No. 02

JAPAN INTERNATIONAL
COOPERATION AGENCY

MINISTRY OF AGRICULTURE
AND COOPERATIVES,
THE UNITED REPUBLIC OF
TANZANIA

THE FEASIBLITY STUDY
ON
LOWER MOSHI INTEGRATED AGRICULTURE
AND
RURAL DEVELOPMENT PROJECT
IN
THE UNITED REPUBLIC OF TANZANIA

ANNEXES (2/2)

JULY 1998

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JULY 1998

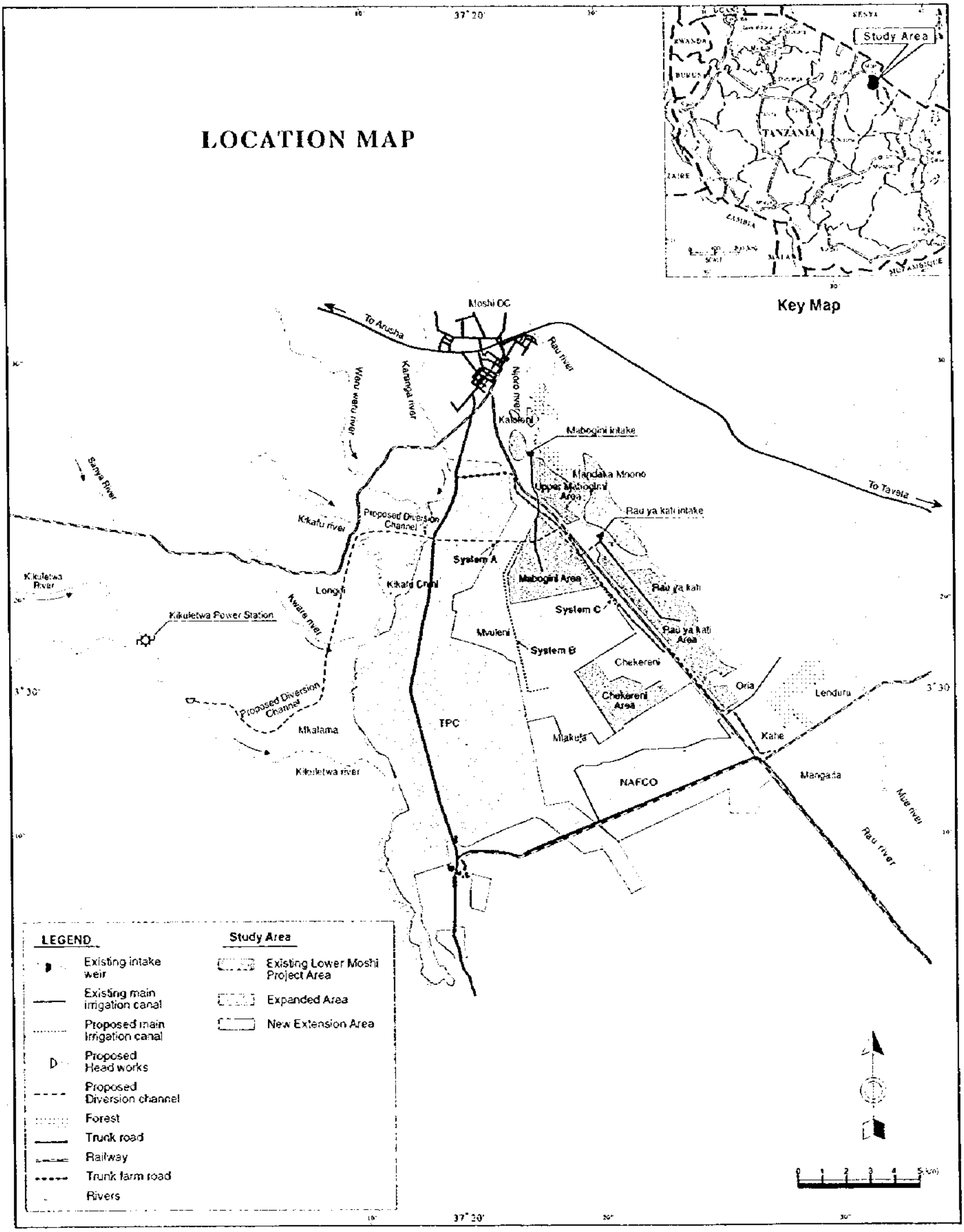
NIPPON KOEI CO., LTD.
PASCO INTERNATIONAL INC.

LIST OF REPORTS

CURRENCY EQUIVALENT
(as of December 1997)

One U.S.Doller (US$1) =Six Hundreds Twenty Tanzanian Shilling (Tsh.620)
=One Hundred Twenty Five Japanese Yen (¥ 125)

LOCATION MAP



ANNEX-J

WATER SOURCE DEVELOPMENT

Annex J

WATER SOURCE DEVELOPMENT

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ANNEX - J

WATER SOURCE DEVELOPMENT

1 INTRODUCTION

This Annex presents the studies for the water source development plan, which describes candidate water source, water balance study to delineate the beneficiary area as well as the hydrological impact for downstream reach of the Study Area.

Chapter 2 indicates the basic approach for water source development, putting the first priority on the utilisation of the existing water source to the maximum extent, and exploiting a new source for the supplementary purpose .

Chapter 3 discusses the candidate new water source, concluding that the Kikuletwa river is the most promising water source.

Chapter 4 indicates water demands for both irrigation and domestic water, which will be used to determine the project development scale.

Chapter 5 presents the water balance study, showing the condition of the study, irrigable area for each water source. The development scale of the Project were determined based on the available water source and the water demands.

Chapter 6 mentions the change of the river discharge with the project, comparing the discharge with project with the those before the project implementation. It also stresses that the some river flow can be released downstream even after implementation of the Project .

Chapter 7 analyses the effects to the Nyumba Ya Mungu Reservoir, where is located downstream of the Study Area, simulating the reservoir operation study. It is concluded that water abstraction for the project w will never obstruct the dam operation.

Chapter 8 describes the optimum scale of water source facilities including locations of the headworks and the diversion channel, type of the weir, method of canal lining.

Chapters 9 and 10 presents the engineering design of the headworks and the diversion weir, such as design condition, feature of the facilities.

2 BASIC APPROACH FOR WATER SOURCE DEVELOPMENT

The Njoro and Rau rivers have been exploited at the water source for the Existing Lower Moshi Project Area. However, recent vigorous paddy cultivation in upstream areas has preferentially used water of these rivers from its advantageous location, therefore the Existing Lower Moshi Project Area has been obliged to be in a constant water shortage. In order to save the Existing Lower Moshi Project Area from water shortage, and also to release the surrounding areas from unstable rainfed cultivation and domestic water insufficiency, it is essential to look for a new additional water source near these areas, say the Study Area.

The basic approach for formulation of the water source development plan, is as follows:

(a) First priority shall be placed upon the maximum use of the existing water sources of the Njoro and Rau rivers.

(b) The water deficit after the maximum use of the existing water sources shall be supplemented through exploitation of new water sources, to minimise the cost.

(c) The exploitation of new water sources shall be considered from the viewpoint of application of gravity irrigation to the Study Area.

3 CANDIDATE NEW WATER SOURCES

There are five rivers flowing near the Study Area : the Kikuletwa river, Kikafu river and its tributaries: such as the Karanga, Weruweru and Kware rivers. Those rivers are considered as candidate new water sources. Hydrological analysis has been made for those rivers as shown in Section 3.3. The result showed that except the Kikuletwa and Kikafu rivers, the rivers have runoff as small as 20 to 40 million m^3 in a year. In addition, those are seasonal rivers and their discharge becomes very small in the dry season. Judging from water availability, the possible new water sources would be narrowed down to the Kikuletwa and Kikafu rivers. The Kikuletwa river has a minimum monthly discharge of 10 m^3/s at as observed at IDD54 because much spring water flows into the Kikuletwa river. On the other hand, the Kikafu river shows a considerable annual variation in runoff, reflecting rainfall pattern, and its mean monthly discharge becomes 1.5 m^3/s only in October at IDD8 whereas it is 18 m^3/s in May. It means that the Kikafu river could not be expected as a new water source if a dam with a regulatory capacity is not available.

All the five rivers have been preliminarily examined about a possibility of constructing a dam with a regulatory function, using the available topographic map on a scale of 1/50,000 and through site inspection. The Kikuletwa river originates from the Mt. Meru, and the Kikafu river and its tributaries from the Mt. Kilimanjaro. Those rivers flow on the highland vastly extending at the foot of the said mountains. The highland, which geologically consists of tuff breccia, has a comparatively flat surface. The Kikuletwa and Kikafu rivers form a v-shaped river course in the highland due to steep riverbed gradient. Such topographic and morphological conditions do not present any suitable dam site where the expected reservoir capacity can be created.

4 WATER DEMAND

4.1 Irrigation Water

Water demand for irrigation use in the Study Area, is estimated as shown below:

(a) Rainy season paddy

(Unit : l/s/ha)

Item	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.
Area - 1	0.1	1.2	2.2	1.9	1.8	1.0	0.1
Area - 2	0.1	1.4	2.3	2.0	1.8	1.0	0.1

Area - 1 : Upper Mabogini, Lower Mabogini (a part), and Expanded Area

Area - 2 : Remaining area of Existing Lower Moshi Project Area and New Extension Area

(b) Dry season paddy

(Unit : l/s/ha)

Item	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.
Area - 1	0.1	1.6	2.1	2.3	2.2	1.2	0.2
Area - 2	0.1	1.2	2.3	2.4	2.3	1.2	0.2

Area - 1 : Upper Mabogini, Lower Mabogini (a part), and Expanded Area

Area - 2 : Remaining area of Existing Lower Moshi Project Area and New Extension Area

(c) Alfalfa

(Unit : l/s/ha)

Item	Oct.	Nov.	Dec.	Jan.	Feb.
Area - 1	0.3	1.0	1.2	1.3	0.4
Area - 2	0.3	1.0	1.2	1.4	0.5

Area - 1 : Upper Mabogini, Lower Mabogini (a part), and Expanded Area

Area - 2 : Remaining area of Existing Lower Moshi Project Area and New Extension Area

(d) Pilot farm and sugar estate

The pilot farm including KATC and sugar estate shall be supplied with irrigation water at a rate of 130 l/s and 70 l/s, respectively.

4.2 Domestic Water

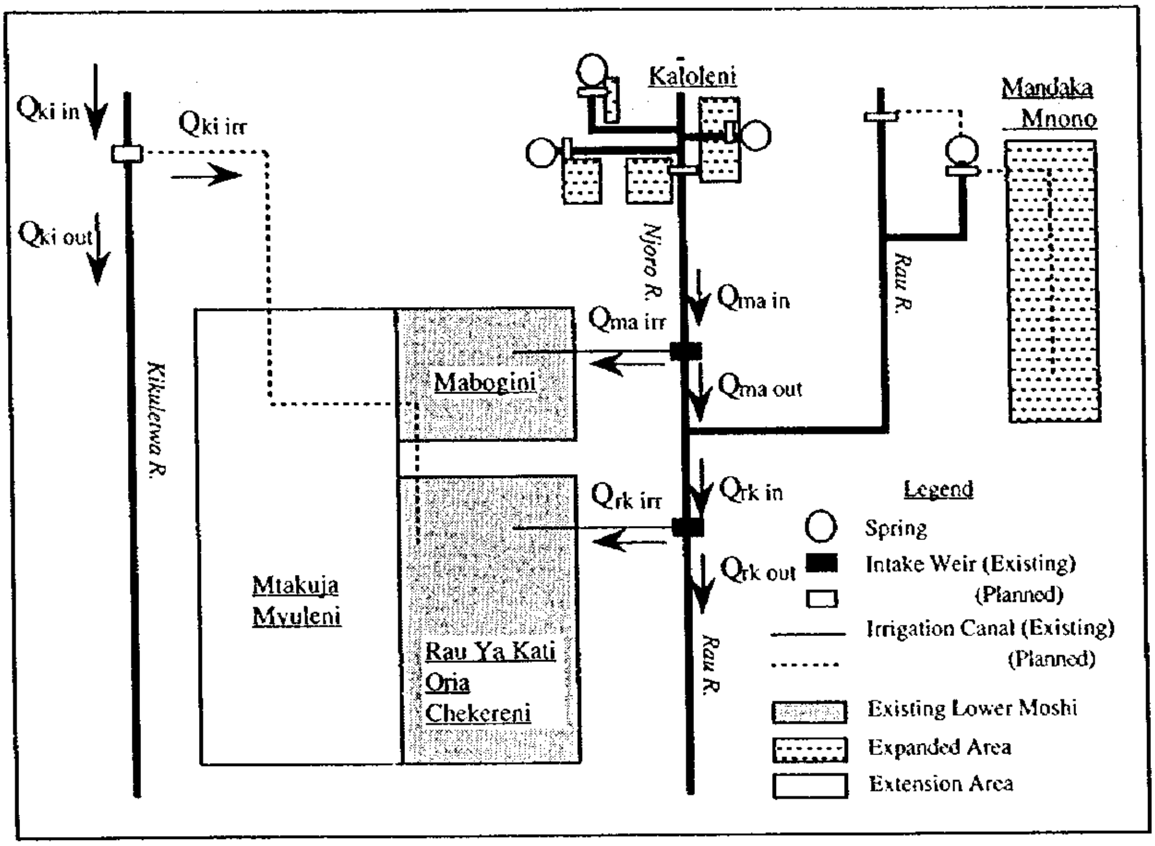
The required domestic water quantity is calculated based on the forecast population and livestock in 2015. The results are as follows:

Item	Calculation	Demand	
Human use	35,400 pers. x 50 l/day /86,400 =	20 l/s	
Cattle	40,300 heads x 20 l/day /86,400 =	9 l/s	
Goats and sheep	97,100heads x 3 l/day /86,400 =	3 l/s	
Total		32 l/s	say 40 l/s

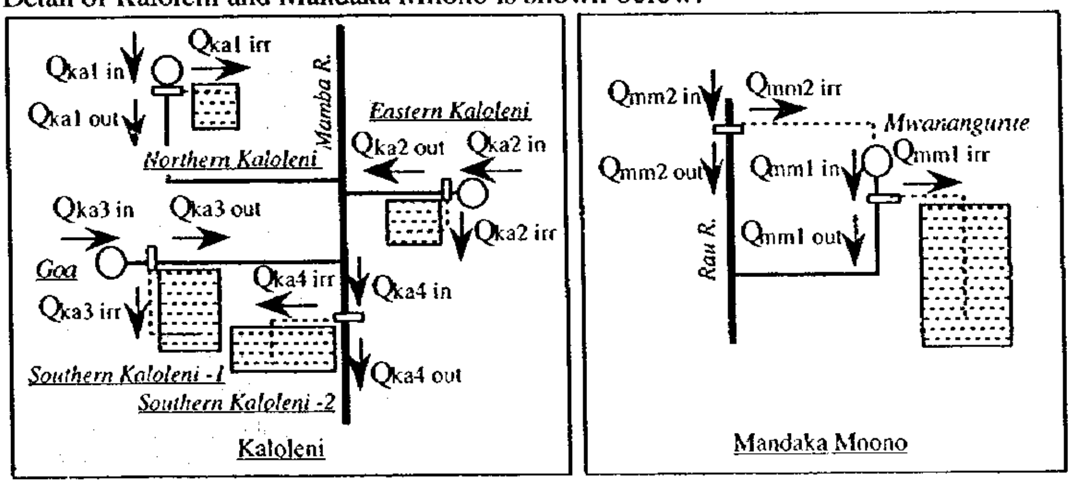
5 WATER BALANCE STUDY

5.1 Conditions of the Water Balance Study

A water balance study was conducted based on such river discharge data, as those of the Rau, Njoro, and Kikuletwa rivers, and the water demand estimated above, to assess the irrigation area relative to the available water source. The schematic chart of the Study Area and the basic conditions applied in the water balance study are shown below.



Detail of Kaloleni and Mandaka Mnono is shown below.



Water Source for each Area

Based on the field investigation in Phase-II study, the water source for each area was determined as shown below.

Area	Primary Water source	Secondary Water Source
Kaloleni	Njoro river with springs	-
Mandaka Mnono	Mwananguruе Spring	Rau river
Mabogini	Njoro River	Kikuletwa River
Rau Ya Kati, Chekereni, Oria	Rau River	Kikuletwa River
Mtakuja, Mvuleni	Kikuletwa River	-

In the Kaloleni, three intake weirs will be constructed near springs while one intake will be on the Njoro River. The Mwananguruwe spring, the primary water source for the Mandaka Mnono area, will be fully utilised for irrigation in the area, supplementing water from the Rau River in the event that the water demand exceeds the discharge of the spring. Thus, an intake weir will be proposed on the Rau River in the Rau Forest Reserve.

The Existing Lower Moshi Project Area, Mabogini, Rau Ya Kati, Chekereni, and Oria, will depend primary water source on the Rau and Njoro rivers, aiming to the maximum utilisation of their water source. Secondary, the water deficit after the maximum use of the rivers will be supplemented from the Kikuletwa River, constructing a new intake facility and a diversion channel.

The irrigation water of the extension area, such as Mtakuja and Mvuleni areas, will be diverted from the Kikuletwa River.

Discharge Records

The discharge records used for the water balance study are outlined below.

		Description	Records Used
Northern Kaloleni	Qka1 in	Inflow at intake site	Qka1 in =30 l/s from the field investigation
	Qka1 irr	Diversion water requirement	Qka1 irr = Unit water requirement x Irrigated area
	Qka1 out	Overflow at intake site	Qka1 out = Qka1 in - Qka1 irr
Eastern Kaloleni	Qka2 in	Inflow at intake site	Qka2 in =100 l/s from the field investigation
	Qka2 irr	Diversion water requirement	Qka2 irr = Unit water requirement x Irrigated area
	Qka2 out	Overflow at intake site	Qka2 out = Qka2 in - Qka2 irr
Southern Kaloleni-1	Qka3 in	Inflow at intake site	Qka3 in =80 l/s from the field investigation
	Qka3 irr	Diversion water requirement	Qka3 irr = Unit water requirement x Irrigated area
	Qka3 out	Overflow at intake site	Qka3 out = Qka3 in - Qka3 irr
Southern Kaloleni-2	Qka4 in	Inflow at intake site	Qka4 in = Estimated Monthly discharge at 1DC35
	Qka4 irr	Diversion water requirement	Qka4 irr = Unit water requirement x Irrigated area
	Qka4 out	Overflow at intake site	Qka4 out = Qka4 in - Qka4 irr
Mabogini	Qma in:	Inflow at intake site	Qma in = Estimated Mabogini intake discharge - Qka1 irr - Qka2 irr - Qka3 irr - Qka4 irr
	Qma irr:	Diversion water requirement	Qma irr = Unit water requirement x Irrigated area
	Qma out	Overflow at intake site	Qma out = Qma in - Qma irr
Mandaka Mnono (Primary)	Qmm1 in:	Inflow at intake site	Qmm1 in =300 l/s from the field investigation
	Qmm1 irr:	Diversion water requirement	Qmm1 irr = Unit water requirement x Irrigated area
	Qmm1 out	Overflow at intake site	Qmm1 out = Qmm1 in - Qmm1 irr
Mandaka Mnono (Secondary)	Qmm2 in:	Inflow at intake site	Qmm2 in = Estimated monthly discharge at 1DC5
	Qmm2 irr:	Diversion water requirement	Qmm2 irr:=Qmm1 irr - Qmm1 in if Qmm1 irr > Qmm1 in
	Qmm2 out	Overflow at intake site	Qmm2 out = Qmm2 in - Qmm2irr
Rau Ya Kati	Qrk in	Inflow at intake site	Qki in = Estimated monthly discharge records of the Rau River at confluence with the Njoro River - Qmm1 irr: + Qma out
	Qrk irr	Diversion water requirement	Qrk irr = Unit water requirement x Irrigated area
	Qrk out	Overflow at intake site	Qrk out = Qrk in - Qrk irr
Kikuletwa	Qki in	Inflow at intake site	Qki in = Monthly discharge records at 1DD54
	Qki irr	Diversion water requirement	Qki irr = Unit water requirement x Irrigated area
	Qki out	Overflow at intake site	Qki3 = Qki1 - Qki2

The monthly records from 1967 to 1992 (26 years) were used for the study taking into consideration the availability of the discharge data.

Cropping Pattern

The proposed cropping pattern was set for the water balance study as shown below.

Rainy season	Dry season
paddy 100%	paddy 50%, alfalfa 20%

Irrigable Area

According the land use plan, the maximum irrigable area was set as follows:

	Area Name	Maximum Irrigable Area (ha)
Expanded Area	Mandaka Mnono	360
	Kaloleni	100
	Northern Kaloleni	4
	Eastern Kaloleni	27
	Southern Kaloleni-1	32
	Southern Kaloleni-2	37
Existing Lower	Upper & Lower Mabogini	885
Moshi Area	Rau Ya Kati & Oria	1,265
Extension Area		2,400
Total		5,010

Aiming at the maximum use of available water source, the irrigable area of each abstraction point were estimated. The irrigable area in the rainy season was set up as irrigation area.

5.2 Result of the Water Balance Study

Based on the aforementioned conditions, the irrigation area with a 80% dependability was estimated for both the Rau and Njoro Rivers, and the Kikuletwa River. The study results are summarised below:

Irrigable Area per Water Source

(Unit : ha)

Area	River	Abstraction point	Rainy season	Dry season
Expanded Area	Rau	Mandaka Mnono	360*	252
	Njoro	Kaloleni**	100*	70
Existing Lower	Njoro	Mabogini Intake Weir	257	180
Moshi Area	Rau	Rau Ya Kati Intake Weir	160	294
Extension Area	Kikuletwa		4,133	2,711
Total			5,010	3,507

Note : * Maximum Development Area, ** Total of 4 systems

70 ha of sugar estate and 80 ha of KATC farm can also be irrigated.

The water balance study results indicate that the whole area of 5,010 ha would be irrigated in the rainy season and 3,507 ha in the dry season.

The diversion water requirement at each location is shown below.

Diversion Water Requirement

(Unit : m^3/s)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Northern Kaloleni	0.00	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.01	0.01	0.01	0.00
Eastern Kaloleni	0.01	0.04	0.06	0.05	0.05	0.03	0.01	0.02	0.03	0.03	0.04	0.02
Southern Kaloleni 1	0.02	0.04	0.07	0.06	0.06	0.03	0.01	0.03	0.03	0.04	0.04	0.03
Southern Kaloleni 2	0.02	0.05	0.08	0.07	0.07	0.04	0.01	0.03	0.04	0.04	0.05	0.03
Mabogini	0.19	0.40	0.64	0.56	0.53	0.33	0.11	0.28	0.34	0.38	0.41	0.29
Mandaka Mnono	0.17	0.46	0.79	0.69	0.65	0.36	0.06	0.29	0.38	0.44	0.47	0.30
Rau Ya Kati	0.17	0.23	0.35	0.29	0.27	0.16	0.04	0.34	0.44	0.51	0.55	0.35
Kikuletwa	2.05	6.34	9.67	8.43	7.60	4.30	0.77	2.49	4.62	5.04	5.39	3.42

It is remarked that, on the Kikuletwa River, the diversion water requirement in March and November exceed the discharge applied by the provisional water right of the Project, 9 m^3/s in the rainy season, and 5 m^3/s in the dry season. Thus, the balance study was carried out repeatedly so that the diversion water requirements on the Kikuletwa River are not more than that indicated in the water right application. The final results are shown below:

Irrigable Area per Water Source

(Unit : ha)

Area	River	Abstraction point	Rainy season	Dry season
Expanded Area	Rau	Mandaka Mnono	360*	252
	Njoro	Kaloleni**	100*	70
Existing Lower	Njoro	Mabogini Intake Weir	257	180
Moshi Area	Rau	Rau Ya Kati Intake Weir	160	294
Extension Area	Kikuletwa		3,823	2,494
Total			4,700	3,290

Note : * Maximum Development Area, ** Total of 4 systems
70 ha of sugar estate and 80 ha of KATC farm can also be irrigated.

The water balance study results indicate that the whole area of 4,700 ha would be irrigated in the rainy season and 3,290 ha in the dry season.

The diversion water requirement at each location is shown below.

Diversion Water Requirement

(Unit : m^3/s)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Northern Kaloleni	0.00	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.01	0.01	0.01	0.00
Eastern Kaloleni	0.01	0.04	0.06	0.05	0.05	0.03	0.01	0.02	0.03	0.03	0.04	0.02
Southern Kaloleni 1	0.02	0.04	0.07	0.06	0.06	0.03	0.01	0.03	0.03	0.04	0.04	0.03
Southern Kaloleni 2	0.02	0.05	0.08	0.07	0.07	0.04	0.01	0.03	0.04	0.04	0.05	0.03
Mabogini	0.19	0.40	0.64	0.56	0.53	0.33	0.11	0.28	0.34	0.38	0.41	0.29
Mandaka Mnono	0.17	0.46	0.79	0.69	0.65	0.36	0.06	0.29	0.38	0.44	0.47	0.30
Rau Ya Kati	0.17	0.23	0.35	0.29	0.27	0.16	0.04	0.34	0.44	0.51	0.55	0.35
Kikuletwa	1.90	5.87	8.96	7.81	7.04	3.99	0.72	2.30	4.26	4.65	4.97	3.16

Taking into consideration effective operation of the small-scale hydropower at the diversion channel, it is proposed the diversion from the Kikuletwa River will be set at 9 m^3/s throughout the year, and then 4 m^3/s will return the Kikuletwa river via the Kikafu river in the dry season from June to November.

The results of the balance study of the Mabogini, Rau Ya Kati, and Kikuletwa with the above figures are presented in Figures 5.1 to 5.3.

5.3 Delineation of the Project Area

After estimating the maximum irrigable area per abstraction point as mentioned in the previous sections, a study on water allocation from the water sources, the Rau and Njoro rivers, and Kikuletwa river, was carried out as described in this sub-section. Firstly, the Expanded Area and the Existing Lower Moshi Area are to be irrigated by water from the Rau and the Njoro rivers. Secondly, the Existing Lower Moshi Area is to be developed with water from the Kikuletwa river so as to attain the maximum development of the area. Finally, the remaining water from the Kikuletwa river would be utilised to irrigate the Extension Area. The following tables present the calculated irrigation development areas by area and water source.

Irrigation Development Area by Area and Water Source

(Unit : ha)

Area		Rainy season (paddy)			Dry season (paddy + alfalfa)		
		Rau & Njoro	Kikuletwa	Total	Rau & Njoro	Kikuletwa	Total
Expanded Area	Mandaka Mnono	360	0	360	252	0	252
	Kaloleni	100	0	100	70	0	70
Existing Lower Moshi Area	Mabogini	257	628	885	180	440	620
	Rau Ya Kati	160	1,105	1,265	294	591	885
Extension Area		0	2,090	2,090	0	1,463	1,463
Total		877	3,823	4,700	796	2,494	3,290

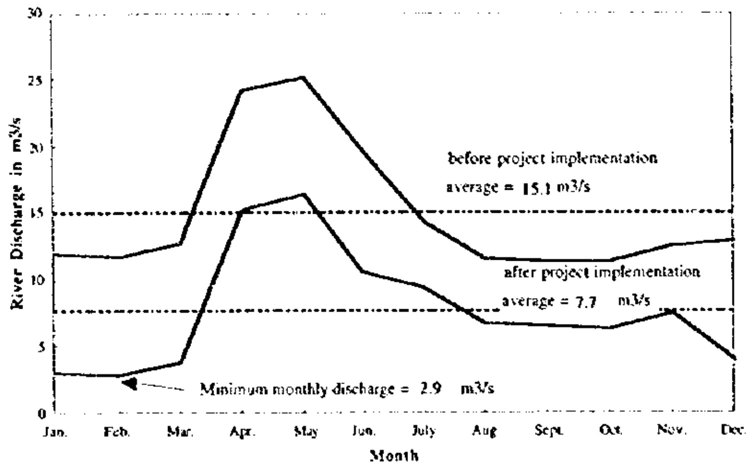
Note: Following areas can also be irrigated:
70 ha of sugar estate by the Mabogini Intake
80 ha of KATC farm by the Kikuletwa Intake

6 RIVER DISCHARGE CHANGE WITH THE PROJECT

6.1 Change of River Discharge

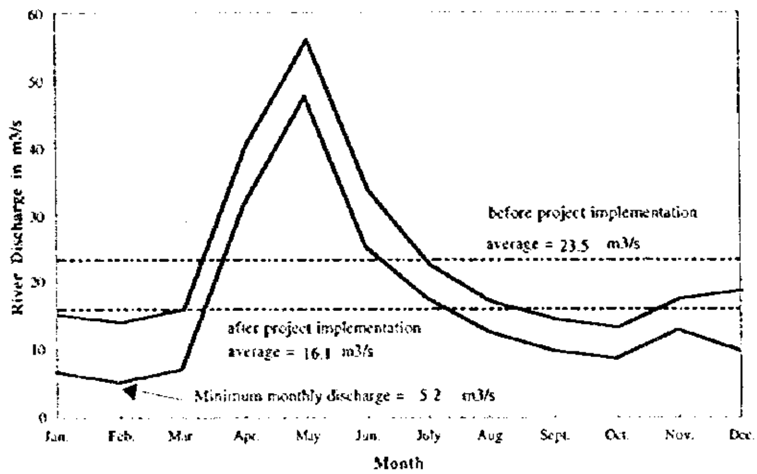
This sub-section describes the changes in the Kikuletwa river discharge after implementation of the Project. The diverted discharge from the intake site is 9 m^3/s from December to June, and 5 m^3/s from July to November.

The changes of river discharge at IDD54 and IDD1 are illustrated below.



Change of Mean Monthly Discharge with the Project Implementation Station : IDD54

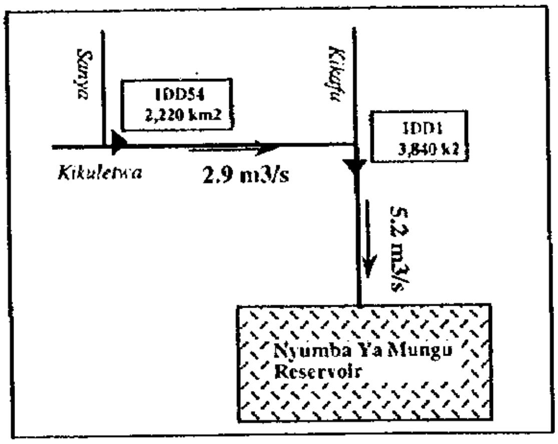
The mean minimum flow, which may take place in February, at the IDD54 station will be decreased to 2.9 m^3/s compared to 11.9 m^3/s at present. The mean annual discharge will be also decreased by 7.4 m^3/s, i.e. 15.1 m^3/s at present and 7.7 m^3/s after completion of the Project.



Change of Mean Monthly Discharge with the Project Implementation Station : IDD1

As for the station IDD1, the mean minimum flow, which may take place in February, will be decreased to 5.2 m^3/s compared to 14.2 m^3/s at present. Mean annual discharge will be also decreased by 7.4 m^3/s; 23.5 m^3/s at present and 16.1 m^3/s after completion of the Project.

The monthly mean minimum flow of the Kikuletwa river, in summary, is graphed below.



Mean Minimum Discharge of Kikuletwa River
after Implementation of the Project

6.2 Water Use Downstream of Abstraction Points

There exist two villages, Kiruani and Chemchem, between the junction point of the Kikuletwa and Kikafu rivers, and the reservoir of Nyumba Ya Mungu dam. The results of interview with villagers indicate that the Kikuletwa river is presently used for maize cultivation, but not paddy cultivation. Paddy cultivation has been discontinued since 1994 because priority has been placed upon production of maize as staple food. Domestic water for 300 families is obtained from the spring located at the hill foot through a tap system installed by the Regional Water Office.

On the other hand, it has been found that the Chemchem village with 450 families in total, cultivates paddy on about 500 acres constantly using the Kikuletwa river. Further, as mentioned in Annex A, the provisional water right for the irrigation are was applied in May, 1997, ensuring 100 l/s of irrigation water from the Kikuletwa River at the Maitu wa Tembo Village in the Arusha region. However, the sum of the water abstraction will be within the minimum monthly discharge of the Kikuletwa River even the Project will be implemented.

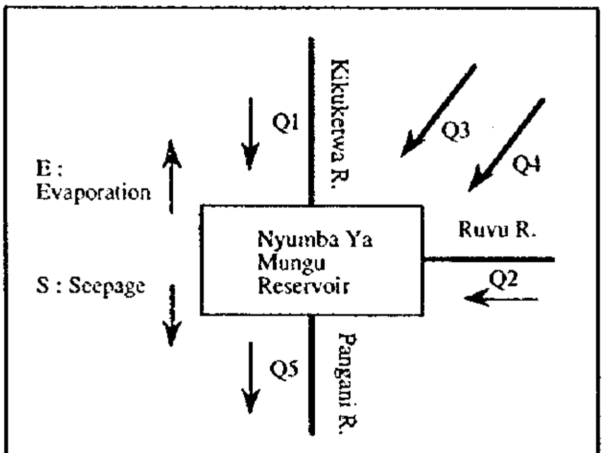
From these results, it is deemed that any problem would not occur in these areas after water abstraction from the Kikuletwa river for the Study Area.

7 EFFECT ON THE NYUMBA YA MUNGU RESERVOIR

7.1 Water Balance Model for the Reservoir

A water balance study was carried out based on the Kikuletwa river discharge data and the water demand estimated above, to assess the utilisable water sources for the Study Area taking into account present water use in downstream areas.

In order to perform the water balance study, a water balance model was prepared as described below.



Water Balance Model for Nyumba Ya Mungu Reservoir

The following formula was used for preparing the water balance model:

$$S2 - S1 = Q1 + Q2 + Q3 + Q4 - Q5 - E - S$$

Where,

S1 :	Dam storage at the beginning of the month
S2 :	Dam storage at the end of the month
Q1 :	Inflow from the Kikuletwa River
Q2 :	Inflow from the Ruvu River
Q3 :	Return flow through groundwater after irrigation
Q4 :	Inflow from area downstream of 1DD1
Q5 :	Outflow from the dam
E :	Evaporation from the reservoir
S :	Seepage from the reservoir

Each parameter is described below:

Q1 : Inflow from the Kikuletwa River

First, the estimated mean monthly discharge at the 1DD1 gauging station in the last ten years was used for the water balance study as shown below.

Estimated Kikuletwa River Discharge at IDD1 1987 - 1996

(Unit : m^3/s)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1987	15.60	12.80	18.84	22.03	46.62	23.75	14.21	15.33	13.49	12.88	12.55	12.29
1988	12.28	12.18	16.37	55.94	41.83	28.10	20.01	14.66	14.99	14.10	15.49	15.27
1989	16.49	15.59	13.76	44.60	75.02	41.76	25.32	18.39	15.97	13.60	16.40	20.07
1990	18.61	14.25	35.15	68.01	90.18	41.50	21.17	16.30	14.10	13.68	20.94	17.02
1991	13.47	13.23	15.75	21.46	55.24	28.72	16.44	14.36	14.80	14.04	15.15	19.89
1992	13.52	13.48	12.26	41.93	51.85	34.39	20.41	16.63	13.97	12.66	13.78	13.98
1993	24.17	22.14	14.81	16.92	21.06	15.01	15.32	13.39	11.60	12.12	12.17	12.22
1994	12.10	12.41	18.42	21.58	49.24	21.76	15.90	13.21	12.73	13.29	14.05	22.15
1995	13.75	12.37	14.12	46.34	60.70	40.25	20.00	14.78	13.56	13.40	12.84	13.08
1996	12.62	13.31	14.56	66.76	69.89	38.90	23.43	16.64	14.54	12.49	13.59	12.38

Secondly, the inflow from the Kikuletwa river to the reservoir can be estimated by subtracting the water quantity abstracted for the Study Area.

Q2 : Inflow from the Ruvu River

The estimated mean monthly discharge at the IDC1 gauging station in the last ten years was used for the water balance study as indicated below.

Estimated Ruvu River Discharge at IDC1 1987 - 1996

(Unit : m^3/s)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1987	12.81	10.10	7.23	13.12	36.35	17.02	14.67	12.55	9.13	8.42	5.15	5.26
1988	2.17	8.58	4.11	13.12	25.43	18.62	12.33	11.02	8.85	8.42	5.15	10.22
1989	8.50	8.58	7.16	13.12	25.43	18.62	12.33	11.02	8.85	8.42	5.15	10.22
1990	8.50	8.58	7.16	13.12	25.43	18.62	12.33	11.02	8.85	8.42	5.15	10.22
1991	5.77	8.58	7.16	12.73	24.54	24.68	17.20	16.79	12.94	9.74	2.33	12.59
1992	10.08	5.63	5.86	13.09	30.04	15.87	12.11	10.43	6.64	8.42	8.76	10.22
1993	17.05	18.93	10.79	8.97	10.86	18.62	8.42	8.54	7.95	8.01	5.06	10.19
1994	4.88	5.48	7.16	10.46	24.76	16.12	10.61	9.32	8.76	8.57	6.58	19.26
1995	10.91	5.32	7.60	11.69	23.65	18.73	11.80	10.83	8.94	8.13	4.34	7.27
1996	4.32	6.05	7.35	21.73	27.89	19.28	11.52	8.73	7.64	7.67	3.85	6.73

Q4 : Inflow from the Residual Area

The inflow from residual area downstream of 1DD1, some 800 km^2 in catchment area, is to be considered. It is estimated as follows:

$$Q4 = \frac{CA \bullet R \bullet 1000 \bullet f}{86400 \bullet 30}$$

Where,

CA : Catchment area of the residual area (km^2)

R : Monthly rainfall at Nyumba Ya Mungu Dam (mm)

f : Runoff Coefficient

The runoff coefficient of the area is estimated at 30 %, taking into consideration the relation between monthly discharge and areal rainfall of the Ruvu river basin. The estimated discharge is as follows:

Estimated Discharge from Residual Area, 1987 - 1996

(Unit : m^3/s)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1987	0.00	0.00	0.00	6.11	7.57	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1988	0.00	0.00	14.21	6.91	0.00	0.00	0.00	0.00	0.00	0.00	4.67	0.00
1989	5.65	0.00	4.71	15.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.11
1990	5.06	0.00	17.48	18.24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1991	0.00	0.00	0.00	5.46	5.20	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1992	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	7.41
1993	0.00	6.21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1994	0.00	0.00	6.18	8.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	10.99
1995	0.00	0.00	13.68	10.64	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1996	5.72	5.88	4.98	6.31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Q5 : Outflow from the Dam

The discharge records from 1971 to 1987 of the gauging station named ID8C, which is located some 2 km downstream of the dam, were collected from the Regional Water Office. As for flow to be released to the downstream reach, in respond to the inquiry by the JICA Study Team, the water office of the water office replied that that is 14 m^3/s. This discharge data was adopted for the water balance study.

E : Evaporation from the Reservoir

Mean monthly evaporation data from the Chekereni station are used for evaporation from the reservoir. The FAO, Irrigation and Drainage Paper No. 27, Agro-Meteorological Field Stations, explained that the evaporation from the lake would be generally about 70 % of that of Class A pan, due to the storage of heat in the large water volume of the lake which results in evaporation not being in phase with climatic, and advective heat difference between lake and pan. On the other hand, average daily evaporation in the country ranges from 5 to 6 mm/day, and that for Chekereni is estimated at 5.3 mm/day. If that at the Nyumba Ya Mungu reservoir is 6 mm/day, the evaporation from the reservoir would be 4.2 mm/day which is lower than at Chekereni. From these study results, it could be said that the use of evaporation data at Chekereni would be more conservative for water balance study. The evaporation data at the Chekereni station are shown below.

(Unit : mm/d)

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Evaporation	7	8	7	4	3	3	3	4	5	7	7	6

Source Chekereni, KADP (1981 - 1996)

S : Seepage from Reservoir

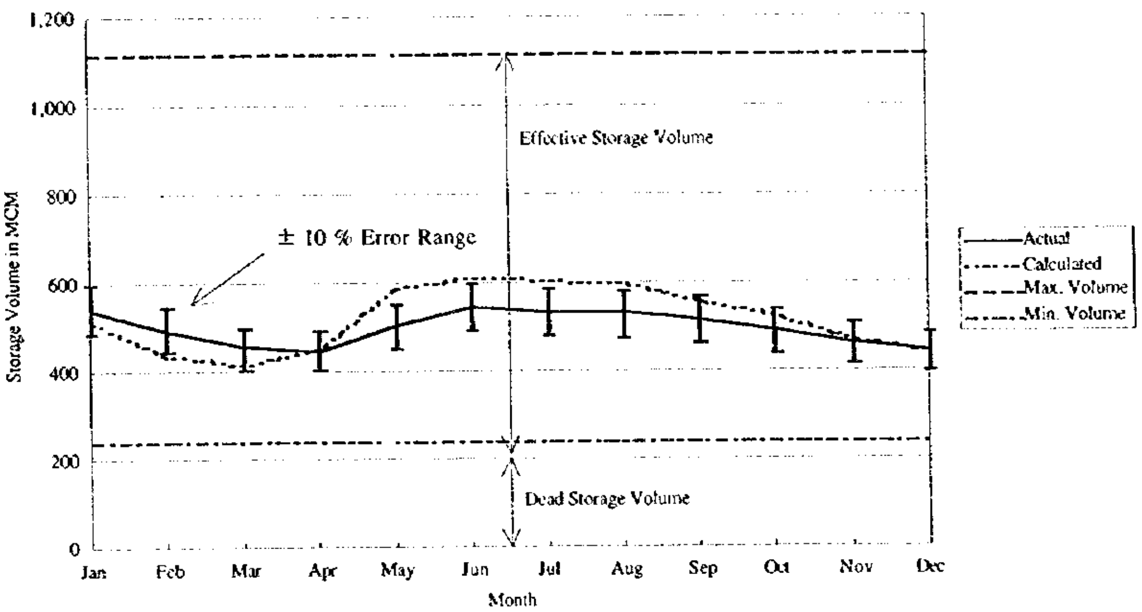
Seepage from the reservoir is assumed at 0.5 % of the storage volume.

Data on the Nyumba Ya Mungu Dam

On 27th May, 1997, JICA Study Team visited the Pangani Basin Water Office at Hale, Tanga Region to collect data and information in relation to the Nyumba Ya Mungu Dam. Salient features of the Nyumba Ya Mungu Dam including water level - storage - area curve, minimum water level and maximum water level, were collected Further, weekly water level records of the reservoir could also be obtained from the office. The water level - storage - area curve of the reservoir is shown in Figure J.7.1. Water level records of the reservoir on a monthly basis from 1971 to date are tabulated in Table J.7.1.

7.2 Verification of the Model

A trial calculation of water balance was carried out to verify authenticity of the aforementioned model, by using of observed discharges as well as water levels of the reservoir in 1987. The result is graphed below.



Trial Water Balance Study Based on Records in 1987

It is confirmed that the model could be applied for the water balance study because the deviation of the calculated storage volumes is within 10 % of the actual values.

7.3 Return Flow Effect

(1) Mechanism of Gross Irrigation Requirement

The gross irrigation water requirement consists of evapo-transpiration (ET), percolation (P), and overall irrigation efficiency (IE). ET is consumed by crops and P and IE are supplied to groundwater. According to the measurement results for paddy at the pilot farm of KADP, the ratio of ET to (ET + P) would be about 60% on an average, and the an overall irrigation efficiency is 69% for paddy. With these figures, the actual water consumption by paddy is calculated to be about 40% of the irrigation water requirement, and the remaining 60% would be supplied to groundwater and finally to the Nyumba Ya Mungu dam located downstream. In addition, the difference between the abstracted water and irrigation water requirements below the peak one will be released into the Rau river and/or drains, and they will finally flow into the reservoir, too. The following table indicates that the annual mean flow of the return flow is estimated at 5.5 m^3/s.

Return Flow Effect

(Unit : m^3/s)

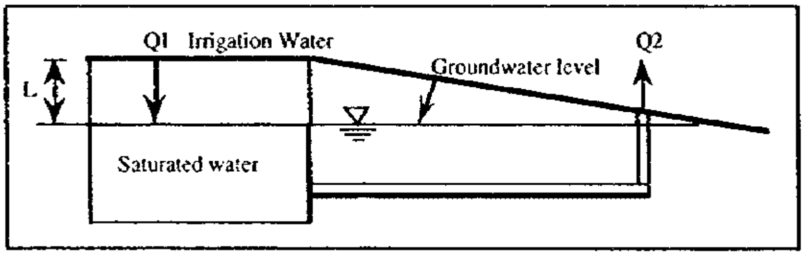
		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
(A) Abstracted discharge		9.00	9.00	9.00	9.00	9.00	9.00	5.00	5.00	5.00	5.00	5.00	9.00
(B) Water demand		1.90	5.87	8.96	7.81	7.04	3.99	0.72	2.30	4.26	4.65	4.97	3.16
(C) Actual water consumption	40% of B	0.76	2.35	3.58	3.12	2.82	1.59	0.29	0.92	1.70	1.86	1.99	1.26
(D) Water to ground water	60% of B	1.14	3.52	5.37	4.69	4.23	2.39	0.43	1.38	2.56	2.79	2.98	1.89
(E) Flow to the Rau River	A - B	7.10	3.13	0.04	1.19	1.96	5.01	4.28	2.70	0.74	0.35	0.03	5.84
(F) Return flow to the reservoir	D+E	8.24	6.65	5.42	5.88	6.18	7.41	4.71	4.08	3.30	3.14	3.01	7.74

(2) General Hydrogeotectonic structure of the Lower Moshi Alluvial Fan

The Study Area including the intake site and diversion channel route extends over the Lower Moshi alluvial fan of which area would be 518 km^2 and depth 90 m as shown in Figure J.7.2. Figure J.7.3 shows the net flow of groundwater in the alluvial fan. This alluvial fan is enclosed by hills, and opens only to the Nyumba Ya Mungu dam located in the south. The groundwater below the alluvial fan is supplied from the Kibo lava belt to the north-west, and the Mawenzi lava belt to the north-east within the volcano area of the Mt. Kilimanjaro. In addition, it is also supplied from surface water, that is from many rivers such as the Kikuletwa, Kikafu, Karanga, Weruweru, Kware, Rau and Ruvu rivers flowing in the fan.

As seen in Figure J.7.3, the groundwater flows toward the ground surface due to the protruded Usagaran layer group at the southern end, and comes out and eventually flows into the Nyumba Ya Mungu reservoir. This phenomenon is proved by the fact that this southern end area is always swampy. The water volume in the same basin is immutable if there is no leakage and/or no diversion to another basin.

Following figure shows schematic model of groundwater movement with an irrigation.



At the initial time irrigation, the remaining 60% of the water go down until it reaches the groundwater table. The time to reach the groundwater table is estimated by use of the Darcy Formula as follows:

$$T = \frac{L * 100}{i * k * 3600}$$

Where,

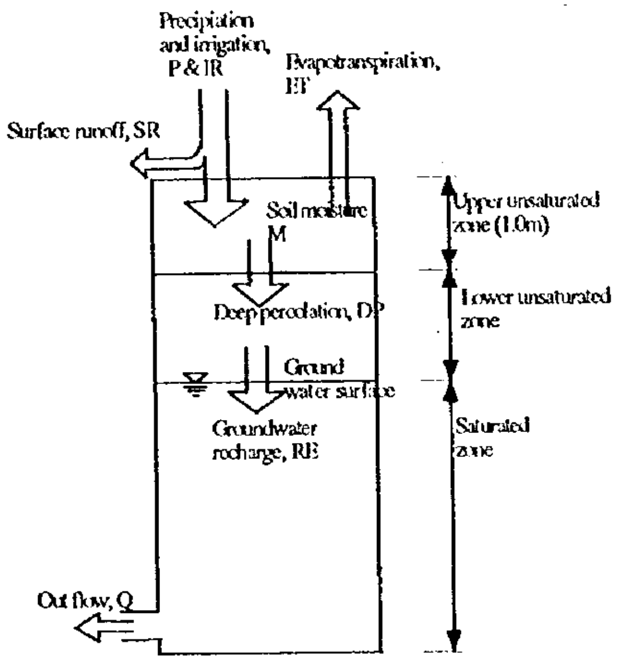
T : Time to reach the groundwater table (hours)
L : Groundwater table in the Study Area (m)
i : Hydraulic gradient (=1)
k : Permeability coefficient (cm/s)

The result of the core drilling in the Study and the information from the Regional Water Office indicated that the L ranges from 9 m throughout a year while the value of k is 10^{-3} cm/s. The T value ,thus, will be estimated at some 10 days. Once a certain amount of water (Q1) flow into the saturated groundwater, the water table goes up temporary and the force to restore the water level works. This movement will make the discharge water (Q2) come out instantly. As long as the irrigation is carried out, the coming out of the water of Q2 could take place continuously. Thus, the time lag would be estimated at only 10 days at the initial irrigation period.

Accordingly, it could be said that about 60 % of the abstracted water from the Kikuletwa river, which is estimated above, would flow into the Nyumba Ya Mungu reservoir with time lag of 10 days.

(3) Estimation of deep percolation

Groundwater recharge (RE) from irrigation (IR) and rainfall (P) can be estimated from a water balance around a vertical soil column of unit area cross-section, extending from the ground surface to the saturated zone. The components of the water balance are illustrated below:



Water balance in the single aquifer system

The soil column is divided into two zones, the upper unsaturated zone and the lower unsaturated zone. The upper unsaturated zone is characterized by the fact that it loses water by evapotranspiration (ET), while any input into the lower saturated zone will finally end up as the groundwater recharge. By taking the water mass balance over the upper unsaturated zone, the deep percolation (DP) to the lower unsaturated zone is determined. The groundwater recharge differs from the deep percolation only by delay and temporal spreading. Also the behavior of the outflow Q can be nearly similar to that of the groundwater recharge because the aquifer under the project area is considered to be surrounded by no flow boundaries, meaning no leakage to the deeper layer and no flow to the other basin as described before.

The water balance over the upper unsaturated zone for a time interval (t, t+Dt) is described by:

$$M(t+Dt) = M(t) + (IR+P-ET-SR)\ Dt \qquad (7.1)$$

Where,

M:	moisture content (mm/day)
IR:	net irrigation requirement (mm/day)
P:	precipitation (mm/day)
ET:	evapotranspiration (mm/day)
SR:	surface runoff (mm/day)
Dt:	1 day

The storage behavior of the upper unsaturated zone is simulated by assuming that seepage will only take place if the input (IR+P-ET-SR) Dt increases the stored water volume above the field capacity FC.

Thus,

$$DPDt = 0 \qquad \text{if } M(t)+(IR+P-ET-SR)\ Dt < FC$$
$$DPDt = M(t)+(IR+P-ET-SR)\ Dt-FC \qquad \text{if } M(t)+(IR+P-ET-SR)\ Dt \geq FC \qquad (7.2)$$

The daily water balance over the upper saturated zone for the project area was calculated for one year from February 1 to January 31 of the next year because the projected irrigation will start in February. This successive calculation of the daily deep percolation needs (1) an initial value of the soil moisture content on February 1, (2) the field capacity and (3) daily data for P, ET, SR, and IR as input.

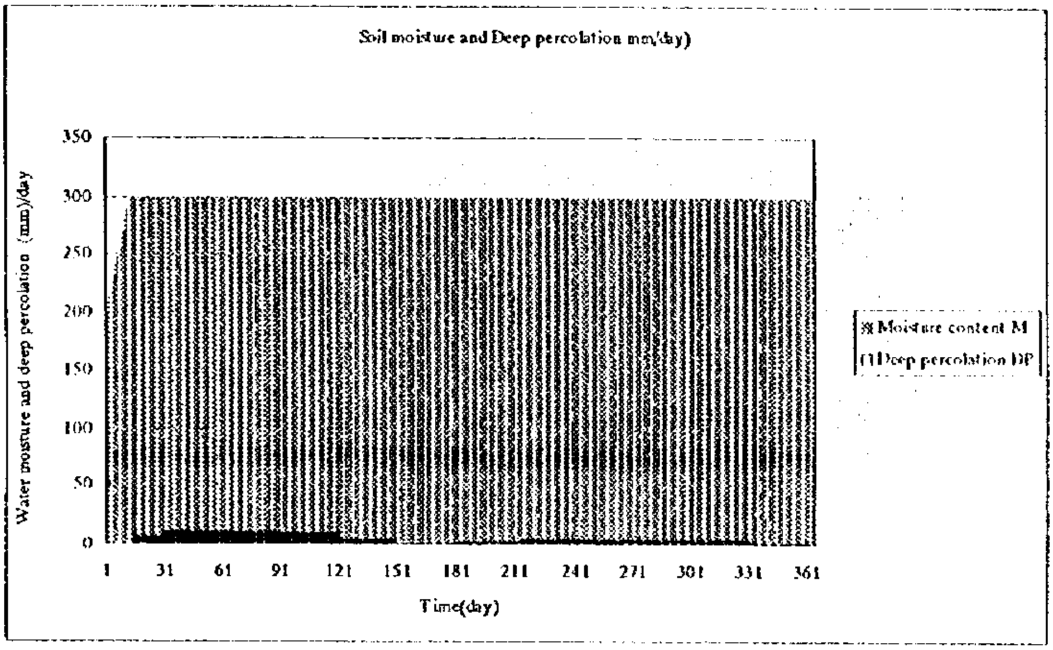
From the field survey results, the soil moisture content on February 1 and the field capacity are assumed 20% and 30%, respectively. Also the upper saturated zone was assumed 1.0 m depth and therefore the field capacity and the initial moisture content are 300 mm (30%*1.0m) and 200mm (20%*1.0m), respectively.

The daily data for P, ET, SR, and IR were simply estimated by dividing those monthly data by days of each month. The monthly average rainfall at Chekelenni was involved in the calculation. The surface runoff was estimated 10% of the precipitation. The evapotranspiration was estimated from the potential evaporation and crop coefficients, considering the projected cropping pattern. The monthly average of surface runoff, evapotranspiration, precipitation, and net irrigation requirement are shown below.

Input data for water balance over the upper unsaturated zone

month	Irrigation requirement IR (mm/month)	Precipitation P (mm/month)	Surface runoff SR (mm/month)	Evapotranspiration ET (mm/month)
Jan.	69.5	32.0	3.2	43.8
Feb.	239.2	38.0	3.8	51.9
Mar.	428.0	68.0	6.8	145.5
Apr.	349.0	160.0	16.0	165.0
May.	338.0	120.0	12.0	136.9
Jun.	183.0	29.0	2.9	63.0
Jul.	38.0	11.0	1.1	8.1
Aug.	111.0	10.0	1.0	16.9
Sep	204.0	9.0	0.9	61.5
Oct	233.3	20.0	2.0	102.5
Nov	236.2	28.0	2.8	121.4
Dec	143.3	63.0	6.3	86.3
total	2572.5	588.0	58.8	1002.6

The following figure shows the daily change of moisture content M and deep percolation DP. The moisture content reaches the field capacity and the deep percolation takes place on February 13, 13 days after the irrigation starts.



Daily change of deep percolation DP and soil moisture content M

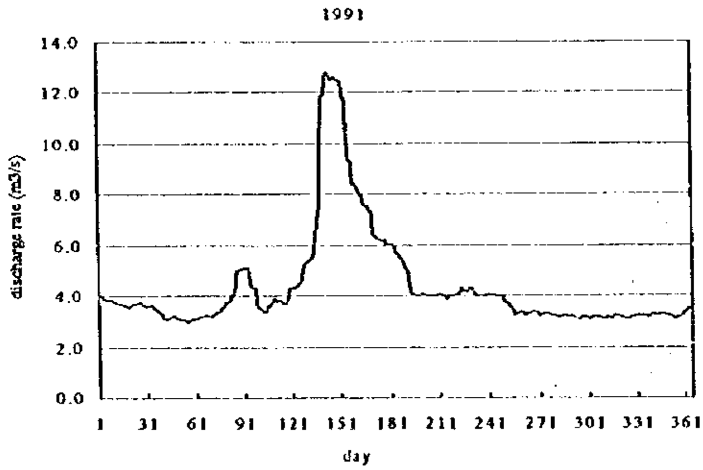
(4) Outflow from the aquifer under the project area

The above deep percolation is a rough approximation. In reality, the excess water leaves the upper unsaturated zone while exponentially decreasing its rate possibly over a longer time. Assuming that excess water percolates down into the lower unsaturated zone during the time interval Dt, a single seepage event over a time interval (t, t+Dt) causes an outflow Q (t). Q (t) can be described by:

Q (t)=0	for t <t
Q (t)=AR*DP*(1-exp (-a(t-t)))	for t $\leq$ t $\leq$ t+Dt
Q (t)=AR*DP*(1-exp (-aDt))exp(-a (t-(t+Dt)))	for t < t+Dt

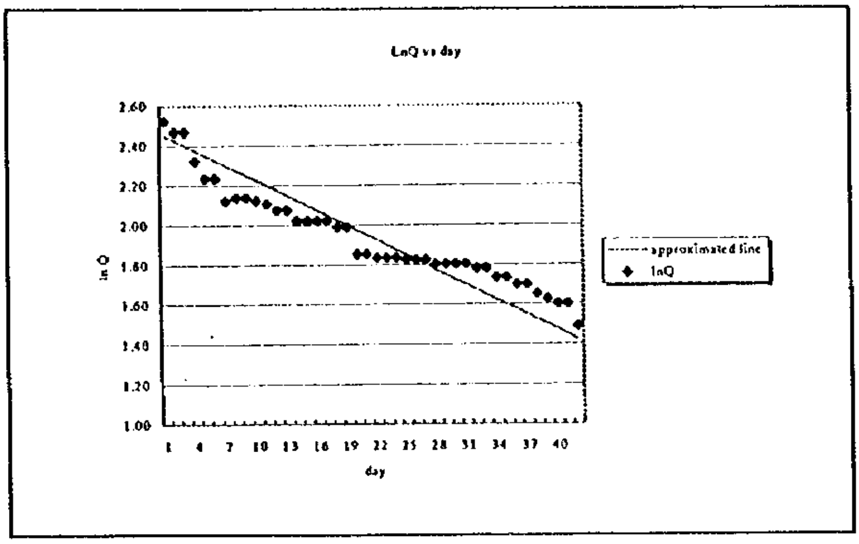
Where,
AR: irrigation area
DP: deep percolation estimated from the water balance
a: exponentially decreasing factor
t: time

The factor a, having dimension 1/time, depends on the geometry of the aquifer and is assumed constant over time. This factor can be estimated using the daily discharge records. The daily discharge record of 1991 at the gauging station IDC6 was selected for this estimation because most of discharge at IDC6 comes from the springs through the aquifer (not including surface runoff).



Daily discharge (Q) in 1991 at IDC6

Over the period (day 152-193) when the daily discharge Q is decreasing, ln Q was plotted as shown below.



ln Q vs d

The relation between ln Q and day can be linear.

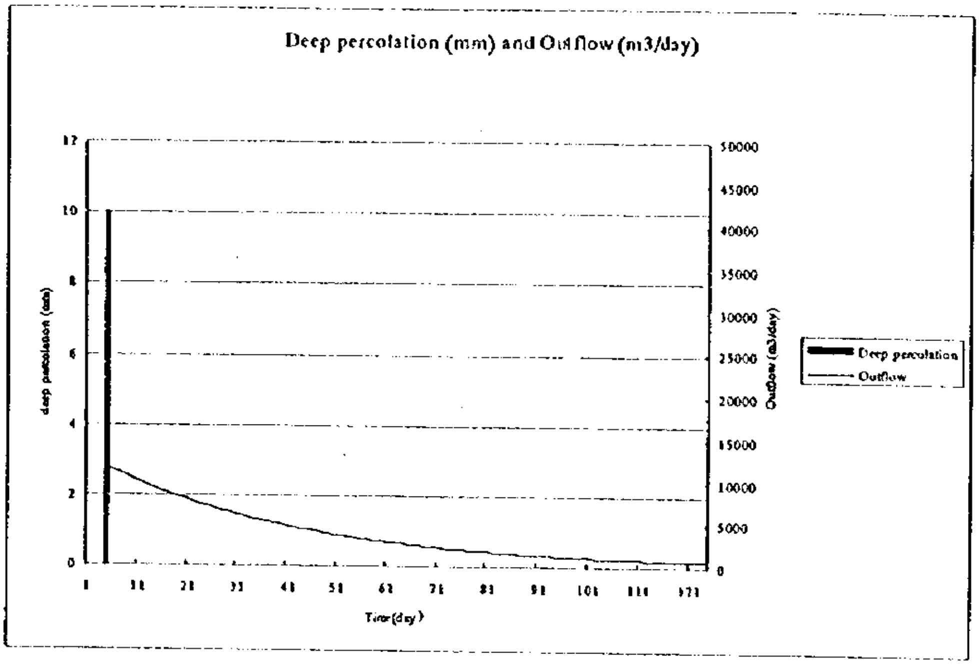
$\ln Q/Q_0 = -at$

$Q = Q_0 e^{-at} = Q_0 e^{-t/Tc}$

Where,
Q and Q_0 : daily discharge rate
Tc: 1/a

From the above figure, a is estimated 0.025 and Tc is 40 days.

For the project area, the behavior of the outflow Q for a single seepage event was simulated. Here, a is 0.025. The irrigation area AR is 4,700 ha. A single deep percolation DP is 10 mm. The daily changes of outflow (m^3/day) for 10 mm of DP on day 5 are shown below.



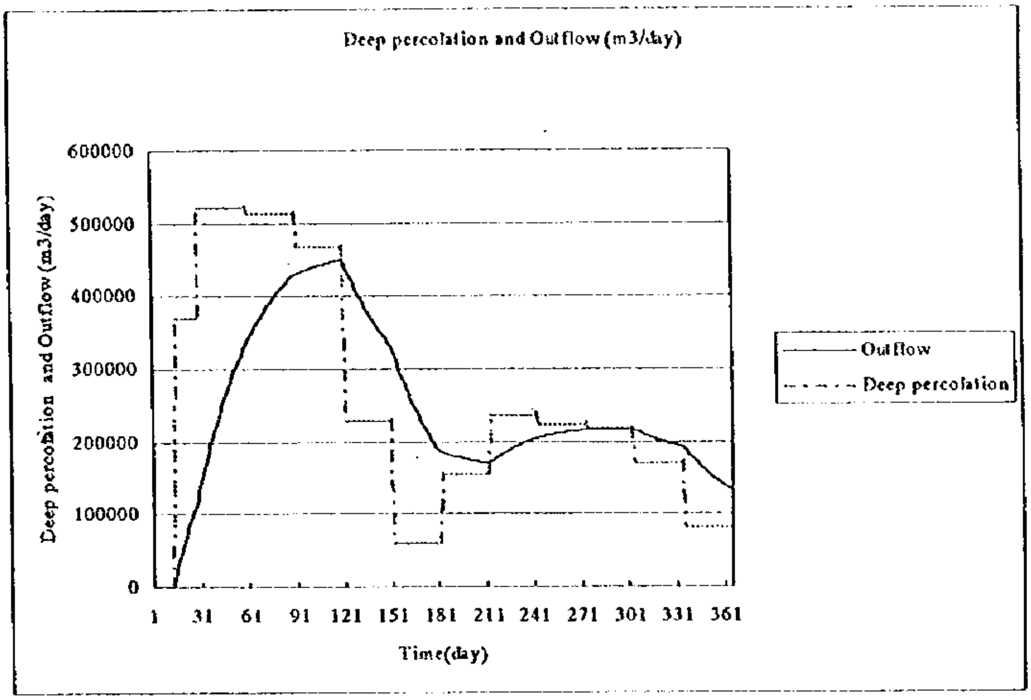
Daily change of outflow for a single seepage event

For the single seepage event, the outflow from the aquifer continues for a long time while gradually decreasing. The cumulative outflow (mm) and the ratio of the cumulative outflow (mm) to the deep percolation of 10mm are summarized in the table shown below. On Day 15, 10 days after the seepage event to the saturated zone, 2 mm (20% of the deep percolation) leaves the aquifer. On Day 45, 40 days after the seepage event, 6.3 mm (63% of the deep percolation) of water leaves the aquifer. On day 95, 90 days after the seepage event, 9.2 mm (92% of the deep percolation water) leaves the aquifer.

Cumulative outflow for a single seepage event

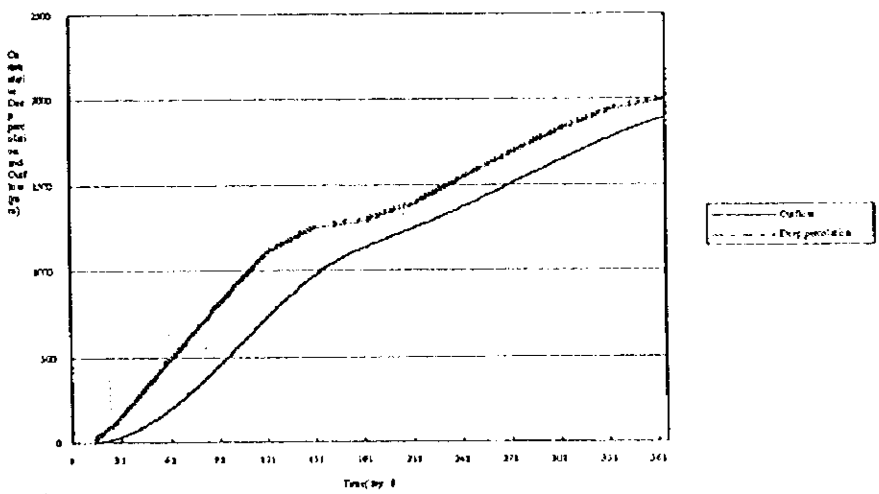
Passing time after seepage event (days)	10	20	30	40	50	60	70	80	90
Cumulative outflow (mm), Sum Q	2.0	3.9	5.3	6.3	7.13	7.8	8.7	9.0	9.2
Ratio of Sum Q to DP	20%	39%	53%	63%	71%	78%	87%	90%	92%

Similarly, the behavior of outflow was simulated for consecutive seepage events, estimated from the water balance with the projected irrigation. The simulation was made for one year, starting on February 1 and ending January 31 of the next year. The following figure shows the simulation results, the daily changes of the deep percolation and the outflow from the aquifer.



<u>**Daily changes of deep percolation DP and outflow Q**</u>

The cumulative amounts of the outflow and deep percolation for simulated one year are illustrated in the following figure. On January 31 of the next year (day 365), the total of deep percolation is 1999.1 mm while the total of outflow reaches 1886.8 mm. This means 94% of the deep percolation leaves from the aquifer within one year.



<u>**Cumulative amount of deep percolation and outflow**</u>

It seems that the outflow takes place with a certain time lag, comparing with the successive seepage events. Here, we define that the time lag is the difference in days between the date when the cumulative amount of the deep percolation reaches a certain amount and the date when the cumulative amount of outflow reaches that amount. The following table summaries the time lags. By this definition, the time lag is estimated around 30 to 40 days.

Sample calculation for time lag

	Date when the cumulative amount reaches a certain amount		
Cumulative amount	Deep percolation	Outflow	time lag
100 mm	26	46	20
500 mm	63	97	34
1000 mm	110	154	44
1500 mm	234	271	37
1800 mm	297	338	41

7.4 Result of the Reservoir Operation

The water balance study was carried out based on observed records in the period from 1987 to 1996. The water abstraction for the Study Area is set up as shown below.

(Unit : m^3/s)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
Abstraction	9.00	9.00	9.00	9.00	9.00	9.00	5.00	5.00	5.00	5.00	5.00	9.00

Firstly, as mentioned in the previous sub-section, the model was run taking into account outflow of 14 m^3/s from the reservoir. The result shows that the water abstraction for the Study Area would never obstruct the dam operation because the simulated water storage in the reservoir is always larger than the minimum volume, which corresponds to the minimum water level for the dam operation under the condition of a 14 m^3/s outflow from the reservoir. Secondly, a maximum 26 m^3/s outflow from the reservoir could be utilised for power generation throughout the year while sustaining water abstraction for the Study Area with the aforementioned discharge. The reservoir operation results with the Project are given in Figure J.7.4. Figure J.7.5 indicate the results of water balance study for the cases of 1) outflow from the reservoir: 14 m^3/s, and 2) maximum water release from the reservoir without water abstraction for the Project. The result indicates that a maximum 28 m^3/s of outflow could be released from the reservoir.

A comparison was made between the "with abstraction" and "without abstraction" as explained above. The result presented that there was only small difference between them. The annual mean inflow to the reservoir will be decreased by 2 m^3/s on in total.

8 OPTIMUM SCALE OF WATER SOURCE FACILITY

8.1 General

The water source facility consists of a headworks and a diversion channel to lead the intaked water from the Kikuletwa river to the Extension Area and the Existing Lower Moshi Area.

A study on the water source facilities has been carried out based on the topography maps with scale of 1/5,000, geology investigation results and site reconnaissance to determine the location and height of a headworks and the route of diversion channel.

8.2 Alternative Study on Headworks Location and Diversion Channel Route

8.2.1 Criteria for Selection and Alternative Sites

As for the selection of headworks site and the diversion channel route, following conditions are applied to the basic criteria of selection:

(1) Headworks

- Headworks composes of diversion weir, sand flush sluice and intake facility
- Intake water level was decided taking into account the highest elevation of the beneficiary area(not less than WL.746.0m)
- Construction workability, especially river diversion works during construction were considered
- Intake water level was raised as higher as possible to reduce the rock excavation for the diversion channel at high land area
- Water level elevation at the crest of headworks should not influence the tail water level of the Existing Kikuletwa No.1 of TANESCO power station(WL.817.4m)

(2) Diversion Channel

- A route selection was executed putting a care upon the topography and geology conditions from the view points of cost saving as well as structural stability
- Diversion channel was provided with lining to reduce channel section and maintenance cost during operation
- Water level of the channel are principally lower than the original ground level to protect the illegal water tapping by inhabitants, especially on the route of the low land area
- Inspection road is to be provided for easy operation and maintenance of the diversion channel

8.2.2 Alternative Sites

Following five(5) alternative plans were set taking into consideration the technical and economic viewpoints.

Alternative	Location of Intake Weir (km) (approximate distance in from the TANESCO power station)	Length of Diversion Channel (km) Total	High land	Low land
Alternative-A	1km	22	10	12
Alternative-B	2km	28	16	12
Alternative-C	4km	23	14	9
Alternative-D	6km	28	16	12
Alternative-E	9km	24	12	12

The locations and routes of the alternative plans are shown in Figure J.8.1.

8.2.3 Selection of Site and Route

An alternative study was carried out based on the said criteria and conditions and study results are shown in Table J.8.1. and summarised as below:

<u>Summary of Evaluation Result</u>

	Alternative plan	A	B	C	D	E
1.	Diversion weir					
1.1	Topography	good	good	steep	deep valley	deep valley
1.2	Geology	hard	hard	hard	hard	weathered
1.3	Workability	good	good	right side steep	steep	good
1.4	Cost	585	1,589	3,300	6,428	33,702
2.	Diversion Channel					
2.1	Topography	high hill	high hill	good	steep slope	steep slope
2.2	Geology	hard & stiff	hard & stiff	hard & stiff	weathered	weathered
2.3	Workability	good	good	good	less	less
2.4	Cost(1,000US$)	36,994	29,284	10,720	58,868	11,334
3	Overall					
3.1	Topography	good	good	good	less	less
3.2	Geology	good	good	good	fairy good	fairy good
3.3	Workability	good	good	good	less	less
3.4	Cost(1,000US$)	37,578	30,873	14,020	65,296	45,036
3.5	Ranking	3	2	1	5	4

As seen in the above summary, the Alternative-C is selected from technical and economical view points.

8.3 Topography and Geology of Selected Site and Route

8.3.1 Headworks Site

(1) Topography and Geology

The headworks site is located at the Kikuletwa river approximately 4 km downstream from the confluence with the Kware river. The Kikuletwa river flows down on the tableland, forming a v- shaped valley. The average gradients of the Kikuletwa river ranged from 1/30 to 1/50 at both sites, and the riverbed elevations is El. 787.0 m. The abutment at the site is characterised by a steep slope with almost vertical or 1 : 0.1 to 1 : 0.2, especially right side abutment is so steep. The site is composed of tuff breccia, and bedrock and basement rocks are found hard. Although unconsolidated deposits are observed, it is extremely thin.

(2) Engineering Assessment on Geology

To grasp the geological conditions for selection of the proposed Intake weir site, 5 numbers of the core boring have been made at the proposed site. The result of core borings indicates the tuff breccia comprising the left and right abutments of the both sites, and this geological condition presents fully satisfactory bearing capacity and water tightness. Permeability test results show that Tuff Breccia gives small permeability coefficients of 10^{-4} to 10^{-5} cm/s, indicating less leakage seams, however, gas holes were found at the boring holes .

8.3.2 Diversion Channel

(1) Topography and Geology

The topography and geology conditions on the diversion channel routes are broadly classified into two categories as shown below.

(a) High land area

Length of the channel on high land area is estimated at 12 km, passing a gently sloped tableland with elevation from El. 840.0 m to 760.0 m. Deep excavation of 10 m to 15 m depth would be required for some 1 km. On the route, there are 2 rivers: one is the seasonal river having width about 15m and 5 m depth, and the other is the Longoi river, which forms valley width with 20 m at bottom and 120 m at top, and 25 m deep. From the geology view points, the both routes are composed of tuff breccia which is generally hard. Unconsolidated deposits and alluvium soils are observed on the routes, however, have been extremely thin with maximum depth less than 1 m.

(b) Low land area

The length of the channel in the low land area is at 12 km after passing the high land area with elevation from El. 755.0 m to 740.0 m. On the route, there are 2 river crossings at the Kikafu river and Weruweru river and 2 road crossings at TPC area and the Existing Lower Moshi Project Area. The route is fully covered with hard alluvium deposits about 5 to 8 m thick in top layer followed by gravel /sand layer.

(2) Engineering Assessment on Geology

To grasp the geology conditions of the route, total 10 numbers of core boring were carried out; 4 numbers for the high land area, and 6 numbers for the low land area. The results of boring for the high land area are deemed that tuff breccia is distributed at the foundation level of channel. This suggests that no serious problem may take place in terms of stability of the channel including slope stability of excavated section. As for the low land area, on the other hand, the standard penetration test (SPT) was executed to clarify the bearing capacity of foundation layers. The test results show the N-value of more than 30 within 2 m depth are observed at all boring sites. The foundation layer of the low land area, thus, are judged to be fully satisfactory on structural stability including earth work as well as channel dike.

8.4 Selection of Headworks Type

The study on selection of structure type of headworks; gravity dam type and fill type dam, was made considering workability, diversion work during construction period, topography condition of the proposed site. As a result, the concrete gravity type was adopted from the following reasons:

(1) Ratio of width(L) and height(H)

The proposed site is found very deep and narrow topography condition and ratio of width and height (L/H) is about 1 or less. It is understood that the concrete gravity type would be more economical than the fill type in such topography.

(2) Component of required structures

The concrete gravity type could be constructed without diversion tunnel during construction period. While, the fill type would require a diversion tunnel with more than 100 m long. In addition, the spillway structure is to be constructed on the right or left side abutment with concrete construction.. From the above, construction cost of fill type would be much costly than gravity type. In addition, the required construction period of fill type weir would be more than 2 times comparing with that of concrete gravity type.

(3) Workability

The topography condition of proposed site is found very narrow (less than 50m in width at maximum portion). From the viewpoints of workability of construction equipment as well as construction period required, such narrow condition is more favourable for concrete gravity type than fill type.

(4) Hydrogeotectonnic Stability

The both sides the Kikuletwa river at the proposed site are sharp-cut cliff, so that contact of abutments with impervious zone (core zone) would be hardly possible to ensure the hydraulic stability, and thus many curtain grouting as foundation treatment would be required in case of fill type.

The headworks is composing of weir body, sand flush gate and intake facility. For the diversion of river water during construction time, it is proposed to employ the open type diversion channel at one side.

8.5 Diversion Channel

8.5.1 Necessity of Lining

The lining work was employed for the diversion channel from the following viewpoints:

- Reduction of hydraulic head loss and seepage
- Prevention of weeds growing
- Less cost in operation and maintenance

As for selection of lining type, a comparative study was carried out for shotcrete lining, cast in-situ concrete lining, and precast concrete block lining. Consequently, it was decided to employ the shotcrete lining at high land area and precast concrete block lining in the low land area from the following viewpoints.

(1) High land area

Shotcrete lining is possible to construct the channel with more steep side slope than the channel with the cast in-situ concrete lining. It would bring reduce of the rock excavation and construction cost. (The preliminary estimate shows the volume of rock excavation may reduced by 12%, the cost for rock excavation by 5%). Meanwhile, the precast concrete block lining would have some difficulty in construction, especially for back-filling behind the block.

(2) Low land area

The cost for precast concrete block lining is slightly lower than that for cast-in-situ concrete lining; the cost for precast concrete block lining and cast-in-situ concrete lining are estimated at US$61.0 and US$63.0, respectively. Besides, no skill or special equipment is necessary for construction and maintenance. Further, quality control of lining material can be easily and effectively made, since concrete blocks are manufactured by a factory of block making plant.

8.5.2 Related Structures

Deep rock excavation, say 10m to 15m, would be required around 1km from the headworks site on the channel route. Except this section, a route would run along the contour line as much as possible to reduce rock excavation volume. A superpassage structure would be required at place where the channel crosses with seasonal streams because its water level is lower than the ground level. Either aqueduct or siphon will be necessary for the channel to cross comparatively large rivers such as Longoi, Kikafu and Weruweru. 2 numbers of chute structure of about 10m drop height in total would be provided at the 5 km and 11 km points. As for its connection with the existing canals in the Existing Lower Moshi Project Area, attention shall be paid on the junction structure in order to make dual functions of smooth water distribution and proper measurement of water.

8.6 Alternative Study on Headworks Height

(1) Comparison Study

On Alternative-C, further 5 alternative cases have been studied to determine the headworks height mainly from economic viewpoints. The route of diversion channel on respective headworks heights are presented in Figure J.8.2, and summarised below. In this study, the crest elevation of the headworks was set lower than EL.814.0m, taking into account the tail water level (EL.817.4 m) of TANESCO No.1 power station and the overflow depth of design flood of 234m^3/s of 100 years return period.

Alternative Case	Crest elevation	Weir Height
C-1	806.00	19
C-2	808.00	21
C-3	810.00	23
C-4	812.00	25
C-5	814.00	28

The preliminary construction cost was estimated for respective headworks heights. The results are presented in Table J.8.2. The table relates that Alternative C-5 of crest elevation with EL. 814.00 m was most economic case.

(2) Determination of Design Condition

Based on the respective study carried out of the said section, basic design conditions of the headworks and the diversion channel are determined as follows:

(a) Headworks

Type	:	Concrete gravity
Intake water level	:	EL.813.90m
Lowest elevation of river bed	:	EL.789.00m
Length	:	70 m
Design flood discharge	:	Q=234m^3/sec(return period: 100 years)
Type of stilling basin	:	Roller bucket type(ski jump type)

(b) Diversion channel

Design discharge	:	Q=9.0m^3/s.	
Concrete lining	:	Shotcrete lining at high land area	
Precast concrete block lining at low land area			
Canal section	:	Trapezoidal	
Low land area	:	1: 0.3	
High land area	:	1: 1.25	
Longitudinal gradient	:	1/1,000 - 1/2,000	
Bottom width and height	:	Type-I	: 2.00(w)m x 2.60 (h)m
		Type-II	: 2.00(w)m x 2.00(h)m
		Type-III	: 1.90(w)m x 1.90(h)m
Velocity(max.)	:	Not less than 1.50 m/s.	

9 ENGINEERING DESIGN OF THE KIKULETWA HEADWORKS

9.1 Component of Headworks

The location of the headworks was determined at approximately 4 km downstream from the existing TANESCO power station. The Kikuletwa headworks is broadly divided into 2 components, namely i) diversion weir and ii) intake structure, and respective components are further consisting of followings:

(a) Diversion weir

- Non-overflow section
- Overflow section(spillway section) with stilling basin at end
- Scouring sluice section with gate facility

(b) Intake structure

- Intake gate with gate facility
- Connection channel to diversion channel
- Measuring device

9.2 Basic Design Condition

9.2.1 Hydraulic Conditions

(1) Flood discharge

Based on the hydrology study mentioned in ANNEX-A, the flood discharge was applied for Q=234 m^3/s(100 years return period).

(2) Crest elevation of overflow section(spillway section)

As described in Chapter-8, the full water level of diversion weir was determined at WL.814.00m.

(3) Intake discharge and intake water level

Based on the irrigation development plan, the intake discharge was applied for Q=9.0 m^3/s and intake water level was determined at WL.813.90m(FWL. 814.00m-0.10m).

9.2.2 Foundation Level

Through the examination of geology boring at sites such as bearing capacity, permeability and groutability to the foundation rock, the lowest excavation line of the river bed was set at EL.789.00m.

9.3 Weir Section

9.3.1 Flood Water Level and Height of Headworks

(1) Overflow depth

Overflow depth(h) above spillway crest was calculated by following formula:

$$h= (Q/CB)^{2/3}$$

where, h: overflow depth(m)
Q: flood discharge(= 234 m3/s)

C: discharge coefficient by overflow crest shape(= 2.10)
B: length of overflow weir(= 40.0 m)

therefore, $h = (234.0/2.10x40.0)^{2/3}$
$= 1.98m$: say 2.00m

Flood water level(FWL): WL.814.00 + 2.00 = 816.00 m

(2) Crest EL. of non-overflow section(TEL)

TEL= FWL + Fb.(free board =1.0m, no-gated spillway below Q=500m3/s)
= 816.00 + 1.00 = EL. 817.00 m

(3) Height of headworks(H)

Height of headworks(H) is: TEL - Lowest foundation level
H= 817.00 - 789.0 = 28.00 m

9.3.2 Section of Diversion Weir

(1) Slope of section

Upstream slope(m) and downstream slope (n) were determined based on the coefficient of horizontal seismic force(Kh), shearing force of bed rock, and external forces acting on weir body. Based on the stability analysis made in preliminary design level, the section of weir was determined as follows:

Upstream slope(m)	: 1:0.12
Downstream slope(n)	: 1:0.80
Width of crest at non-overflow section	: 4.0m

(2) Spillway

Non-gated type spillway was designed taking into purpose of weir(intake purpose only, non-flood control). Type of stilling basin at the end portion of spillway was selected "Roller Bucket type(ski jump type)" taking into consideration of topography and geology conditions of site.

(3) Scouring sluice

To keep the river water course to the intake structure side and to scour the sediments load to the downstream, 2 sets of roller gates with 3 m height and 2 m width were provided.

9.3.3 Stability Analysis

(1) Design condition

(a) Calculation condition: Under full water level and seismic force toward from upstream to downstream.

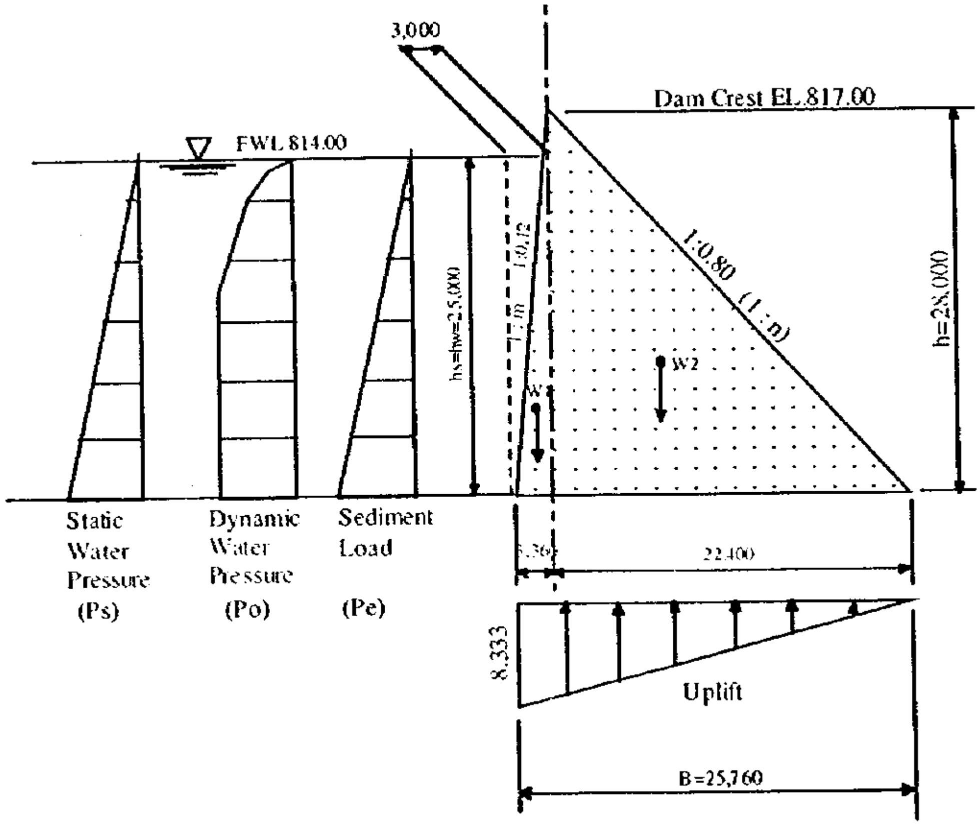
(b) Water level, unit weight, coefficient , etc.

h	: dam height(28.0m)
hw	: max. water depth under full water level(25.0m)
hs	: height of sediment load(25.0m)
m,n	: slope of upstream section(m) and downstream section(n) (m=0.12, n=0.80)
Wc	: unit weight of concrete(2.30tf/m3))
Ww	: unit weight of water(1.00tf/m3)

Ws	: Unit weight of sediment load under submerged condition(1.00tf/m3)
Kh	: coefficient of seismic force(0.12)
Up	: coefficient of up-lift (0.33)
Ce	: coefficient of sediment load pressure(0.60)

where,
Static water pressure (Ps) $Ps=W_w \times h_w$
Horizontal force of sediment pressure (Pe): $Pe=Ce \times Ws \times hs$
Water pressure by seismic force (Po): $Po=0.875 \times Ww \times kh \times (Hh)^{1/2}$
(by Westergard formula)

Calculation model is illustrated as below.



(c) Calculation of load and moment

Calculation of loads and moment. Calculation of loads moments is made under full water condition acting on seismic force calculation sheet is shown in Table J.9.1 and summarised as follows:

Total vertical load:	$\Sigma V = 797.14$ (tf / m)
Total horizontal load:	$\Sigma H = 665.16$ (tf / m)
Total resistance moment :	$\Sigma MR = 13,338.25$ (tfm / m)
Total turning moment :	$\Sigma MT = 5,751.92$ (tfm / m)

(d) Stability Analysis

1) Eccentric distance(e)

$$e = B/2-(\Sigma MR-\Sigma MT / \Sigma V)$$
$$= 25.76 / 2 - (13,338 - 5,533 / 797)$$
$$= 3.087 < B / 6 = 4.293\text{m} \ldots\ldots \text{O.K}$$

2) Stress at ends

$$\sigma_{1,2} = \Sigma V / B(1 \pm 6 \times e / B)$$
$$= 797 / 25.76 \ (1 \pm 6 \times 3.087 / 25.76)$$
$$= 8.7 \text{ or } 53.2 \ (tf / m^2)$$

(σ_1:Upstream end, σ_2:Downstream end)

3) Safety factor for shearing (n)

$$n = (\Sigma V \times f + t \times B) / \Sigma H$$
$$= (797 \times 0.88 + 170 \times 25.76) / 643$$
$$= 7.90 > 4.0 \ \ldots\ldots \ \text{O.K}$$

(where, $f=\tan 41.5^{\circ}=0.88$, $t=170tf/m^2$)

From the above, stability against i)overturning and ii) foundation rock were satisfied. The typical cross section of the weir is shown in Figure J.9.1.

9.3.4 Foundation Treatment

To improve the foundation stability against i)cracks/seams and ii)permeability ,consolidation grout and curtain grout were provided for the foundation rock with following specification:

(a) Consolidation grout:

- pitch : 2.5m alternative arrangement
- depth : 5.0m
- pressure : Max. 5.0kgf/cm^2

(b) Curtain grout:

- pitch and row : 2.5m alternative arrangement, 2 rows
- depth : 10.0m
- pressure : Max. 7.5 kgf/cm^2(by stage method)

9.4 Intake Structure

(1) Component of Structure

Intake facility is composed of i)intake gate, ii) connecting channel to diversion channel and iii) measuring facility at end portion of structure.

(2) Intake Gate

Intake gate was provided just upstream of weir portion of left side of the Kikuletwa river. Width of intake structure was designed based on the velocity of intake water within 1.0m/s. Number of gate is 2 sets with 2.30 m width and 3.00 m height. Intake gate tower is of reinforced concrete structure with 5.30m height and 5.60 m width. Gate facility were roller gate type sluice gate with spindle and hoist. In front of intake gate, steel made trash rack was provided to avoid entering the floating debris to the intake.

(3) Connecting channel

After intake gate, a basin with 30.0 m length and 5.60m width of reinforced U-type concrete flume structure was provided.

(4) Measuring Facility

At the end of connecting channel, measuring facility was provided to control of intake discharge. A type of measuring facility is "45 feet type Parshall Flume".

(5) Protection Facility from Crocodile

To protect crocodile from entering into the diversion channel, a protection wall of masonry was provided on the left side of Headworks. Length of wall is 200 m long and 1.5 m high.

(6) Control Facility

For the operation and maintenance of headworks, a control house was provided near intake structure. A control house is reinforced concrete construction with necessary facilities such as furniture, communication equipment, etc. Area of control house is approximately $50m^2$.

10. ENGINEERING DESIGN OF DIVERSION CHANNEL

10.1 General

As described in Chapter 8, the final route of the diversion channel was selected after several alternative studies(Refer to Figure J.8.1 and following design conditions were determined.

1) Design discharge : 9.0m3/s
2) Section of channel :

Type	Width(m)	Gradient	Height(m)	Side slope	Lining	Water depth(m)	Velocity(m/s)
I	2.00	1/1000	2.60	1:0.30	shotcrete	2.20	1.54
II	2.00	1/2000	2.00	1:1.25	concrete block	1.66	1.33
III	2.00	1/1500	1.90	1:1.25	concrete block	1.55	1.48

10.2 Channel

(1) Lining material

Lining materials of channel were studied from the view points of geology conditions and following materials were selected:

(a) Shotcrete : -Strength at age of 28 days : 180 kgf/cm2

(Type-I)
- Ratio of water and cement : 45 %
- Unit cement quantity : 360 kg/m³
- Max. size of gravel : 15 mm
- Ratio of fine aggregate : s/a=55 %
- Agent for rapid cementation : 5.5 % of cement weight
- Thickness : design 0.07m, payment 0.10m

(b) Concrete block lining:
- Strength at age of 28 days : 180 kgf/cm²
- Max. size of coarse aggregate : 25 mm
- Thickness : 0.10 m
- Joint : by mortar

(c) Bottom concrete:
- Strength at age of 28 days : 180 kgf/cm²
- Max. size of coarse aggregate : 25 mm
- Thickness : 0.10 m

(2) Inspected road

Along the channel, inspection road was provided with following conditions:

- Total width : 5.0 m
- Effective width : 4.0 m
- Pavement : by Laterite, thickness t= 0.10 m

10.3 Related Structures

10.3.1 General Concepts of Design

Deep rock excavation with 10 m to 15 m would be required around 1 km from the headworks on the channel route. Except this portion, a route would be run along the contour line as much as possible to reduce rock excavation volume and head loss. A super-passage structure would be required at place where the channel crosses with seasonal streams because it water level is lower than the ground level. Either aqueduct or siphon structure will be necessary for the channel to cross comparatively large rivers such as Longoi, Kikafu and Weruweru. Chute structures of about 11 m and 34 m drop height would be provided at 5 .4 km point on the route of high land area and at the transfer point from high land to low land. As for connection with the existing canals in the Existing Lower Moshi Project Area, attention shall be paid on the bifurcation structure in order to make dual function of smooth water distribution and proper measurement of irrigation water. The followings are major related structures designed and provided for the diversion channel.

10.3.2 Structures

(1) Longoi river siphon

(a) Superstructure

Longoi river forms deep valley with 30 m depth and 40 m width. In the rainy season, river water flow down with high velocity with many cobbles and debris. The structural system for river crossing was considered with alternatives taking into account of; i)safety against river flow during flood season , ii)construction cost and iii)construction workability. Alternative design plans were i)steel pipe inverted type siphon, ii) box culvert type concrete siphon, iii) concrete aqueduct with approximately 30 m span, and iv) combined type with steel pipe and concrete box culvert structure. As a results of comparison study to the said structure systems, combined type siphon with steel pipe and concrete box culvert is selected from the view points of i) total water head acting to structure at deepest portion of structure(approximately 30 m) and ii) construction cost. Longoi siphon has a total length of 130 m. Inclined portion of right and left side of river are to be reinforced concrete structure with barrel size of 2.40 m x 2.40 m rectangular section and 55.00m length. Centre portion was designed of steel pipe(diameter; 2,200 mm) structure supported by steel girder with 2.50 m height. Maximum span of siphon at steel pipe portion is 25.0 m and total length of steel pipe portion is 55.00 m length. To connect the diversion channel at inlet and outlet of siphon. transition structures with each 10.0 m length were provided. Steel girder is designed as the main beam of supporting structure, and functioned as T-beam of slab bridge for inspection road.

(b) Substructure

Superstructure is supported by the substructure of reinforced concrete construction. The foundation of pier was designed as a direct foundation on the rigid rock. The foundation level of pier was set at approximately EL.741 m, and a height of pier is 11.0 m. At the joint portion between the steel pipe and the concrete box barrel, a gravity type of anchor block structure was provided.

(2) Kikafu river siphon

Width and depth of Kikafu river are found approximately 20 m and 3m at crossing point with the diversion channel. For the selection of river crossing structure, i)concrete box siphon and ii)aqueduct structure were studied. In case of aqueduct structure, no clearance between bottom level of aqueduct and flood water level is kept, therefore, a concrete siphon with total length of 66.0 m including inlet and outlet structure was selected. The size of barrel is 2.40 m x 2.40 m, rectangular shape. Existing elevation of river bed is found approximately EL.749 m to EL.750 m, and keeping the covering depth of 2 m from top of siphon structure, centre elevation of barrel at the river crossing portion was set at EL.745.0m. Apart from 15 m to upstream side, inspection bridge with total length of 36 m was provided. To protect the structures from the river flow, protection with masonry works were provided at right and left banks.

(3) Weruweru river aqueduct:

(a) Superstructure

Width and depth of Weruweru river are found approximately 50 m and 5m, respectively at crossing point with the diversion channel. For the selection of crossing structure, i)concrete box siphon and ii)aqueduct structure were studied. As a result of alternative study, aqueduct with total length of 60.0 m including inlet and outlet transition was selected. The size of barrel is 3.50 m width and 2.50 m height and 12.0 m span. Structure system of aqueduct is to be reinforced concrete and side wall is designed as a main beam of structure.

(b) Substructure

2 number of abutments and 3 number of piers were provided. Type of abutment was designed as concrete gravity type structure with 3.5 m height, and piers were of reinforced concrete with 6.50 m height. Apart from 15 m to upstream side, inspection bridge with total length of 48 m was provided.

(4) Road crossing structure

On the route of the diversion channel, there are many crossing portion with the existing roads such as village roads, provincial roads, and Project road, etc. To minimise the head loss and construction cost, reinforced concrete box culvert with 3.50 m width and 2.50 m height were provided. Structure is composed of inlet and outlet transitions and culvert portion. Length of barrel is determined in accordance with width of existing roads.

(5) Railway crossing structure

Near the trunk road of the Existing Lower Moshi Area, the diversion channel crosses the national railway. Crossing structure is selected box culvert type culvert with 3.50 m width and 2.50 m height. Structure is composed of inlet and outlet transition and barrel portion.

(6) Drop and chute

A vertical and chute type drops were provided at steep gradient section to adjust excessive hydraulic head and to maintain velocity of channel within the allowable limit. The applied criteria was that a vertical type drop should be less than 2 m drop height, and a chute type is more than 2 m and composed of following structure system.

<u>Vertical drop:</u>

a) inlet structure with transition
b) basin for energy dissipater
c) outlet structure with transition

<u>Chute:</u>

a) inclined flume
b) stilling basin
c) outlet structure with transition

(7) Bifurcation

At 21.4 km point, irrigation water from the Kikuletwa headworks is bifurcated into the irrigation command area of i)Existing Lower Moshi Area and ii)Extension Area in accordance with irrigation plan. The distribution of irrigation water to respective areas is controlled by gates and measuring facility(Parshall flume type). Design discharge of bifurcation canal of left side(Existing Lower Moshi Area) is 5.30 m^3/s and right side canal(Extension Area, System A area) is 3.70 m^3/s. Both bifurcation structures are designed of reinforced concrete flume with 5.30m at left side and 3.70 at right side, respectively. Height of both flumes are 3.0 m height.

(8) Cross drain

To cross the seasonal and small river portions, cross drain structure underneath of channel was provided. Structure system is applied for concrete pipe culvert type considering flood discharge of seasonal and small rivers with 5 years return period. Size and number of pipes were determined based on the design discharge at the respective catchment area.

(9) Footpath bridge

For the village people along the channel and for connection of the existing roads of village road class, footpath bridges were provided as required. Width of footpath is 3.0 m and of reinforced concrete slab.

(10) Wasteway structure to Kikafu river

During dry season, surplus discharge with 4.0m3/s (9.0m3/s-5.0m^3/s=4.0m^3/s) is to be returned to the Kikuletwa river through the Kikafu river after generation of electricity at No.2 hydropower station. Wasteway structure was provided at 13.5 km point on the diversion channel and providing approximately 250 m length wasteway canal and finally flow into the Kikafu river. Wasteway canal is of concrete block lining with design discharge of 4.0 m^3/s. Control of discharge is made by control gate and measuring facility.

(11) End structure at confluence of Rau river

At the end portion of the diversion channel, end structure was provided. End structure is composed of i)box culvert underneath of existing Rau Ya Kati main irrigation canal and flood dike of right side of Rau river and ii) connecting channel in the high water channel of the Rau river.

(12) Bridge

On the diversion route, 3 bridges were designed at Longoi, Kikafu and Weruweru river. Structure system of bridges at respective rivers were designed as follows.

Longoi bridge:
a) Span: 25.0 m
b) Width: total; 4.0m, effective: 3.50m
c) Superstructure: slab; RC, girder; steel girder with 2.50m high
d) Substructure: RC pier, 11 m high

Kikafu, Weruweru bridge:
a) span: 12.0m
b) Width: total; 4.0m, effective: 3.50m
c) Superstructure: slab; RC, girder; RC T-beam
d) Substructure: RC pier, 6.0 to 6.5 m high

A list of related structure with features is shown in Table J.10.1.

Tables

Table J.7.1 Water Level of Nyumba Ya Mungu Reservoir

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1971	687.53	687.29	686.75	686.63	688.10	689.29	689.25	689.13	688.94	688.58	688.08	687.68
1972	687.45	687.18	686.89	686.85	687.37	688.33	688.29	688.12	687.94	687.71	687.82	687.82
1973	687.88	687.69	687.36	687.14	688.26	688.54	688.57	688.31	688.02	687.64	687.21	686.80
1974	686.37	685.92	685.41	686.61	687.17	687.25	687.46	687.32	687.01	686.60	686.08	685.60
1975	685.18	684.68	684.08	684.19	684.57	684.66	684.60	684.26	684.08	683.68	683.12	682.64
1971-75	686.88	686.55	686.10	686.28	687.09	687.61	687.63	687.43	687.20	686.84	686.46	686.11
1976	682.11	681.56	680.93	680.88	681.28	681.49	681.71	681.62	681.50	681.33	680.63	680.19
1977	679.97	679.81	679.99	681.78	684.07	684.96	685.15	684.80	684.47	683.86	683.32	683.13
1978	683.49	683.67	684.40	686.12	687.56	688.29	688.75	688.90	688.78	688.33	687.94	688.33
1979	688.44	688.70	688.95	689.64	689.69	689.66	689.36	689.12	688.94	688.68	688.28	687.92
1980	687.59	687.36	687.16	687.28	689.02	689.25	689.12	689.01	688.82	688.42	688.02	687.78
1976-80	684.32	684.22	684.29	685.14	686.32	686.73	686.82	686.69	686.50	686.12	685.64	685.47
1981	687.67	687.27	687.06	688.27	689.43	689.31	689.15	688.88	688.62	688.38	688.04	687.77
1982	687.27	686.80	686.27	686.20	686.93	687.10	687.16	686.98	686.73	686.89	687.32	687.54
1983	687.26	686.90	686.40	686.16	686.95	687.38	687.34	687.13	686.80	686.49	686.01	685.76
1984	685.45	684.98	684.37	684.45	685.24	685.34	685.39	685.28	684.94	684.57	684.61	684.75
1985	684.40	684.20	683.71	684.65	685.71	686.07	686.00	685.76	685.42	685.08	684.73	684.45
1981-85	686.41	686.03	685.56	685.95	686.85	687.04	687.01	686.81	686.50	686.28	686.14	686.05
1986	684.46	684.08	683.47	683.86	684.95	686.09	686.07	685.70	685.31	684.85	684.27	684.25
1987	683.96	683.37	682.80	682.70	683.49	684.01	683.87	683.80	683.65	683.32	682.98	682.67
1988	682.33	681.95	682.29	684.38	685.46	685.90	685.97	685.77	685.55	685.25	684.94	684.73
1989	684.67	684.45	684.12	685.37	687.03	687.84	687.99	687.68	687.27	686.82	686.40	686.28
1990	686.21	685.80	686.34	688.75	689.71	689.36	689.12	688.93	688.70	688.46	688.36	688.25
1986-90	684.33	683.93	683.80	685.01	686.13	686.64	686.60	686.38	686.10	685.74	685.39	685.24
1991	687.94	687.57	687.06	686.92	687.42	687.63	687.41	687.09	686.74	686.27	685.68	685.49
1992	684.96	684.41	683.68	684.11	684.98	685.57	685.47	685.19	684.67	684.03	683.34	682.81
1993	682.91	683.00	682.96	682.96	683.17	683.31	683.51	683.68	683.71	683.76	683.74	683.69
1994	683.63	683.60	683.78	684.06	685.36	685.73	685.82	685.85	685.85	685.81	685.58	685.67
1995	685.30	684.85	684.77	685.47	686.58	687.46	687.25	686.76	686.29	685.85	685.29	684.81
1991-95	684.95	684.69	684.45	684.70	685.50	685.94	685.89	685.71	685.45	685.14	684.73	684.49
1996	684.35	684.08	683.77	684.82	686.29	687.19	687.28	686.97	686.64	686.22	685.66	685.14
1997	684.56	683.91	683.29	684.41								

Resource : Pangani Basin Water Office

Table J.8.1 Comparison Table for Alternative Plan of Water Source Facility(1/2)

Description	Alternative-A	Alternative-B	Alternative-C	Alternative-D	Alternative-E
1. Headworks					
1.1 Intake WL	EL.817	EL.815	EL.802	EL.761	EL.760
1.2 EL. of river bed	EL.815	EL.810	EL.789	EL.752	EL.730
1.3 Topography condition					
- river width(m)	12	15~20	15~20	45~50	60~70
- abutment slope	1:02	1:02	1:0.2~0.3	1:02	1:02
1.4 Geology condition					
- river bed	hard	hard	hard	hard	hard
- abutment	hard	hard	hard	hard	weathered
1.5 Diversion weir					
- height(m)	4	7	16	11	33
- length(m)	15	60	30	50	140
- excavation volume(m3)	4,500	11,500	13,000	45,000	32,000
- concrete volume(m3)	1,100	3,600	7,500	18,000	85,500
- diversion method	open channel	open channel	open channel	open channel	open channel
- workability	good	good	too steep slope	low(deep valley)	good
- estimated cost(US$)	649,920	1,777,200	2,934,000	7,214,400	36,252, 000
1.6 Evaluation					
- advantage	low cost	good workability	stable foundation	less	less
- disadvantage	less	length of weir	low workability	high cost	high cost
2. Diversion Channel					
2.1 High land portion					
2.1.1 Topography	EL.850 to 760	EL.840 to 760	EL.840 to 760	EL.770 to 755	EL.770 to 755
2.1.2 Geology	hard	hard	hard	seam and disturbed	seam and disturbed
2.1.3 River crossing	2 places	1 place	1 place	2 places	2 places
2.1.4 Ground water level	low	low	low	low	low
2.1.5 Dimension					
- length(km)	10	16	14	16	12
- bottom width(m)	2	2	2	2	2
- height(m)	2.7	2.7	2.7	2.7	2.7
- side slope	1:0.25	1:0.25	1:0.25	1:0.25	1:0.25
- velocity(m/s)	1.53	1.53	1.53	1.53	1.53
2.1.6 Workability					
- depth of excavation(m)	15(average)	5~10(max. 20)	5~10(max. 15)	5~8	5~8
- hardness of bedrock	hard	hard	hard	weathered	weathered

(to be continued)

Table J.8.1 Comparison Table for Alternative Plan of Water Source Facility(2/2)

Description	Alternative-A	Alternative-B	Alternative-C	Alternative-D	Alternative-E
2.1.7 Work quantity					
- excavation, rock(m3)	1,350,000	988,000	340,000	2,214,000	261,000
- excavation, soil(m3)	27,000	12,000	8,000	44,000	52,000
- lining concrete(m3)	6,500	10,400	9,500	10,400	7,800
2.1.8 Estimated cost(US$)	34,356,000	26,654,000	10,830,000	56,270,400	8,728,800
2.1.9 Evaluation					
- advantage	less	less	low cost	less	low cost
- disadvantage	high cost	deep excavation	less	too high cost	low workability
2.2 Low land portion	(All alternative plans are same route and conditions)				
2.2.1 Topography	EL.755~740				
2.2.2 Geology	Alluvial soil/stiff				
2.2.3 River crossing	2 places				
2.2.4 Ground water level	generaly low, 3 km from beginning of lowland portion is same level of ground				
2.2.5 Dimension					
- length(km)	12				
- bottom width(m)	1.7				
- height(m)	2.2				
- side slope	1:1.0				
- velocity(m/s)	1.42				
2.2.6 Workability					
- depth of excavation(m)	4~5				
- hardness of soil layer	N>30				
2.2.7 Work quantity					
- excavation, rock(m3)	5,000				
- excavation, soil(m3)	80,000				
- embankment(m3)	30,000				
- lining concrete(m3)	10,800				
- estimated cost(US$)	3,278,400				
2.2.8 Evaluation	good				
2.3 Overall channel					
2.3.1 Estimated cost(US$)	37,634,400	29,932,800	14,108,400	59,548,800	12,007,200
2.3.2 Overall evaluation	high cost	high cost	low cost	too high cost	low workability
3. Evaluation as Water Source Facility					
3.1 Topography	good	good	good	steep at high land	steep at high land
3.2 Geology	hard & stiff	hard & stiff	hard & stiff	seam and weathered	seam and weathered
3.3 Workability	good	good	good	low	low
3.4 Construction cost(US$)	38,284,000	31,710,000	17,042,000	66,763,000	48,259,000
3.5 Ranking	3	2	1	5	4

Table J.8.2 Preliminary Cost Estimate for Route A, B, C, D and E

	Work Item	Unit Price	C-1 Intake EL. EL.806		C-2 Intake EL. EL.808		C-3 Intake EL. EL.810		C-4 Intake EL. EL.812		C-5 Intake EL. EL.814	
		(Yen)	Working Volume	Amount (Yen)	Working Volume	Amount (Yen)	Working Volume	Amount (Yen)	Working Volume	Amount (Yen)	Working Volume	Amount (Yen)
1.	**Headworks**											
1-1	Rock Excavation (HW)	3,600	27,000 (m^3)	97,200,000	22,000 (m^3)	79,200,000	18,000 (m^3)	64,800,000	16,000 (m^3)	57,600,000	11,000 (m^3)	39,600,000
1-2	Concrete (HW)	19,000	8,000 (m^3)	152,000,000	10,000 (m^3)	190,000,000	11,000 (m^3)	209,000,000	13,000 (m^3)	247,000,000	16,000 (m^3)	304,000,000
1-3	Form (HW)	1,500	24,000 (m^2)	36,000,000	30,000 (m^2)	45,000,000	33,000 (m^2)	49,500,000	39,000 (m^2)	58,500,000	48,000 (m^2)	72,000,000
1-4	Reinforcement Bar (HW)	96,000	320 (ton)	30,720,000	400 (ton)	38,400,000	440 (ton)	42,240,000	520 (ton)	49,920,000	640 (ton)	61,440,000
1-5	Others	(20% of above)		63,184,000		70,520,000		73,108,000		82,604,000		95,408,000
	Sub-total 1			379,104,000		423,120,000		438,648,000		495,624,000		572,448,000
2.	**Head Race Canal**											
2-1	High Land Portion											
2-1-1	Rock Excavation	2,400	290,000 (m^3)	696,000,000	236,000 (m^3)	566,400,000	222,000 (m^3)	532,800,000	180,000 (m^3)	432,000,000	145,000 (m^3)	348,000,000
2-1-2	Soil Excavation	500	28,500 (m^3)	14,250,000	36,000 (m^3)	18,000,000	28,860 (m^3)	14,430,000	29,000 (m^3)	14,500,000	29,000 (m^3)	14,500,000
2-1-3	Soil Embankment	800	39,000 (m^3)	31,200,000	30,000 (m^3)	24,000,000	21,000 (m^3)	16,800,000	23,000 (m^3)	18,400,000	40,000 (m^3)	32,000,000
	Gravel Pavement	1,200	4,400 (m^3)	5,280,000	4,400 (m^3)	5,280,000	3,500 (m^3)	4,200,000	3,700 (m^3)	4,440,000	3,700 (m^3)	4,440,000
2-1-4	Shotcreat Concrete	27,000	8,900 (m^3)	240,300,000	9,000 (m^3)	243,000,000	7,000 (m^3)	189,000,000	7,000 (m^3)	189,000,000	7,000 (m^3)	189,000,000
2-1-5	Structures and Others	(20% of above)		197,406,000		171,336,000		151,446,000		131,668,000		117,588,000
	Sub-total 2.1			1,184,436,000		1,028,016,000		908,676,000		790,008,000		705,528,000
2-2	Low Land Portion											
2-2-1	Rock Excavation	2,400	5,000 (m^3)	12,000,000	5,000 (m^3)	12,000,000	5,000 (m^3)	12,000,000	5,000 (m^3)	12,000,000	5,000 (m3)	12,000,000
2-2-2	Soil Excavation	500	124,000 (m^3)	62,000,000	124,000 (m^3)	62,000,000	124,000 (m^3)	62,000,000	124,000 (m^3)	62,000,000	124,000 (m3)	62,000,000
2-2-3	Embankment	800	37,000 (m^3)	29,600,000	37,000 (m^3)	29,600,000	37,000 (m^3)	29,600,000	37,000 (m^3)	29,600,000	37,000 (m3)	29,600,000
	Gravel Pavement	1,200	2,700 (m^3)	3,240,000	2,700	3,240,000	2,700 (m^3)	3,240,000	2,700 (m^3)	3,240,000	2,700 (m3)	3,240,000
2-2-4	Lining Concrete	18,000	9,500 (m^3)	171,000,000	9,500 (m^3)	171,000,000	9,500 (m^3)	171,000,000	9,500 (m^3)	171,000,000	9,500 (m3)	171,000,000
2-2-5	Structures and Others	(20% of above)		55,568,000		55,568,000		55,568,000		55,568,000		55,568,000
	Sub-total 2.2			333,408,000		333,408,000		333,408,000		333,408,000		333,408,000
	Su-total 2			1,517,844,000		1,361,424,000		1,242,084,000		1,123,416,000		1,038,936,000
Ground Total				**1,897,000,000**		**1,785,000,000**		**1,681,000,000**		**1,620,000,000**		**1,611,000,000**

Table J.9.1 Calculation Sheet for Stability of Headworks Weir

Load	Mark	Force		Arm length		Moment
		Formula	Force(tf)	Formula	Length(m)	(tf.m/m)
1. Vertical Load						
Self-weight	W1	0.5 x 28.0 x 3.36 x 2.3	108.19	1/3 x 3.36 + 22.48	23.60	2,553.33
	W2	0.5 x 28.0 x 22.4 x 2.3	721.28	2/3 x 22.40	14.93	10,771.11
Water weight	Ww1	0.5 x 25.0 x 3.00 x 1.00	37.50	25.76 - 1/3 x 3	24.76	928.50
Sediment	We1	0.5 x 25.0 x 3.00 x 1.00	37.50	25.76 - 1/3 x 3	24.76	928.50
Up-lift	U	-0.5 x 8.333 x 25.76	-107.33	2/3 x 25.76	17.17	-1,843.20
I. Total			**797.14**			**MR=13,338.25**
II. Horizontal Load						
Dynamic load by seismic	P1	108.19 x 0.12	12.98	1/3 x 28.00	9.33	121.17
	P2	721.28 x 0.12	86.55	1/3 x 28.00	9.33	807.83
Static water pressure	Ps	0.5 x 25.0 x 25.0 x 1.00	312.50	1/3 x 25.00	8.33	2,604.17
Sediment pre pressure	Pe	0.5 x 0.6 x 25.0 x 25.0 x 1.00	187.50	1/3 x 25.00	8.33	1,562.50
Dynamic water pressure	Pd	0.875 x 0.12 x 25.0 x 25.0	65.63	2/5 x 25.00	10.00	656.25
II. Total			**665.16**			**MT=5,751.92**

Table J.10.1 List of Related Structures of Diversion Channel

	Name of Structure	Location / Number	Dimension / Scale
1.	**Cross Section of Channel**		
(a)	High land	BP - 11,650m, l=11,650m	B=2.0m, H=2.6m, 1:m=1:0.3, shotcrete lining
(b)	Low land-I	11,868m - 18,200m, l=6,400m	B=2.0m, H=2.0m, 1:m=1:1.25, precast concrete block lining
(c)	Low land-II	18,200m - 24,484m, l=6,284m	B=2.0m, H=1.9m, 1:m=1:1.25, precast concrete block lining
2.	**Major Related Structures**		
(a)	Measuring Device	BP of Diversion Channel	12feet type Parshall flume
(b)	No.1 Chute	BP+5,350m	B=3.00m, L=200.00m.
(c)	Longoi Syphon	BP+7,550m	Concrete & Steel pipe Combination Type
	-do-		B x H=2.40m x 2.40m,D=2.20m, L=130m
(d)	Longoi Bridge	BP+7,550m	B=4.00m, L=55.00m I=1/16
(e)	No.2 Chute	BP+11,650m	B=3.00m, L=150.00m, I=1/4
(f)	Kikafu Wasteway	BP+13,450m	B=2.00-3.00m, L=150.00m, 12feet Parshall flume is Provided
	-do-		12feet type Parshall flume is Provided
(g)	Kikafu Syphon	BP+13,880m	Concrete Box Culvert
	-do-		B x H=2.40m x 2.40m, L=36.00m
(h)	Kikafu Bridge	BP+13,880m	B=4.00m, L=48.00m
(i)	Weruweru Aqueduct	BP+16,169m	B=3.50m, H=2.10m, L=60.00m
(j)	Weruweru Bridge	BP+16,169m	B=4.00m, L=60.00m
(k)	Bifurcation	BP+20,560m	Q1=5.30m3/sec, Q2=3.70m3/sec
	-do-		12feet type Parshall Flume is Provided
	-do-		10feet type Parshall Flume is Provided
3.	**Other Related Structures**		
(a)	Cross Drain	15places	Concrete Pipe
	-do-		(d=600mm x 1row):1no.
	-do-		(d=800mm x 1-4row):14nos.
(b)	Box Culvert	16places	B x H=2.60m x 3.00m :1no.
	-do-		B x H=2.10m x 3.50m :15nos.
(c)	Foot Path Bridge	32places	High land portion :12nos.
	-do-		Low land portion :20nos.
(d)	Drop	7places	H=1.0m and 1.5m

Figures

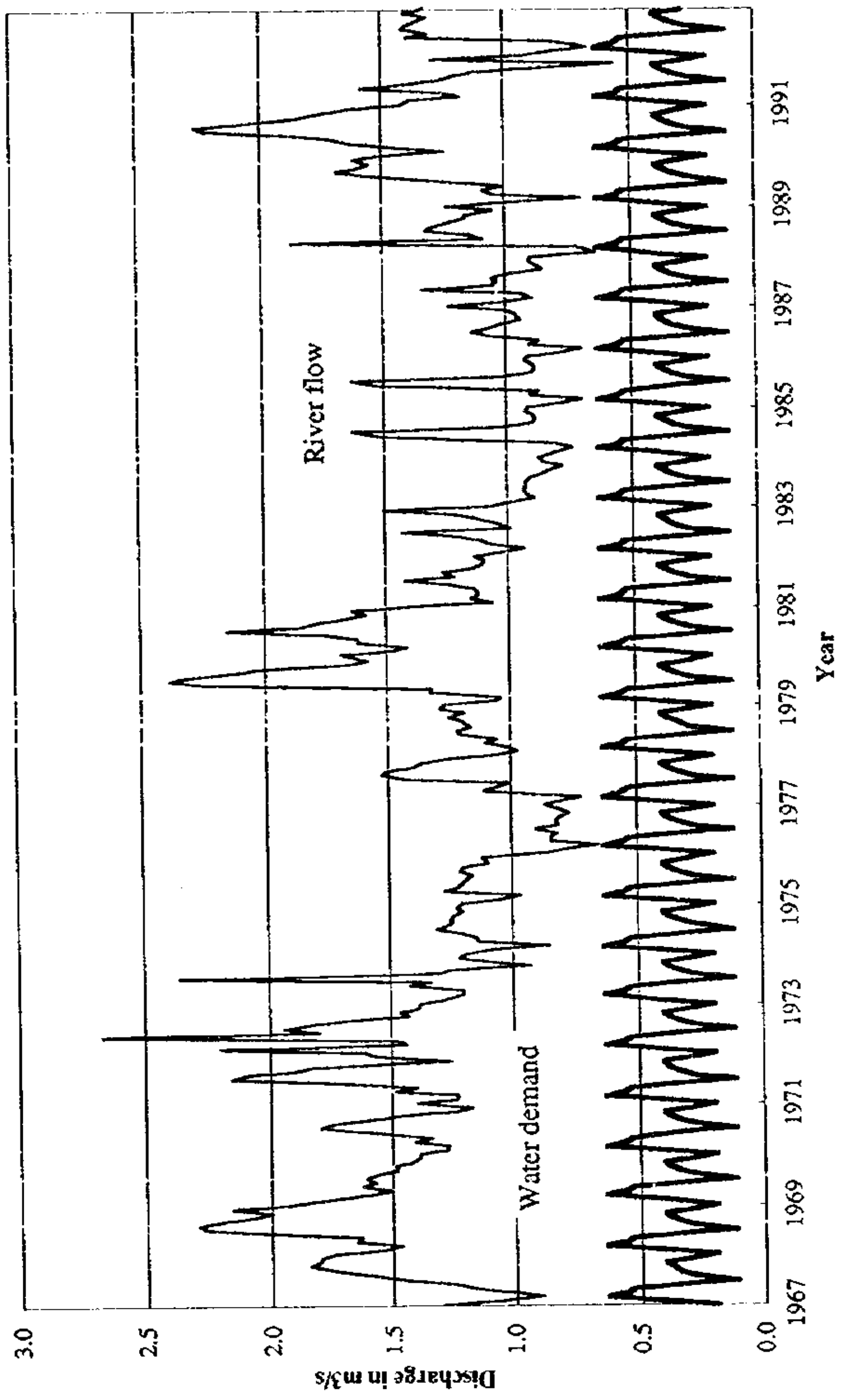


Figure J.5.1 **Water Balance Study** **at the Mabogini Intake Weir**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania** Japan International Cooperation Agency

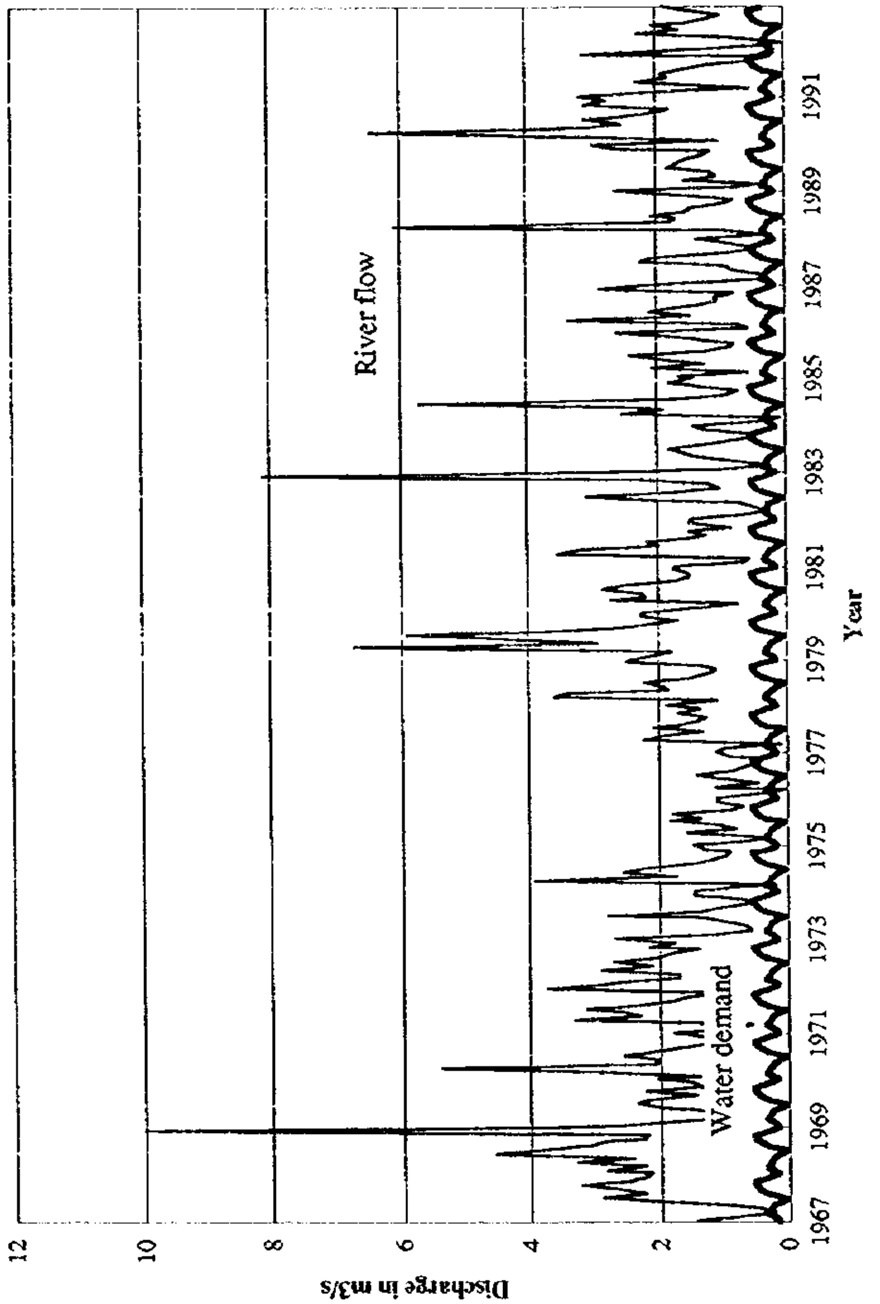


Figure J.5.2 **Water Balance Study** **at the Rau Ya Kati Intake Weir**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania** Japan International Cooperation Agency

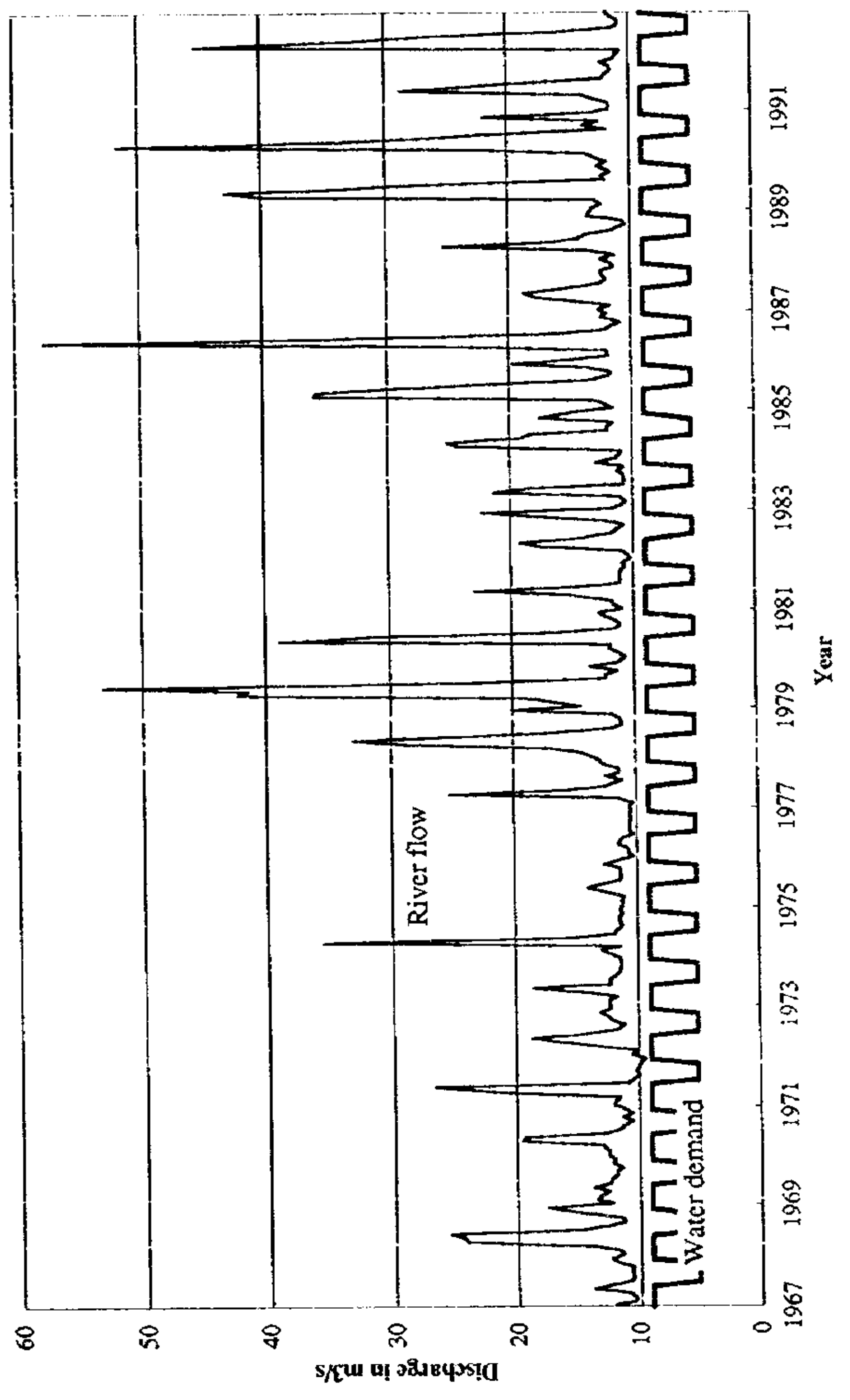


Figure J.5.3 **Water Balance Study** **at the Kikuletwa Intake Weir**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania** Japan International Cooperation Agency

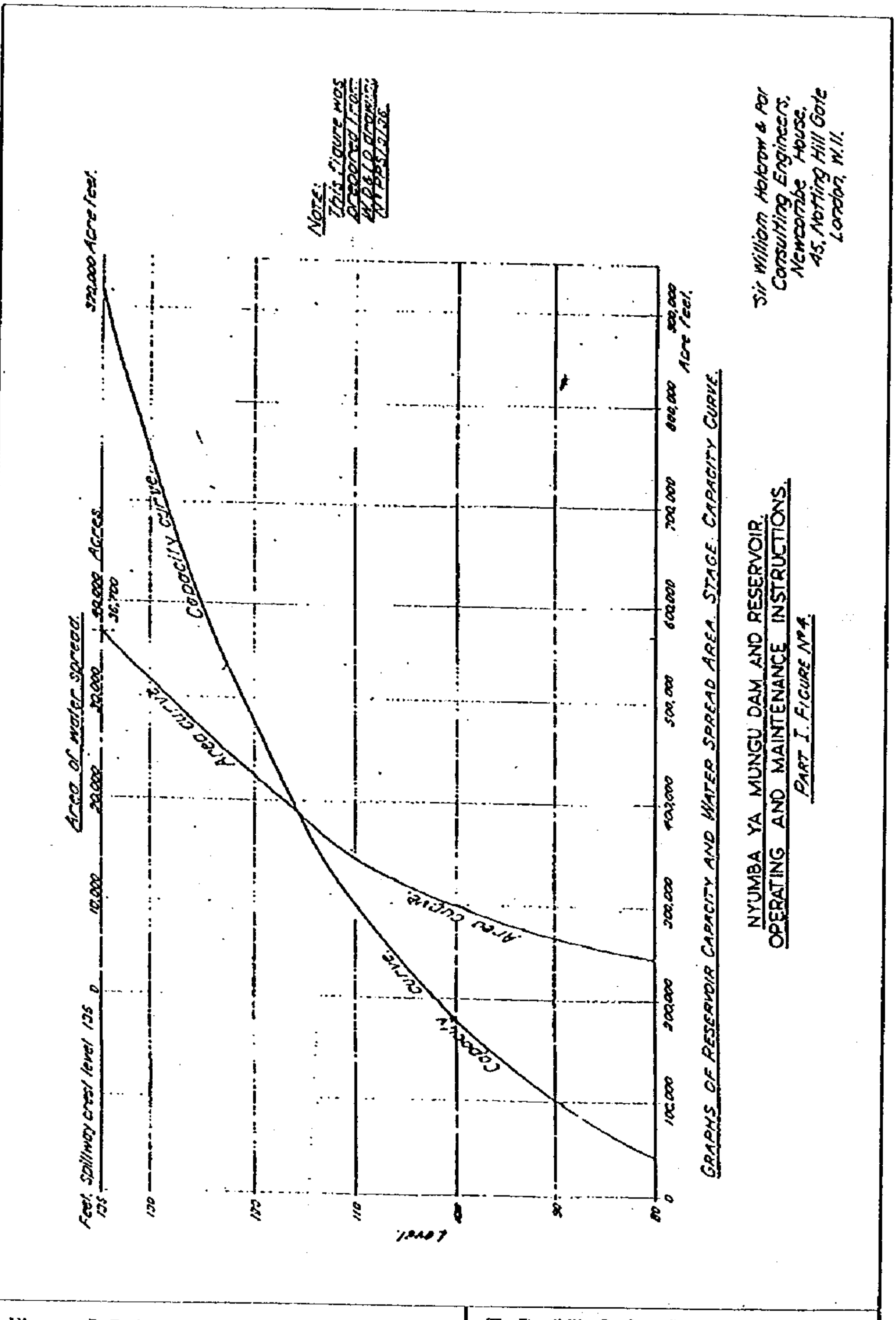


Figure J.7.1 **H-Q-A Curve of Nyumba Ya Mungu Reservoir**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania** Japan International Cooperation Agency

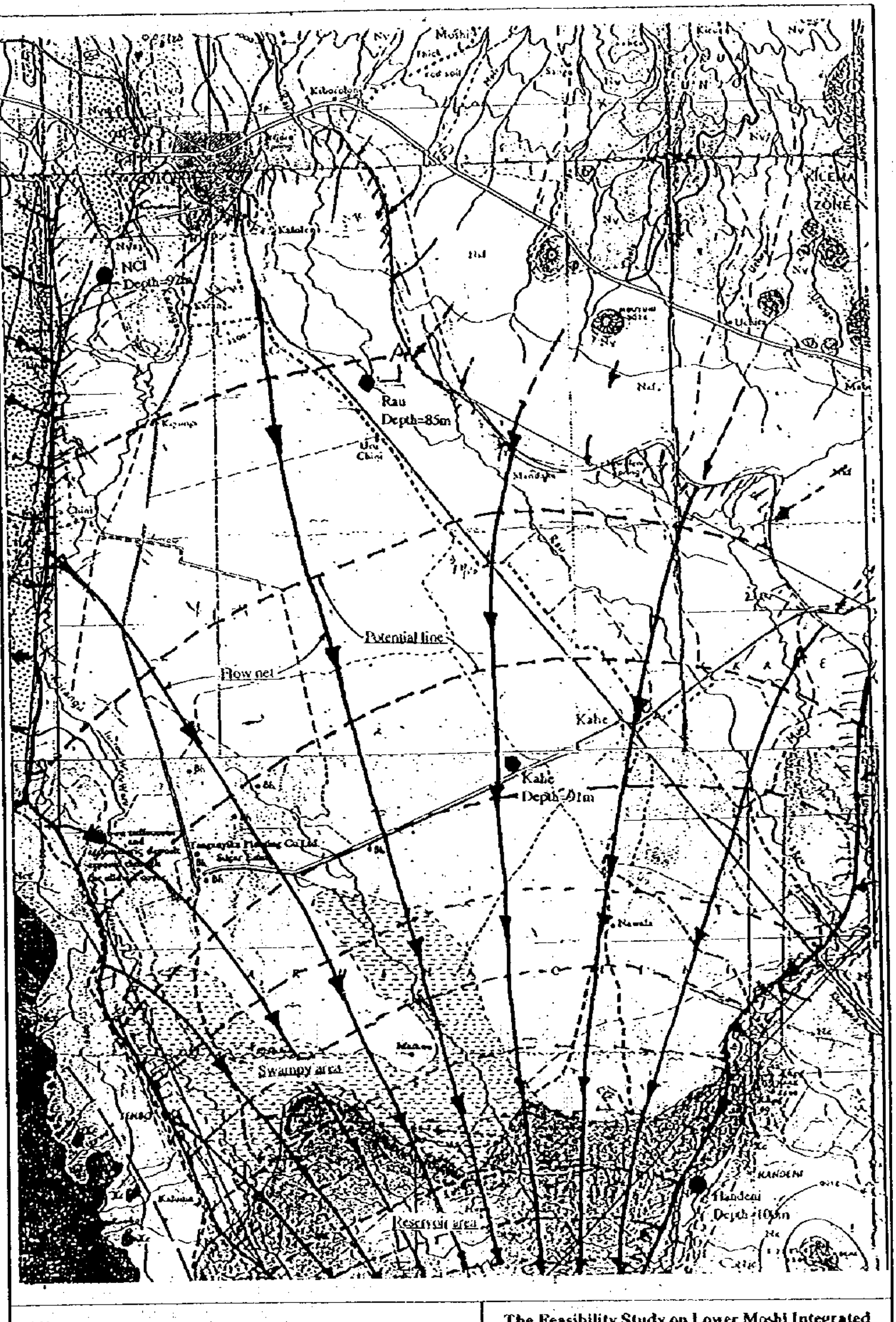


Figure J.7.2

General Hydrogeotectonic Plan of Lower Moshi Alluvial Fan

The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania

Japan International Cooperation Agency

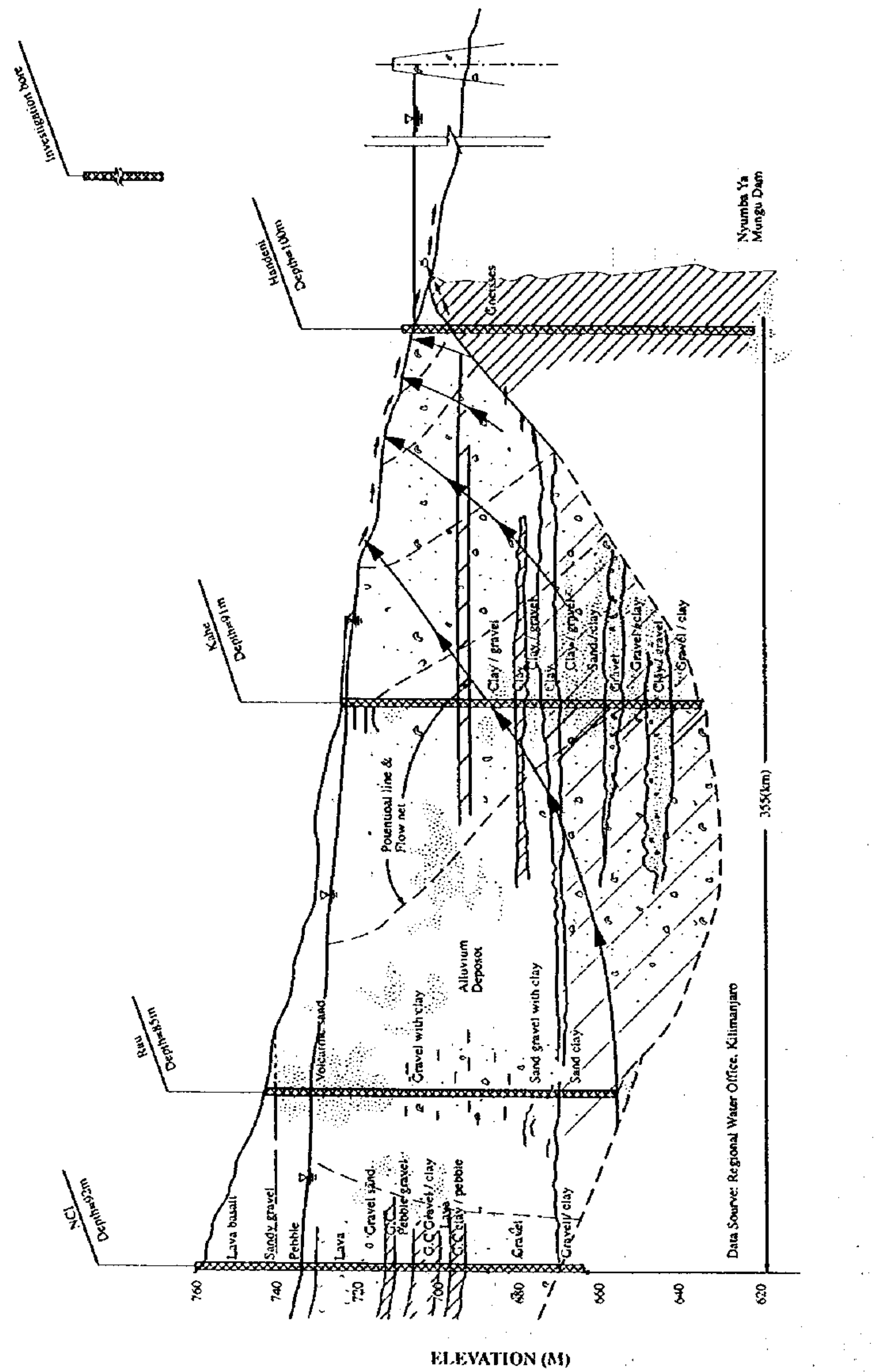
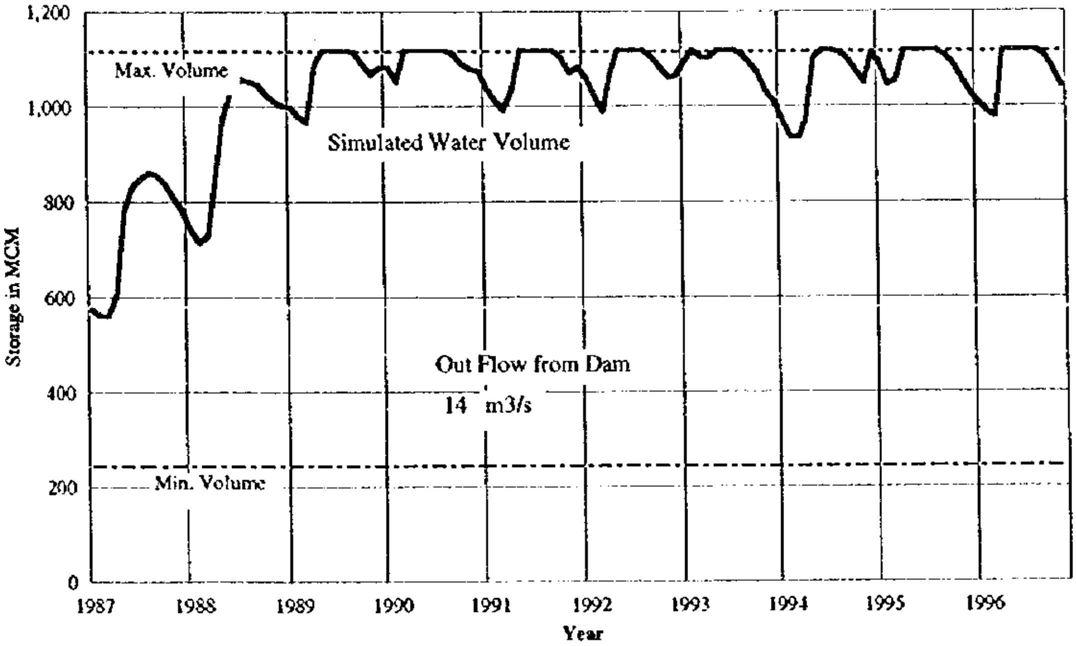
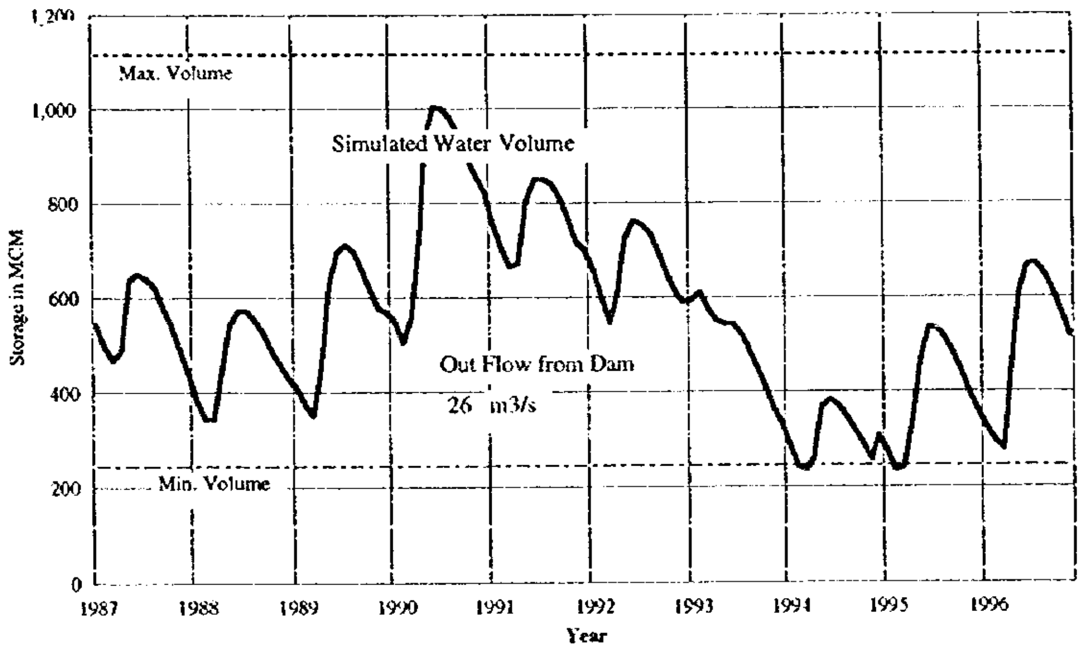


Figure J.7.3 General Hydrogeotectonic Profile of Lower Moshi Alluvial Fan	The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania
	Japan International Cooperation Agency

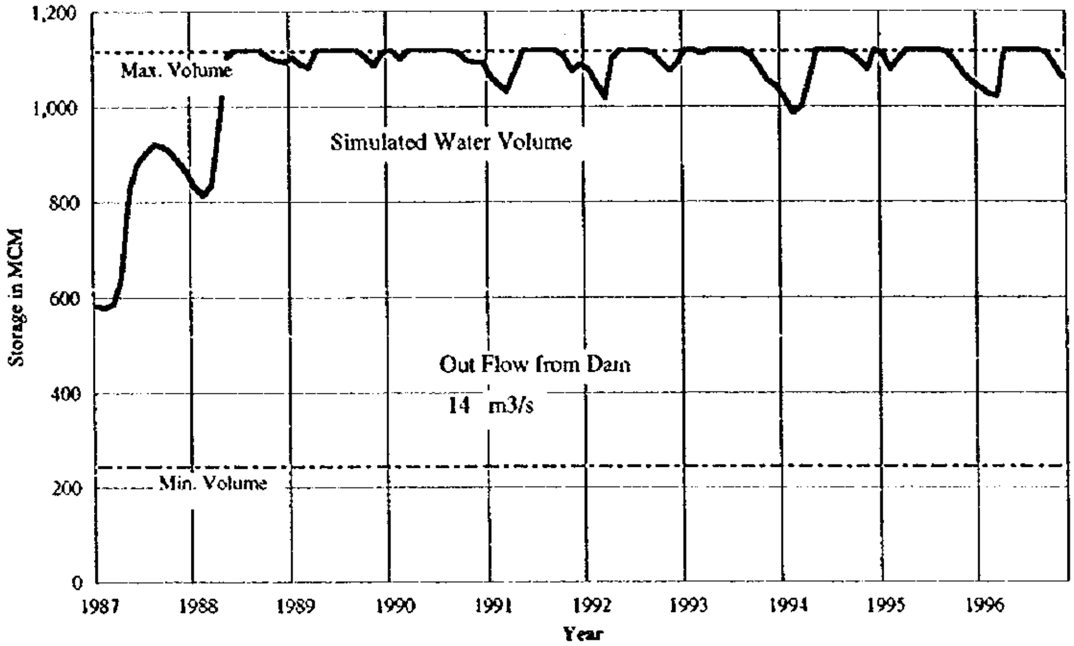


Nyumba Ya Mungu Reservoir Operation
Outflow from Reservoir : 14 m3/s

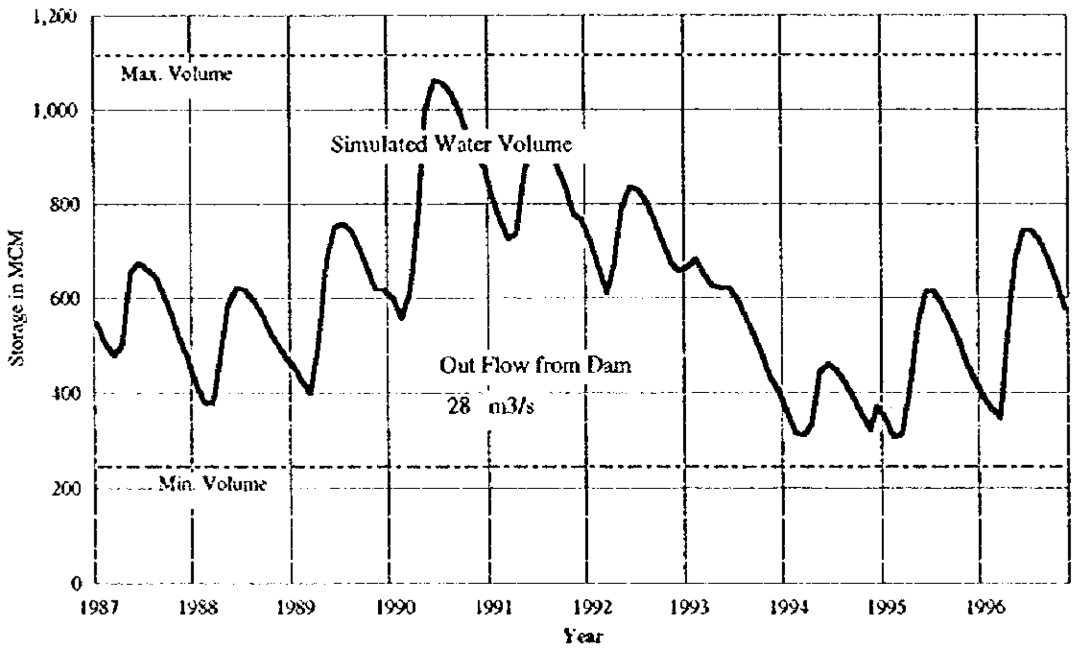


Nyumba Ya Mungu Reservoir Operation
Outflow from Reservoir : 26 m3/s

Figure J.7.4 **Nyumba Ya Mungu Reservoir Operation with the Project**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania** Japan International Cooperation Agency



Nyumba Ya Mungu Reservoir Operation
Outflow from Reservoir : 14 m3/s



Nyumba Ya Mungu Reservoir Operation
Outflow from Reservoir : 28 m3/s

Figure J.7.5 **Nyumba Ya Mungu Reservoir Operation without the Project**	**The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania**
	Japan International Cooperation Agency

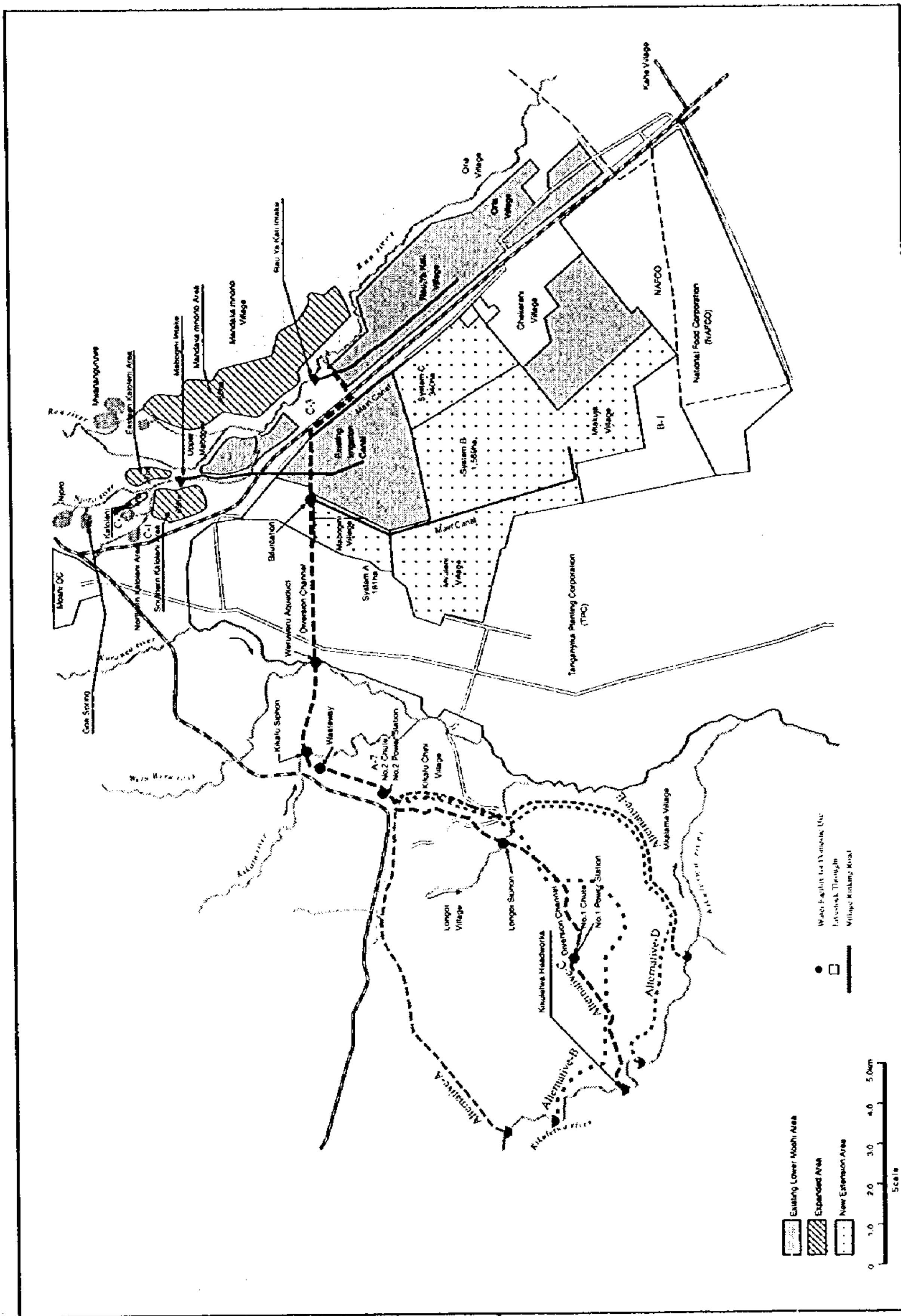


Figure J.8.1
Location and Routes of the Alternative Plan

The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania

Japan International Cooperation Agency

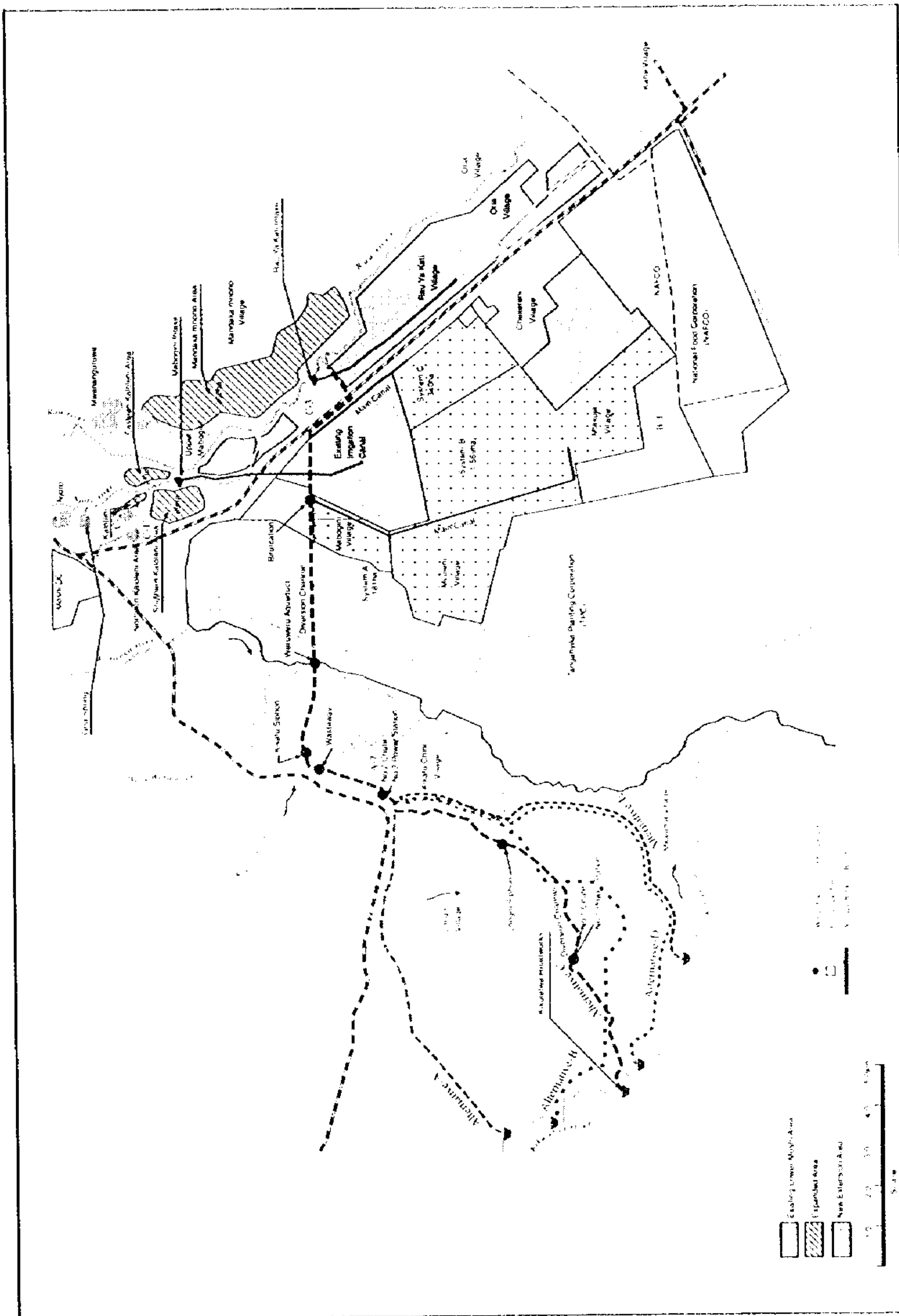


Figure J.8.1
Location and Routes of the Alternative Plan

The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania

Japan International Cooperation Agency

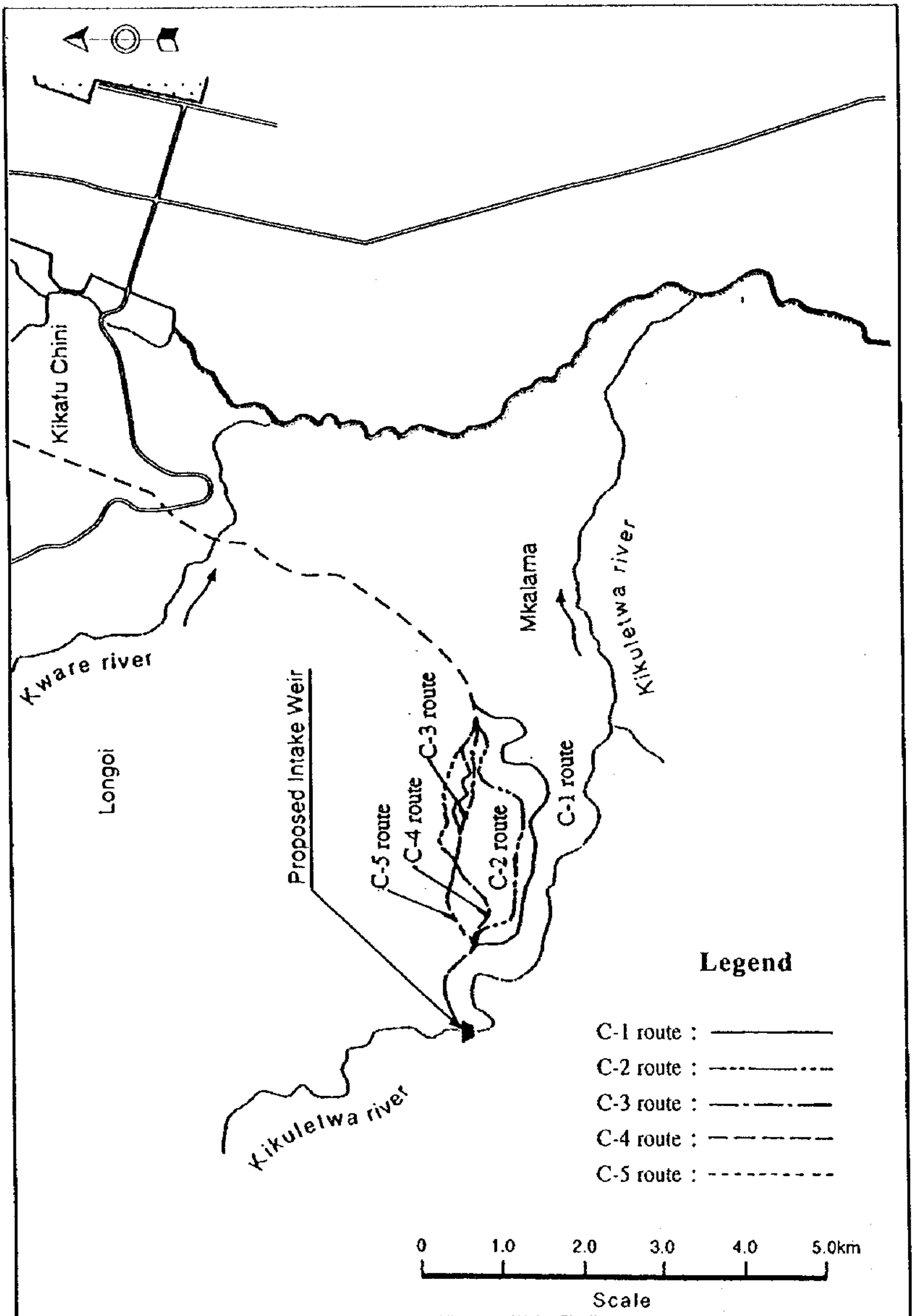


Figure J.8.2
Route for Respective Weir Heights

The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania

Japan International Cooperation Agency

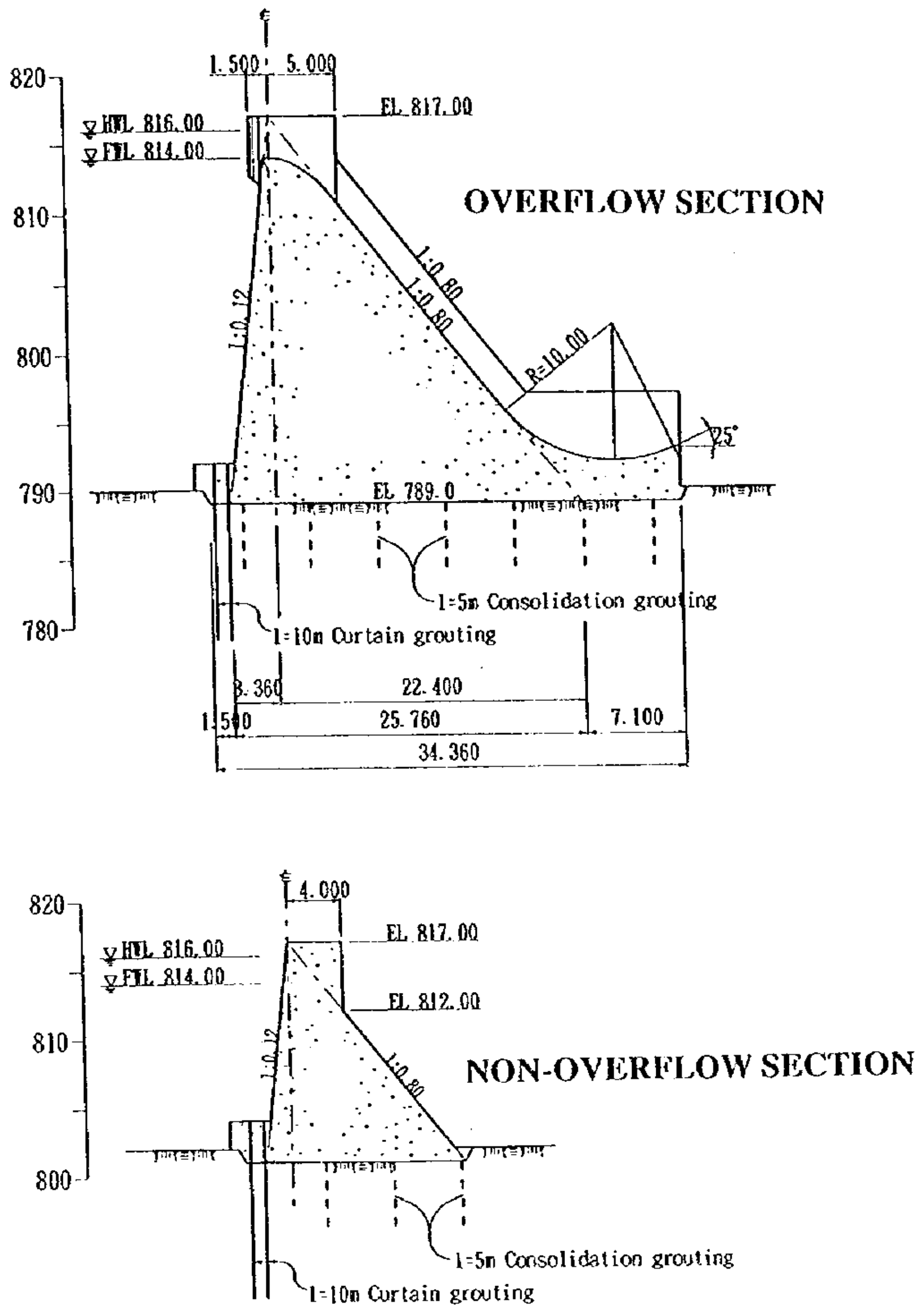


Figure J.9.1
Typical Cross Section of Diversion Weir

The Feasibility Study on Lower Moshi Integrated Agriculture and Rural Development Project in the United Republic of Tanzania

Japan International Cooperation Agency