0-10.5)m

Figure 2.3.3 Log of Boring (PB-01) at pump station

2.4 Riverbank Protection

(1) Installation range

For the river bank of Bahr el Jebel, there is no protection works provided in general, and there can be seen some parts eroded by the river flow with higher velocity during the floods. The pump station sites are located on the gentle curve line river bank but not on the water colliding river bank. Due to the possible erosion by river flow, however, the suction pipe embedded underground might be exposed and the safety of the pipe could be endangered with the trashes/drifts clinched. This requires the riverbank protection works for attaining sustainable operation of the pump station. The extent of the riverbank protection shall cover 20 m each of both upstream and downstream directions from the center of suction pipe considering the position of suction pipe embedded. Also for connecting the riverbank protection with the present bank, 2 m width space shall be secured.

(2) Structure type

The structure of mortar masonry retaining wall by using natural stones shall be adopted for the protection works, considering the gentle slope of 1:2.0, safety against the effect of water flow, availability of required materials, other viewpoints including landscape evaluation, economy and easiness in construction etc. For the connecting work with the present riverbank and protection work for embankment slope, gabion works with high flexibility shall be adopted.

(3) Foundation

In order to average the variable velocity distributions during the suction, it is necessary to secure sufficient space for the head of suction pipe. Also, it is preferable to provide structures to fix the suction pipe so as to protect the suction pipe with considerable length exposed from various actions/effects by the river flow. In view of the above considerations, retaining wall of plain concrete which will satisfy the both requirements as discussed shall be provided for the foundation portion for the riverbank protection works around the suction pipe.

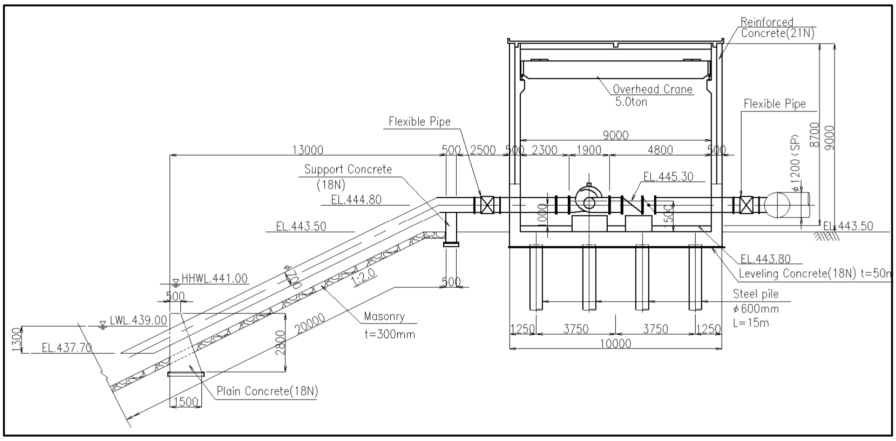


Figure 2.4.1 Plan and Section of Pump Station Building (Profile)

2.5 Pipeline

2.5.1 Pipeline from pump station to canal

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 1200mm 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.

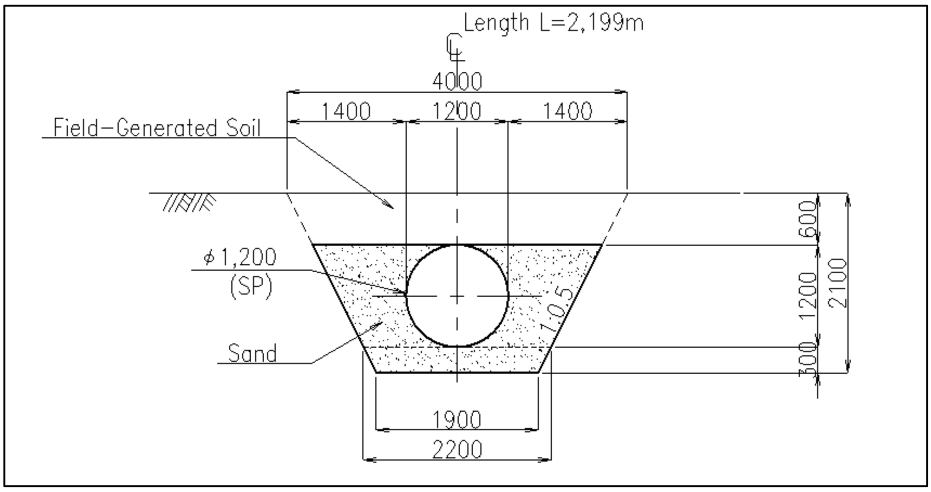


Figure 2.5.1Typical Section of Pipeline

2.5.2 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.

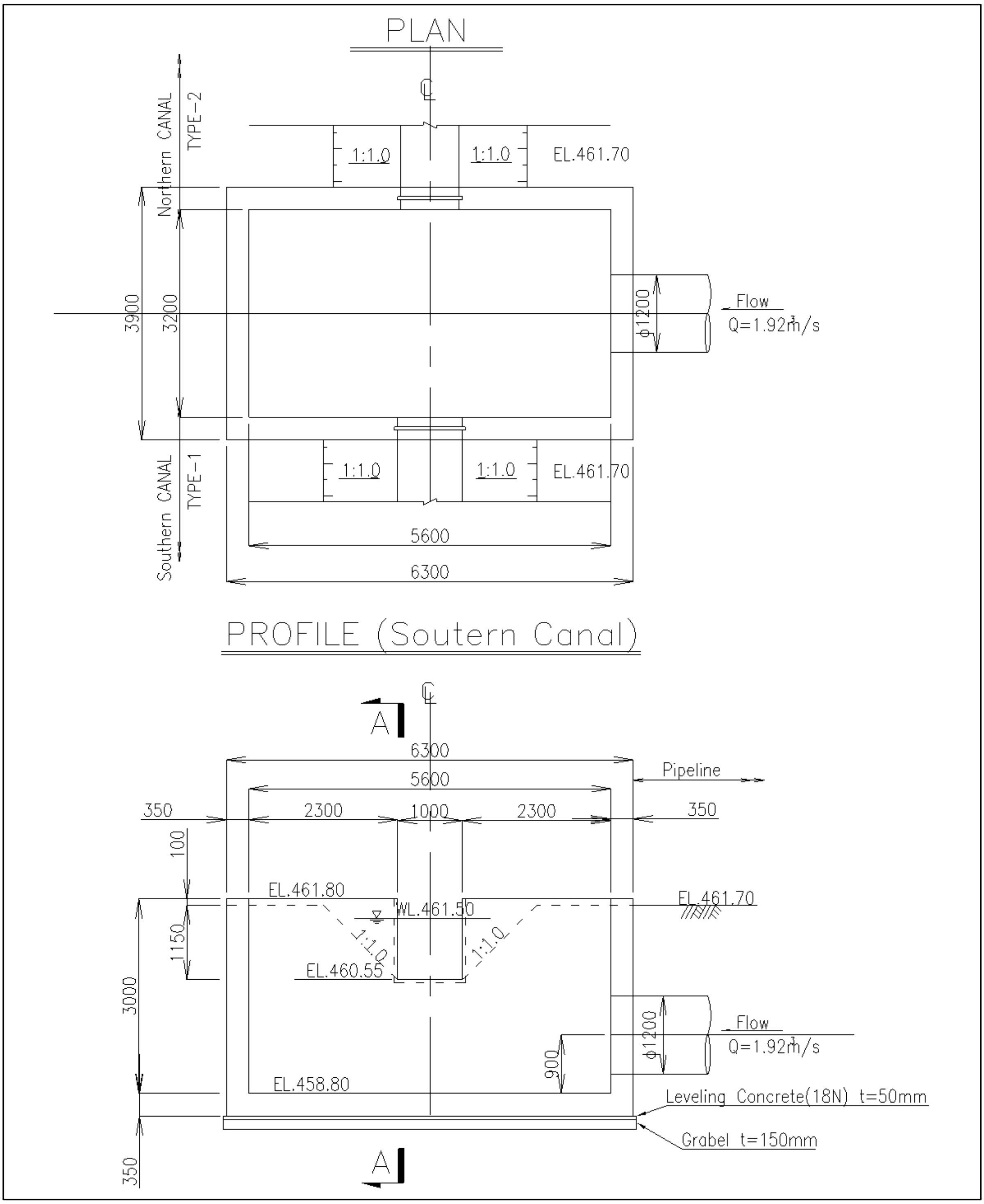


Figure 2.5.2 Discharge Chamber

CHAPTER 3 MAIN CANAL

3.1 Location

Main canal shall be planned to conduct the irrigation water from the discharge chamber at the end of pipeline to the command area. Also main canal is planned in the east side of command area which is featured as the high land comparatively. The main canal is separated to two (2) routes, one is the northern canal which is the length of 6.4km, and another is the southern canal which is the length of 7.0km.

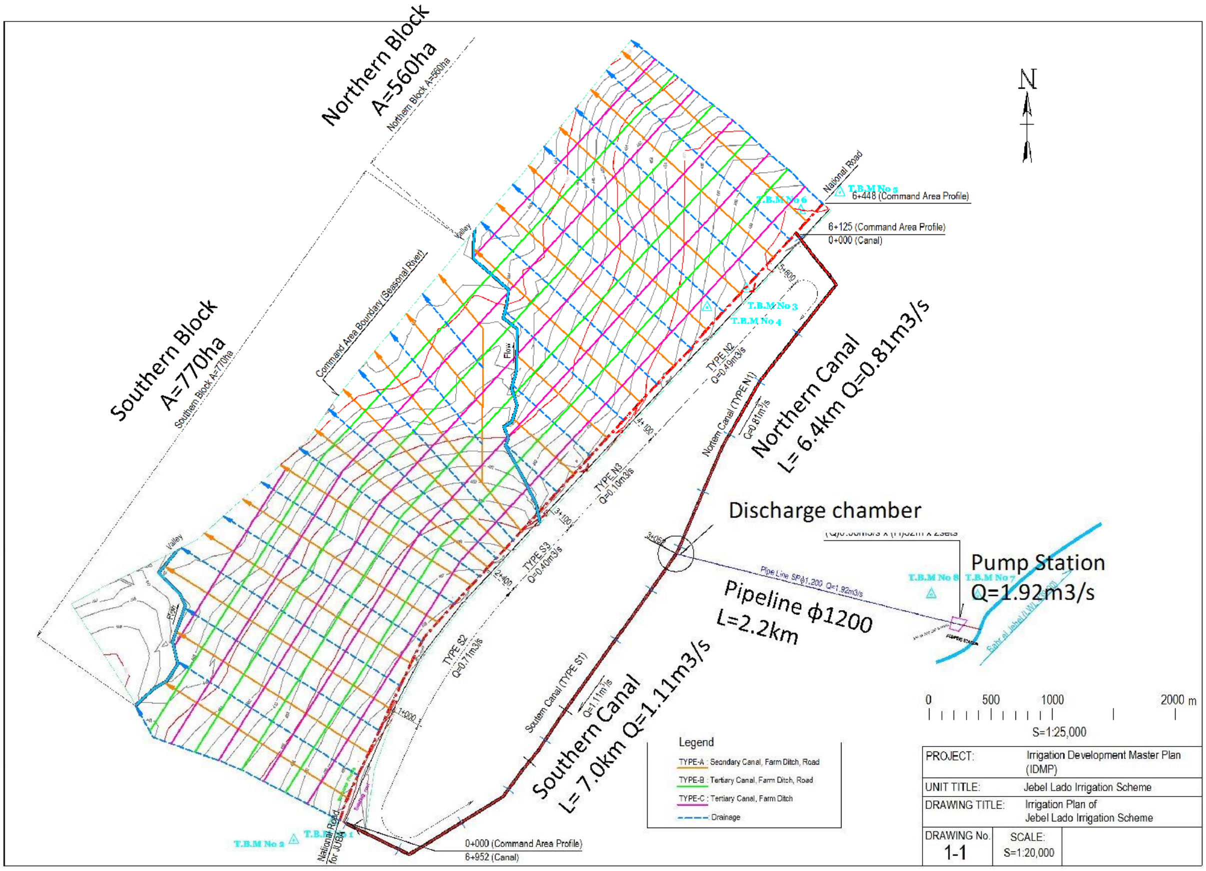


Figure 3.1.1 Location Map

The station number, length, and design discharge of each section is shown in the Table 3.1.1

Table 3.1.1 Main canal

Type	Station	Length (m)	Design Discharge (m3/s)
Northern Canal		6.400	
N1	3+052 + 0.000, and 6+125 to 5+800	3,052 325	0.81
N2	5+800 to 4+100	1,700	0.49
N3	4+100 to 3+100, and 6+125 to 6+448	1,000 323	0.10
Southern Canal		7,000	
S1	3+052 to 6+952, and 0+000 to 1+000	3,900 1,000	1.11
S2	1+000 to 2+400	1,400	0.71
S3	2+400 to 3+100	700	0.40

3.2 Design Discharge

Unit water requirement was estimated at 1.44 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 (0.00144 m^3/s)

A: Subject area (ha)

Design discharge

- Northern Main irrigation canal Type-1: Q= 0.81m^3/s (=0.00144×560ha)
- -do- Type-2: Q= 0.49m^3/s (=0.00144×(70ha+271ha))
- -do- Type-3: Q= 0.10m^3/s (=0.00144×70ha)
- Southern Main irrigation canal Type-1: Q= 1.11m^3/s (=0.00144×770ha)
- -do- Type-2: Q= 0.71m^3/s (=0.00144×(158ha+121ha+216ha))
- -do- Type-3: Q= 0.40m^3/s (=0.00144×(158ha+121ha))

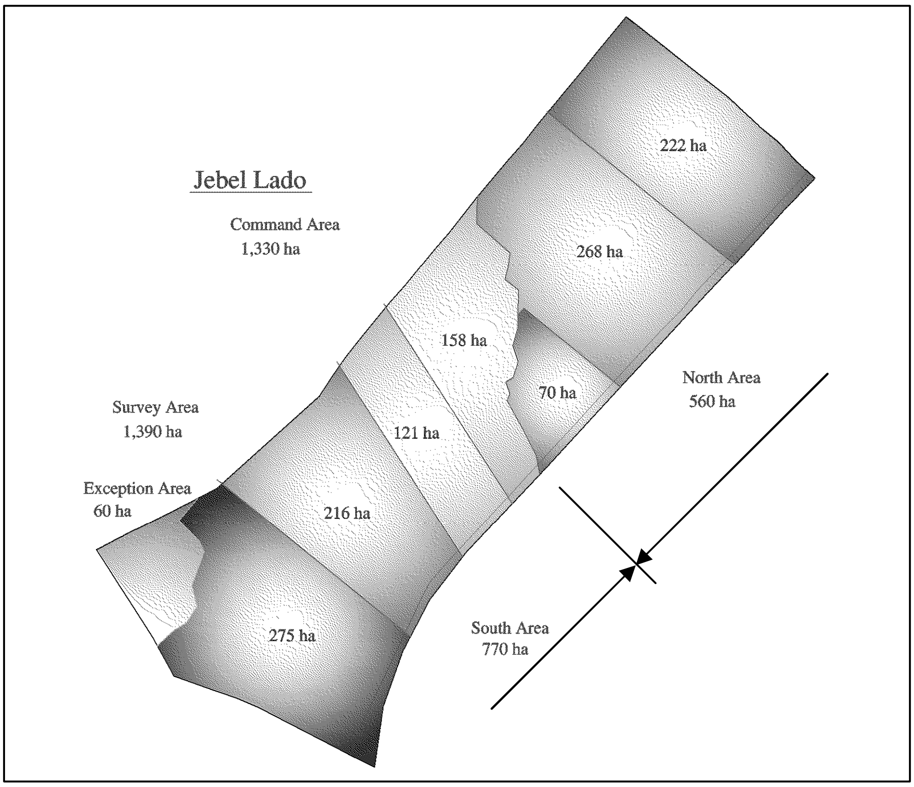


Figure 3.2.1 Area Map

3.3 Examination Method of Canal Capacity

Main canal is designed of the plain concrete lining, considering hydraulic characteristics, conveyance efficiency, durability, and maintenance. The required function of canal is to convey the irrigation water properly with the required water level and water volume supplied from the pump station. The volume of the required water is determined based on the irrigation area or each irrigation blocks as divided by the regulator. 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);

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

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

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.

$Fb = 0.05d + \beta \cdot hv + hw$

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 3.3.1 Calculation of Main Canal Section

Items		N1	N2	N3	S1	S2	S3
Design discharge	Q (m^3/s)	0.81	0.49	0.10	1.11	0.71	0.40
Width of canal bed	B (m)	0.90	0.50	0.30	1.00	0.60	0.40
Water depth	d (m)	0.843	0.401	0.280	0.948	0.531	0.347
Bank slope	1:N	1.0	1.0	1.0	1.0	1.0	1.0
Cross-sectional area of flow	A (m)	1.469	0.361	0.162	1.847	0.601	0.259
Wetted perimeter	P (m)	3.284	1.634	1.092	3.681	2.102	1.381
Hydraulic mean depth	R (m)	0.447	0.221	0.148	0.502	0.286	0.187
Coefficient of roughness	n	0.015	0.015	0.015	0.015	0.015	0.015
Canal bed slope	I (%)	0.02	0.315	0.11	0.02	0.167	0.50
Mean velocity	V (m/s)	0.551	1.361	0.620	0.595	1.181	1.544
Velocity head	hv (m)	0.016	0.094	0.020	0.018	0.071	0.122
Free board	Fb (m)	0.207	0.249	0.170	0.202	0.219	0.253
Height of canal	H	1.05	0.65	0.45	1.15	0.75	0.60

Canal profile and canal section are shown in the Figure 3.3.1, Figure 3.3.2 and Figure 3.3.3 respectively.

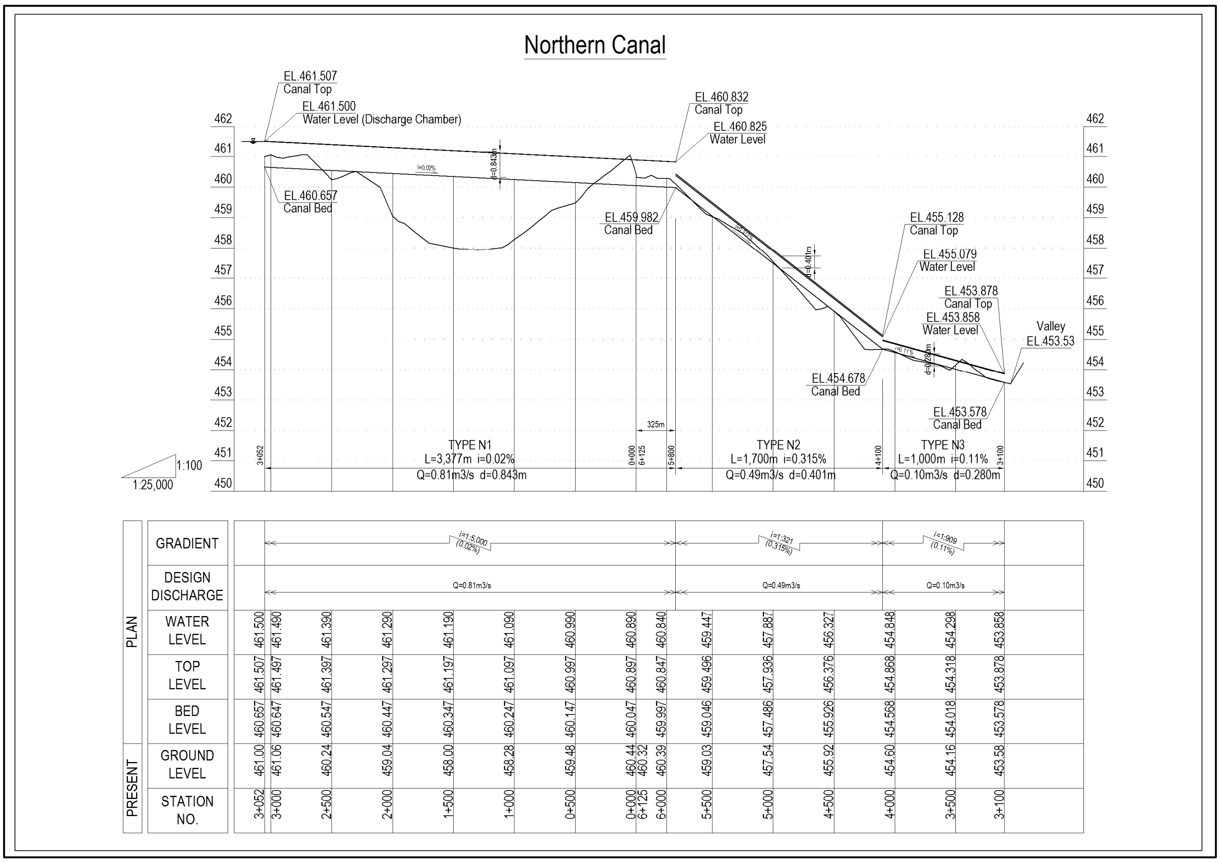
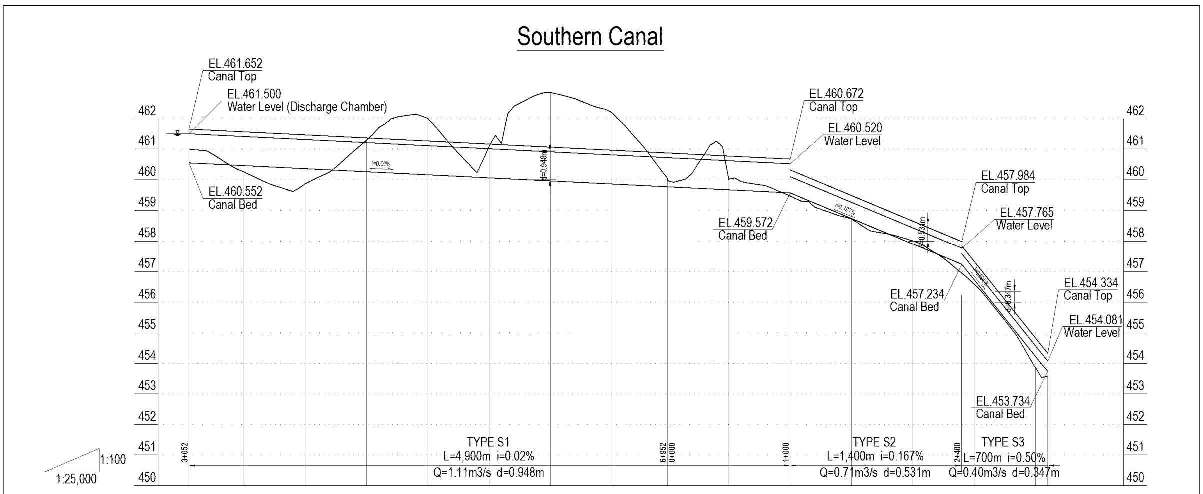
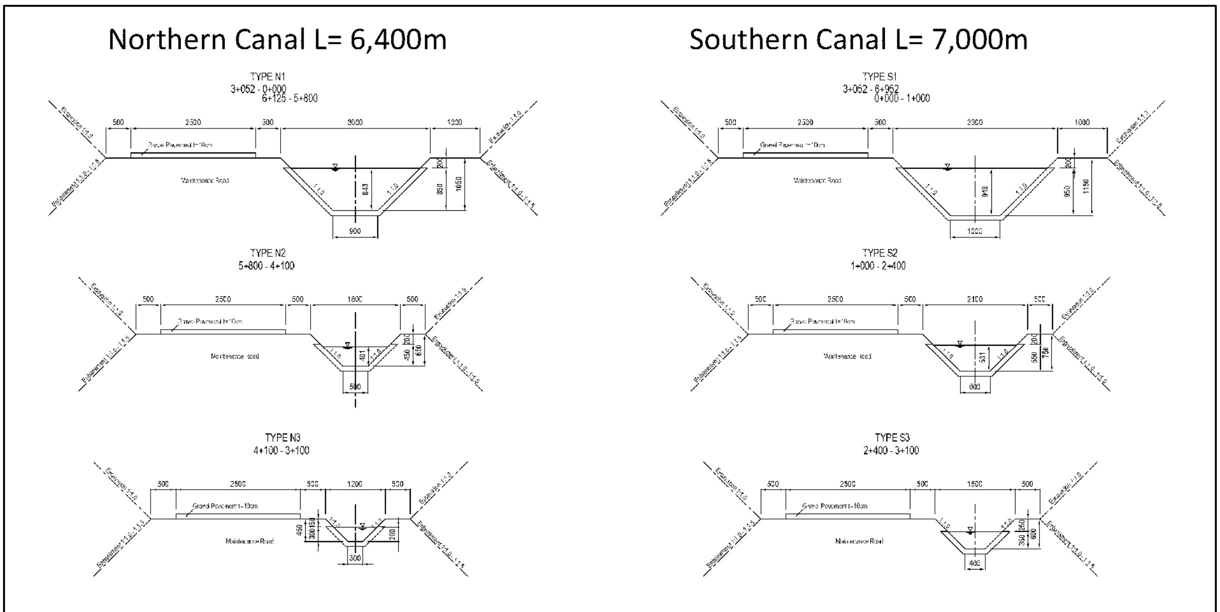


Figure 3.3.1 Northern Canal Profile



		3+052	3+500	4+000	4+500	5+000	5+500	6+000	6+500	6+952 0+000	0+500	1+000	1+500	2+000	2+500	3+000 3+100
PLAN	GRADIENT	i=1:5,000 (0.02%)										i=1:600 (0.167%)			i=1:200 (0.50%)	
	DESIGN DISCHARGE	Q=1.11m3/s										Q=0.71m3/s			Q=0.40m3/s	
	WATER LEVEL	461.500	461.410	461.310	461.210	461.110	461.010	460.910	460.810	460.720	460.620	460.520 460.103	459.268	458.433	457.081	454.581 454.081
	TOP LEVEL	461.652	461.562	461.462	461.362	461.262	461.162	461.062	460.962	460.872	460.772	460.672 460.322	459.487	458.652	457.334	454.834 454.334
	BED LEVEL	460.552	460.462	460.362	460.262	460.162	460.062	459.962	459.862	459.772	459.672	459.572	458.737	457.902	456.734	454.234 453.734
PRESENT	GROUND LEVEL	461.00	460.26	459.87	461.29	462.01	461.10	462.86	462.20	460.05 459.97	460.02	459.47	458.73	458.01	456.59	453.86 453.58
	STATION NO.	3+052	3+500	4+000	4+500	5+000	5+500	6+000	6+500	6+952 0+000	0+500	1+000	1+500	2+000	2+500	3+000 3+100

Figure 3.3.2 Southern Canal Profile



- Concrete Lining Canal
- Maintenance Road W= 3.5m (Gravel pavement)

Figure 3.3.3 Typical Cross section of Main Canal

3.4 Relative structures

In general the relative structures such as diversion gate, canal spillway, drop, water measurement facilities, cross culvert and siphon etc. are required in the canal system if necessary. They shall be designed considered the canal system and the terrain around canal in the future design stage.

3.5 Recommendation

As shown the canal profiles of Figure 3.3.1 and 3.3.2, the route of northern canal in the survey work is located in the low land and the numerous earthworks for embankment are required in this section. On the other hand, the route of southern canal in the survey work is located in the high land and the numerous earthworks for excavation are required in this 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 4 IRRIGATION AND DRAINAGE SYSTEM IN FARMLANDS

4.1 Outline of Command Area

The command area is divided along the valley located at the middle of command area to the northern block (A=560ha) and the southern block (A=770ha). The ground gradient is shown comparatively gentle about 0.9% in northern block and about 0.8% in southern block toward northwest from southeast.

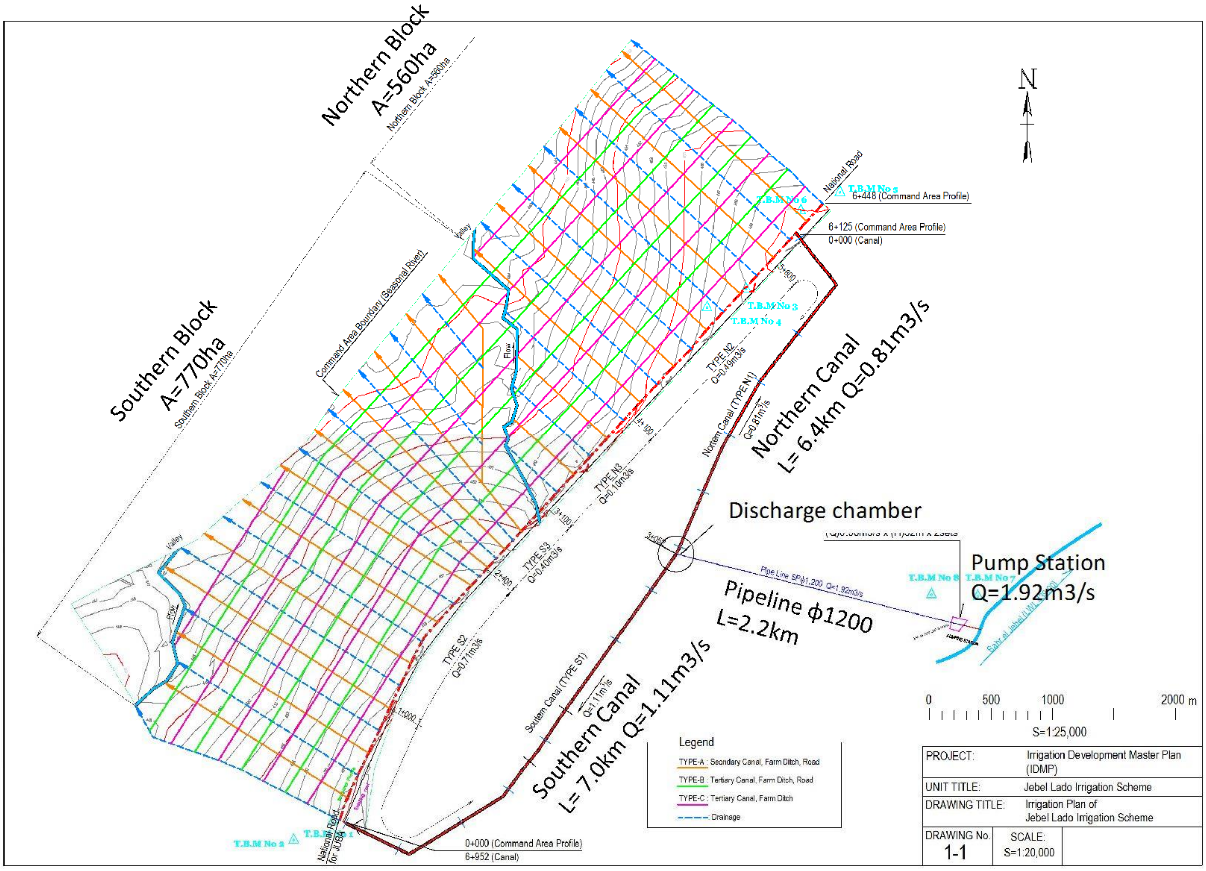


Figure 4.1.1 Location Map

The major facilities such as the secondary canal, drainage and road are generally arranged from highland toward lowland. The tertiary canal is planned to branch off from the secondary canal, also the feeder canal is planned to branch off from the tertiary canal for the distributing the irrigation water to the furrows. The canal is made of the earth because of a small size.

The drainage is allocated between the both secondary canals. The surplus water from tertiary canal and farm flows down to the drainage. The drainage is planned to be the earth canal.

The road used for farming and maintenance for facilities shall be planned along the secondary canal and tertiary canal. The road crossing is placed at the crossing point between the canal and road.

The length of furrow of 100m is assumed based on the soil survey results conducted in command area and table 9.1 shown below. That survey results that the soil classification is mainly loam.

Table 4.1.1 Example of Maximum Furrow Length of Different Soil

Soil	Root zone depth (m)	One-time irrigation volume (mm)	Maximum furrow length (m)
Sandy soil	40	16	4
Volcanic ash soil	40	44	29
Sandy loam	40	34	36
Loam	40	38	99
Clay	40	44	121

Note: Furrow inclination is 10%

Source: Engineering Manual for Irrigation & Drainage, Upland Irrigation (1990), The Japanese Institute of Irrigation and Drainage

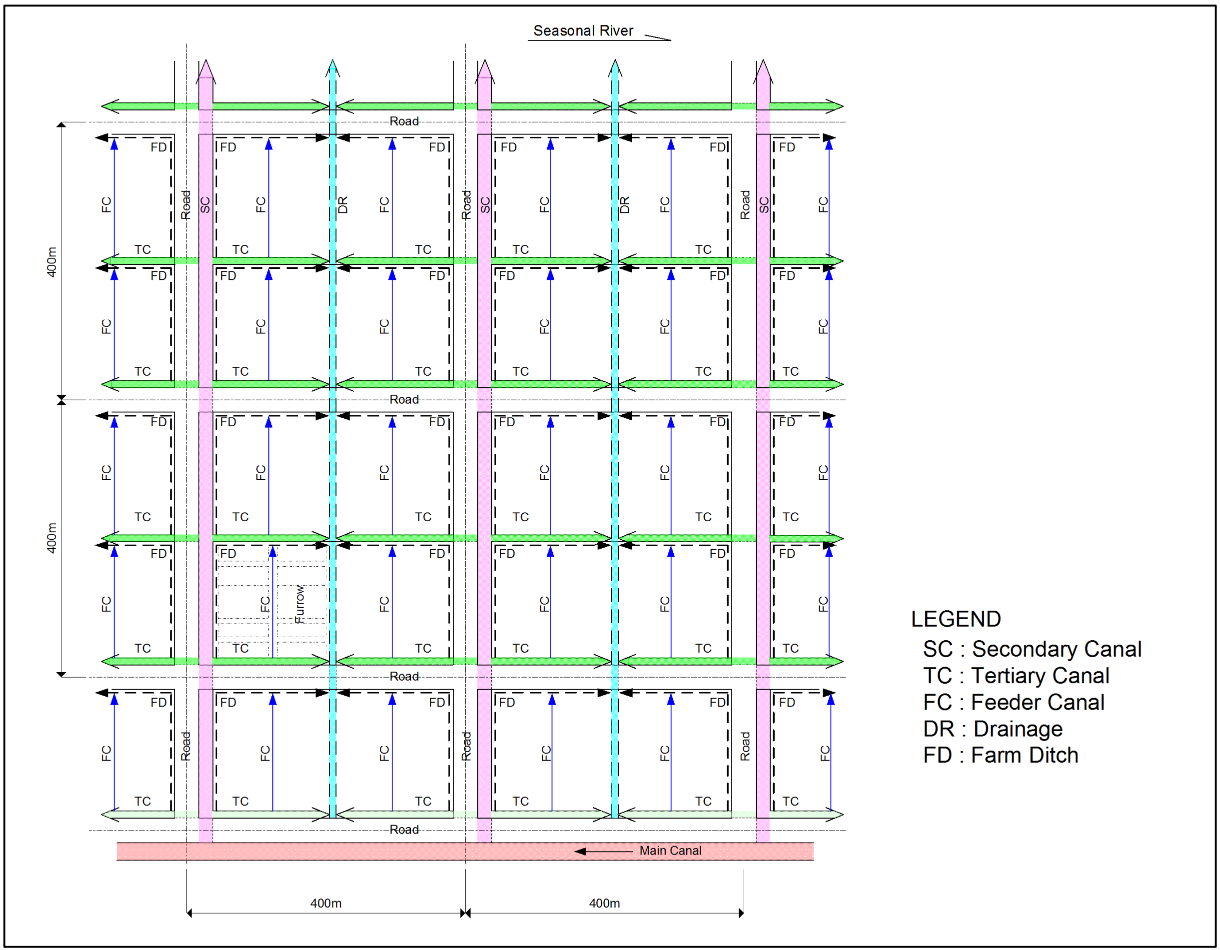


Figure 4.1.2 Layout of Irrigation and drainage Facilities in Command Area

4.2 Design Discharge of Canal and Drainage

4.2.1 Irrigation Canal

Unit water requirement was estimated at 1.44 l/s/ha (0.00144m^3/s/ha), depending on the calculation results in the table 2.5.1 in 9.2.3 Irrigation and Drainage Plan.

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 (0.00144 m^3/s)

A: Subject area (ha)

- Secondary canal: Q= 0.12m^3/s (=0.00144×averagely 80ha)
- Tertiary canal and Feeder canal: Q= 0.023m^3/s (=0.00144×16ha)

4.2.2 Drainage

Unit area drainage discharge was estimated at 0.045m3/s/ha, depending on the calculation results as below.

Return period T=5 year: outflow Q= 60.4m^3/s (catchment area A= 13.3km^2)

Unit area drainage discharge: q= Q/A = 4.5 $m^3/s/km^2$ = 0.045m^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)

- Drainage : Q= 3.78m^3/s (=0.045× averagely 84ha)
- Farm ditch: Q= 0.090m^3/s (=0.045×averagely 2ha)

<u>Unit area drainage discharge (5 year return period)</u>

・Rainfall intension

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

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

R_{24} : 24 hours rainfall (94.4 mm for 5 year return period)

t : Rain fall duration or period of flood concentration (= 1 hr)

$$r_t = \frac{R_{24}}{24} \cdot \left(\frac{24}{t}\right)^{2/3} = \frac{94.4}{24} \cdot \left(\frac{24}{1}\right)^{2/3} = 32.7$$

Å 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 (13.3 km^2) → 1330ha

r_e: average effective rainfall intensive in the catchment within the lag time of flood (32.7 mm/hr) $r_e = f_p \cdot r_t = 0.5 \times r_t$

$$Q_p = \frac{1}{3.6} \times 0.5 \times 32.7 \times 13.3 = 60.4 (m^3/s)$$

Therefore, Unit area drainage discharge: q= 60.4m^3/s / 1330ha = <u>0.045 m^3/s/ha</u>

Table 4.2.1 Annual Maximum Rainfall in Juba

Year	(mm)	Year	(mm)	Year	(mm)	Year	(mm)
1970	66.7	1981	66.5	1992	52.0	2003	missing
1971	86.7	1982	131.8	1993	115.0	2004	missing
1972	46.4	1983	71.7	1994	106.5	2005	68.0
1973	64.0	1984	81.0	1995	55.0	2006	95.0
1974	68.5	1985	87.9	1996	70.0	2007	73.5
1975	88.3	1986	67.5	1997	75.5	2008	62.0
1976	83.9	1987	80.0	1998	59.0	2009	67.2
1977	41.0	1988	137.5	1999	119.5	2010	60.0
1978	82.9	1989	74.5	2000	66.8	2011	73.0
1979	107.0	1990	111.5	2001	61.2	-	-
1980	68.2	1991	51.5	2002	76.0	-	-

Table 4.2.2 Return Period Probability

Return Period T Year	ζ	1/a・ζ	Average Y+1/a・ζ	X+b	Return Period Probability (m3/s) X
2	0	0	1.7718	59.1326	74.4
5	0.5951	0.12644	1.8983	79.1167	94.4
10	0.9062	0.19254	1.9643	92.1229	107.4
20	1.1631	0.24710	2.0189	104.4552	119.7
30	1.2967	0.27551	2.0473	111.5160	126.8
50	1.4520	0.30851	2.0803	120.3189	135.6
100	1.6450	0.34951	2.1213	132.2333	147.5
200	1.8215	0.38702	2.1588	144.1590	159.5
500	2.0350	0.43238	2.2042	160.0310	175.3
1000	2.1850	0.46425	2.2361	172.2165	187.5

4.3 Examination Method of Canal Capacity

All of canals in the command area are designed of the earth canal, considering the economical reason. The required function of canal is to convey the irrigation water properly with the required water level and water volume supplied from the pump station. The volume of the required water is determined based on the irrigation area or each irrigation blocks as divided by the regulator. 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 canals : n = 0.015

and for earth canals : 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.

$Fb = 0.05d + \beta \cdot hv + hw$

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 4.3.1 Calculation of Irrigation Canal Section and Farm ditch

Items		Secondary canal	Tertiary canal	Feeder canal	Farm ditch
Design discharge	Q (m^3/s)	0.12	0.023	0.023	0.090
Width of canal bed	B (m)	0.30	0.30	0.30	0.30
Water depth	d (m)	0.270	0.174	0.092	0.182

Bank slope	1:N	1.0	1.0	1.0	1.0
Cross-sectional area of flow	A (m)	0.154	0.082	0.036	0.088
Wetted perimeter	P (m)	1.064	0.792	0.560	0.815
Hydraulic mean depth	R (m)	0.145	0.104	0.064	0.108
Coefficient of roughness	n	0.025	0.025	0.025	0.025
Canal bed slope	I (%)	0.50	0.10	1.0	1.0
Mean velocity	V (m/s)	0.780	0.279	0.642	0.907
Velocity head	hv (m)	0.031	0.004	0.021	0.042
Free board	Fb (m)	0.180	0.126	0.203	0.168
Height of canal	H (m)	0.45	0.30	0.30	0.35

Table 4.3.2 Calculation of Drainage Section

Items / Type		SD-1	SD-2	SD-3	SD-4	SD-5
Design discharge	Q (m^3/s)	3.78	2.88	2.16	1.44	0.72
Width of canal bed	B (m)	1.30	1.10	1.00	0.80	0.60
Water depth	d (m)	1.233	1.074	0.931	0.786	0.585
Bank slope	1:N	1.5	1.5	1.5	1.5	1.5
Cross-sectional area of flow	A (m)	3.883	2.912	2.231	1.555	0.864
Wetted perimeter	P (m)	5.746	4.972	4.357	3.634	2.709
Hydraulic mean depth	R (m)	0.676	0.586	0.512	0.428	0.319
Coefficient of roughness	n	0.025	0.025	0.025	0.025	0.025
Canal bed slope	I (%)	0.100	0.127	0.143	0.167	0.200
Mean velocity	V (m/s)	0.974	0.990	0.968	0.927	0.835
Velocity head	hv (m)	0.048	0.050	0.048	0.044	0.036
Free board	Fb (m)	0.217	0.226	0.219	0.214	0.165
Height of canal	H (m)	1.45	1.30	1.15	1.00	0.75

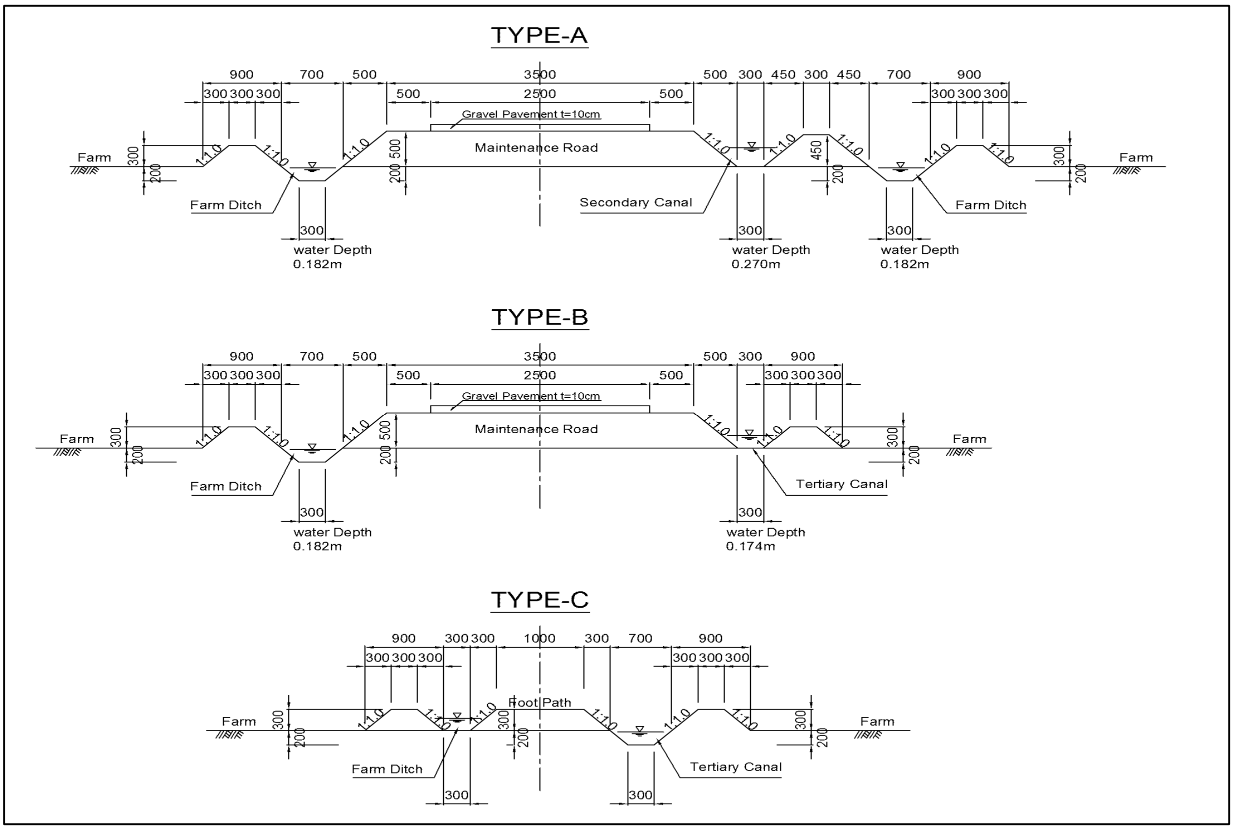


Figure 4.2.1 Typical Cross section of Irrigation and drainage Facilities in Command Area

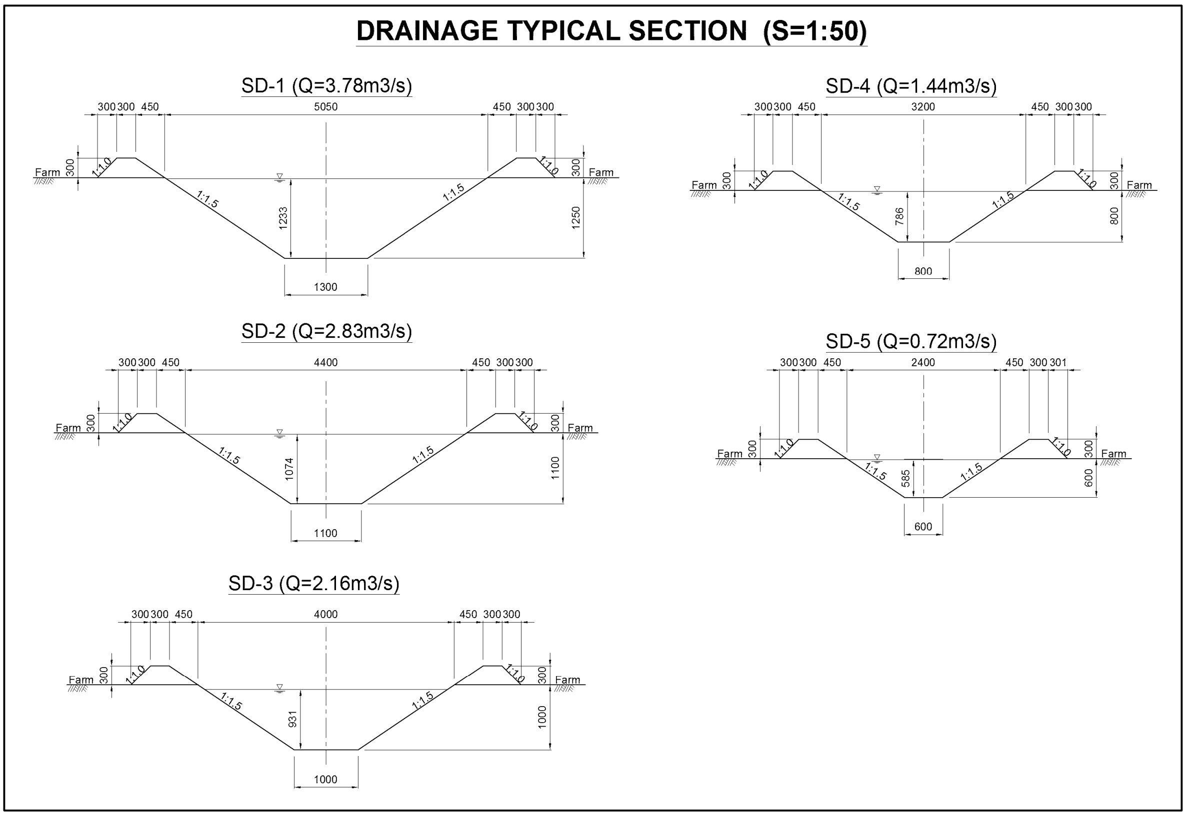


Figure 4.2.2 Typical Cross section of Drainage in Command Area

4.4 Furrow Irrigation methods

Surface irrigation methods are in general classified to four (4) methods. In this Jebel Lado area, the furrow irrigation methods are adequate depending on the conditions of the terrain and soil characteristic.

Surface Irrigation Methods

1) Furrow irrigation methods

The method irrigates plant roots by water permeated from the side of the furrow. Supply channels are arranged at certain intervals between the moderately sloped furrow, and cause a fixed amount of water to flow. Water is retained for the minimum necessary time to secure water depth downstream to supply sufficient water to the roots, while upstream where water is retained for an excessive time, water penetration loss to the deeper layer cannot be avoided. The irrigation efficiency is influenced by geographical features, intake rate, furrow length, and discharge amount. To make a uniform slope of a furrow, construction is required.

2) Border irrigation methods

The field is divided into bands by low boundary ridges, and sloped to cause water to flow as a thin laminar flow. The deep layer penetration loss and irrigation efficiency are similar to those of the furrow irrigation method. Compared with the furrow irrigation method, it requires less labor force; whereas it requires greater amount of water and as the limitation factor is the slope, land levelling over a wider area is indispensable. It is often used for irrigation pasture land.

3) Contour ditch irrigation methods

A ditch to introduce water is prepared with a slope of 1/1000 along the contour line, and water is supplied from the turnout provided at the ditch. The method is applicable even on relatively irregular land, but the irrigation efficiency is low.

4) Basin irrigation methods

According to this method, farm land will be flatten and enclosed by ridges. The irrigation water will be conveyed through canals or pipelines to irrigate the farm land intermittently.

4.5 Relative structures

In general the relative structures such as diversion gate, drop, water measurement facilities, cross culvert and siphon etc. are required in the canal system if necessary. They shall be designed considered the canal system and the terrain around canal in the future design stage.

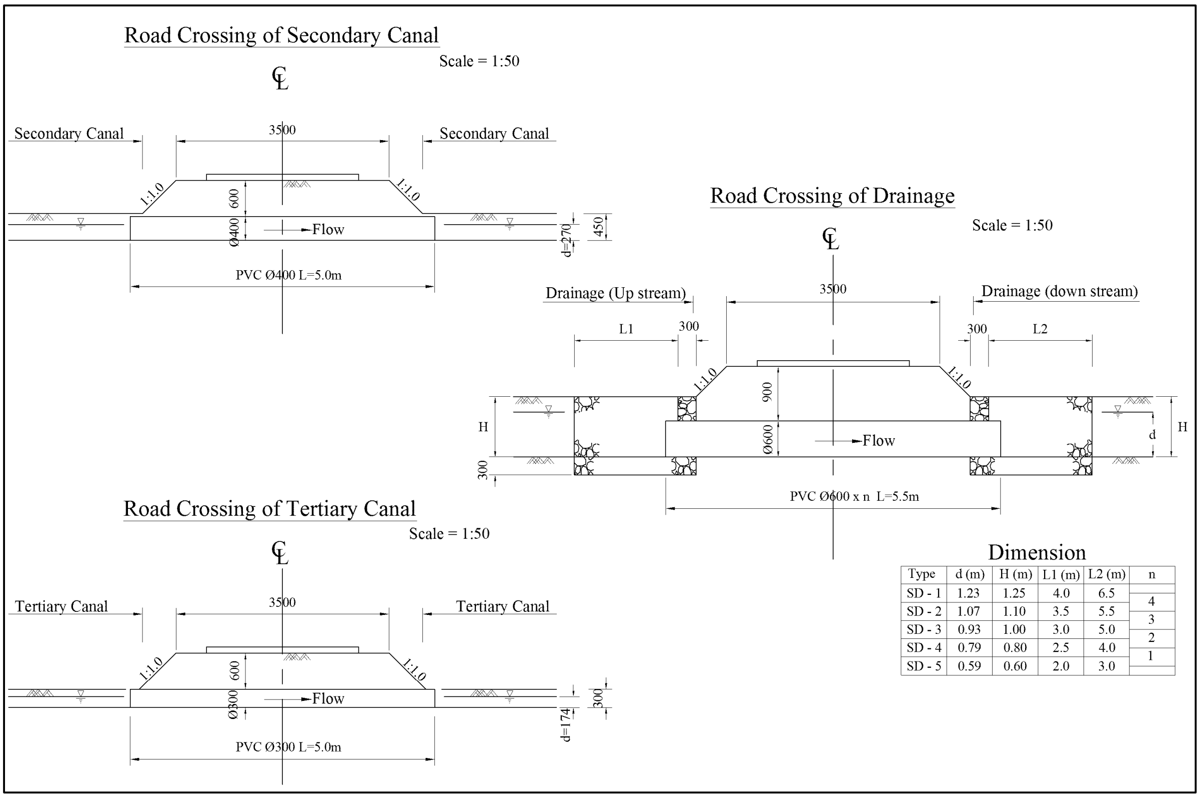


Figure 9.3 Typical Cross section of Road Crossing in Command Area

4.6 Recommendation

(1) Design of Upland Irrigation and Intake Rate

In the future design stage, more investigation is required to carry out the design of upland. For example, the intake rate is very important factor to make a plan of irrigation system. The intake rate is the rate for irrigation water or rainwater infiltration into soil under the specific conditions, and generally measurement in term of mm/hr. As an index of water permeability in unsaturated soil, it is an important factor to be considered in deciding the irrigation method and the appropriate irrigation intensity for upland irrigation.

The intake rate is measured either by the cylinder intake rate or by the furrow intake rate, depending on the purpose of the measurement. For furrow irrigation, the intake rate is measured by the furrow intake rate.

(2) Drainage in Command area

As shown in the table 4.3.2 of calculation of drainage section, the canal bed slope is determined gentler than the existing ground gradient of 0.9% so that the velocity of canal does not exceed the maximum allowable velocity of 1.0m/s by clay. In this case, many drop structures are required on the drainage line at the site. It is recommended to study the canal type more or compare with the concrete lining canal from the viewpoint of economical efficiency. Moreover, it is recommended to review the unit area drainage discharge whether it could be reduce or not.

CHAPTER 5 REFERENCE: DAM

- **Dam specification:**

Table 5.1 Specifications

Facility	Items	Specification	Remarks
General	Location	25 km upstream from Jebel Lado	
	River name	Koda River	
	Purpose	Irrigation	
Reservoir	Catchment area	1,063.1 km^2	
	Reservoir area at FWL	12.2 km^2	
	Total storage capacity	36,500,000 m^3	
	High water level	H.W.L. 487.5 m	
	Full water level	F.W.L. 487.0 m	
	Minimum operation water level	L.W.L. 483.6 m	
	Available depth	3.4 m	
Dam	Dam type	Fill type (Homogeneous)	
	Dam height	11.25 m	
	Dam length	2,330 m	
	Dam crest width	5 m	
	Dam crest level	E.L. 489.25 m	
	Foundation treatment	Cutoff	
	Dam volume	366,000 m^3	

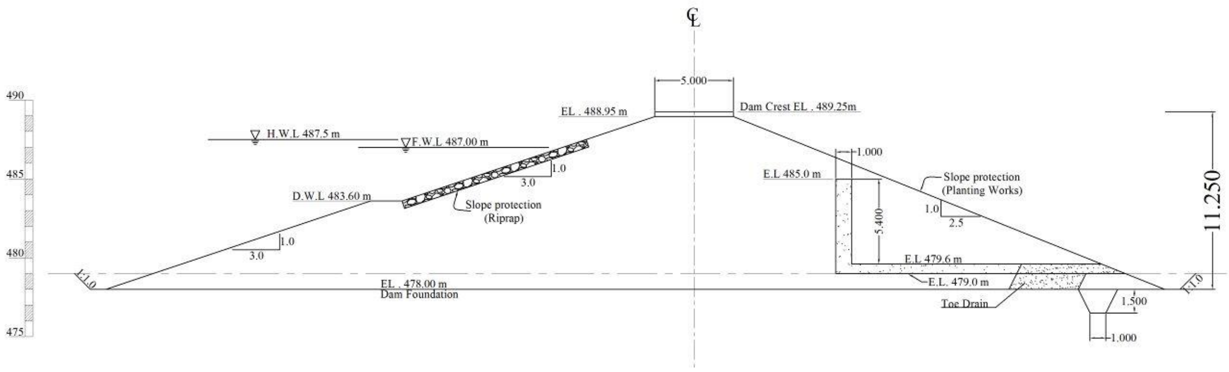


Figure 5.1 Typical Cross Section of Dam

- **Construction Cost**

The construction cost of 30 million US$ is the direct cost without including the overhead cost.

- **Conclusion**

There is a candidate site for dam but the site is very far of 25km from the subject command area and the construction cost of the dam and the distribution canal requires numerous expenses. Therefore it is judged that the irrigation method operated by the dam is dismissed due to its economic efficiency.

5.1 Planning of Reservoir

5.1.1 Location of Dam Site

In order to send the irrigation water by gravity to the command area, the intake elevation at the dam site must be higher enough than the highest elevation in the command area, which is EL. 474.0 m according to the Google Earth. Moreover, it is desirable to give a suitable inclination to the conveyance canal to flow down the water economically. From such view point, the dam site location is selected at the point just upstream of Wulikare village, on the Koda River westward about 25 km in direct distance from the command area. The intake elevation at this point is estimated to be EL.482m.

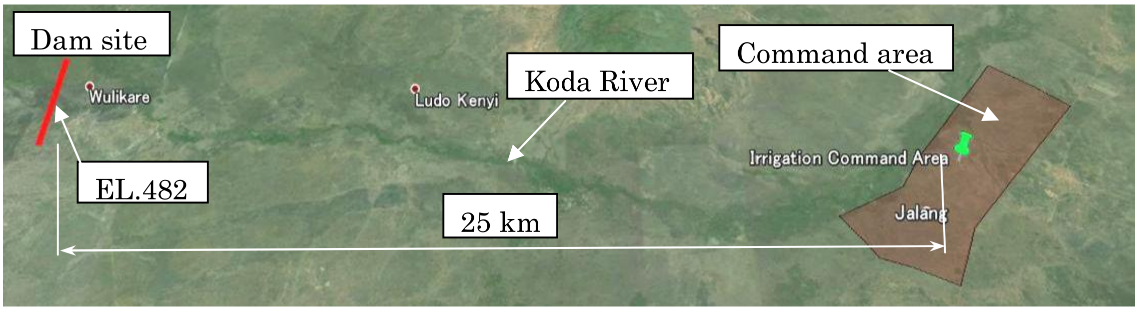


Figure 5.1.1.1 location of the dam site

5.1.2 Dimensions of Reservoir

(1) H~Q curve

Google Earth has the function of indicating the elevation of the cursor point and measuring the area of an arbitrary polygon. By using these functions, we can get the H~Q curve, i.e. the relationship between the stored water level and the reservoir capacity through the following process.

- Setting out the elevations of stored water level every suitable period of meters
- In descending/ascending order of the elevations, decide the shape of polygon composed of the elements of same elevation including the elevation point on the dam axis, left/right side as the starting/ending point.
- The area corresponding to the decided polygon is shown at the same time.
- The volume of stored water between the two elevations can be calculated through multiplying the mean area of two areas by the differential in height between two elevations.
- Consequently, the accumulative volumes at each step of stored water surface can be calculated; and then we can get the relationship between the stored water surface level and the stored water capacity at the corresponding water level.

The followings are the polygons of stored water surface shown on the Google Earth , the H~Q curve and the calculation table

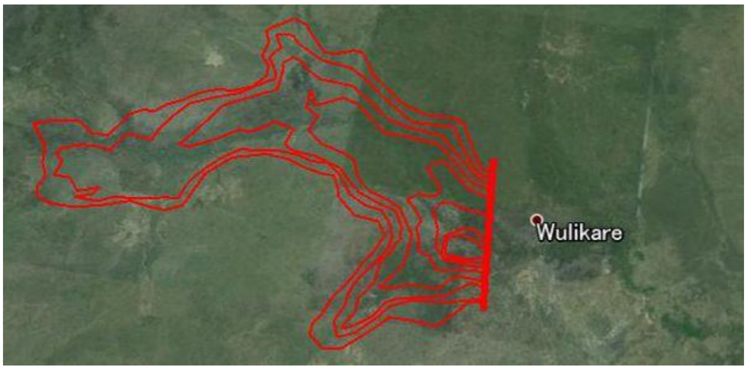


Figure 5.1.2.2 Water surface contour drawn on the Google Earth

Water surface	Water depth	Area (m2)	Area(mean) (m2)	sectional depth(m)	sectional volume(m3)	accumulative volume(m3)	accumulative volume (million m3)
478	0.0	0				0.0	0.00
479	1.0	230500	115250	1.0	115250	115250.0	0.12
480	2.0	385100	307800	1.0	307800	423050.0	0.42
481	3.0	898400	641750	1.0	641750	1064800.0	1.06
482	4.0	1916400	1407400	1.0	1407400	2472200.0	2.47
484	6.0	5387200	3651800	2.0	7303600	9775800.0	9.78
486	8.0	9409300	7398250	2.0	14796500	24572300.0	24.57
488	10.0	15438500	12423900	2.0	24847800	49420100.0	49.42
490	12.0	20355400	17896950	2.0	35793900	85214000.0	85.21

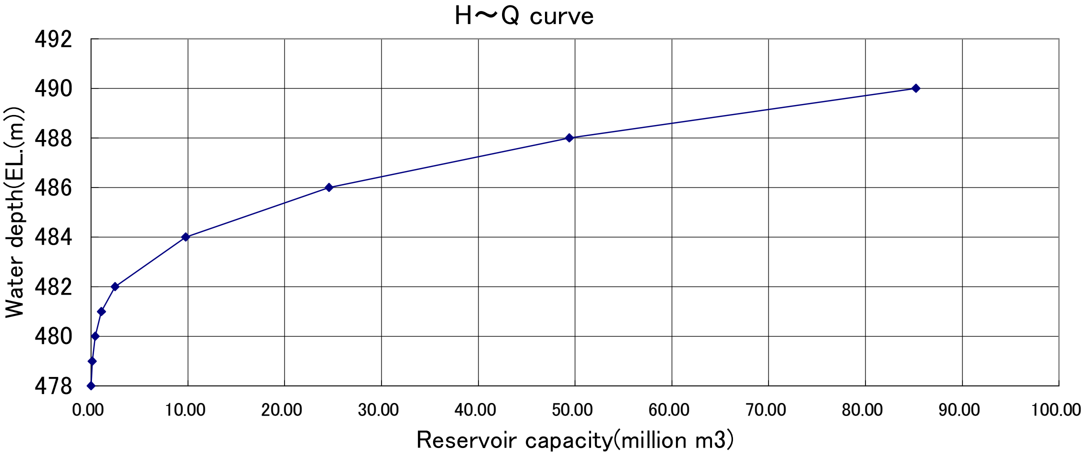


Figure 5.2.2.1 H~Q curve of the Jebel Lado reservoir

(2) Catchment area and its water resource

Google Earth has the function of indicating the linear distance and the cross-sectional profile between two points, i.e. ñpass functionò, and of marking the specified point, i.e. ñmarking functionò. By using these functions and the polygon function, we can draw the border line of watershed and get the catchment area concerned through the following process.

・Finding out a valley/river adjacent to the valley/river consisting of the watershed concerned.

・Setting out two points, i.e. a straight line, on the above boundary hill or mountain range.

・The cross-section shall be given on the screen through the ñpass functionò.

・Find out the dividing ridge on the cross-section and give the mark on that point.

・Repeat the above work at the interval of suitable distance all around the watershed.

・Draw the border line of watershed by connecting the markings and making the polygon of watershed; then the screen shows the catchment area.

The catchment area of the Jebel Lado reservoir drawn through the process above is shown in Fig. 2.3.1.4 together with the catchment areas of the surrounding dam site candidates.

The largeness of the catchment area of the Jebel Lado reservoir is 1,061.3 km^2. The quantity of water resource at the dam site is estimated to be 62.5 million m^3 in annual average through multiplying 1,063.1 km^2 by the specific runoff yield of 58.9 mm.

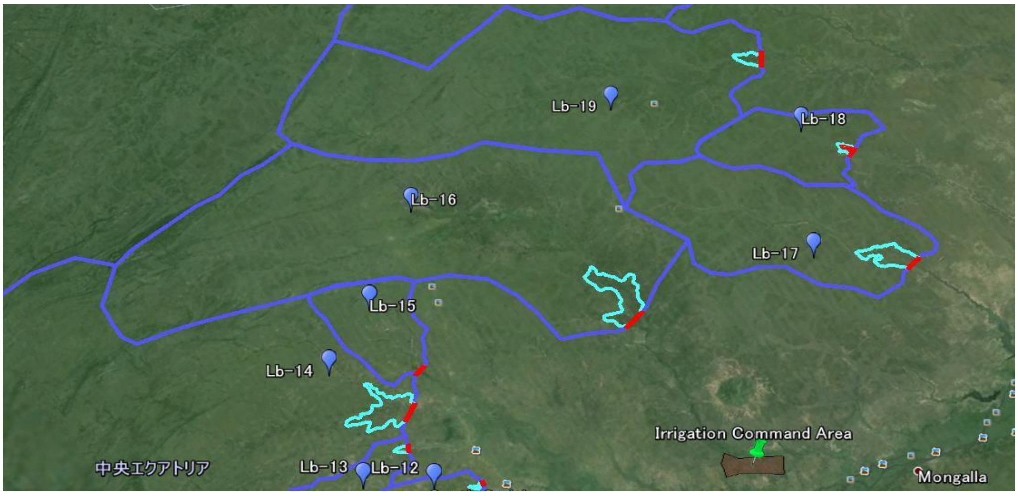


Figure 5.1.2.3 Catchment area drawn on the Google Earth

(3) Total capacity of the reservoir and the full water level

The total capacity of the reservoir is the sum of the effective capacity of the reservoir and the design sediment quantity. According to the estimation result of water requirement in Section 1.2, the annual water requirement to the reservoir is 24,300,000 m^3. Under consideration of allowance, 30,000,000 m^3 shall be adopted as the effective capacity of the reservoir.

The design sediment quantity is estimated to be 6,500,000 m3 according to Section 2.3.2 (3) so that the total capacity of the reservoir becomes 36,500,000 m3; and the Full Water Level of the reservoir given by the H~Q curve as the water level corresponding to this total capacity of the reservoir is EL.487.0.

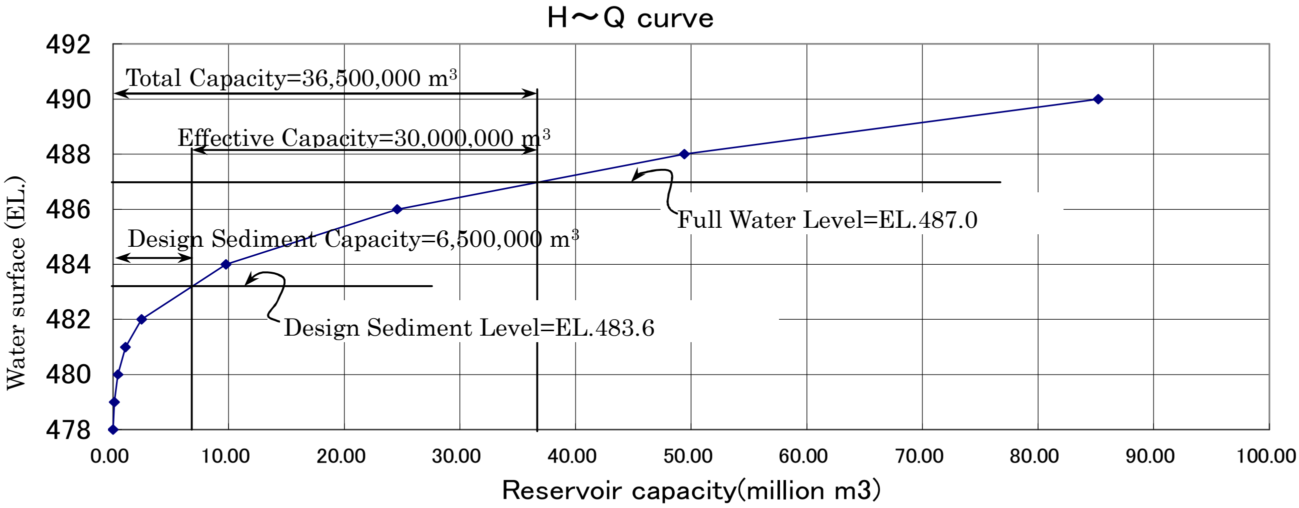


Figure 5.1.2.4 Deciding of reservoir dimensions on H~Q curve

(4) Water surface area

The surface area at Full Water Level is 12,200,000 m2 based on the H~A curve.

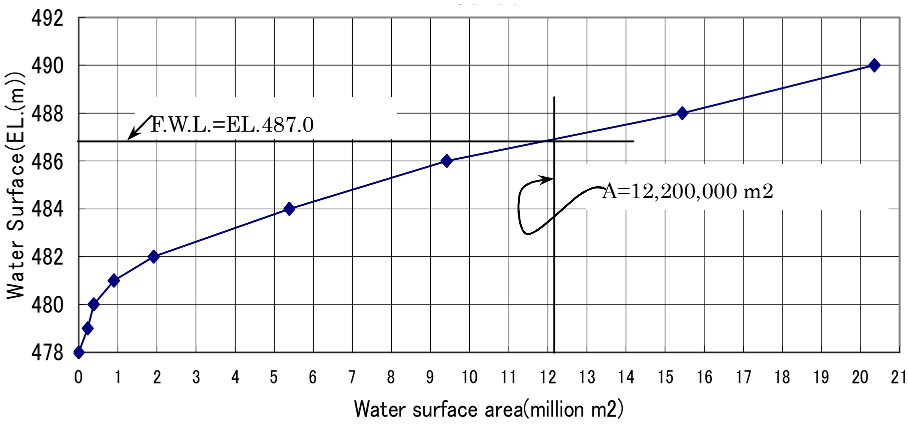


Figure 5.1.2.5 H~A curve of the reservoir

5.2 Design of Dam

5.2.1 Dam Type

(1) General classification and its applicability of dams

The classification of dams and its applicability are summarized as shown below. To the dam planned here, the homogeneous type shall be applied because of its height being less than 15 m, thick soil layers being assumed as the foundation, and the thick soil layers including clay being assumed to cover the surrounding ground surface.

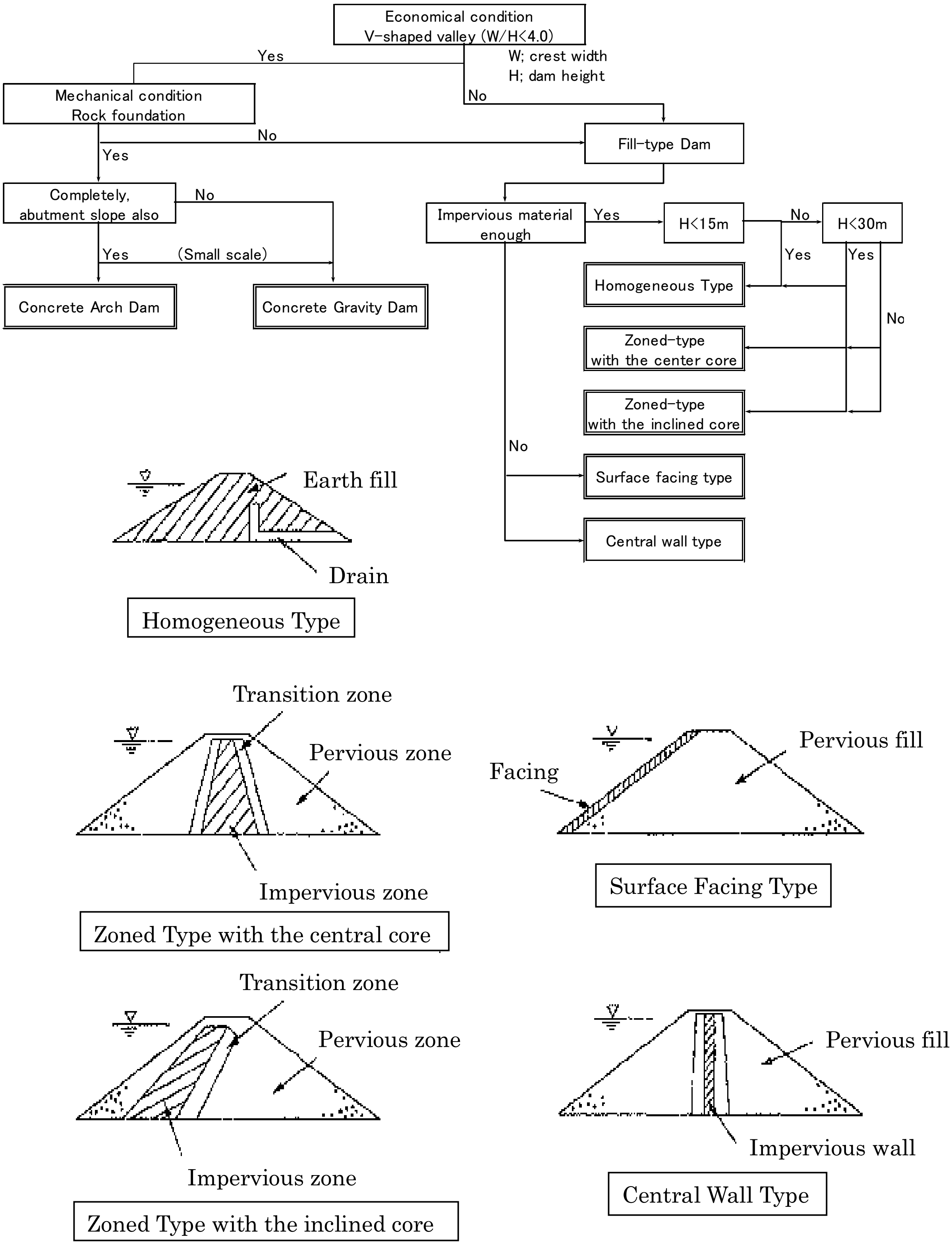


Figure 5.2.1.1 Selection chart of dam type

5.2.2 Design Flood Discharge

In case of the catchment area being vast, it does not rain uniformly and widely but does partially by partially; and the river flow rate changes slowly and in a long span of time so that it is essential to estimate the river flow rate based on the observation results. There is a set of observation data of Sue River, which is old to be obtained from 1953 to 1963, with the rainfall data observed at several stations in the catchment area. Based these data, the runoff ratio is estimated as shown below.

Fig. 2.3.2.2 shows the relationship between the monthly total rainfall multiplied by the catchment area and the observed discharge of Sue River. It is natural to consider that the peak of the rainfall is reflected to the peak of the discharge; then the runoff ratio is estimated to be 0.12 through the calculation of 879/7284.

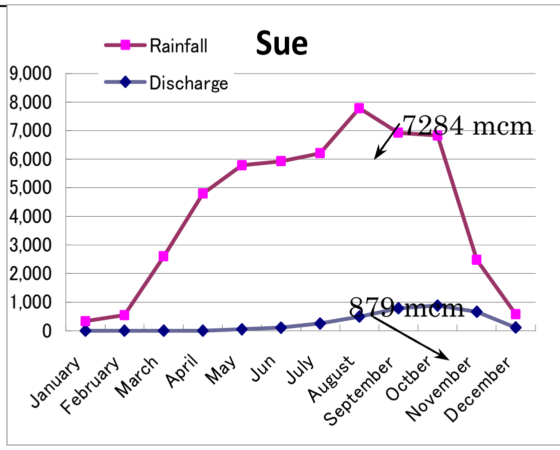


Figure 5.2.1.2 Rainfall and runoff of Sue River

The nearest meteorological weather station to the dam site is Juba station which has the observation data of rainfall ranging from 1901 to 2012. The average monthly rainfall observed there is shown as Fig. 2.3.2.3. The monthly rainfall of May is the maximum so that the discharge at the dam site shall be studied to the observation data of May.

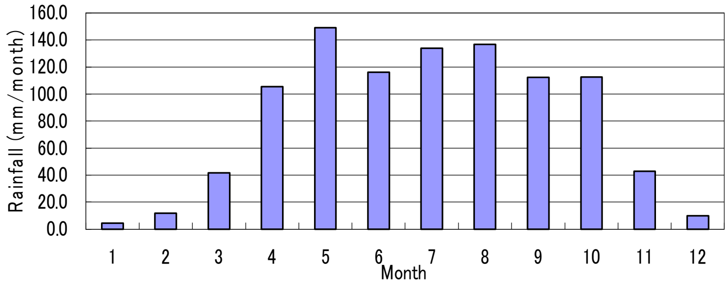


Figure 5.2.1.3 Average monthly rainfall in Juba

- Statistic monthly rainfall with the exceedance probability of 200 years

  The number of sample runs up a large amount of 112; and the reliability of probability value is considered to be high so that the value with exceedance probability of 200 years shall be applied.

  Design monthly rainfall; 396.5 mm

Table 5.2.1.4 Probability estimation of the monthly rainfall in Juba

Occurrence year	Probability calculation process				Exceedance probability value
T year	ξ	1/a・ξ	AverageY +1/a・ξ	x+b	x
1	0.0000	0.0000	2.1801	151.4	140.221
2	0.0000	0.0000	2.1801	151.4	140.221
5	0.5951	0.1406	2.3207	209.3	198.080
10	0.9062	0.2140	2.3942	247.8	236.658
20	1.1630	0.2747	2.4548	285.0	273.805
50	1.4520	0.3430	2.5231	333.5	322.310
100	1.6450	0.3885	2.5687	370.4	359.219
200	1.8215	0.4302	2.6104	407.7	396.536
500	2.0350	0.4807	2.6608	457.9	446.736
1000	2.1850	0.5161	2.6962	496.8	485.659

- Monthly runoff yield in the catchment area of the dam site

  This runoff yield shall be assumed as follows.

  Monthly runoff yield=396.5 mm×1063.1 km2=421.5×10^6 m^3

- Monthly discharge

  The monthly discharge with 200 year exceedance probability shall be assumed to be as follows.

  Monthly discharge=monthly runoff yield×runoff ratio

  $=421.5\times10^6\ m^3\times0.12$

  $=\underline{50.6\times10^6\ m^3}$

- Peak discharge

  It is assumed that this monthly discharge shall be mainly composed of the flood discharge which shall occur and continue for five days and of which total amount shall correspond to 80 percent of the monthly discharge.

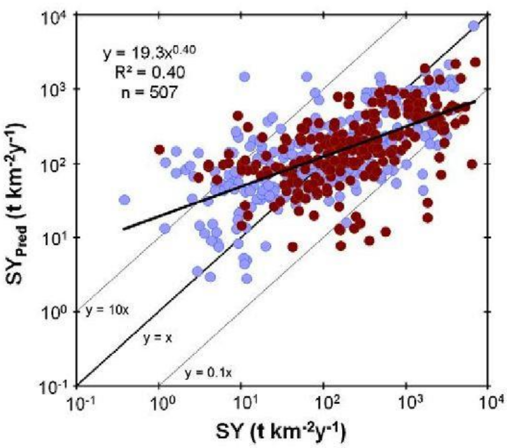
  Peak discharge$=50.6\times10^6$ m3$\times0.80/(5\times86400)$

  $=93.7\ m^3/sec$

  $\underline{100\ m^3/sec}$ shall be applied as the design discharge quantity.

5.2.3 Design Sediment Quantity

There are many study achievements regarding the estimation method of sediment nation by nation; but there are not so many in terms of sediment in African rivers and it would be able to say that the literature ñSediment yield in Africaò by Matthias Vanmaercke, Jean Poesen, Jente Broeckx and Jan Nyssen (Earth-Science Reviews 136 (2014)) is the most recent, comprehensive, thorough and continent-wide study achievement that compiled and analyzed the observation data until then. Here, the design sediment yield shall be estimated by the following equation, which they showed in the literature together with the graph at right where the observed data and the calculated values by the equation are correlated highly.



$$SY_{Pred}=1.49\times e^{1.24PGA}\times MLR^{0.66}\times e^{-0.05TreeCover}\times Ro^{0.24}$$

Here, SY_{Pred} ; sediment yield in ton $km^{-2}\ y^{-1}$

PGA ; the average expected Peak Ground Acceleation with an exceedance probability of 10% in 50 years

MLR ; the average height difference within a radius of 5 km

TreeCover ; the estimated percentage of the catchment covered by trees

Ro ; the estimated average annual runoff depth in mm y^{-1}

Now, PGA=1.6 m/sec^2 (the project area corresponds to the yellow area in Fig. 5.2.3.1)

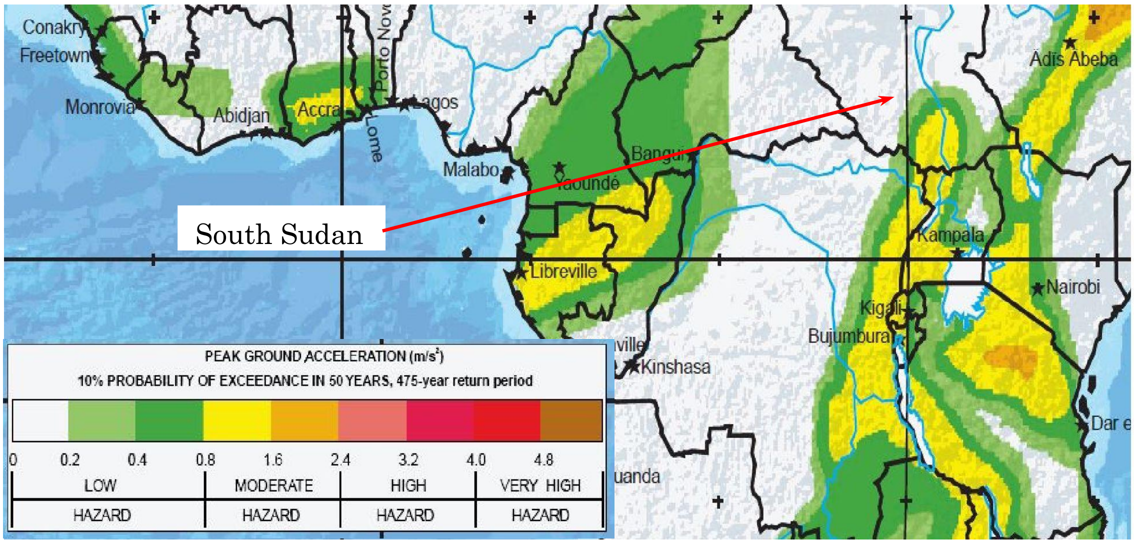


Figure 5.2.3.1 Expected Peak Ground Acceleration(Seismic Hazard Map of the World, by Andrew Alden)

MLR=77.8 m ; refer to Table 5.2.1.4

TreeCover=40 % ; refer to Fig. 5.2.3.1

Ro=58.9 mm y^{-1} ; from the assessment of water resource development potential in South Sudan in IDPM

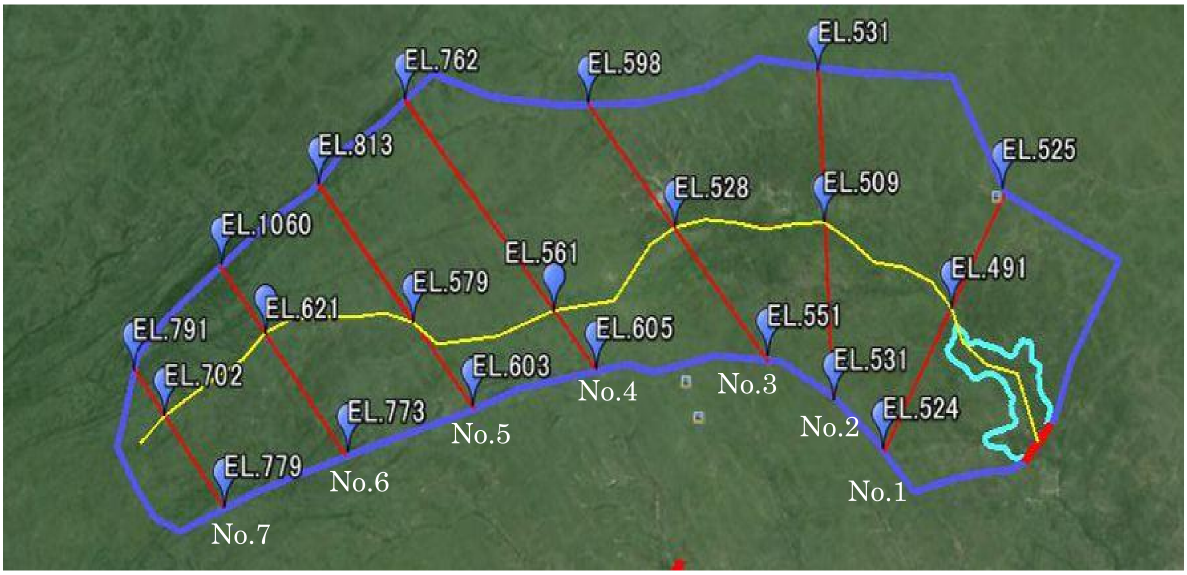
Then, $SY_{Pred}=1.49\times e^{1.24PGA}\times MLR^{0.66}\times e^{-0.05TreeCover}\times Ro^{0.24}$

$=1.49\times e^{1.24\cdot 1.6}\times 77.8^{0.66}\times e^{-0.05\cdot 40}\times 58.9^{0.24}$

$=71.4$ ton/km^2/year$=71.4\times 1/1.2$ m^3/km^2/year$=59.5$ m^3/km^2/year

Table 5.2.3.1 Degree of height difference in the watershed

Section		River bed (EL. m)	Ridge peak (EL. m)	Bed～Peak Diff. (m)	Bed～Peak Dis. (km)	Diff. / 5km (m)
Left Bank Side	No.1	491	525	34	7.66	22.2
	No.2	509	531	22	9.01	12.2
	No.3	528	598	70	9.23	37.9
	No.4	561	762	201	15.98	62.9
	No.5	579	813	234	10.38	112.7
	No.6	621	1060	439	4.90	448.0
	No.7	702	791	89	3.21	138.6
Right Bank Side	No.1	491	524	33	9.14	18.1
	No.2	509	531	22	9.67	11.4
	No.3	528	551	23	9.70	11.9
	No.4	561	605	44	4.34	50.7
	No.5	579	603	24	6.44	18.6
	No.6	621	773	152	8.75	86.9
	No.7	702	779	77	6.69	57.5
Mean						77.8



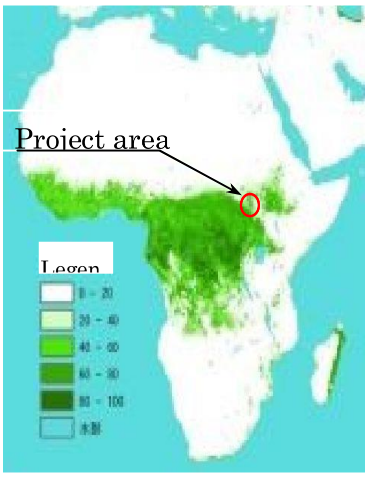


Figure 5.2.3.2 Tree Cover Ratio (Geospatial Information Authority of Japan, 2008)

The design sediment quantity is calculated as follows based on this sediment yield.

Design sedimentn quantity: $Q_{sd}=SY_{pred}\cdot A\cdot Y$

Here, Q_{sd} ; design sediment quantity

A ; catchment area : A=1,063.1 km^2

Y ; durable years of the reservoir : 100 years (commonly used)

Then, $Q_{sd}=SY_{pred}\cdot A\cdot Y$

$=59.5\ m^3/km^2/year\times 1{,}063.1\ km^2\times 100\ year$

$=6{,}325{,}000\ m^3$

$=6{,}500{,}000\ m^3$

5.2.4 Dimensions of Dam

(1) Full water level / flood water level

The full water level of the reservoir is EL.487.0 according to Fig. 2.3.1.5.

The flood water level is the water level which is raised and suspended together with the design flood flowing down through the spillway

The overflow capacity of the weir is given by the following formula.

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

Here, Q ; overflow quantity (m3/sec)

C ; overflow coefficient of the weir

L ; weir length (m)

H ; overflow depth (m)

Considering the relationship between the weir height and the improvement of the overflow coefficient, the overflow depth is designed to be as same as the weir height, i.e. P/Hd=1.0 (P: weir height, Hd: overflow depth). The front wall of the weir is designed to be perpendicular for the convenience sake of construction. Then the overflow coefficient is 2.14 corresponding to P/Hd=1.0.

The relationship between the overflow depth and the weir length under the design flood discharge, Q=100 m^3/sec, can be obtained as follows by applying the formula above.

Q : Design flood discharge (m3/sec)	C: overflow coefficient	L : weir length (m)	H : overflow depth (m)
100	2.14	50	0.96
100	2.14	75	0.73
100	2.14	100	0.60
100	2.14	125	0.52
100	2.14	132.2	0.50
100	2.14	140.5	0.48
100	2.14	150	0.46

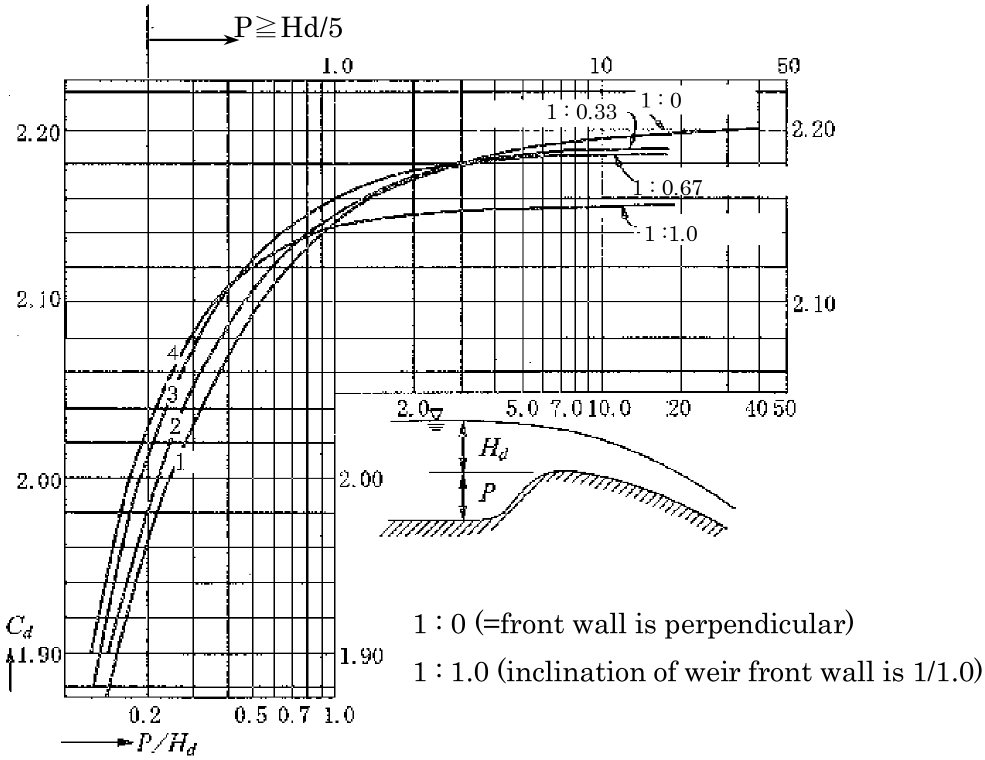


Figure 5.2.4.1 Overflow conditions and the overflow coefficient

Based on the calculation above, the dimension of the weir is designed to be 133 m of the weir length, 0.50 m of the weir height and 0.50 m of the overflow depth. Then the flood water level (H.W.L.) becomes EL.487.50 m (=EL.487.0 m+0.50 m).

(2) Freeboard of the dam crest

The design standard by the Ministry of Agriculture, Forestry and Fishery of Japan recommends applying the homogeneous type to the fill-type dam less than 15 m in height based on the following views.

・In case of small dams, the homogeneous type is preferable for the convenience sake of construction.

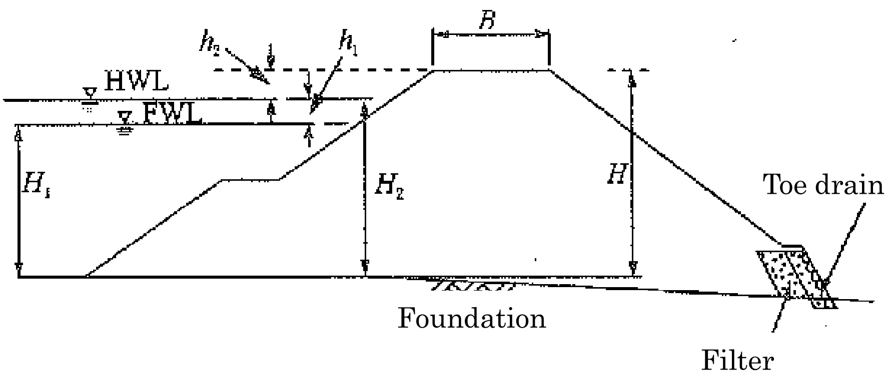
・The wide bottom of the impervious embankment given by the homogeneous dam body is effective to reduce the seepage quantity through the foundation.

・The homogeneous type can adapt to wide foundation conditions and has higher adjustability to the deformation/displacement of the foundation.

・Earth materials are the easiest and the cheapest to be obtained.

The freeboard given to the crest portion of the dam is defined as the height to be prepared for the wave over-topping and unanticipated settlements of the dam crest. The Ministry of Agriculture, Forestry and Fishery of Japan sets the design standard of small dams apart from the fill-type dam design standard. The freeboard of the dam crest shall be studied based on this design standard for small dams, where a smaller allowance is allowed to apply in deciding the freeboard instead of the spillway being not allowed to be provided with a gate for flood discharge.

In the homogeneous damôs case, specifications of the dam body are defined as follows.



H; dam height

B; dam crest width

HWL; high water level (the maximum water level at the time of the design flood overflowing the spillway weir)

FWL; full water level (the maximum water level at the time of daily storage behavior)

H_1; water depth at the time of FWL

H_2; water depth at the time of HWL

h_1; overflow depth at the time of the design flood overflowing the spillway weir

h_2; freeboard (margin/additional height) of the dam crest to HWL

h_2 is given as follows.

In case of R≦1.0m・・・・・・h_2=0.05 H_2+1.0

or h_2=1.0 (only to the case of H being lower than 5m, by judging the damage level at the time of failure)

In case of R＞1.0m・・・・・・h_2=0.05 H_2+R

Here, R is the wave height that includes the height of wave swash on the slope, and estimated by the following diagram.

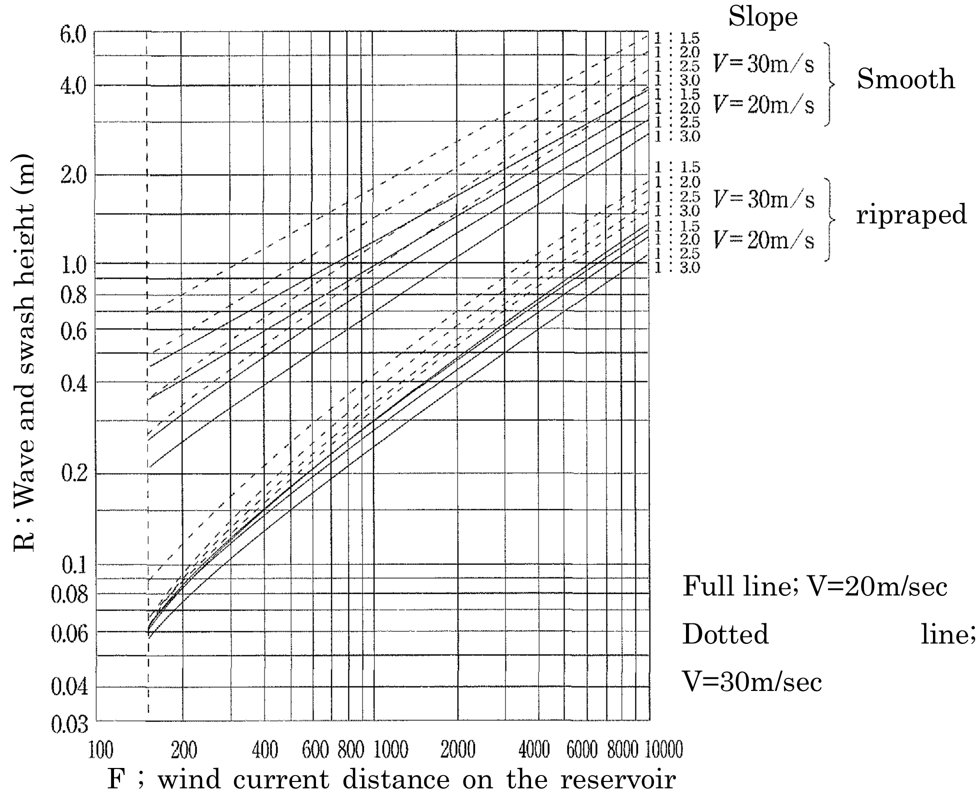
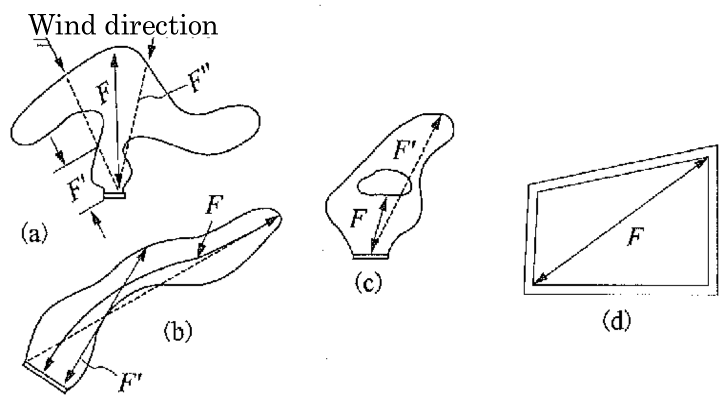
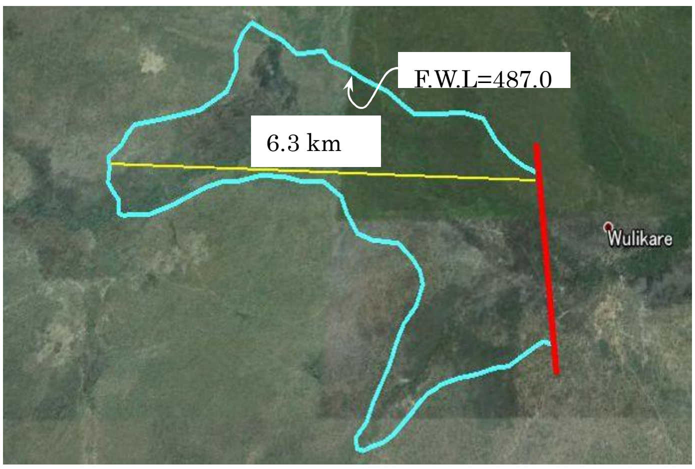


Figure 5.2.4.2 Wave and swash height ~ Wind current distance



F is adopted, not F' nor F"

Now, F, wind current distance, is 6.3 km as shown below.



But the average wind velocity in Juba is very small to be less than 1 m/sec in from March to May, the period with strongest wind, as shown below so that R shall be treated to be less than 1.0 m.

Then, the freeboard is estimated to be 1.45 m from the calculation below.

$$h_2=0.05 \cdot H_2+1.0=0.05\times(EL.487.0-EL.479.0+0.50)+1.0=1.425$$

This is the mean monthly wind speed (meters per second)

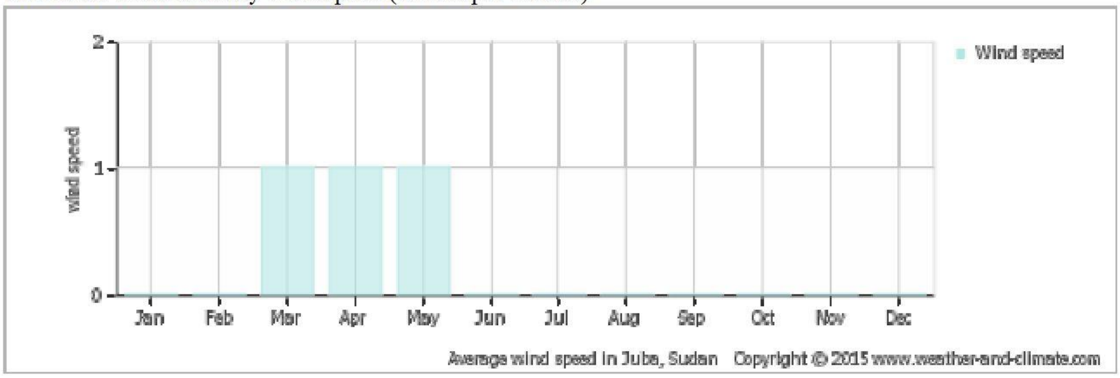


Figure 5.2.4.3 Wind velocity in Juba

(3) Dam crest

1) Crest width

The design standard for small dams, Japan, shows the formula for deciding the crest width together with the description that the width shall be decided under consideration of O & M works and vehicular traffic.

Crest width B ;B=0.2・H+2.0 (m) (H ; dam height)

In this damô s case, the crest length is more than 2 km so that the vehicle is essential for the O & M works. It is adequate to apply 5 m as the crest width.

2) Protection work to the dam crest

The protection work shall be provided to the embankment crest to avoid the damage by erosive actions of intense rainfall, the injury by vehiclesô wheels, the deterioration of quality by repetition of wetting and drying, and injurious actions of insects.

The compacted sand-and-gravel shall be suitable from the view point of resisting ability against erosive actions of rainfall and so on, especially injurious actions of insects. Thickness 30 cm shall be applied.

Figure 5.2.4.4 Nests of ants on the dam crest in Rwanda

3) Crest elevation and dam height

The crest elevation is EL.489.23 through the following calculation.

Crest elevation=H.W.L + additional height + thickness of crest protection

=EL.487.50 + 1.45 +0.30

=EL.489.25

The dam height is m through the following calculation.

Dam height =Crest elevation－(Foundation elevation－Excavation thickness)

=EL.489.25－(EL.479.0－1.0)

=11.25 m

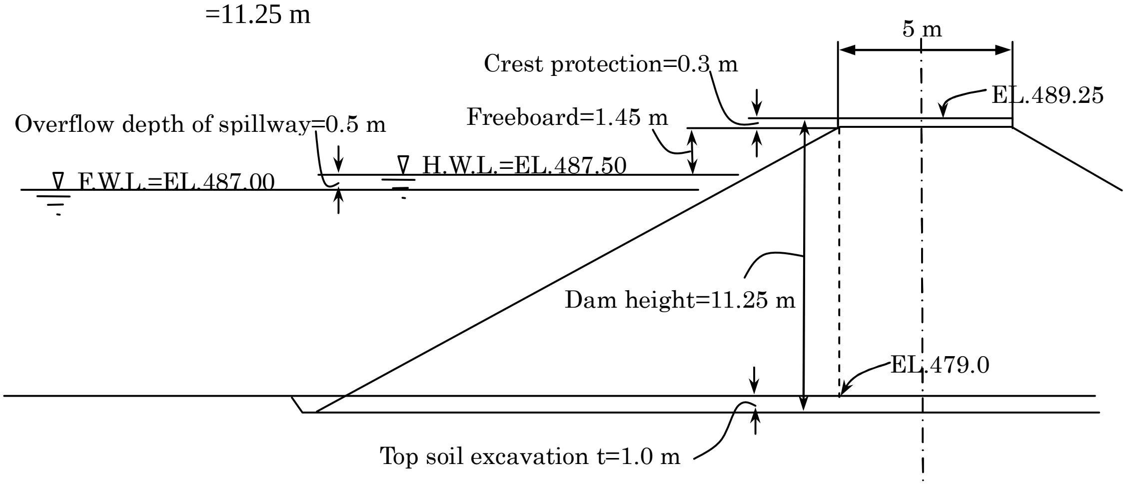


Figure 5.3.4.5 Planned elevations and the dam height

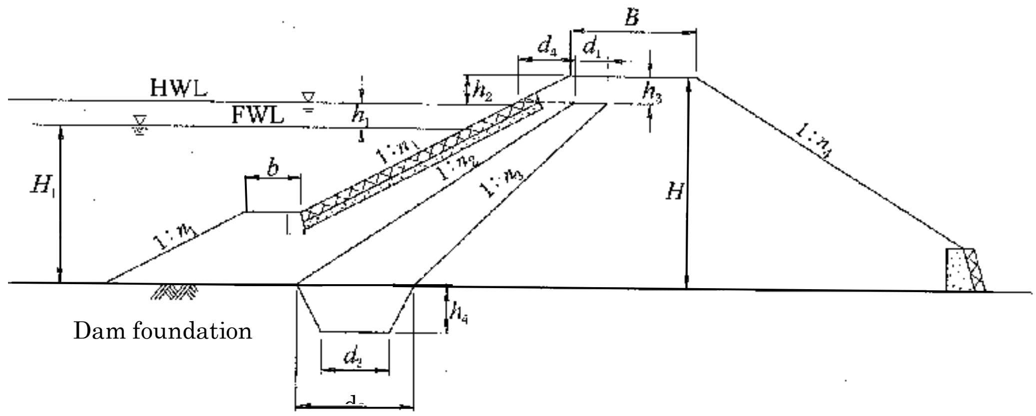
4) Crest length

The shape of the longitudinal section is obtained by Google Earth as follows. The crest length is 2.33 km.

Figure 5.2.4.6 Longitudinal cross-section of the dam

The design standard for small dams, Japan, shows the dimensions of the homogeneous type dam with an inclined core for reference.

Considering the dam height and referencing dimensions, the inclination of 1 : 3.0 to the upstream slope and 1 : 2.5 to the downstream slope shall be applied. The function of the step is to enhance the safety of the embankment against the sliding circle that passes through soft soil layers in the foundation. In this dam's case, the foundation shall be composed not of rock or weathered rock beddings but soil layers that might include weak portions mechanically, so that a 2 m wide step shall be provided to the upstream slope.



Dam height H (m)	Water depth H_1(m)	Overflow depth h_1(m)	Additional height h_2(m)	Crest width B (m)	Front slope		Inclined core zone						Back slope inclination n_4
					Inclination n_1	Step width b (m)	to the crest h_3(m)	Crest width d_1(m)	to the slope d_4(m)	Core trench upper d_2(m)	Core trench lower d_3(m)	Core trench Depth h_4(m)	
～5	～3.3	0.3～0.5	1.0～1.2	2.0～3.0	1.5～1.8	～1.5	0.3～0.5	1.5～1.8	1.5	Calculation through $n_2=n_1-0.1$ $n_3=n_2-0.2$	$d_3=1/2d_2$	1.1～1.3	1.5～1.8
5～10	3.3～7.8	0.5～0.8	1.2～1.4	3.0～4.0	1.8～2.1	1.5	>=0.5	1.8～2.1	>=1.5			1.3～2.1	1.8～2.1
10～15	7.8～12.2	0.8～1.2	1.4～1.6	4.0～6.0	2.1～3.0	2.0	>=0.5	2.1～3.5	>=1.5			2.1～3.2	2.1～2.5
Note		depend on the structure	calculated by the formula			minimum 1.0 m						depend on the geological condition	

Figure 5.2.4.7 Referencing dimensions of small dams

5.2.5 Design of Embankment

(1) Seepage processing works

The dams are not the structures that shut out the seepage flow from the dam and from the foundation but the structures that allow the existence of seepage flow within the limits of the reservoir functioning as the reservoir and the dam body being not damaged due to the existence of seepage flow. A large part of leakage does not occur through the dam body but occurs through the foundation, so that the subject of the reservoir functioning as the reservoir is the matter of foundation treatment; and this matter shall be treated in the next section.

From the view point of the relationship between the seepage flow through the embankment and the safety of the embankment, such points as shown below are important.

① To prevent the seepage flow from appearing on the downstream slope, of which saturated condition caused by the seepage decreases the safety factor against sliding

② To protect the upper surface of the foundation not to have the piping phenomenon, which tends to occur on/around the spring-out portion of the seepage water

③ To provide the toe of the downstream slope with the leakage measurement system in order to grasp the circumstances around leakage

To meet the item ①, the vertical drain shall be provided in the downstream side embankment.

To meet the item ②, the foundation surface of the slope toe shall be covered by the drain which is composed of sand and gravel mixture.

To meet the item ③, the leakage measurement system composed of the drain, the impervious dike, drainage pipes and conveyance pipes.

These are illustrated as follows.

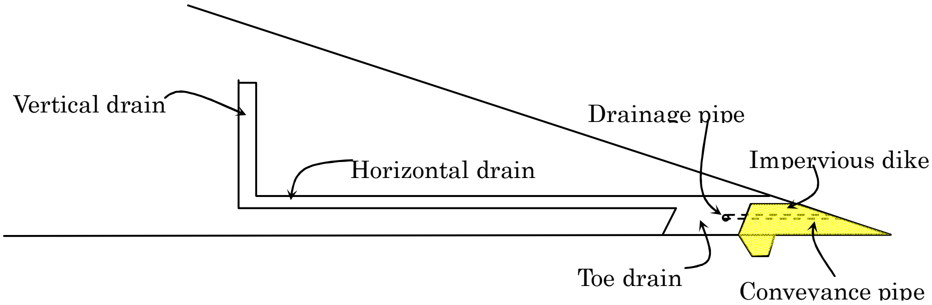


Figure 5.2.5.1 Illustration of seepage processing works

(2) Seepage line in the dam body

The seepage line in the homogeneous fill type dam is estimated by the method suggested by A. Casagrande. In embankments banked layer by layer through spreading and compaction process, there is a differential in permeability between the horizontal direction and the vertical direction. It is said empirically that the ratio of the horizontal permeability coefficient (k_h) to the vertical permeability coefficient (k_v) is 5.0 (k_h / k_v =5) in case of the compaction being done by tamping rollers. Before applying the method of A. Casagrande, the cross-section of dam body must be transformed from such anisotropic conditions into isotropic conditions; this transformation can be done through multiplying the horizontal length/distance of dam body by 1/√5. A. Casagrande showed the way of setting the seepage line by a fundamental parabola and the way to modify this parabola in terms of the flowing-in portion and the flowing-out portion. The fundamental parabola is given by the following formula based on the illustration below.

$$y = \sqrt{2y_0 x + y_0^2} \qquad y_0 = \sqrt{h^2 + d^2} - d$$

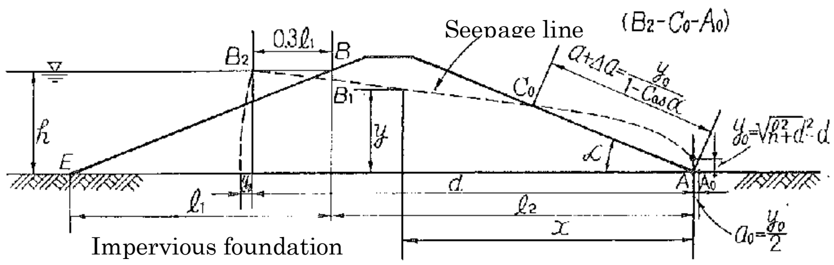
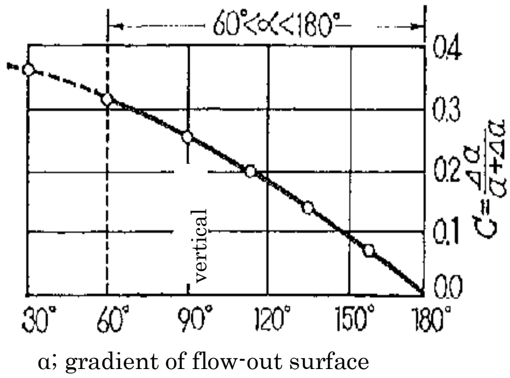


Figure 5.2.5.2 Seepage line setting by the fundamental parabola

Modifications of the flowing-in portion and the flowing-out portion are done as follows by the equation and illustrations.

$$a+\Delta a=\frac{y_0}{1-\cos\alpha}$$



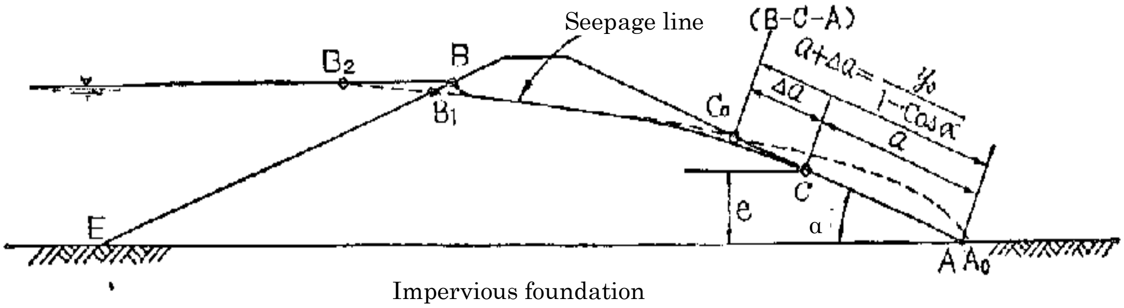
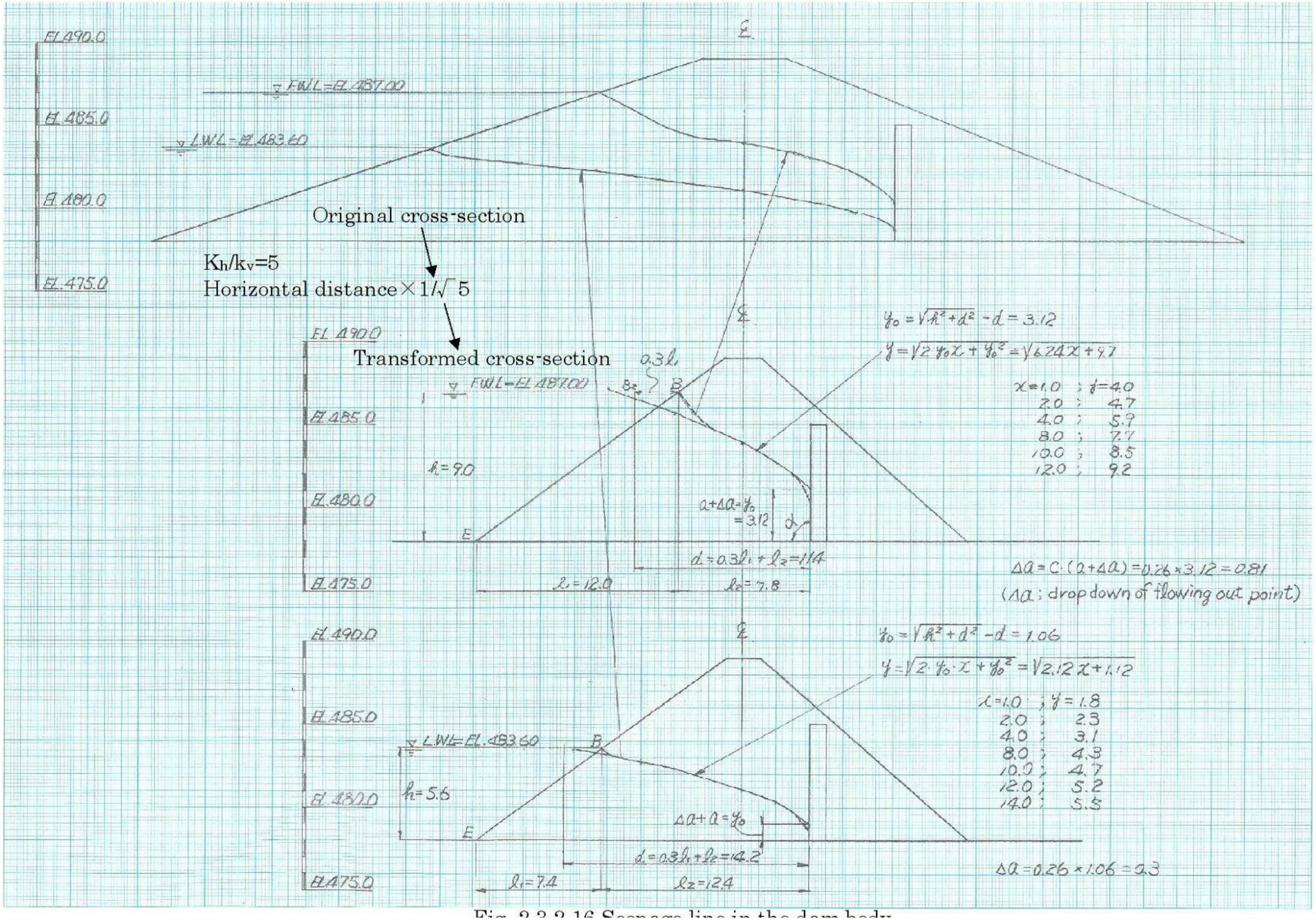


Fig. 5.2.5.3 Modification of the parabola seepage line

Fig 5.2.5.4 shows the seepage lines obtained by A. Casagrande Method to the F.W.L. (EL.487.0) and L.W.L.(EL.483.6).



Fiure. 5.2.5.4 Seepage line in the dam body

(3) Protection works

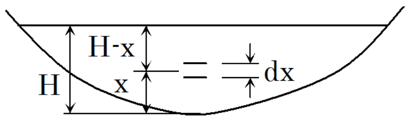
The upstream slope and the downstream slope of the embankment shall need the protection works to avoid deteriorated conditions by rainfall erosive actions and the repetition of wetting and drying even though the influence of wave actions to the upstream slope is low.

To the upstream slope, the layer of 40 cm thickness composed of gravels, sand-and-gravels, lateritic gravels, weathered rocks or compacted soil-cement shall be provided as the protection. Identification of the materials shall be done through the investigation in future.

To the downstream slope, the vegetation cover shall be provided.

(4) Countermeasure to settlement

The embankment materials consist of cohesive soils so that the consolidation shall not finish completely within the period of construction but some portion is postponed after the end of the construction term. The additional embankment is provided to avoid the shortage of freeboard of the dam crest due to this postponed settlement. The adequate height of additional embankment can be evaluated based on the relation curve between the load and the settlement ratio obtained from the consolidation test result as follows. Here, the consolidation test result in Rwanda irrigation project conducted to the samples that shall have the similar gradational conditions to the materials in this site shall be referred.



When assuming an element with the thickness of ñdxò at the position ñxò m above the embankment bottom, here H: dam height and ɔ: unit weight of embankment, the load ñLò acting on the element ñdxò is expressed as ñL=(H－x)・ɔò.

Now a settlement ratio ñyò is introduced; ñyò is expressed as the function of load ñLò. Then, the settlement of ñdxò is expressed as ñy・dxò and the settlement at the dam crest can be calculated by integrating ñy・dxò from the bottom (x=0) to the dam crest (x=H). When expressing the relation between ñLò and ñyò by a linear approximate equation ñ$y=aL$ò(refer to Fig.2.3.2.17), the settlement of dam crest is shown as follows.

$$\int_0^H y\,dx = \int_0^H aL\,dx = a\int_0^H (\gamma H - \gamma x)\,dx = a\left[\gamma Hx - \gamma\frac{x^2}{2}\right]_0^H = \frac{a\gamma H^2}{2}$$

Here, y=aL, a=6.0$^{\%}$/(220^{kPs}) , ɔ=1.97tf/m^3, H=11.25m

Then, the settlement of dam crest=1/2*(0.06/22.0tf/m^2)*(1.97tf/m^3)*(11.25m)^2

=0.34m

The consolidation settlement also occurs during the construction term. In case of the construction work being carried out for 3 years or so intermittently due to the break period of rainy season, it is said empirically that the percentage of the postponed settlement to the total settlement is about 35% or so. From this concept, the needed additional embankment height is very small; but the additional embankment height of 30 cm shall be applied considering the accuracy of the embankment works.

The additional height 30cm is provided to the central section longitudinally and is gradually decreased toward the abutment.

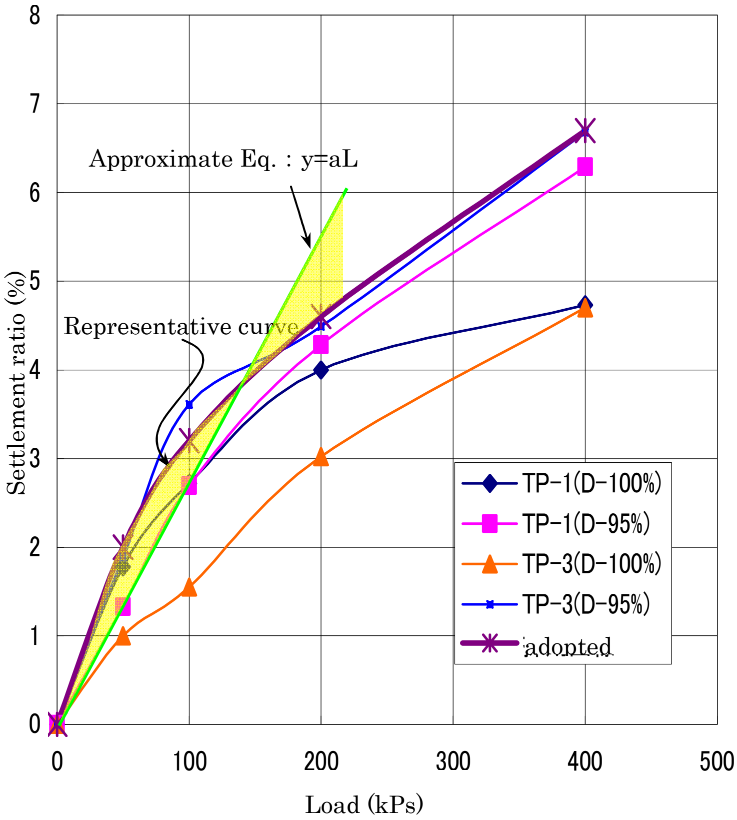


Figure 5.2.5.5 Load ～ settlement ratio curve

(5) Typical cross-section of the dam body

The typical cross-section of the dam body is shown as Fig. 2.3.2.18.

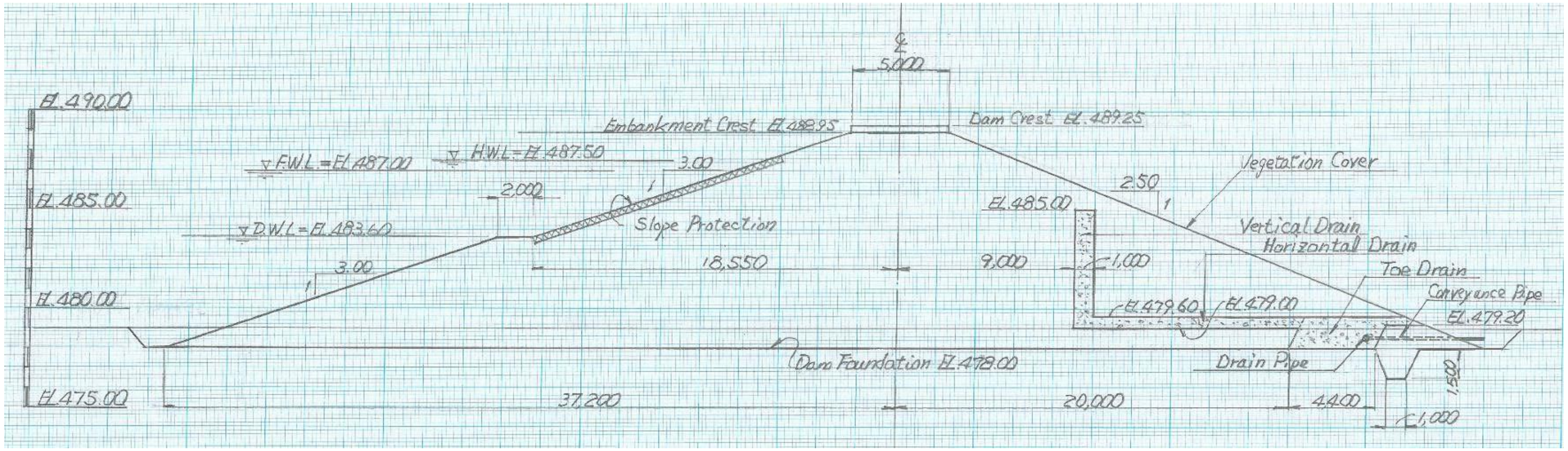


Figure 5.2.5.6 Typical cross-section of Jebel Lado Dam

5.2.6 Foundation Treatment

(a) Treatment method

1) General

Usually a large part of leakage from reservoirs, say 80 % to 90 %, occurs from the dam foundation so that the foundation treatment to decrease the leakage within an allowance limit is very important.

The outline or generalities in terms of treatment methods are as shown below.

Table 5.2.6.1 Introduction of foundation treatment methods

Item	Grouting method	Earth blanket method	Underground continuous wall method
Outline of the method	Pressurized cement milk is injected into the cracks in rock formations through borehole walls drilled into the foundation. The improvement by this method is understood in the formula ñQ*=k・i・Aò that the decrease of k through injection brings the decrease of Q.	The pervious layer/layers of the dam foundation is/are covered by the impervious earth blanket. The improvement by this method is understood in the formula ñQ*=k・i・Aòthat the decrease of i (=H/L) through the increase of L brought by the blanket covering brings the decrease of Q.	The pervious layer in the foundation is shut out by a underground continuous wall such as sheet piles or cast-in-place soil-cement columns.
Applicable range	Rock foundation ~Weathered rock foundation	Highly weathered rock foundation ~River bed san-and-gravel	Pervious foundation layer bedded by impervious formations
Degree of effect	Rock foundation; k ≦ 2×10^{-5}cm/sec, Weathered rock 〃; k ≦5×10^{-5}cm/sec	The longer the blanket length extends, the lower the effect of extension becomes.	Applicable or not depending on the feasible depth of the work.
Problems experienced	Highly weathered portions can not be improved; to cover this, additional grouting works are needed next by next and eventually the construction cost increases a lot.	A large differential of water pressure rises between the upper surface of the blanket and the lower surface of the blanket. In case of a large void existing in the foundation layer below the blanket, water penetrates and flushes out the blanket into the void.	In case the work is imperfect, a concentrated seepage flow, i.e. extreme hydraulic gradient, rises at this portion; and a crisis situation occurs.
Construction examples	This method is the most popular treatment method so that construction examples are uncountable.	The largest dam provided with the earth blanket is Tarbela Dam in Pakistan, of which dam height is 143.26 m and the blanket length is 2 km.	There are scarce construction examples in Japan. It is said that there are not a few examples in Myanmar.

2) Jebel Ladoôs case

It is assumed that the river-bed area is thickly covered by sediment layers that would be able to function mechanically as the embankment foundation so that the embankment shall be constructed on the foundation composed of sediment layers. Sediment layers do not have cracks so that the grouting method does not work as the foundation treatment but works the earth blanket method only. The earth blanket method shall be applied to this dam.

(b) Design of the earth blanket

1) Basic equations

$$q_f = \frac{k \cdot h \cdot d}{x_r + x_d} \qquad x_r = \frac{e^{2ax} - 1}{a\,(e^{2ax} + 1)} \qquad a = \sqrt{\frac{k_1}{t \cdot k \cdot d}}$$

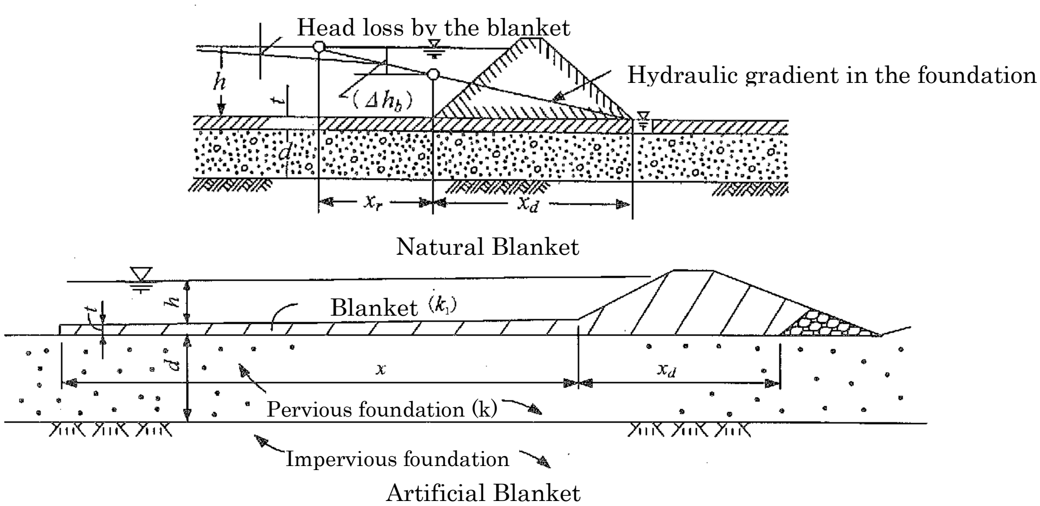


Fig. 5.2.6.1 Dimensions for designing the earth blanket

Here,

q_f; seepage quantity through the foundation layer (m^3/sec)

h; differential between the reservoir water level and the downstream water level (m)

x_r; effective seepage length (m)

x_d; bottom length of the dam body (m)

x; required length of the blanket (m)

k; permeability coefficient of the foundation layer (m/sec)

k_1; permeability coefficient of the blanket and the dam body (m/sec)

t; thickness of the blanket (m)

d; thickness of the foundation layer (m)

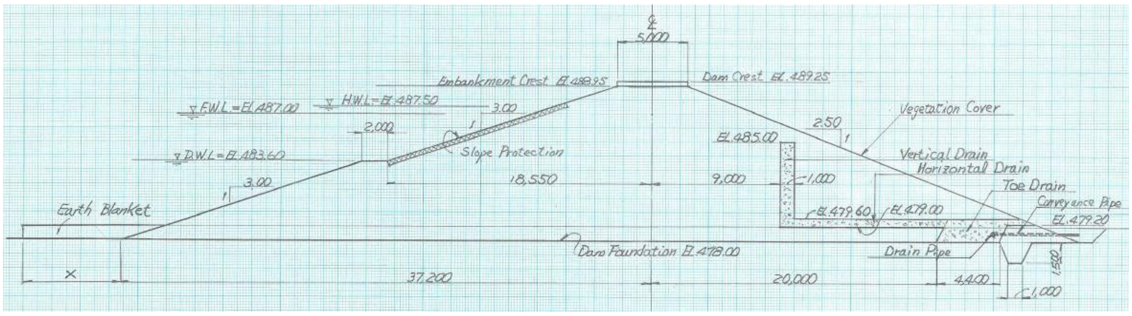
2) Permeability coefficient of the foundation

The permeability coefficient of the foundation shall be assumed and estimated to be 5×10^{-4} cm/sec judging from the river-bed near the command area being composed of dense silty sand layers.

The thickness of the foundation layer shall be given to be 10 m based on the experience of a large part of seepage passing the shallow portion and of the analysis model having the foundation thickness similar to the dam height.

3) Analysis model

The cross-section of the analysis model is shown below.



4) Trial calculation and adoption of blanket length ñxò

To the cases of the blanket thickness: 1.0m, 1.5m, 2.0m, the seepage quantity qf is calculated to the given value of x by the former basic equations. The calculated result is shown in Table 2.3.2.4 and summarized on Fig 2.3.2.20.

Here, the allowable leakage quantity per day is set to be 0.05% of the total reservoir capacity.

Then the allowable quantity = 36,500,000m^3×0.05/100＝18,250 m^3/day

The allowable quantity per meter longitudinally = 18,250 m^3/day×1/2,330m

= 7.8 m^3/day/m

=9.1×10^{-5} m^3/sec/m

The trial calculation result says the blanket length of zero (0) m can satisfy the allowable quantity and the embankment bottom has the enough seepage path length that makes the hydraulic gradient small and reduces the leakage quantity effectively. The earth blanket is not necessary to be provided.

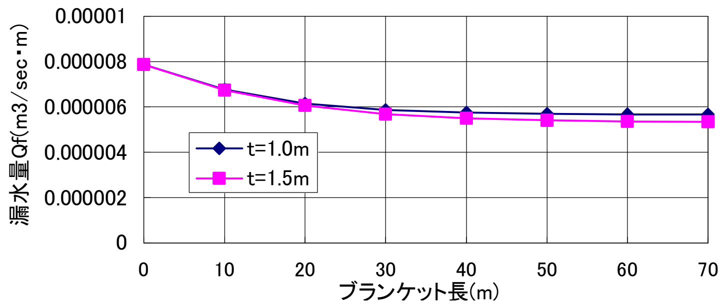


Figure 5.2.6.2 Blanket length and the leakage quantity

Table 5.2.6.1 Trial calculation of leakage quantity

t(m)	k	d(m)	k1	a	x(m)	e2ax	xr(m)	qf(m3/sec)
1	0.000005	10.0	1E-07	0.044721	0	1	0	7.86713E-06
1	0.000005	10.0	1E-07	0.044721	10	2.445934	9.382673	6.75852E-06
1	0.000005	10.0	1E-07	0.044721	20	5.982595	15.95599	6.15124E-06
1	0.000005	10.0	1E-07	0.044721	30	14.63303	19.49998	5.86702E-06
1	0.000005	10.0	1E-07	0.044721	40	35.79144	21.14514	5.74381E-06
1	0.000005	10.0	1E-07	0.044721	50	87.54351	21.8556	5.6922E-06
1	0.000005	10.0	1E-07	0.044721	60	214.1257	22.15279	5.67088E-06
1	0.000005	10.0	1E-07	0.044721	70	523.7374	22.27545	5.66213E-06
1.5	0.000005	10.0	1E-07	0.036515	0	1	0	7.86713E-06
1.5	0.000005	10.0	1E-07	0.036515	10	2.075696	9.578046	6.73874E-06
1.5	0.000005	10.0	1E-07	0.036515	20	4.308516	17.06832	6.05911E-06
1.5	0.000005	10.0	1E-07	0.036515	30	8.943171	21.8776	5.69061E-06
1.5	0.000005	10.0	1E-07	0.036515	40	18.56331	24.58638	5.50214E-06
1.5	0.000005	10.0	1E-07	0.036515	50	38.53179	26.0006	5.40861E-06
1.5	0.000005	10.0	1E-07	0.036515	60	79.98031	26.70976	5.3629E-06
1.5	0.000005	10.0	1E-07	0.036515	70	166.0148	27.05818	5.34073E-06

(c) Stability against the seepage failure

The stability against the seepage failure means the stability against the occurrence of piping phenomenon and the stability of the appearing hydraulic gradient against the critical hydraulic gradient.

The maximum hydraulic gradient appears around the seeping-out mouth and is about 3 to 5 times bigger than the average hydraulic gradient according to the finite element analysis empirically. Here in this case, the average hydraulic gradient ($i_{avr.}$) along the embankment bottom is 0.16 by the calculation of (EL.487.0 －EL.478.0)/(37.2 + 20.0).

The critical hydraulic gradient is shown by Terzhagi as follows.

$$i_c = \gamma'_s/\gamma_w = (1-n)\left(\frac{\rho_s}{\rho_w}-1\right) = \frac{\rho_s/\rho_w-1}{1+e}$$

Here γ'_s : Unit weight of soil in water (mmN/cm³)

γ_w、ρ_w : Unit weight of water (mmN/cm³) and density (g/cm³)

n : Porosity of soil

e : Void ratio of soil

ρ_s : Density of soil particle (g/cm³)

Provided ρ_s=2.65, ρ_w=1.0, e=0.70, then i_c=0.97

Safety factor (S.F.) of the appearing hydraulic gradient against the critical value;

S.F.=i_c/(5×$i_{ave.}$)=0.97/(5×0.16)=1.21

The safety factor is enough against the occurrence of piping phenomenon.

5.3 Appurtenant Facilities

5.3.1 Spillway

(1) Location

The spillway shall be constructed on the left side hill slope from the reasons below.

・To avoid the crossing of alignments between the spillway and the conveyance canal of which alignment shall be chosen on the right side hill slope because of the command area existing on the right bank side.

・To utilize the downstream branch valley, which exists on the left side hill, as the drainage waterway.

(2) Structural planning

1) In-let waterway

The elevation of the crest of the overflow weir is El.487.0 m which corresponds to the full water level of the reservoir.

The channel bed elevation of the in-let waterway is EL. 486.5 m, which is 0.5 m lower than the crest of the overflow weir, EL.487.0 m.

2) Overflow weir

As for the weir, the quarter circular weir shall be applied considering its simplicity in shape and the easiness in construction.

3) Conveyance channel

The transition section shall be inserted after the weir and the channel width shall be adjusted to 80 m of the main conveyance section. The channel bed shall be provided with 30 cm thick soil-cement covering as the protection work.

4) Chute channel

The chute channel connected to the conveyance channel shall be placed on the slope toward the valley. The channel bed shall be provided with 30 cm thick soil-cement covering as the protection work.

5) Sidewall of the channel

The sidewall of the channel shall be protected by the leaning retaining wall made of compacted soil-cement.

6) Stilling basin

The stilling basin shall be constructed to dissipate the flow energy form the chute channel in the valley bottom. At the end of the stilling basin, a weir shall be provided to make the stilling basin to be a pond into which the flow rushes and its energy shall be absorbed.

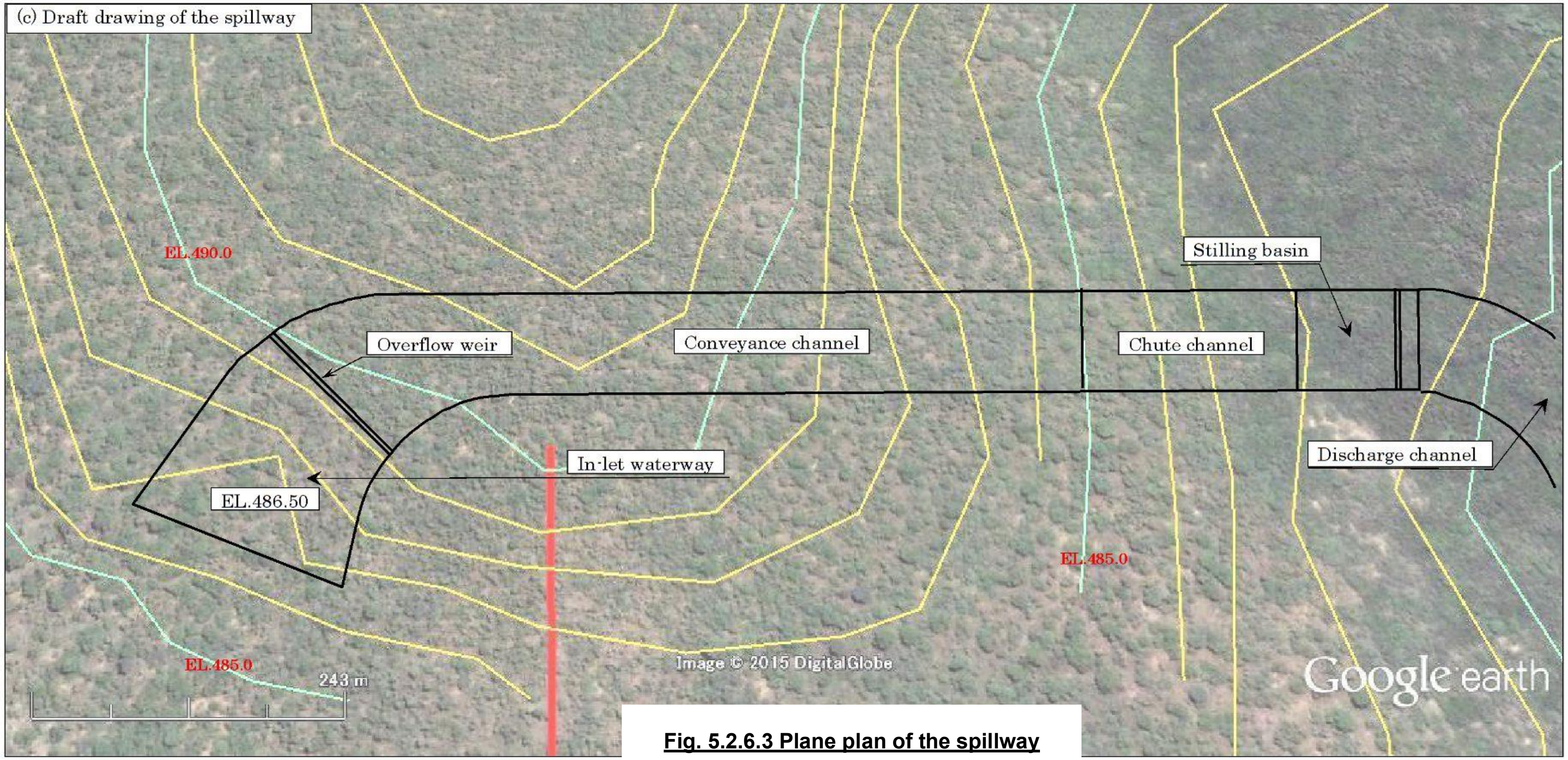


Fig. 5.2.6.3 Plane plan of the spillway

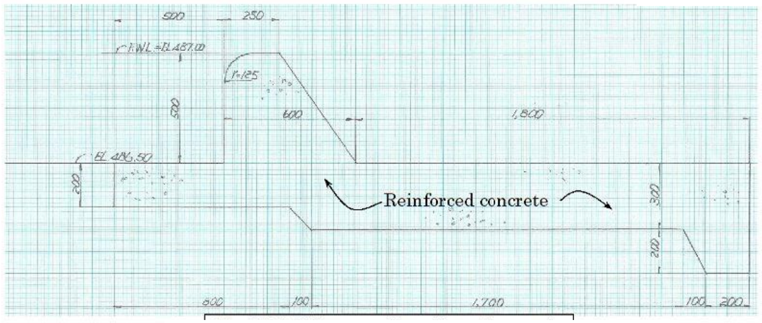


Fig. 5.2.6.4 Cross section of overflow weir

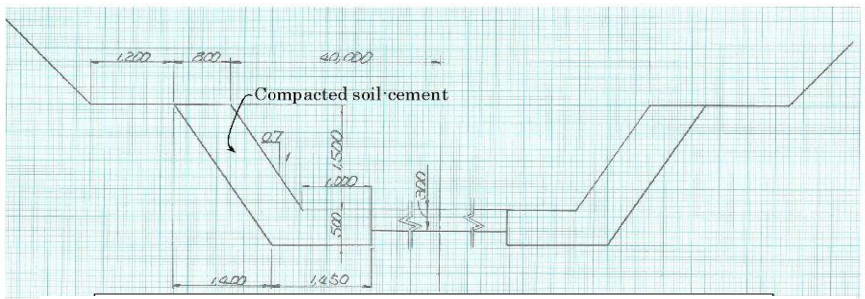


Fig. 5.2.6.5 Typical cross section of the conveyance channel ~ the stilling basin

5.3.2 Intake

(1) Location

The intake facilities shall be constructed on the right abutment slope considering the low water level of the reservoir being EL. 483.60 m and the command area lying on the right bank side.

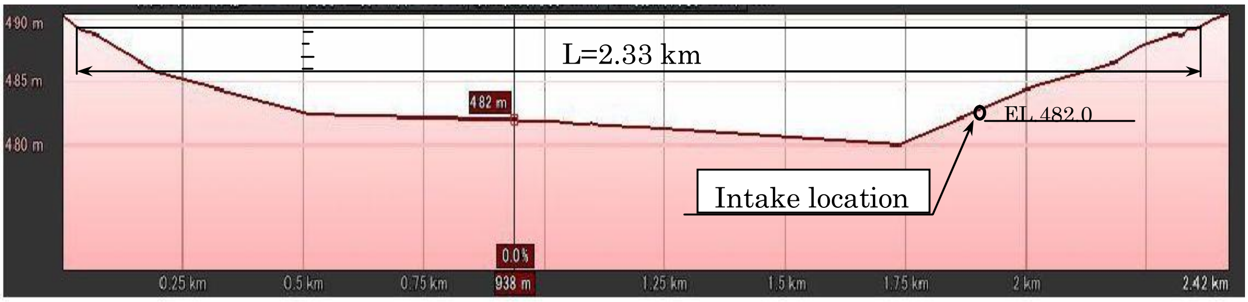


Figure 5.3.2.1 Intake location

(2) Structural planning

1) In-let mouth

The in-let mouth shall be installed on the wall of in-let tower constructed at the upstream slope toe of the embankment and provided with a slide gate.

2) Out-let mouth

The out-let mouth shall be installed at the downstream slope toe of the embankment, provided with a suitable gate for the discharge control, and connected to the conveyance canal toward the command area.

3) Waterway under the embankment

As the waterway under the embankment, the steel conduit line shall be applied which is rolled up by reinforced concrete for the sake of tight contact between the embankment materials and the wall surface of the conduit.

4) Diameter of the conduit

Hydraulic aspect shall be studied by Hazen-Williams formula. The design intake quantity is set to be 2.0 m^3/sec considering 1.96 m^3/sec of the design pump discharge.

Hazen-Williams formula ; $V=0.849 \cdot C \cdot R^{0.63} \cdot I^{0.54}$

Here, V : average velocity (m/sec)

C : velocity coefficient, C=100 corresponding to steel conduit

R : hydraulic radius (m)

I : hydraulic gradient

The formula above is transformed as the relational expression between Q and D shown bellow.

$$Q=0.279 \cdot C \cdot D^{2.63} \cdot I^{0.54}$$

Here, Q : flow rate (m^3/sec)

D : diameter of the conduit (m)

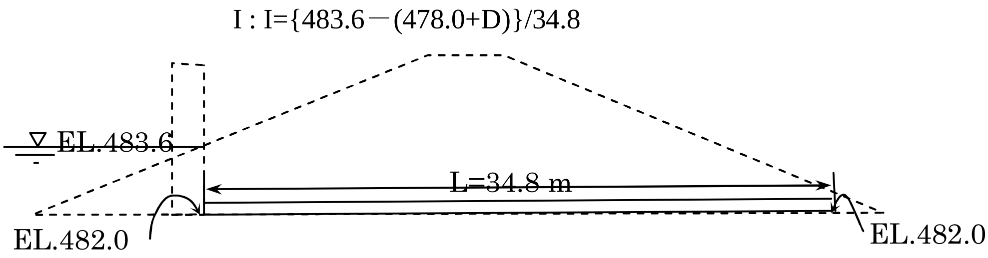


Table 5.3.2.1 Trial calculation of the conduit discharge capacity

D (m)	C	$D^{2.63}$	I	$I^{0.54}$	Q (m3/sec)
0.30	100.00	0.04	0.0374	0.17	0.20
0.40	100.00	0.09	0.0345	0.16	0.41
0.50	100.00	0.16	0.0316	0.15	0.70
0.60	100.00	0.26	0.0287	0.15	1.07
0.70	100.00	0.39	0.0259	0.14	1.52
0.80	100.00	0.56	0.0230	0.13	2.02
0.90	100.00	0.76	0.0201	0.12	2.57
1.00	100.00	1.00	0.0172	0.11	3.11
1.10	100.00	1.28	0.0144	0.10	3.63
1.20	100.00	1.62	0.0115	0.09	4.04

The diameter of the conduit is chosen to be 0.80 m to the design intake quantity 2.0 m^3/sec.

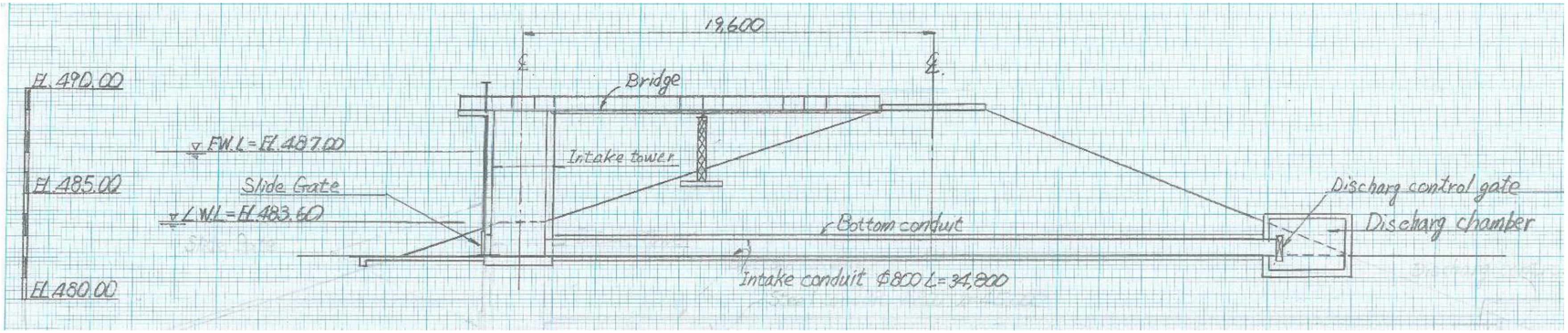


Figure 5.3.2.2 Longitudinal section of intake facilities

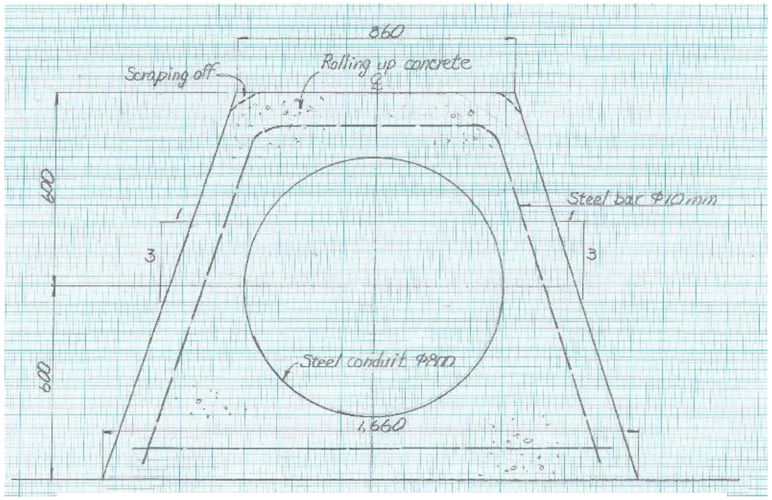


Figure 5.3.2.3 Cross-section of the bottom conduit

5.3.3 Conveyance Canal to Command Aera

(1) Canal route

The canal route is set up as shown below considering the topography and the uniform change of gradient of the canal alignment. The length of the alignment is measured to be 23.7 km.

Fig. 5.3.3.1 Conveyance canal route

(2) Structural planning

1) Formation of the canal

The lined formation by the concrete bed and wall shall be applied to avoid a large amount of conveyance loss due to the long conveyance distance.

2) Narrow and deep waterway

The canal cross-section shall be the inverted trapezoid shape with a narrow water surface to reduce the evaporation loss on the way of conveyance.

3) Work balance between excavation and embankment

From the view point of economy and preventing rain water from flowing into the canal, the structural cross-section shall be composed of a excavated portion and embankments; there, the embankment volume is equivalent to the excavated volume.

4) Cross-section of the waterway

The hydraulic aspect of the canal is studied by Manning formula.

$$Q = V \cdot A \qquad V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$$

Here, Q : flow rate (m^3/s)

V : average velocity (m/s)

R : hydraulic radius (m)＝A/P

〔A : cross-section area of flow (m^2), P : wetted perimeter length (m)〕

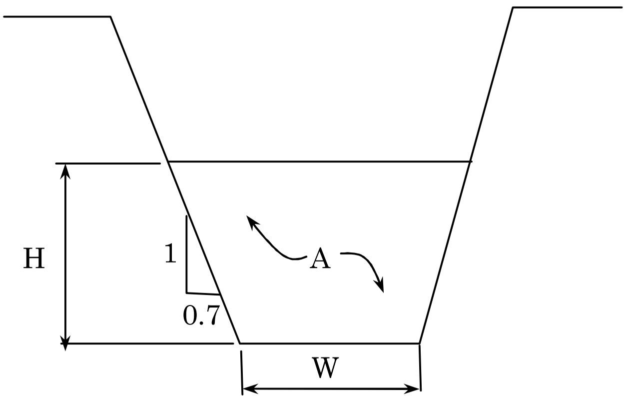
I : channel bed gradient I=1/5,000=0.0002

n : roughness coefficient n=0.016 (refer to Table 2.3.3.2)

Table 5.3.3.1 Manning's roughness coefficient

Material and the material condition	Roughness coefficient		
	min.	mean	max.
Concrete (in-situ casted flume, culvert, etc.)	0.012	0.015	0.016
Concrete (by sprqying.)	0.016	0.019	0.023
Concrete (ready-made flume, pipe, etc.)	0.012	0.014	0.016
Concrete (reinforced concrete pipe)	0.011	0.013	0.014
chanell wall of concrete blocks	0.014	0.016	0.017
Cement (mortal)	0.011	0.013	0.015
Steel (rock-bar, welding)	0.010	0.012	0.014
Steel (rivet)	0.013	0.016	0.017
Smooth steel surface without painting	0.011	0.012	0.014
Smooth steel surface and pipe with painting	0.012	0.013	0.017
Corrugated steel plate	0.021	0.025	0.030
Cast iron without painting	0.011	0.014	0.016
Cast iron plate/pipe with painting	0.010	0.013	0.014
Polyvinyl chloride pipe		0.012	
Reinforced plastic pipe		0.012	
Pottery pipe	0.011	0.014	0.017
Earth lining		0.025	
Asphalt with smooth surface		0.014	
Asphalt with coarse surface		0.017	
Wet massonry	0.017	0.025	0.030
Dry massonry	0.023	0.032	0.035
Tonnel with rock walls without covering	0.030	0.035	0.040
Tonnel with rough rock walls and concrete bed	0.020	0.025	0.030
vegetation-covered surface	0.030	0.040	0.050

Now, given the channel bed width to be 0.6 m, 0.8 m, 1.0 m, 1.2 m and the side wall inclination to be 1 : 0.7, the flow rate is calculated as bellow.



$A=1/2 \cdot (W+W+0.7 \cdot H \cdot 2) \cdot H$

$P=(1+0.7^2)^{1/2} \cdot H \cdot 2+W$

$R=A/P$

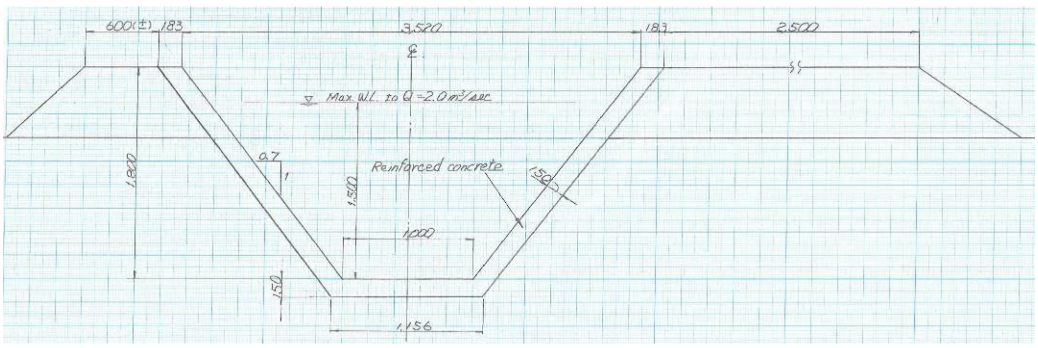
From the calculation result shown in Table 2.3.3.3, the pair of W and H that satisfies the required flow rate 2.0 m3/sec is 0.6m~1.8m, 0.8m~1.6m, 1.0m~1.5m and 1.2m~1.4m.

The plan of 1.0m~1.5m shall be suitable from the safety of farmers escaping from drowning.

Table 5.3.3.2 Flow rate capacity of the canal

W	1/n(n=0.016)	H	A	P	R	$I^{0.5}$	Q
0.6	62.5	1	1.3	3.04	0.427632	0.0141	0.650079
0.6	62.5	1.1	1.507	3.284	0.458892	0.0141	0.789902
0.6	62.5	1.2	1.728	3.528	0.489796	0.0141	0.945982
0.6	62.5	1.3	1.963	3.772	0.520414	0.0141	1.118984
0.6	62.5	1.4	2.212	4.016	0.550797	0.0141	1.30956
0.6	62.5	1.5	2.475	4.26	0.580986	0.0141	1.518353
0.6	62.5	1.6	2.752	4.504	0.611012	0.0141	1.745995
0.6	62.5	1.7	3.043	4.748	0.640901	0.0141	1.993108
0.6	62.5	1.8	3.348	4.992	0.670673	0.0141	2.260306
0.8	62.5	1	1.5	3.24	0.462963	0.0141	0.790878
0.8	62.5	1.1	1.727	3.484	0.495695	0.0141	0.953014
0.8	62.5	1.2	1.968	3.728	0.527897	0.0141	1.132569
0.8	62.5	1.3	2.223	3.972	0.559668	0.0141	1.330173
0.8	62.5	1.4	2.492	4.216	0.591082	0.0141	1.54645
0.8	62.5	1.5	2.775	4.46	0.622197	0.0141	1.782018
0.8	62.5	1.6	3.072	4.704	0.653061	0.0141	2.037486
0.8	62.5	1.7	3.383	4.948	0.683711	0.0141	2.313454
1.0	62.5	1	1.7	3.44	0.494186	0.0141	0.936209
1.0	62.5	1.1	1.947	3.684	0.528502	0.0141	1.121339
1.0	62.5	1.2	2.208	3.928	0.562118	0.0141	1.325053
1.0	62.5	1.3	2.483	4.172	0.595158	0.0141	1.547946
1.0	62.5	1.4	2.772	4.416	0.627717	0.0141	1.79061
1.0	62.5	1.5	3.075	4.66	0.659871	0.0141	2.053636
1.0	62.5	1.6	3.392	4.904	0.69168	0.0141	2.337609
1.0	62.5	1.7	3.723	5.148	0.723193	0.0141	2.643108
1.2	62.5	1	1.9	3.64	0.521978	0.0141	1.085242
1.2	62.5	1.1	2.167	3.884	0.55793	0.0141	1.293977
1.2	62.5	1.2	2.448	4.128	0.593023	0.0141	1.522472
1.2	62.5	1.3	2.743	4.372	0.627402	0.0141	1.771283
1.2	62.5	1.4	3.052	4.616	0.661179	0.0141	2.040968
1.2	62.5	1.5	3.375	4.86	0.694444	0.0141	2.332089
1.2	62.5	1.6	3.712	5.104	0.727273	0.0141	2.645204
1.2	62.5	1.7	4.063	5.348	0.759723	0.0141	2.980871

(3) The draft drawing of the conveyance canal



5.4 Quantity Calculation

2.3.4 Quantity calculation

Item	Calculating formula	Quantity	unit	Note
(1) Dam				
Embankment	The embankment vokume is to be calculated by the equation as shown below.			

$$V=\frac{1}{2}\cdot B\cdot H\cdot(L_1+L_2)+\frac{1}{6}\cdot(m+n)\cdot H^2\cdot(L_1+2L_2)$$

V : Embankment volume of the dam body (m3)

B : Width of the dam crest (m)

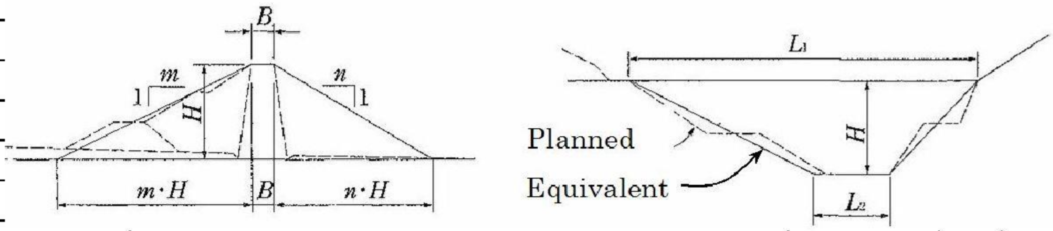
H : Dam height (m)

L_1 : Dam crest length (m)

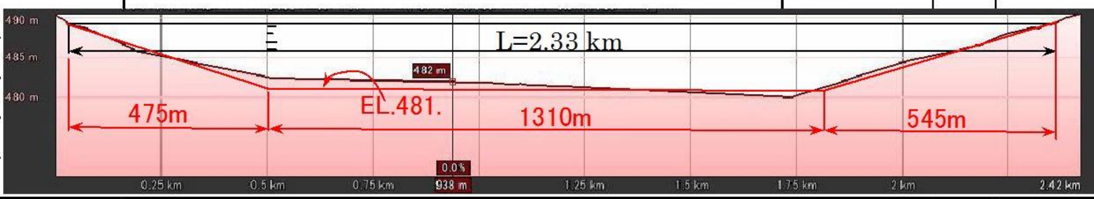
L_2 : Width of the embankment bottom (m)

m : Average inclination of the upstream slope (m)

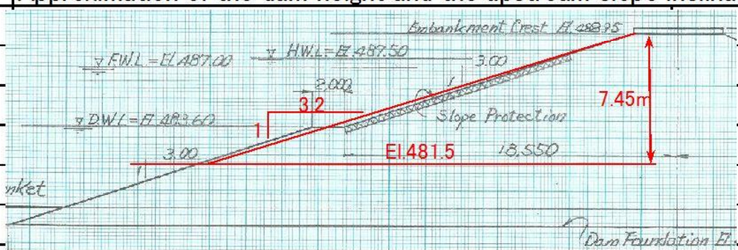
n : Average inclination of the downstream slope (m)



Approximation of the longitudinal shape and the dam foundation



Approximation of the dam height and the upstream slope inclination



Item	Calculating formula		Quantity	unit	Note
Total embankment volume					
	1/2×5.0×7.45×(2330+1310)+1/6×(3.2+2.5)×7.45^2×(2330+2×1310)				
Drain		=	328795.506	m^3	
Vertical	1.0×6.0×(1310+1/3×475+1/3×545)	=	9900	m^3	
Horizontal	0.6×10.5×(1310+1/3×475+1/3×545)	=	10395	m^3	
Toe	(4.4×1.6+0.4×1.5)×(1310+1/3×475+1/3×545)	=	12606	m^3	
Drain total			**32901**	m^3	
Slope protection	13.2×0.4×(1310+4/5×475+4/5×545)	=	**11225**	m^3	
Earth Embankment	328796−32901−11225	=	**284670**	m^3	

Item	Calculating formula	Quantity	unit	Note
Crest protection	0.3 × 5.0 × 2330 =	**350**	m^3	
Vegetation cover	(20.0+4.0) × 1310+(20.0+4.0) × 1/2 × (475+545)=	**43680**	m^2	
Foundation excavation				
	1.0 × 49.2 × 1310+1.0 × (49.2+7.0) × 1/2 × (475+545) =	**93114**	m^3	
(2) Spillway				
Excavation	The excavated soils shall be used as the embankment materials so that this work shall not be counted as an work item.			
Overflow weir	{0.20 × (0.50+0.60+1.80)+1/2 × (2.10+2.00) × 0.10+ 1/2 × (0.30+0.20) × 0.20+1/2 ×			
	(0.25+0.60) × 0.50−(0.125^2−3.14 × 0.125^2 × 1/4)} × 133.0 =	**138.3**	m^3	reinforced c
Retaining wall				
	{0.80 × (1.50+0.50)+0.50 × 1/2 × (1.00+1.00−0.50 × 0.7)} × 2 × 110			
	=	**443.3**	m^3	
Channel bed protection	(40.00−1.50 × 0.7−1.00) × 0.30 × 2 × 110 =	**2504.7**	m^3	
(3) Intake facilities				
Steel conduit	ϕ 800mm	**34.8**	m	
Rolling up concrete (reinforced)				
	{1/2 × (1.66+0.86) × 1.20−3.14 × 0.40^2} × 33.9=	**34.2**	m^3	
Intake tower (reinforced concrete)				
	3.102 × 0.30+(3.10^2−2.50^2) × (488.95−482.00)−1.002 × 0.30+0.20 × 3.10 × 1.20			
	=	**26.7**	m^3	
In-let channel (reinforced concrete)				
	1/2 × 1.4 × 4.0 × 0.2 × 2+0.2 × 6.0 × 2.9+0.2 × 0.2 × 2.9 =	**4.7**	m^3	
Slide gate	1.2 × 1.2=1.44 m2	**1**	Nod.	iron
Discharg control gate		**1**	Nod.	iron
Discharg chamber				
	4.2 × 3.1 × 0.3+(4.2 × 3.1−3.6 × 2.5) × 2.5+4.2 × 3.1 × 0.2−3.14 × 0.4^2 × 0.3			
	=	**16.4**	m^3	
Bridge	L=14.8m	**1**	Nod.	iron
(4) Conveyance canal				
Excavation	1/2 × (1.156+3.04) × 1.35 × 23700 =	**67125.5**	m^3	
Embankment	1/2 × (1.156+3.04) × 1.35 × 23700 =	**67125.5**	m^3	
Concrete lining	{0.183 × 1.80 × 2+1/2 × (1.00+0.183 × 2+1.156) × 0.15} × 23700			
	=	**20096.4**	m^3	
Vegetation cover	(0.84 × 2+0.6+2.5) × 23700 =	**113286.0**	m^2	

5.5 Construction Cost

This construction cost of 29 million US$ is the direct cost without including the overhead cost.

Item	Unit price(US$) evaluation formula	Price	Quantity	Unit	CostUS$	Note
(1) Dam						0.9; Bulking factor
Earth embankment		36.0	316300	m^3	11,386,800	284670/0.9=316300
Drain		40.0	36557	m^3	1,462,280	32901/0.9=36557
Slope protection		50.0	12472	m^3	623,600	11225/0.9=12472
Crest protection		50.0	389	m^3	19,450	350/0.9=389
Vegetation cover		9.0	43680	m^2	393,120	
Foundation excavation		10.0	93114	m^3	931,140	
Sub total					14,816,390	
(2) Spillway						
Overflow weir	concrete, iron bar, frame, all included	440.0	138.3	m^3	60,852	
Retaining wall		440.0	443.3	m^3	195,052	
Channel bed protection		440.0	2504.7	m^3	1,102,068	
Sub total					1,357,972	
(3) Intake						
Steel conduit	φ800mm	1000.0	34.8	m	34,800	
Rolling up concrete	concrete, iron bar, frame, all included	440.0	34.2	m^3	15,048	
Intake tower	concrete, iron bar, frame, all included	440.0	26.7	m^3	11,748	
In-let channel	concrete, iron bar, frame, all included	440.0	4.7	m^3	2,068	
Slide gate	1.2×1.2=1.44 m2		1	Nod	15,000	all included
Discharg control gate			1	Nod	100,000	all included
Discharg chamber	concrete, iron bar, frame, all included	440.0	16.4	m^3	7,216	
Bridge			1	Nod	50,000	all included
Sub total					235,880	
(4) Conveyance canal						
Excavation		10.0	67125.5	m^3	671,255	
Embankment		14.0	67125.5	m^3	939,757	
Concrete lining	concrete, iron bar, frame, all included	350.0	20096.4	m^3	7,033,740	
Vegetation cover		9.0	113286	m^2	1,019,574	
Sub total					9,664,326	
Construction cost					26,074,568	
Temprary Works	10%				2,607,457	
Total					28,682,025	