SOMALI DEMOCRATIC REPUBLIC
SETTLEMENT DEVELOPMENT AGENCY

HOMBOY IRRIGATED SETTLEMENT PROJECT

Design Criteria Irrigation and Drainage Works

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INTRODUCTION

1.1 General

This note describes the design criteria used in the preparation of tender documents for Contract Nr H1, and Nominated Sub-Contracts Nr H1/1 and H1/2 of the Homboy Irrigated Settlement Project, Somalia.

1.2 Background

In July 1978 the Settlement Development Agency of the Somali Government commissioned Hunting Technical Services Ltd. to carry out a Phase I Reconnaissance Study for the identification of 24 000 ha of land suitable for agricultural development. Of this, some 16 000 ha in the Homboy - Burgaan area was considered the most favourable for irrigation, and this was studied in more detail to investigate the feasibility of a 9 000 ha net irrigation project.

The Reconnaissance Report was submitted in December 1978 and Phase II of the Study comprising the preparation of reports covering soils, agriculture, groundwater, villagisation and relocation, and a pre-design engineering survey was completed in October 1979.

Sir M. MacDonald and Partners were requested by Hunting Technical Services Ltd to carry out the designs for about 9 000 ha net of irrigated land in accordance with the Terms of Reference agreed with the Settlement Development Agency. Design work started at the beginning of August 1979 and the tender documents were completed in January 1980.

1.3 Brief Description of the Project

The project area is located on the eastern side of the River Jubba and stretches north-east from Kamsuuma to the new Jilib/Golweyn road, a distance of approximately 30 kilometres. The gross project area is some 14 200 ha.

The project area will be split up into ten irrigation blocks and associated with each block will be built a new village to house the settlers working on the scheme. The villages have been located with regard to favourable soil conditions, ease of access and minimum walking distance to the fields of the irrigation blocks they serve.

The soils of the project area are predominantly fine textured Shabeelle alluvium deposited by flooding from the Shabeelle river and the Harar Naga and Kormajirto depressions that run through the area. This strip of land is relatively narrow and is bounded on either side by very fine textured and saline Marine Plain soils. These are considered unsuitable for either irrigated or rainfed agriculture and thus development has been restricted to the Shabeelle alluvium plus small areas of channel and beach remnant soils.

Mixed cropping will be carried out on most of the project, although certain areas which are low lying or exhibit poor drainage characteristics have been identified as suitable for paddy rice only. In addition, areas unsuitable for irrigation due to topographic reasons have been marked as areas for rainfed cultivation.

Details of the ten block areas are given in Table 1.1

TABLE 1.1

Details of Block Areas

Block Nr	Mixed crop (ha net)	Paddy rice (ha net)	Total irrigated (ha net)	Rainfed (ha net)	Total (ha net)
1	550	50	600	40	640
2	775	50	825	150	975
3	675	25	700	235	935
4	925	0	925	315	1 240
5	1 025	0	1 025	220	1 245
6	700	100	800	485	1 285
7	1 125	0	1 125	360	1 485
8	550	25	575	360	935
9	1 025	500	1 525	195	1 720
10	550	200	750	315	1 065
TOTAL	7 900	950	8 850	2 675	11 525

The works covered by this design note include:

(a) flood protection works

(b) offtake from the Fanoole Main Canal, approximately 15 km of Supply Canal, 21 km of Main Canal and 9 km of branch canals

(c) complete irrigation and drainage works for about 8 850 ha net of agricultural land

(d) surfaced roads.

CHAPTER 2

FLOOD PROTECTION

2.1 Introduction

To protect the project area from flooding from the Shabeelle River and Harar Naga depressions, flood prevention works are necessary, and an analysis was carried out to determine the required form of those works. The analysis comprised three stages:-

(a) Determination of flood hydrographs from rainfall records

(b) Flood routing through the proposed works

(c) Historical simulation of the proposed operational procedures for the works.

These three stages are now described.

2.2 Determination of Flood Hydrographs

The available hydrologic records consisted solely of daily rainfall records from three stations in the area, namely Alessandra, Bardheere and Kismaayo. The records of these stations were combined to form an estimate of the 24 hour point rainfall over the catchment. To determine the return period associated with any particular rainfall the annual maximum 1 day, 3 day and 5 day rainfalls were ranked and plotted on a Gumbel Distribution to yield the results given in Table 2.1.

TABLE 2.1

Maximum Rainfall

Rainfall return period (years)	Flood peak return period (years)	1 day rainfall (mm)	3 day rainfall (mm)	5 day rainfall (mm)
1 in 35	1 in 20	132	174	209
73	50	145	194	
1 in 140	1 in 100	163	211	252
1 in 1 000	1 in 1 000	204	263	312

The flood peak return periods associated with the rainfall return periods have been determined from the Flood Studies Report Vol. I Hydrological Studies - Natural Environment Research Council 1975. As no run-off records were available, the run-off volumes and hydrographs were determined by the method developed by the US Soil Conservation Service. This method utilises an empirical relationship between rainfall and run-off which depends on soil type, land use and treatment and antecedent moisture condition. The method also incorporates the use of a synthetic unit hydrograph which is derived from the catchment characteristics of length and slope.

The catchment was sub-divided into seven hydrologically independent sub-catchments and the US Soil Conservation Service method applied to each one. The resultant hydrographs were routed to the outflow point of the overall catchment by determining the time of travel and allowing for flow attenuation due to seepage and evaporation in the river channels.

It was found that a 3 day storm was the critical one both for run-off volume and peak flow rate.

2.3 Flood Routing

To protect the irrigation area from flooding, the flood flows must be either contained within a diversion canal or stored behind a flood bund. In this case storage is the only feasible solution, as the cost for a diversion channel to take the maximum 1 in 1000 year run-off of 665 m^3/s (see Table 2.2) would be prohibitive. Two suitable storage areas were selected (the Northern and Eastern Reservoirs) and their associated stage-storage characteristics determined from contour plans of the area as shown on Figure 2.1. This includes the curve for the Southern Reservoir which will be used to store internal drainage water as described in Section 3.10.

A computer program was written to test the response of the storage system to the critical storms for the three return periods: 20, 100 and 1 000 years. The program determined the instantaneous outflow from, and storage volume in, the Northern Reservoir for a given type of outflow structure. Several computer runs were done to determine the optimum sizing of the reservoirs and the required outflow structure. It was found that a system of gated culverts provided a flexible system of control which allowed the introduction of a simple operational procedure for controlling all the critical storms.

The results are summarised in Table 2.2.

TABLE 2.2

Northern Reservoir Flood Routing

Flood return (years)	Run-off volume (m^3x10^6)	Maximum inflow (m^3/s)	Maximum outflow (m^3/s)	Stored volume (m^3x10^6)	Maximum water level of Northern Reservoir (m)	Outflow volume (m^3x10^6)	Maximum water level of Eastern Reservoir (m)
1 in 20	132	346	140	68	21.7	64	17.4
1 in 100	175	479	154	105	22.3	70	17.5
1 in 1 000	248	665	331	117	22.8	131	18.5

The inflow/outflow hydrograph for the 1 in 1 000 year flood is shown in more detail in Figure 2.2.

RESERVOIR STAGE–STORAGE CURVES

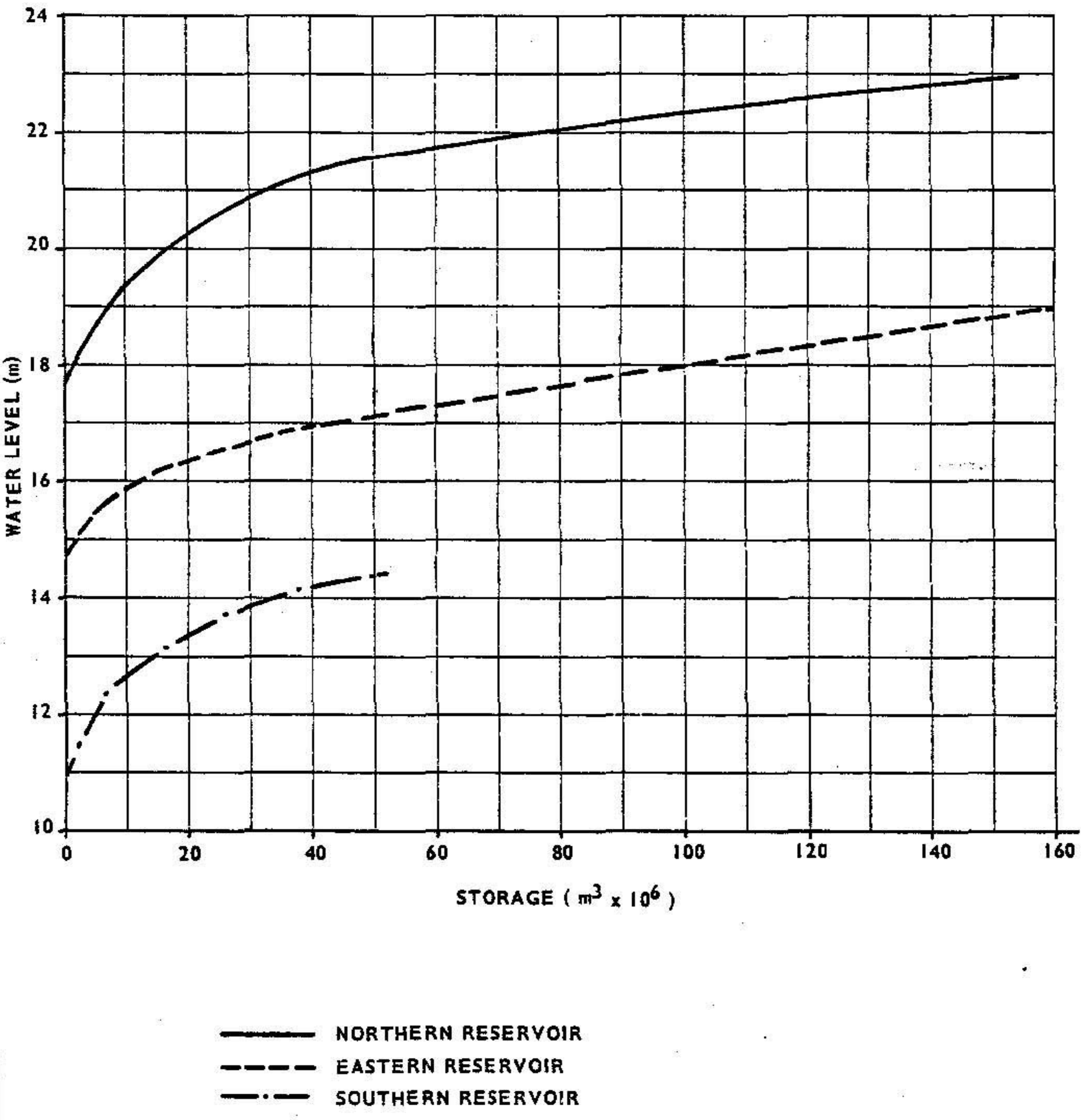
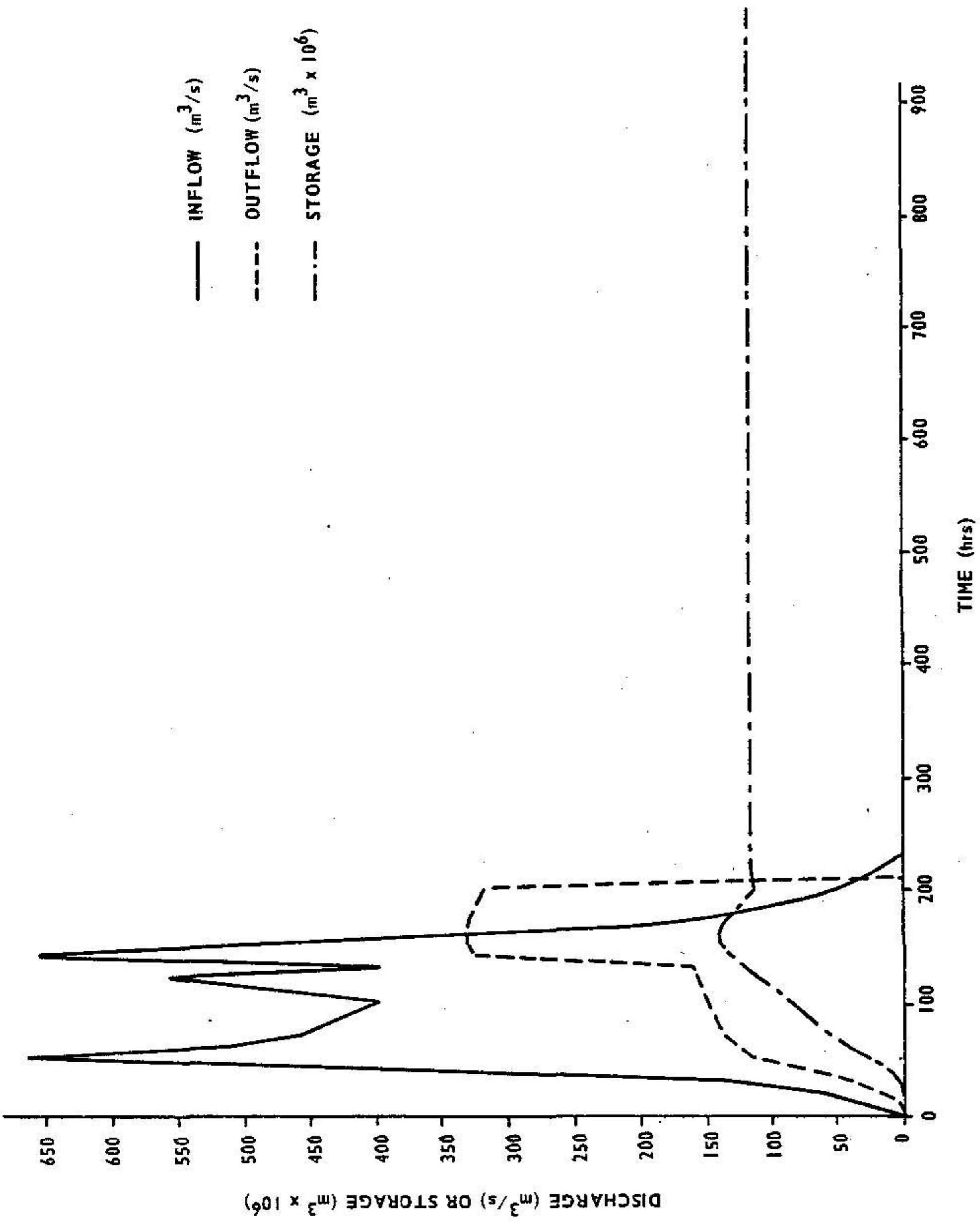


FIGURE 2.2

HYDROGRAPH FOR 1 IN 1000 YEAR FLOOD AT NORTHERN RESERVOIR



2.4 Historical Simulation

The flood routing programme is only applicable to the simulation of one theoretically determined storm. To verify that the flood protection works would successfully operate for a succession of real storms, a second computer program was written. This program operates with a historical record of daily rainfall and river flow in the Jubba River. Concurrent records of rainfall and river flow in the Jubba were available for the period 1953 to 1959 and this contains the year (1957) in which the maximum 3 day rainfall of the entire record (1930 to 1939, 1953 to 1960) occurred. The program calculates daily run-off volumes from the Shabeelle catchment and routes them through the Northern and Eastern Reservoirs, the stored water being discharged into the Jubba via the Lower Outfall Drain at a maximum rate of 10 m^3/s. This can only be achieved when the stage in the Jubba river is equivalent to a level not higher than 11.7 m at Kamsuuma bridge, the corresponding flow in the Jubba being 150 m^3/s. This generally only occurs during January, February, March and December, and during the rest of the year all run-off must be stored in the Eastern and Northern Reservoirs. The program simulates this condition and also calculates losses due to evaporation which is dependent on the surface area of water in the two reservoirs.

The results of the historical simulation model demonstrated that the flood protection works successfully stored all run-off from the Shabeelle catchment and discharged it into the Jubba River at times of low flow without ever exceeding the maximum storage volumes available in the two reservoirs.

For the critical year of 1957 the two reservoirs taken together were never more than 70% full. This demonstrates that the flood protection works are capable of routing a succession of extreme storm events even if they occur when the two reservoirs are not empty and it is not possble to discharge water into the River Jubba.

A simplified summary of the required operational procedure is as follows:-

(a) The Eastern Reservoir Outlet Structure is to be open whenever discharge into the Jubba River is possible.

(b) The Eastern Reservoir should normally be maintained at a maximum level of 17.5 m to facilitate emptying of the Northern Reservoir. If the water level in the Northern Reservoir exceeds 22.5 m, water must be released to the Eastern Reservoir until its maximum level of 18.5 m is reached. The maximum design water level in the Northern Reservoir is 22.5 m, although this may rise to 22.8 m under the 1 in 1 000 year storm conditions.

2.5 Use of Flood Flows for Irrigation

The use of flood water for irrigation purposes was examined in some detail. It was assumed that supplies would be taken from the Eastern Reservoir, since according to the flood protection operational procedure this contains water more often than the Northern Reservoir.

Computer simulation studies were carried out for the years 1953 to 1959 for which adequate data were available. It was assumed that the average daily irrigation requirement was 7 m^3/s and the table below gives the number of days in each year when it was possible to take water from the Eastern Reservoir for irrigation purposes.

TABLE 2.3

Use of Flood Flows for Irrigation

Year	Number of days
1953	22
1954	25
1955	83
1956	127
1957	238
1958	63
1959	117

As can be seen from the table, supplies are not reliable enough to consider using them for irrigation. In addition a pump station and link canal would be required to lift water from the Eastern Reservoir into the Main Canal with associated capital and running costs, although this would be partly offset by the costs associated with the Supply Canal. As it is the Somali Government's intention to use gravity supplies from the Fanoole Main Canal, the possibility of using flood water flows may be rejected due to unreliability and the disadvantages and expense of providing and maintaining a large pump station.

CHAPTER 3

IRRIGATION AND DRAINAGE SYSTEM

3.1 Land Use

The boundaries of the irrigated and rainfed areas are taken from the HTS Soils Report (September 1979). Within the project area certain small areas of potentially irrigable land which are not irrigated due to topographic or other reasons (in particular, 'fartas' or old river meanders) have been classed as suitable for rainfed development. In addition low lying areas which are not drainable have also been classified as rainfed.

3.2 Survey Information

All designs have been based on the 0.25 m interval 1 : 10 000 contour maps produced by HTS and the pre-design survey carried out by MMP in June/July 1979. The datum for the survey work was the Survey and Mapping Department's BM 140 located on the existing Mogadishu road approximately 4.2 km east of Jilib. The value of this bench mark was taken as 22.772 m.

3.3 Watercourse Unit

Each watercourse unit is designated as suitable for either mixed crops or paddy rice in accordance with the HTS Soils and Agriculture Reports. Due to the irregular topography of the project area it was found impossible to standardise on unit dimensions, although the area of both types of unit has been kept constant at 25 ha net, (approx. 30 ha gross) with farmers being allocated 1 ha net each.

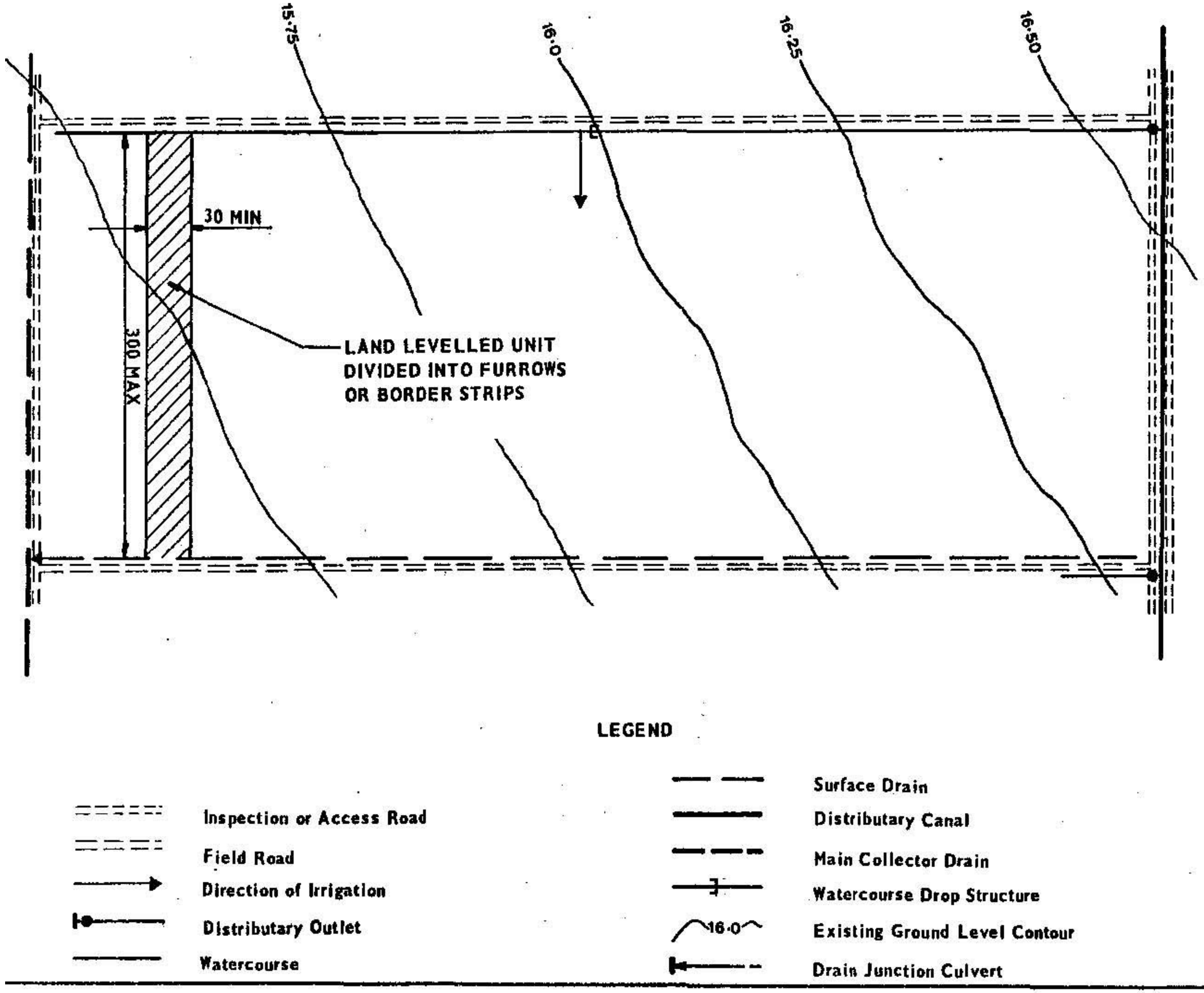
The mixed crops will be grown in either border strips about 10 m wide or furrows at approximately 0.8 m spacings. The maximum border strip or furrow length has been taken as 300 m with a maximum slope of 0.3%. The border strips or furrows are aligned in the direction that gives the most acceptable slope thus reducing land levelling costs, and a watercourse unit may be divided into several sections to achieve this. The watercourse itself may sub-divide in order to facilitate irrigation, the flow being controlled by portable checks or earth bunds. Checks will also be used to maintain a water level in the watercourse at the area under irrigation.

Paddy rice will be grown in basins of, normally, 1 ha net or proportions thereof, and the basins should be flat in both directions. The minimum basin width will be 30 m which is the minimum allowable for land levelling operations, with a maximum length of 200 m.

Sample field layouts are given in Figures 3.1, 3.2 and 3.3.

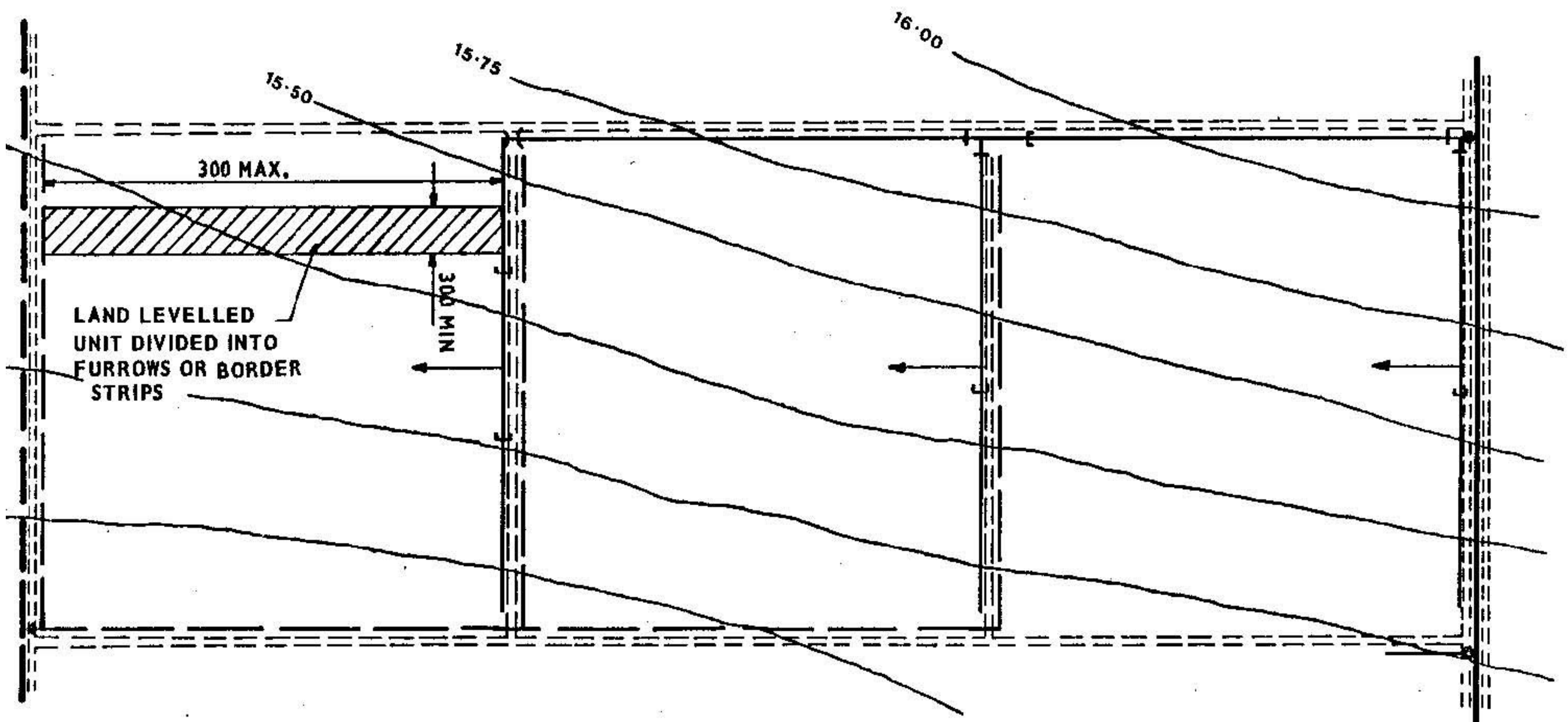
3.4 Cropping Pattern

The areas designated as suitable for paddy rice will grow two crops a year whenever possible with no rotation. On the mixed crop areas the following cropping pattern has been proposed by HTS:



TYPICAL MIXED CROP WATERCOURSE UNIT I

FIGURE 3.1

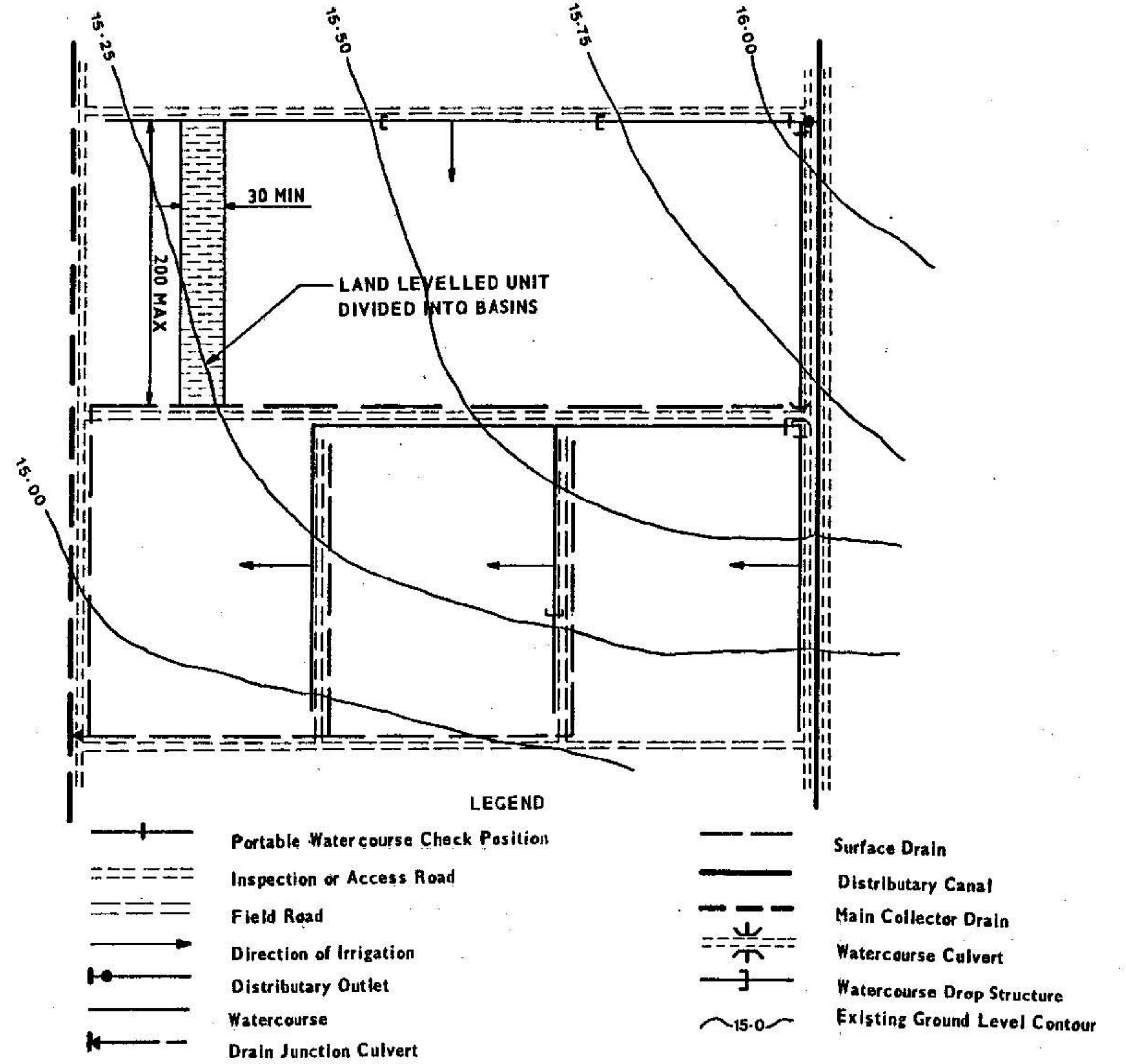


LEGEND

- Portable Watercourse Check Position
- Inspection or Access Road
- Field Road
- Direction of Irrigation
- Distributary Outlet
- Watercourse
- Drain Junction Culvert
- Surface Drain
- Distributary Canal
- Main Collector Drain
- Watercourse Culvert
- Watercourse Drop Structure
- 15·0 Existing Ground Level Contour

TYPICAL MIXED CROP WATERCOURSE UNIT 11

FIGURE 3.2



TYPICAL PADDY RICE WATERCOURSE UNIT

FIGURE 3.3

TABLE 3.1

Proposed Cropping Pattern

Crop	Gu season %	Der season %
Groundnuts	15	-
Vegetables	5	5
Upland rice	20	20
Maize	20	20
Sesame	-	15
Cotton	-	40
TOTAL	60	100

The overall cropping intensity is therefore 160%. It is proposed that for ease of operation a single crop is grown on each watercourse unit and each distributary canal at any one time. The cropping pattern would be based therefore on rotation at distributary canal level only. There is however, sufficient flexibility in the system to vary the cropping pattern or grow more than one crop on one distributary canal or even one watercourse unit if required.

Each of the ten irrigation blocks has been designed as a self contained unit irrigated by one or more distributary canals. No distributary canal irrigates more than one block.

3.5 Water Requirements

Water requirement calculations are presented in the final Agricultural Report (HTS 1980) and are reproduced and extended in this note to give the required discharge at watercourse unit level.

(a) Mixed Crops

The monthly net water requirements for each crop, allowing for effective rainfall, are given in Table 3.2.

The watercourses and distributary canals are designed for a maximum of 163.1 mm per month which is equivalent to upland rice in October. Assuming a 12 hour watering day, this is equivalent to a net requirement for a 25 ha watercourse unit of:

$$\frac{171.3}{31 \times 12 \times 3\,600} \times 25 \times 10^4 \text{ l/s} = 32 \text{ l/s}$$

Assuming 60% field efficiency and 10% watercourse losses, the required watercourse discharge is:

$$\frac{32}{0.6} \times 1.1 \text{ l/s} = 58.7, \text{ say } 60 \text{ l/s}$$

TABLE 3.2

Monthly Water Requirements for Mixed Crops (mm)

		J	F	M	A	M	J	J	A	S	O	N	D
Groundnuts	- gu (15%)	-	-	-	5.3	75.5	115.5	96.0	4.4	-	-	-	-
Vegetables	- gu (5%)	-	-	-	-	30.3	99.9	77.1	8.8	-	-	-	-
Vegetables	- der(5%)	99.5	-	-	-	-	-	-	-	-	48.4	89.4	141.1
Upland rice	- gu (20%)	-	-	-	-	77.1	108.9	110.6	22.5	-	-	-	-
Upland rice	- der(20%)	-	-	-	-	-	-	-	-	153.9	171.3	139.2	72.3
Maize	- gu (20%)	-	-	-	-	36.3	111.5	114.6	7.8	-	-	-	-
Maize	- der(20%)	67.2	-	-	-	-	-	-	-	-	56.6	102.6	154.1
Sesame	- der(15%)	-	-	-	-	-	-	-	-	12.2	118.9	129.6	30.7
Cotton	- der(40%)	89.9	-	-	-	-	-	-	16.0	105.3	163.1	138.6	155.1
TOTAL		49.4	-	-	0.8	37.9	66.8	65.0	14.3	82.4	137.2	130.1	115.5

Note: These figures allow for effective rainfall and the totals assume the worst case of upland rice being substituted for vegetables in the proposed cropping pattern.

TABLE 3.3

Monthly Water Requirements for Paddy Rice (mm)

	J	F	M	A	M	J	J	A	S	O	N	D
Paddy rice (gu)												
Net requirement	-	-	-	10.6	114.0	147.8	148.3	32.8	-	-	-	-
Percolation	-	-	-	75.0	155.0	150.0	155.0	75.0	-	-	-	-
TOTAL	-	-	-	85.6	269.0	297.8	303.3	107.8	-	-	-	-
Paddy rice (der)												
Net requirement	-	-	-	-	-	-	-	36.2	187.4	218.7	169.9	33.3
Percolation	-	-	-	-	-	-	-	75.0	150.0	155.0	150.0	75.0
TOTAL	-	-	-	-	-	-	-	111.2	337.4	373.7	319.9	108.3

Note: These figures allow for effective rainfall

The branch, Main and Supply canals are designed for the highest overall crop requirement, which is 137.2 mm in October. This is equivalent to a rate per watercourse unit of:

$$\frac{137.2}{171.3} \times 60 \text{ l/s} = 48.1, \text{ say } 50 \text{ l/s}$$

i.e. $^5/_6$ of the distributary canal discharge.

In addition, as the Supply, Main and branch canals are designed to run for 24 hours a day, their discharge is $^5/_{12}$ of their component distributary canals discharge.

(b) Paddy Rice

The monthly net water requirements for paddy rice, allowing for effective rainfall and 5 mm percolation and evaporation a day from the ponded fields are given in Table 3.3.

The highest requirement is 373.7 mm in October, and assuming a 12 hour watering day and a 25 ha net watercourse unit, this is equivalent to a net requirement of

$$\frac{373.7}{31 \times 12 \times 3\,600} \times 25 \times 10^4 \text{ l/s} = 70 \text{ l/s}$$

Assuming 70% field efficiency and zero watercourse losses (since the field is ponded) the required watercourse discharge is

$$\frac{70}{0.7} \text{ l/s} = 100 \text{ l/s}$$

The water requirements based on a watercourse unit of 25 ha net are summarised in Table 3.4:

TABLE 3.4

Summary of Water Requirements

	Mixed crop units	Paddy rice units
Watercourse capacity (l/s)	60	100

3.6 Field Irrigation

(a) Mixed Crops

Mixed crops will be grown on furrows or border strips fed either directly from the watercourse or a small field channel running parallel to the watercourse.

(i) Furrows

The maximum non-erosive slope of a furrow is given by

$$S = \frac{0.6}{Q}$$

where S = furrow slope (%)
Q = furrow discharge (l/s)

Assuming a maximum furrow slope of 0.3%, then the maximum furrow discharge is 2 l/s which may be provided by 50 mm diameter siphon pipes under a nominal head of 0.2 m. Thirty furrows may therefore be irrigated at one time giving a total width of 24 m with furrows at 0.8 m spacing.

An irrigation cycle of 10 days is recommended and hence the net requirement per irrigation is $^R/_3$ where R is the net monthly requirement in mm as given in Table 3.2. Each set of thirty furrows irrigates an area of 24 x L where L is the length of the furrow, and taking the net application rate as :

$\frac{60}{1.1} \times 0.6 = 32.7$ l/s then the time required to complete one irrigation is:

$$\frac{R \times 24 \times L}{3 \times 32.7 \times 3\,600} \quad h$$

Hence for a watercourse unit growing cotton in October (net requirement 163.1 mm) with a furrow length of 300 m, the time for each irrigation is:

$$\frac{163.1 \times 24 \times 300}{3 \times 32.7 \times 3\,600} = 3.3 \text{ h}$$

(ii) Border Strips

The maximum allowable slope for the border strips will be 0.3%, with zero slope across the strips. Assuming 3 strips each of width 10 m are irrigated at one time, then the time required to complete one irrigation is:

$$\frac{R \times 30 \times L}{3 \times 32.7 \times 3\,600} \quad h$$

where R = net monthly requirement (mm)
L = length of basin (m)

(b) Paddy Rice

Paddy rice will be grown in basins which should be horizontal.The minimum width is 30 m, and the length will be determined by the most economic length for land levelling with a maximum value of 200 m. The bunding around each basin should be at least 0.25 m high in order to retain the maximum ponded depth of 0.10 m.

3.7 Design Flows

From the calculations in Section 3.5, the distributary canals have been designed according to a requirement of 60 l/s for each mixed crop outlet and 100 l/s for each paddy rice outlet. Seepage losses in distributary canals have been allowed for as detailed in Section 4.2.

The distributary canals are designed for 12 hour daytime flow only, whereas the Supply, Main and branch canals will be operated continuously. In addition the Supply, Main and branch canals have been designed for the highest overall requirement based on the recommended cropping pattern which is $^5/_6$ of the highest single crop requirement of 60 l/s per watercourse unit. Paddy rice units will not be rotated and therefore no further reduction has been made for them. The branch and Main canal designs have been based on the following formula:

$$Q = 0.5\left[(q - 0.1\,n) \times {}^5/_6 + 0.1\,n\right]$$

where Q = design offtaking discharge from Main or branch canal (m^3/s)
q = distributary canal design discharge (m^3/s)
n = number of paddy rice units on distributary canal
0.1 = paddy rice unit discharge (m^3/s)

Where watercourse units are fed directly from the Main or branch canals or night storage reservoirs, an allowance of 50 l/s (= 60 l/s x $^5/_6$) for mixed crop or 100 l/s for paddy rice has been made.

3.8 Canal System

Canals are defined briefly as follows:

Supply Canal - the Canal offtaking from the Fanoole Main Canal and bringing water to the project area

Main Canal - the Canal designed to flow continuously through the project area feeding night storage reservoirs and the branch canals

Branch canal - a canal offtaking from the Main Canal designed to flow continuously and feeding night storage reservoirs

Distributary canal	-	a canal offtaking from a night storage reservoir designed to flow for up to 12 hours each day and feeding several watercourses
Watercourse	-	a small channel offtaking from a distributary canal or in some instances from the Main or branch canals or night storage reservoirs. These are the channels which supply water to individual watercourse units.

The canal system is designed for peak requirements with irrigation only taking place during the day for a maximum of 12 hours. During the night, water is diverted from the Main and branch canals into night storage reservoirs at the heads of distributary canals. In the day the distributary canal head regulator gates in the reservoirs are opened, and the combined flows from the Main or branch canal and the reservoir pass into the distributary canal. Watercourses are fed simulataneously, a ten day rotation being carried out within each watercourse unit.

The Supply Canal is aligned parallel to the new Jilib/Golweyn road and the Main and branch canals run through the approximate centre of the project area on the elevated flood plain of the old Shabeelle meander complex. Night storage reservoirs have been grouped at suitable locations with one or more distributary canals offtaking from each reservoir. The distributary canals have generally been aligned along ridges with the watercourses offtaking at right angles.

3.9 Canal Numbering System

All canals are numbered consecutively with reference to the canal or night storage reservoir from which they offtake. Even numbered canals offtake on the right and odd numbered canals offtake on the left.

Branch canals	-	two branch canals offtake from the tail of the Main Canal and are numbered HB1 and HB2
Night storage reservoirs	-	these are numbered with respect to the group number: e.g. N4.2 is the first storage reservoir on the right at the fourth group on the Main Canal
Distributary canals	-	these are numbered with respect to the night storage reservoir group from which they offtake. There are six groups on the Main Canal and one at the tail of each of the two branch canals: e.g. H2/D2 is the first distributary on the right at the second group on the Main Canal, and HB1/D1 is the first distributary on the left at the group on branch canal HB1 Where a distributary canal sub-divides the larger of the two channels has the same name and as the parent canal and the smaller is given a suffix: e.g. H5/D3 sub-divides into H5/D3 and H5/D3.2 which offtakes on the right

Watercourse units	-	Each watercourse unit is numbered with a further suffix: e.g. H2/D3/5 is the third watercourse unit on the left offtaking from distributary canal H2/D3.

3.10 Drain System

The drain system serves two functions - disposing of flood flows from the reservoir areas and also excess surface water from the project area.

Drains are defined briefly as follows:

Upper Outfall Drain	-	the Drain taking flood flows from the Northern Reservoir to the Eastern Reservoir
Lower Outfall Drain	-	the Drain taking flood flows from the Eastern Reservoir to the River Jubba Outfall. This Drain also collects drainage water from the project area
Branch drain	-	the two large drains D2 and D16 running along the western and eastern boundaries of the project area into which main collector drains discharge
Main collector drain	-	a drain into which two or more field drains discharge
Escape drain	-	a drain taking Main or distributary canal escape flow which discharges into the main drainage system
Field drain	-	a shallow vee-shaped depression collecting excess surface water from a watercourse unit. The section is shallow to permit the passage of agricultural vehicles.

Drainage from inside the project area passes into the Lower Outfall Drain by way of branch and main collector drains and from there into the River Jubba when the relative levels are suitable. In order to maximise the periods when this is possible, the Lower Outfall Drain has been designed to be as high as possible to provide gravity drainage to a maximum practical area, and is in fill for part of its length. Gravity flow of drainage water from the project area into the Lower Outfall Drain was not possible in all cases, and four main collectors require pumping. In addition some small areas of very low lying land have been classed as out of drainage command and will be used for rainfed development.

The Southern Reservoir will be utilised when drainage water from the project area cannot be discharged into the River Jubba because the relative levels are unsuitable. In this case the Lower Outfall Drain will overflow into the Southern Reservoir, and a level of 12.5 m can be accommodated without any ponding occurring in the gravity drained areas of the project. The reservoir will empty by flowing back into the Lower Outfall Drain when the river level drops, with any residual water being disposed of by percolation and evaporation. The Reservoir is formed by the escarpment to the east and the existing banana plantation bunds in the west, which may require a small amount of refurbishing in some areas.

3.11 Drain Numbering System

The drain numbering system is referenced to the Lower Outfall Drain. Drains discharging directly into the Lower Outfall Drain are numbered consecutively from the downstream end in a similar manner to canals: e.g. D2 is the first drain on the right (measured from the downstream end) discharging into the Lower Outfall Drain. Further drain sub-divisions are marked by suffixes: e.g. D2/1 and D2/7/1. The large drains D2 and D16 have been classified as branch drains, with the remainder as main collector drains.

CHAPTER 4

CANAL DESIGN

4.1 Canal Hydraulic Design

The following Lacey regime equations have been used for the hydraulic design of the Supply, Main, branch and distributary canals:

$$Dm = 0.525 \left(\frac{Q}{e^2 f}\right)^{1/3}$$

$$W_s = 4.83 \; e \; Q^{\frac{1}{2}}$$

$$S = 0.00030 \; e^{1/3} \; f^{5/3} \; \frac{E}{Q^{1/6}}$$

where Dm = mean depth, $\frac{\text{Area}}{W_s}$ (m)

W_s = water surface width (m)

E = shape factor, $\frac{\text{wetted perimeter}}{W_s}$

V = mean velocity (m/s)

Q = discharge (m^3/s)

S = water surface slope (m/m)

e = width factor

f = Lacey silt factor

The canals have been designed with a trapezoidal cross section and a ratio of bed width to water surface width of 0.8. The width factor e has been taken as 0.83, with the Lacey silt factor f in the range of 0.4 to 1.1 to suit ground slopes.

A minimum bed width of 1.0 m and depth of 0.3 m were taken at the tail of the distributary canals.

The designs were checked to ensure that the unit tractive force did not exceed 2.4 N/m^2. The unit tractive force T is defined as:-

$$T = CWRS$$

where W = water specific weight (9 810 N/m^3)

R = hydraulic radius (m)

S = water surface slope (m/m)

C = coefficient depending on the shape of the channel and part of the channel considered. The maximum value of C = 1 on the canal bed has been assumed.

The canals will generally be unlined except when they pass through sandy areas when clay lining may be required.

A canal design chart is given in Figure 4.1

4.2 Canal Transit Losses

Transit losses in canals have been calculated for each reach at a rate of 2.50 m^3/s per million square metres of wetted perimeter, using the formula:

$$\text{Losses in reach} = 0.012\, Q^{\frac{1}{2}} L$$

where Q = discharge (m^3/s)
L = length of reach (km)

4.3 Canal Radii

The radius of curvature R of the centre line of any canal has been calculated from the formula:

$$R = 128\, Q^{\frac{1}{2}}, \text{ with a minimum of 90 m}$$

where Q = discharge (m^3/s)

4.4 Cross Section Details

Canal cross section details are given in Table 4.1.

TABLE 4.1

Canal Cross Section Details

Canal	Bank top width (m)	Side slopes inside	Side slopes outside	Freeboard (m)	Reservation width (m)
Supply	5.0	1:2	varies	0.55	35.0
Main	5.0	1:2	varies	0.50	30.0
Branch	5.0	1:2	varies	0.50	30.0
Distributary	4.0	1:2	varies	0.40	20.0

The outside bank slopes have been designed to resist a 1 in 7 seepage gradient from design water level to ground level, with a minimum of 1 in 2. The bank tops, which will be used as inspection roads, have been given an outward camber of 1 in 40 to prevent run-off eroding the inside canal slopes.

The canals will generally be in cut and fill or fill, except where the Supply Canal is aligned across the elevated marine plain to the west of the project area. In these cases a small berm and pitched drains have been provided at ground level to limit bank erosion.

4.5 Watercourse Design

Two sizes of watercourse have been designed to feed either paddy rice or mixed crops. Details are given in Table 4.2.

TABLE 4.2

Watercourse Cross Section Details

Watercourse type	Discharge (l/s)	Design depth (m)	Freeboard (m)	Command (m)	Side slopes
Paddy rice	100	0.68	0.30	0.3 - 0.6	1:1.5
Mixed crop	60	0.55	0.25	0.2 - 0.5	1:1.5

The watercourses will be constructed by laying a strip of fill obtained from the adjacent surface drain and then forming a vee-shaped section with a ditcher or grader.

The watercourses have been designed using the Manning equation:

$$Q = \frac{AR^{2/3}S^{\frac{1}{2}}}{n}$$

where
Q = design discharge (m^3/s)
n = Manning coefficient
R = hydraulic radius = A/P (m)
A = area of flow (m^2)
P = wetted perimeter (m)
S = water surface slope (m/m)

A value for n of 0.025 was assumed with slopes in the range 10 to 100 cm/km. A check was made to ensure that at maximum slope the unit tractive force did not exceed 2.4 N/m^2.

4.6 Night Storage Reservoirs

The night storage reservoirs have been designed to store half their offtaking distributary canal 12 hour design requirements. The minimum live storage in the reservoirs has been set at a nominal 0.50 m, although this has been increased where extra head was available to reduce the reservoir area. A minimum dead storage of 0.50 m has also been allowed. The reservoir slopes have been taken as 1 in 3 inside and 1 in 2 outside with a bank top width of 5 m minimum, increased as necessary to achieve a 1 in 7 seepage gradient through the bank from maximum water level to ground level. The freeboard has been taken as 0.5 m minimum, and where the reservoir is adjacent to the Main or branch canal, the canal bank has been utilised as one of the reservoir embankments in order to reduce earthworks.

CHAPTER 5

DRAIN DESIGN

5.1 Drainage Rates

5.1.1 Paddy Rice Areas

The drainage run-off rates for paddy rice areas were calculated assuming a certain amount of storage in the fields - the maximum storage level being no higher than the emergent crop to avoid drowning. The critical case of newly planted rice was taken as described below.

Under the proposed cropping pattern rice will be planted over a one month period, water being applied under a 10 day irrigation cycle. At the end of this one month period the area may be viewed as consisting of 10 equal sized blocks, with stages of growth varying from zero (for newly seeded areas) to seedlings approximately 200 mm high with corresponding irrigation depths varying from zero to 100 mm.

The available storage in the 10 blocks determines the required drainage rate for any given storm and this storage may have a minimum or maximum depending on when the storm occurs in relation to the irrigation cycle. This is shown in Tables 5.1 and 5.2.

TABLE 5.1

Minimum Storage - Paddy Rice Areas

Block Nr.	1	2	3	4	5	6	7	8	9	10
Seedling height (mm)	200	178	156	133	111	89	67	44	22	0
Irrigated depth (mm)	100	89	78	67	56	44	33	22	11	0
Days since irrigation	4	3	2	1	0	9	8	7	6	5
Water loss (mm)	40	30	20	10	0	44	33	22	11	0
Water depth (mm)	60	59	58	57	56	0	0	0	0	0
Available storage (mm)	140	119	98	76	55	89	67	44	22	0

Average storage: 71 mm

The water loss component consists of 5 mm/day infiltration and 5 mm/day evapotranspiration. Available storage for mature rice is less critical than that for growing rice with respect to drainage rates and has not been considered.

A daily water balance of rainfall, infiltration, evapotranspiration, storage and run-off was computed for each block for all significant rainfall events using the Alessandra rainfall records (1930-34, 1953-60). It was assumed that when the available storage for any block was exceeded, run-off to the drainage system occurred.

TABLE 5.2

Maximum Storage - Paddy Rice Areas

Block Nr	1	2	3	4	5	6	7	8	9	10
Seedling height (mm)	200	178	156	133	111	89	67	44	22	0
Irrigated depth (mm)	100	89	78	67	56	44	33	22	11	0
Days since irrigation	9	8	7	6	5	4	3	2	1	0
Water loss (mm)	90	80	70	60	50	40	30	20	10	0
Water depth (mm)	10	9	8	7	6	4	3	2	1	0
Available storage (mm)	190	169	148	126	105	85	64	42	21	0

Average storage: 95 mm

The maximum annual run-off rates were extracted, ranked and plotted using a Gumbel Distribution to determine required drainage capacity for any given return period. These results are summarised in Table 5.3

TABLE 5.3

Drainage Run-off Rates - Paddy Rice Areas

Return period (years)	Run-off for minimum storage (l/s/ha gross)	Run-off for maximum storage (l/s/ha gross)
2	0.76	0.71
4	1.32	1.18
5	1.50	1.34

5.1.2 Mixed Crop Areas

A similar daily water balance method was used for these areas. In this case storage is available to replenish the soil moisture deficit caused by evapotranspiration since the previous irrigation application. It has been assumed that irrigation water is applied to return the soil to field capacity. Further storage is available to bring the soil moisture up to saturation point, but during this stage the infiltration rate is reduced. Allowance for antecedent rainfall was made prior to each significant rainfall event.

To reduce the required drainage capacity it has been assumed that ponding of water on the fields can be tolerated for short periods. Evaporation and infiltration from the ponded areas have been allowed for in the daily water balance.

Run-off rates and retention times of ponded water for several drainage rates were computed for all significant rainfall events. The annual maxima of days of ponding for each assumed drainage rate were extracted, ranked and plotted using a Normal Distribution. The results are summarised in Table 5.4.

TABLE 5.4

Drainage Run-Off Rates - Mixed Crop Areas

Drainage rate (l/s/ha gross)	Return period (years)	2	4	5
1	Field storage (days)	2.72	3.80	4.09
1.5		1.70	2.50	2.69
2.0		1.03	1.81	2.02

An overall run-off figure of 1.5 l/s/ha gross was taken for both paddy rice and mixed crop areas. This is equivalent to a five year return period with minimum storage for the paddy rice areas, and a five year return period with 2.69 days of ponding in the fields for the mixed crop areas. Each watercourse unit has a gross area of about 30 ha, and thus the design run-off from each unit has been taken as 45 l/s. A similar rate has been assumed for the unirrigated land and village sites etc. within the project area.

To allow for the non-uniformity of rainfall over the project area an areal reduction factor of 0.9 has been applied to all main collector drains at their junction with a branch drain or the Lower Outfall Drain.

5.2 Drain Hydraulic Design

The drains (excluding field dains and escape drains) have been designed using the Manning-Lacey design chart as given in Figure 5.1 with Manning's 'n' taken as 0.025 and the Lacey modified silt factor f in the range of 0.2 to 1.2 to suit ground slopes. The cross sections have been taken as trapezoidal with a bed width to depth ratio of 3 and a minimum bed width of 1.0 m.

The drains are generally in cut except for parts of the Lower Outfall Drain and where other drains pass through low lying areas of unirrigated land.

5.3 Drain Transit Losses

No drain transit losses have been considered in the design.

5.4 Drain Radii

The minimum radius of curvature of the centre line of drains has been taken as ten times the water surface width, with a minimum of 50 m.

5.5 Cross Section Details

Drain cross section details are given in Table 5.5.

TABLE 5.5

Drain Cross Section Details

Drain	Bank top width (m)	Side slopes inside	Side slopes outside	Minimum freeboard (m)	Reservation width (m)
Field drain					
(a) in cut	-	1:8	-	0.00	-
(b) in fill	1.00	1:1.5	1:1.5	0.00	-
Escape drains					
(a) in cut	-	1:1.5	-	0.00	-
(b) in fill	4.0	1:1.5	Varies	0.20	-
Main collector or branch drain					
(a) in cut	-	1:1.5	-	0.20	Varies
(b) in fill	4.0	1:1.5	1:1.5	0.50	25.0
Lower Outfall Drain					
(a) in cut	-	1:1.5	-	0.20	25.0
(b) in fill	4.0	1:1.5	1:1.5	0.50	25.0

Upper Outfall Drain - see Section 5.10

The main collector, branch and Lower Outfall Drains have 6 m access roads on both sides and these have been given an outward camber of 1 in 40 to prevent bank erosion.

5.6 Protective Embankments

The Lower Outfall Drain and branch drain D2 have a protective embankment aligned along their unirrigated side, to prevent run-off from the escarpment outside the project area causing flooding. The embankment for the Lower Outfall Drain is not required downstream of the junction with D2 as this would prevent drainage water from entering the Southern Reservoir area.

The height of the protective embankments has been taken as a nominal 1.5 m above drain design water level.

5.7 Field Drains

The field drains collect any excess irrigation water or run-off from the fields and dispose of it into the main drainage system. They have been designed with a depth of 0.30 m minimum and side slopes of 1 in 8 to allow vehicles to cross.

5.8 Escape Drains

The escape drains take either Main Canal or distributary canal escape flow and discharge into a suitable adjacent drain. They have been designed using the Manning formula.

5.9 Lower Outfall Drain

The Lower Outfall Drain has been designed with a canal type section using the criteria given in Chapter 4. This is because, as much of the section is in cut and fill, a canal type design is more appropriate and gives a more economic section.

The Drain serves two purposes - it disposes of flood water from the Eastern Reservoir and also collects drainage water from the project area. As the occurrence of maximum discharge from both the Eastern Reservoir and the project area is small, the Lower Outfall Drain has been designed to carry the maximum discharge from the Eastern Reservoir (10 m^3/s) until the total of the incoming drains from the project area is greater than 10 m^3/s, when this total becomes the design discharge.

5.10 Upper Outall Drain

The Upper Outfall Drain takes water from the Northern Reservoir to the Eastern Reservoir. The Drain has not been designed to take the maximum 1 in 1000 year outflow of 331 m^3/s, but to act as a low flow channel only. When the Drain overtops, water is prevented from entering the project area by the adjacent flood bund. Flow to the east is restricted only by ground levels except for the first section where flooding of the new Jilib/Golweyn road and the village of Dakadai would occur. The Dakadai Flood Protection Embankment has been provided to prevent this. The maximum design water level at the Northern Reservoir Outlet is 20.0 m and the required flow area to pass the maximum discharge has been taken as a nominal 300 m^2 with a water surface slope of about 24 cm/km. Therefore, in addition to providing a low flow channel, the Upper Outfall Drain is required to provide two further functions:

(a) to act as a borrow area for the adjacent flood bund

(b) to provide the minimum flow area of 300 m^2 at maximum design water levels.

The dimensions of the Upper Outfall Drain will be determined from these criteria by the Engineer on site when the ground levels are known more eccurately.

CHAPTER 6

GENERAL STRUCTURAL DESIGN CRITERIA

6.1 Loading

Traffic loading has been taken as HA loading (BS 153), or 10 kN/m^2 in the case of traffic loading on soil surfaces for surcharge calculations. Loading on footbridges has been taken as 4 kN/m^2 based on the gross plan area.

6.2 Stability

The stability of structures or parts of structures has been assessed using the following minimum factors of safety.

Against sliding or overturning	1.5
Against failure by piping, based on the exit gradient (lower values have been allowed where the extreme loading case is considered to be unlikely to occur, with an absolute minimum of 2.5)	5.0

A structure has been considered safe against uplift if the weight of concrete alone is greater than the hydrostatic uplift under the worst possible loading conditions, no allowance having been made for friction at the soil/concrete interface.

6.3 Soil Properties

The following soil properties have been used in the design:

Saturated weight	20 kN/m^3
Submerged weight	10 kN/m^3
Coefficient of active earth pressure	0.4
Coefficient of passive earth resistance	2.5
Coefficient of earth pressure at rest	0.6
Maximum permissible net bearing pressure	70 kN/m^2
Coefficient of base friction	0.4

To allow for cracking at the vertical soil/wall interface no wall friction has been taken into account. Groundwater table level has been generally assumed to be at channel design water level or other appropriate level and the safety of all structures has been checked for a rapid drawdown case where the channel is assumed to empty rapidly leaving an unbalanced hydrostatic pressure on the structure. Soil below water table level is treated as submerged and soil above as saturated. A general traffic surcharge of 10 kN/m^2 has been assumed for all structures where vehicular access is possible.

Active earth pressure conditions have only been used in situations where the structural member is free to move in the direction of pressure, otherwise earth pressure at rest has been assumed. Passive resistance has been assumed to start at finished ground level or top of pitching, and the coefficient has been reduced appropriately in instances where the earth surface slopes away from the member under consideration.

6.4 Concrete and Reinforcement

For reinforced concrete, design has been performed in accordance with CP 110 for concrete grade 20 (characteristic strength 20 N/mm^2) and mild steel reinforcement (yield strength 250 N/mm^2).

For mass concrete design a maximum allowable tensile stress of 0.35 N/mm^2 has been assumed and conventional elastic design theory has been followed.

The following properties of concrete have been used:

Weight of reinforced concrete	23.5 kN/m^3
Weight of mass concrete	22.0 KN/m^3
Modulus of elasticity (for deflections)	23×10^3 N/mm^2
Coefficient of linear expansion	$11 \times 10^{-6}/^{o}C$
Coefficient of shrinkage	300×10^{-6}

A temperature range of 25^{o}C has been assumed.

Cover to reinforcement has been taken as 50 mm except for some small relatively unimportant members.

Laps in bars have been set at a minimum of 40 x bar diameter.

Bar spacings of 100 mm minimum and 300 mm maximum have been adopted and the following bar diameters have been used: 8, 10, 12, 16, 20 and 25 mm.

The following minimum reinforcement percentages were used:

For main steel in the tension face	0.25% effective area
Secondary steel in tension face	0.15% gross area
Compression face (in each direction)	0.15% gross area

The classes of concrete used are as follows :

A	Reinforced concrete - thin sections
B	Reinforced concrete - general use
C	Mass concrete
D	Blinding, infill
AS	Sulphate resisting concrete - thin sections (reinforced)
BS	Sulphate resisting concrete - general use (mass and reinforced)

Sulphate resisting concrete has been used where the structure is liable to sulphate attack from groundwater or soils. Generally drain structures will be constructed using sulphate resisting cement.

6.5 Pipes and Pipe Bedding

All concrete pipes are of the spigot and socket type in the following standard sizes:

0.30 m internal diameter

0.375 m internal diameter

0.45 m internal diameter

0.60 m internal diameter

0.75 m internal diameter

0.90 m internal diameter

1.05 m internal diameter

1.20 m internal diameter

Two classes of pipe have been used: Class H and Class M; and three classes of bedding; Class A2 (reinforced concrete), Class A1 (mass concrete) and Class B (granular).

Table 6.1 gives the ranges of depths of cover appropriate to each size and class of pipe. Generally granular bedding has not been used for gated structures or structures where there is a large head loss through the pipe. This is because the granular bed presents a low resistance seepage path and any significant flow through it could cause piping and subsequent failure at the downstream end. Otherwise the granular bed is preferred since it is much cheaper than the concrete alternatives.

TABLE 6.1

Depth of Cover to Pipes (in metres)

Internal diameter (m)	Pipe class	Bedding class A2	Bedding class A1	Bedding class B
0.30	M	-	-	-
	H	0.9 - 6.8	0.9 - 4.9	0.9 - 3.0
0.375	M	0.9 - 6.6	0.9 - 4.8	0.9 - 2.8
	H	0.9 - 7.9	0.9 - 6.3	0.9 - 4.2
0.45	M	0.9 - 6.6	0.9 - 4.8	0.9 - 2.8
	H	0.9 - 7.6	0.9 - 5.8	0.9 - 4.0
0.60	M	0.9 - 6.3	0.9 - 4.6	0.9 - 2.6
	H	0.9 - 7.6	0.9 - 5.7	0.9 - 3.8
0.75	M	0.9 - 6.1	0.9 - 4.4	0.9 - 2.3
	H	0.9 - 7.6	0.9 - 5.7	0.9 - 3.4
0.90	M	0.9 - 6.7	0.9 - 4.9	0.9 - 2.9
	H	0.9 - 7.6	0.9 - 6.4	0.9 - 4.5
1.05	M	0.9 - 6.5	0.9 - 4.8	0.9 - 2.8
	H	0.9 - 7.6	0.9 - 6.3	0.9 - 4.3
1.20	M	0.9 - 6.4	0.9 - 4.7	0.9 - 2.8
	H	0.9 - 7.6	0.9 - 6.3	0.9 - 4.2

The table gives the minimum and maximum permissible depths of cover based on pipe classes as given in BS 556 and "main road" traffic loading as given in the "Simplified Tables of External Loads on Buried Pipelines" by the Building Research Establishment (published by HMSO).

CHAPTER 7

CANAL STRUCTURES

7.1 Supply Canal Head Regulator

The Supply Canal Head Regulator has been designed to control the flow offtaking from the Fanoole Main Canal into the Supply Canal. It is a three-bayed gated weir structure with three vertical lifting gates each of width 2.5 m and height 2.0 m positioned on the weir crest. The design flow is 11.6 m^3/s and the orifice formula has been used in the design:

$$Q = CA\sqrt{2gH}$$

where
Q = discharge (m^3/s)
C = discharge coefficient = 0.6
H = head over crest (m)
A = area of opening (m^2)

A USBR Type III stilling basin has been used to contain the hydraulic jump, and the following opening conditions were checked:

(a) design flows upstream and downstream

(b) sudden opening to 10% design discharge with downstream dry

(c) sudden opening from 70% to 120% design discharge

It should be noted that the design details of the Fanoole Main Canal were only preliminary when the Homboy designs were carried out (August 1979 - January 1980), and hence the structure may require some modifications when these details are finalised.

7.2 Main Canal Escapes

Two Main Canal escapes have been provided in order to empty the Canal in an emergency or for maintenance purposes. The design discharge of each escape has been taken as equal to the Main Canal discharge at that particular location, as shown below:

TABLE 7.1

Main Canal Escapes

Location (km)	Design discharge (m^3/s)	Number of gates
1.41	10.7	3
19.96	3.9	1

The structures are similar to the Supply Canal Head Regulator except that there is no raised weir crest as this would prevent the Canal from being completely drained. The vertical lifting gates have been standardised at 2.5 m wide by 1.5 m high and the design of the stilling basin follows the procedure outlined for the Head Regulator.

Downstream of the larger structure an escape drain has been provided to take the design discharge. This channel discharges into the Upper Outfall Drain via a triple 1.2 m diameter pipe culvert passing through the adjacent flood bund. Flap gates have been installed at the outlet to prevent return flow from the Upper Outfall Drain.

The smaller escape structure discharges directly into an adjacent farta which is large enough to carry the design capacity. This water then flows into branch drain D2 by means of an enlarged surface water escape.

7.3 Box Culverts

Triple barrelled, reinforced concrete box culverts have been designed to take the Supply or Main Canal under roads where there is no cross regulator to provide a crossing point. The head loss through the structure has been taken as:

(a) inlet and outlet losses: $\frac{V^2}{2g}$

where V = velocity in barrel

(b) frictional loss: from Mannings equation with $n = 0.015$. The depth of flow was taken as 0.1 m below the barrel soffit.

A nominal head loss of 0.1 m was assumed and the culvert size and length calculated accordingly, as shown on Drawing Nr 51101-50.

7.4 Movable Weirs

Movable weir structures have been used as Main Canal cross regulators, branch canal head regulators and night storage reservoir head regulators. Three types of weir have been used - Types A, B and C. The weirs have been designed to have 0.20 m freeboard over upstream design water level at their highest position, and 0.10 m spare downwards travel when passing design discharge at design water levels. The maximum head on the weir is therefore the total weir travel less 0.30 m; these details are given below:

TABLE 7.2

Details of Movable Weirs

Type	Total travel (m)	Maximum head on weir (m)
A	0.65	0.35
B	0.84	0.54
C	1.00	0.70

The discharge formula for these weirs is:

$$Q = CBH^{1.6}$$

where
- Q = discharge (m^3/s)
- B = weir width (m)
- H = head on weir (m)
- C = coefficient of discharge - taken as 2.18 for Type A and 2.30 for Types B and C.

A design table is given below:

TABLE 7.3

Movable Weir Design

Discharge (m^3/s)	Width (m)	Type
0 - 0.40	1.0	A
0.41 - 0.85	1.0	B
0.86 - 1.10	1.3	B
1.11 - 1.70	1.3	C
1.71 - 2.00	1.6	C
2.01 - 2.60	2.0	C
2.61 - 3.25	2.5	C
3.26 - 3.90	3.0	C
3.91 - 6.50	2 x 2.5	C
6.51 - 7.80	2 x 3.0	C
7.81 - 10.40	2 x 4.0	C

The discharge formula is only strictly applicable when the weir is less than 75% submerged, and this criteria has been followed for head regulators. In the case of the Main Canal cross regulators, accurate discharge measurement is not of primary importance, and in order to conserve head the allowable submergence was increased to 80%. At all movable weir structures an additional 0.10 m head loss was added in order to allow for a rise in downstream level due to siltation. The minimum design head losses have been standardised as 0.25 m for cross regulators and 0.28 m for head regulators.

It is recommended also that for accurate measurement the design depth of flow upstream is a minimum of twice the depth of flow over the weir. This criteria has been adopted for the night storage reservoir where accurate flow measurement is essential, but relaxed for the other structures.

Other details of the structure are given below:

(a) At design discharge and levels the weir upstand (where used) is a minimum of 0.40 m below the weir crest to avoid interference with the flow.

(b) The depth from bank top level to structure floor level downstream of the weir must be sufficient to:

(i) permit full lowering of the weir with 0.05 m clearance

(ii) ensure that at minimum downstream water level a sufficient conjugate depth is produced to contain any hydraulic jump within the structure.

7.5 Pipe Regulators

Pipe regulators have been used as distributary canal head and cross regulators. The structures comprise a gated pipe with a reinforced concrete outlet box, and in the case of the head regulators only, a reinforced concrete inlet box.

7.5.1 Head Regulators

The head regulators control the discharge from the night storage reservoirs into the distributary canals. There are a total of 23 structures with discharges in the range of 0.25 to 2.07 m^3/s. The inlet box weirs have been set to give a minimum head of 0.32 m at minimum reservoir water level, with the boxes sized from the weir formula:

$$Q = 1.7 L H^{3/2}$$

where Q = discharge (m^3/s)
L = length of weir (m)
H = head on weir (m)

Assuming the above equation is valid for up to 75% submergence, a minimum head loss of 0.08 m must be allowed. The minimum head loss through the pipes and outlet box has been calculated as 0.04 m, giving a minimum total head loss through the structure of 0.12 m. In many cases this is less than the available head and it was possible to reduce the pipe diameter with a resultant cost saving. The maximum allowable velocity in the pipe was taken as 2 m/s.

7.5.2 Cross Regulators

Cross regulators control the levels in the distributary canals, and have generally been located such that the furthest upstream distributary oulet is less than 1 km away from a cross regulator. The structure has been designed to have a minimum head loss of 0.10 m and a maximum head loss of 0.35 m, and a design chart is given on Drawing Nr 51101-61.

Where a distributary canal sub-divides, a pipe regulator is used to control the flow into each section.

7.6 Canal Cross Drainage Culverts

Canal cross drainage culverts have been provided to take existing natural drainage channels underneath a canal. The structure is basically a Type 2 drain culvert, with membrane lining and warning netting provided in the canal for a distance of 10 m on either side of the structure to prevent an excessive seepage gradient being set up.

Where the Supply Canal is aligned parallel to the new Jilib/Golweyn surfaced road, a toe drain has been provided to intercept flows passing through the various cross drainage culverts in the road. This flow is then discharged under the Supply Canal at km 9.76 via a 2 x 1.20 m diameter cross drainage culvert.

This solution was found to be more economic than providing a culvert under the Supply Canal adjacent to every culvert under the road.

7.7 Distributary Tail Escapes

Each distributary canal has been provided with a tail escape to protect the canal from breaching or overtopping in an emergency. In such a condition the flow passes through the structure into an adjacent escape drain and thence into the main disposal system. It should be noted that tail escapes have been designed as emergency structures only and not as a method of regulating the canal.

The outlet discharge has been standardised at a nominal 0.35 m^3/s with the weir crest at 0.05 m above design water level. The structure consists of an inlet box, three sides of which act as an inlet weir, connected to a 0.45 m diameter pipe passing through the canal bank to the escape drain. An outlet box has been provided to dissipate energy at the pipe outlet.

7.8 Field Outlets

Field outlets take water from the Main Canal, distributary canals or the night storage reservoirs and discharge into the watercourses. The structure consists of a gated pipe through the canal or reservoir bank with an inlet and outlet box. The structure may be required to feed either paddy rice or mixed crop watercourse units and thus two pipe sizes have been used as shown below:

TABLE 7.4

Field Outlet Design

Watercourse unit	Design discharge (l/s)	Pipe diameter (m)
Paddy rice	100	0.375
Mixed crop	60	0.30

The minimum head loss through all the outlets has been taken as 0.10 m.

To allow a degree of flexibility within the system it was decided that each distributary canal outlet should be capable of passing design discharge when the flow in the distributary canal was one third of design flow. A series of backwater curves were carried out, assuming design water level at the control cross regulator, to find the fall in water level upstream. This is shown below, together with the other components necessary to calculate the total command required in the distributary canal over the highest level in the watercourse unit.

TABLE 7.5

Required Command at Distributary Outlets

Watercourse unit	Distance u/s from cross regulator (m)	Loss through outlet (m)	Backwater loss at $^{Q}/_{3}$ (m)	Field ponding (m)	Water-course command (m)	Total command (m)
Paddy rice	0 - 50	0.10	0.00	0.10	0.20	0.40
	50 - 300	0.10	0.10	0.10	0.20	0.50
	300 - 500	0.10	0.15	0.10	0.20	0.55
	500 - 700	0.10	0.20	0.10	0.20	0.60
	700 - 1000	0.10	0.25	0.10	0.20	0.65
Mixed crops	0 - 50	0.10	0.00	-	0.20	0.30
	50 - 300	0.10	0.10	-	0.20	0.40
	300 - 500	0.10	0.15	-	0.20	0.45
	500 - 700	0.10	0.20	-	0.20	0.50
	700 - 1000	0.10	0.25	-	0.20	0.55

In addition a watercourse slope of 0.10 m/km and a watercourse culvert head loss of 0.05 m have been allowed where applicable.

At Main Canal outlets the backwater loss was ignored since it was assumed that the Main Canal will nearly always back up to design levels. At reservoir outlets the backwater loss was not applicable and the command was calculated with reference to the minimum design storage level.

7.9 Watercourse Structures

7.9.1 Watercourse Falls

Watercourse falls have been designed to be simple mass concrete fixed weir structures suitable for pre-casting. Two fall heights of 0.30 and 0.50 m have been accommodated and allowing for the paddy rice and mixed crop watercourse discharge of 100 and 60 l/s, respectively, four standard structures are required.

7.9.2 Watercourse Culverts

Watercourse culverts are required where either surfaced roads or field roads cross the watercourse. The structure consists of a simple pipe culvert with a nominal head loss of 0.05 m. Two pipe diameters have been used, 0.375 m for the paddy rice watercourses and 0.30 m for the mixed crop watercourses.

7.9.3 Cross Drainage Culverts

Cross drainage culverts are required where a natural drainage channel passes underneath a watercourse. The structure consists of a 0.45 m diameter pipe, with a membrane lining provided below the watercourse channel to reduce the seepage gradient.

CHAPTER 8

DRAIN AND RESERVOIR STRUCTURES

8.1 Northern Reservoir Outlet Structure

8.1.1 Introduction

The Northern Reservoir Outlet Structure is required to control flows from the Northern Reservoir into the Upper Outfall Drain. This involves crossing the new Jilib/Golweyn main road and this is achieved by passing the flow underneath the road bridge at km 234.

8.1.2 Design Criteria

From the 1 000 year flood routing analysis, the peak discharge to be accommodated is 331 m^3/s at a reservoir level of 22.8 m, the corresponding level downstream of the structure being 20.0 m.

From the 100 year flood routing analysis, the discharge to be accommodated is 160 m^3/s at a reservoir level of 22.5 m.

Embankment top levels :	Northern Reservoir	24.0 m
	Upper Outfall Drain flood bund	21.0 m

Bridge details :	8 span,	each 14.5 m between pile support lines
	deck width	8.0 m
	pile diameter	0.8 m
	road level	20.0 m
	beam soffit level	19.2 m
	pile feet level	11.64 m

Ground level : 17.5 m approximately

8.1.3 Structure Design

To allow low flows to pass unimpeded and yet be able to cut off all flow, gated orifices are required. These have been designed to pass the peak flow of 331 m^3/s although the alternative solution of providing smaller gated orifices and discharging the rest of the flow over a lowered portion of the bund or via siphons was investigated. However, these alternatives proved both impractical and expensive and it was decided to pass the full discharge through the gated orifices.

The reservoir bund top is almost 7 m above the ground level at the site of the structure. This difference in levels makes it more economical to form culverts under the bund than to substitute a concrete breast wall for the bund which would require long high wing walls.

Downstream of the road bridge the supercritical flow must enter a stilling basin where an hydraulic jump can be induced. As there is a possibility of the jump being forced upstream by high water levels in the downstream channel (particularly if the gates are closed quickly), the culverts have been extended under the bridge, and the bridge protected by a short retaining wall along the tops of the culvert outlets.

To avoid reducing the bearing capacity of the piles supporting the bridge decks, the depth of the structure under the bridge should be fairly small. Therefore, for economical construction, the culverts should be rectangular. The loading on the culverts under the bund is very high so they should not be more than about 4.0 m wide.

Vertical lifting gates with downstream seals were considered unsatisfactory for this structure as there would be a risk of serious damage during floods to the bracing on the front of the gates; the greater frictional forces would require more powerful lifting gear and, because of the considerable contraction of the flow under such gates (coefficient = 0.6), they would have to be much larger than required with streamlined inlets.

For upstream sealing gates, concrete breast walls are required behind which the gates can be raised and these walls influence the discharge relationship. The inlets have been well streamlined, so that the contraction of the flow immediately downstream of the orifice is reduced and a small gate can be used.

The discharge Q through the orifice can be expressed as follows :

$$Q = C\,B\,h_G \sqrt{2g\,(H - C.h_G - h_L)}$$

where
- C = the contraction coefficient
- B = the total width of the gates
- h_G = the height of the gate = height of opening
- H = the upstream total energy head relative to the inlet floor level
- h_L = the energy head lost through the inlet.

The inlet floor level has been set at 17.0 m, similar to the present lowest ground level. With conservative values for C (0.9), h_L (0.3) and a gate width of 4.0 m (corresponding to the maximum culvert width above; smaller widths would require more lifting units and more culvert walls), the gate requirements for peak flow (Q = 331 m^3/s, H = 5.8 m) are determined : six 4.0 m wide, 1.8 m high vertical lifting gates.

The flood routing analysis requires that the structure normally has its gates only partly open, i.e. sufficient for 160 m^3/s to be passed as the reservoir level reaches 22.5 m. The formula above can be used to determine the gate opening; conservative values for this partly open condition are C = 0.6 and h_L = 0.4, giving a gate opening of 1.3 m.

The stilling basin has been designed to St. Anthony Falls stilling basin specifications. This type of basin has been selected because it is significantly shorter than other standard types and it can have diverging sidewalls. To ensure that the stilling basin has an adequate factor of safety it has been designed for the maximum conceivable flow of 415 m^3/s, given by the above formula with C = 1.0 and H - h_L = 6.5.

The overall dimensions of the basin depend on the intensity of the flow entering the basin; the greater the intensity, the lower and longer the floor. Two arrangements with very different flow intensities but with similar basin areas were considered :

(a) two triple culverts passing under two of the 14.5 m bridge spans

(b) six independent culverts, 4 m wide under the flood bund, but expanding symmetically to a total width downstream of the bridge equivalent to three spans.

The first arrangement is probably cheaper than the second, there being less material in the two triple culverts and narrower inlets, although the stilling basin sidewalls are longer and 2 m higher. The second arrangement has been selected however because the much smaller flow intensities both upstream and downstream of the structure make it hydraulically superior, and the spacing of the culverts, by providing greater access, lessen constructional difficulties under the bridge and minimise interference with its pile supports.

The culvert expansion curve is that given by Ven Te Chow (Open Channel Hydraulics eqn. 17-11) for supercritical flows. The glacis profile downstream of the culvert is parabolic to provide the shortest length without flow separation.

The floor thicknesses are sufficient to prevent uplift when water is ponded in the reservoir to a level of 22.5 m with a downstream level of 17.0 m.

The underfloor drainage system has been provided to relieve the high uplift pressures during rapid drawdown following sudden closure of the gates. Several conditions were examined and the critical case found to be the rapid drawdown from about 19.2 m to 17.5 m, which arises after gate closure at the end of a 1 in 100 year flood.

8.2 Eastern Reservoir Outlet Structure

The Eastern Reservoir Outlet Structure discharges water from the Eastern Reservoir into the Lower Outfall Drain at the times when the River Jubba is low enough to permit gravity disposal of the water. The structure consists of a gated 3 x 1.20 m diameter pipe with a reinforced concrete inlet box and a standard Type 4 baffled outlet. The design discharge is 10 m^3/s and an aeration pipe has been provided at the inlet to prevent the pipes from flowing full.

The maximum design water level in the Eastern Reservoir is 18.5 m with a design downstream level of 13.05 m. The bed level upstream is 14.2 m and so the structure will be capable of operation at any water level in the Reservoir.

In order to prevent excessive seepage and uplift pressures downstream, a graded gravel filter has been provided around the outlet box. This solution has been adopted as it is more convenient than adding concrete cut-offs onto the standard outlet.

8.3 Lower Outfall Drain Outfall to River Jubba

This structure has been designed to discharge the flow from the Lower Outfall Drain into the River Jubba whenever the relative levels are suitable. The design discharge is 16.5 m^3/s at a nominal head loss of 0.25 m.

The structure consists of a gated triple 1.8 m square box culvert with an inlet box and breast wall and an enlarged Type 4 outlet. The gates are standard penstocks, and in addition, flap gates have been fitted to the inlet box breast wall to prevent the Jubba backing up the drainage system at times of high flow. It was considered preferable to locate the flap gates upstream since they would be liable to silting up if fitted on the outlet boxes.

The upstream design water level in the Lower Outfall Drain is 11.65 m and hence the structure will be capable of operation, at reduced capacity, as soon as the river level drops below this figure. A maximum river level of 11.7 m at Kamsuuma bridge (equivalent to about 150 m^3/s) is anticipated for this to occur.

The structure has been moved upstream from the river bank for some 400 m, such that it is located at the site of the Kamsuuma/Buulo Mamu road in order to give a crossing point. Although this solution involves more excavation downstream, it was considered preferable to locating the Outfall on the river bank and either providing an extra culvert in the Lower Outfall Drain or re-aligning the road for some 800 m to utilise the Outfall Structure.

8.4 Box Culverts

Box culverts have been provided in cases where the drain design discharge is too large for pipe culverts to be used. The design is similar to the canal box culverts except that in certain cases the head loss through the structure has been reduced from 0.10 to 0.05 m.

Non-return flap gates have been added at the two box culverts at the D2/Lower Outfall Drain junction to prevent any water stored in the Southern Reservoir from backing up the drainage system when the relative levels are suitable.

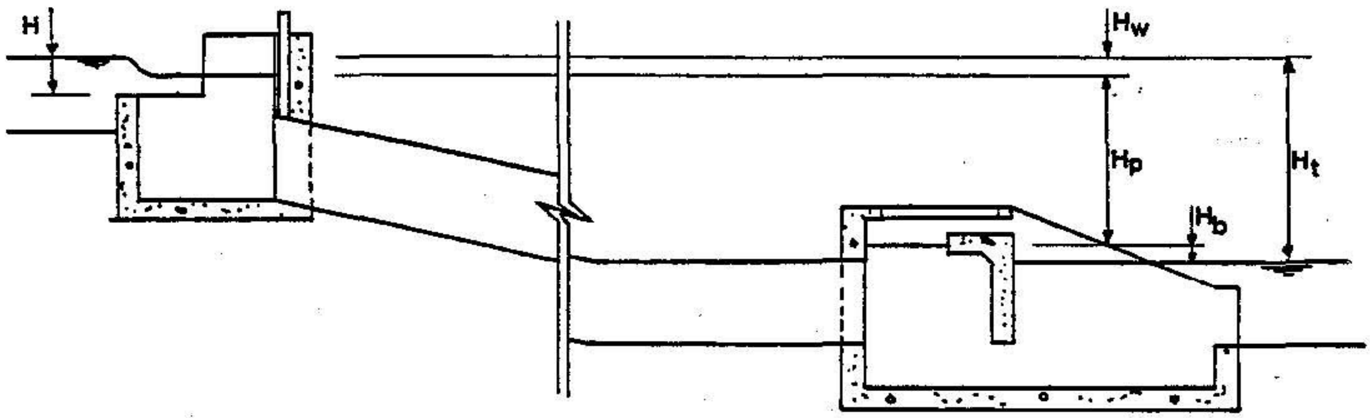
8.5 Drain Pipe Culverts

Four types of pipe culverts have been used in the design of the drainage system (identified as Types, 1, 2, 3 and 4), and they occur at drain junctions and road crossings. They comprise a pipe culvert (single, double or triple) with different inlet and outlet arrangements to suit the discharge and head loss required.

Type 1 culverts are used at the junction between field drains and main collector, branch or Lower Outfall drains. The structures comprise mass concrete inlet and outlet boxes linked by a 0.45 m diameter pipe and serve two purposes. Firstly the orifice type inlet box provides a throttle on the inflow to the drainage system. The culvert is designed to allow 45 l/s to pass from a nominal 0.3 m deep field drain flowing full. Secondly, the culvert is designed as an energy dissipator. The outlet box is set so as to induce an hydraulic jump within the pipe or box for the design flow, with the maximum drop between field and downstream drain bed level and no flow in the downstream drain. The minimum head loss at design discharge is 0.20 m and the maximum head loss checked was 5 m.

Type 2 culverts are used to provide access where there would not be a culvert in the normal design of the drain. They have drystone pitching protection at the inlet and outlet of the pipe and a design head loss of 0.05 m. The maximum discharge is based on two factors. Firstly there is the limitation on velocity imposed by the head loss. The entry and exit losses have been assessed as $V^2/2g$, where V is the velocity in the pipe. The friction head loss was calculated using the Darcy-Weisbach formula with $f = 0.004$ for the concrete pipe. Secondly the velocity has been limited to that below which would cause scour in the drain downstream, assuming a maximum tractive force of 5 N/m^2.

Culvert Types 3 and 4 are provided for cases where head losses greater than 0.05 m are required and for junctions between main collector, branch and Lower Outfall drains. They both have mass concrete inlet boxes incorporating weirs which are set so as to avoid appreciable draw-down or backing-up in the drain upstream. The Type 3 has a depressed mass concrete outlet box, whereas the Type 4 has a baffled reinforced concrete outlet box based on the USBR recommemdations (ref. "Hydraulic Design of Stilling Basins and Bucket Energy Dissipators"). The difference between the two is in the method of pipe sizing. The Type 3 design flows are the same as those for the Type 2 culvert, thus at that discharge, with the pipe flowing full, the exit velocity will again be below the scouring velocity of the drain. The depressed basin accommodates the situation where the pipe flows partly full, ensuring an hydraulic jump within the culvert. The Type 4 outlet box has been shown by model tests to dissipate the entire velocity head of the flow in the pipe. Therefore the velocity is simply that dictated by the available head (with a maximum at pipe full of 3.7 m^3/s, as suggested by the USBR). The total head loss through a Type 4 culvert is calculated as follows:



H_w	=	head loss at inlet weir
H_p	=	pipe inlet, outlet and friction losses
H_b	=	head loss under baffle
H_t	=	$H_w + H_p + H_d$

H_w is taken as 0.5 H

$$Q = 1.8\,LH^{3/2}, \quad \text{where } L = \text{weir length}$$

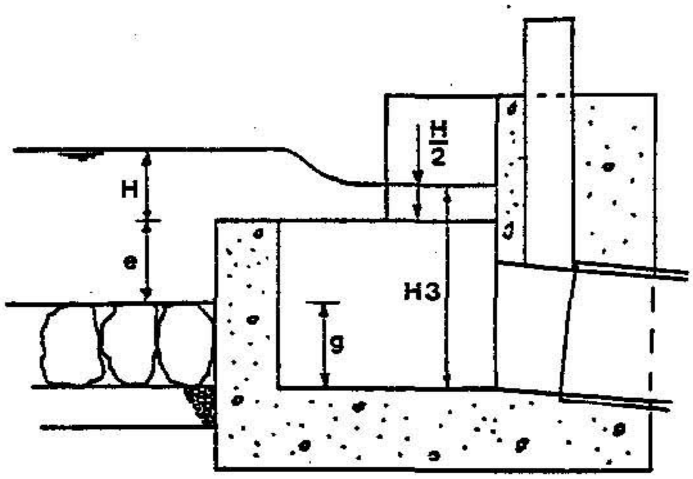
Hence
$$H_w = \frac{1}{2}\left[\frac{Q}{1.8L}\right]^{2/3}$$

$$H_p = 1.4\,\frac{V_p^2}{2g} + \frac{4\,fl}{D}\cdot\frac{V_p^2}{2g}$$

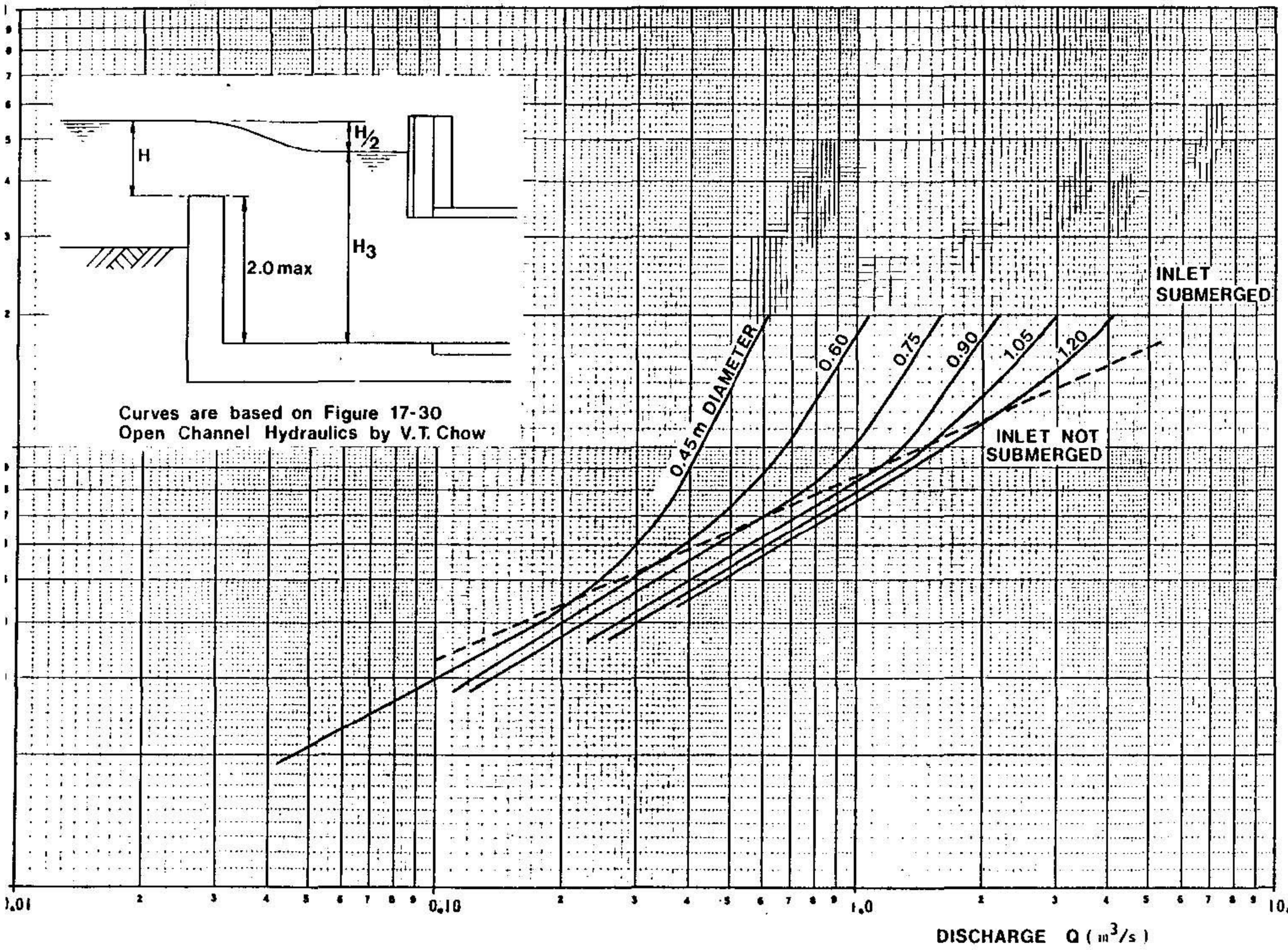
where V_p = velocity in pipe
l = pipe length
D = pipe diameter
f = Darcy-Weisbach friction coefficient (= 0.004)
$H_b = \frac{V_b}{2g}$, where V_b = velocity under baffle

Theoretically the Type 3 and 4 culverts are both satisfactory for a large range of head losses. However, an economic comparison of the two shows that for head losses over 0.20 m, the larger velocities, and so smaller pipes, that can be used in a Type 4 outweigh the greater cost of outlet box and make it the cheaper solution.

The dimensions of the culverts are given on Drawings Nr 51101 - 71, 51101 - 72. The dimensions e and g, of the Type 4 inlet box are calculated as follows:



H is calculated from $Q = 1.8\,L\,H^{3/2}$
and e = u/s depth - H
then H_3 can be taken from Figure 8.1
and $g = H_3 - \frac{H}{2} - e$



TYPE 4 CULVERT INLET BOX DESIGN

FIGURE 8.1

For the Homboy Project the largest size of pipe is 1.2 m diameter and the largest number of parallel pipes taken as 3. This sets an upper limit to the discharge of the pipe culverts. For cases where larger flows need to be accommodated, reinforced concrete box culverts have been designed.

8.6 Footbridges

Footbridges have been provided over drains at suitable locations where there are no nearby culverts. The sites have been selected such that crossing points (either footbridges or culverts) are generally not greater than 2 km apart.

The structure consists of parallel steel universal beams spanning between mass concrete abutments and reinforced concrete piers. The deck itself is formed by laying pre-cast concrete slabs on the universal beams. The width of the deck is 1.5 m, although the handrailing flares outwards to allow the easy passage of cattle.

8.7 Drain Underpasses

Drain underpasses have been provided to convey main collector or branch drains beneath a canal. The structure is basically a Type 2 drain culvert with the head loss increased from 0.05 to 0.10 m to allow for the extra length of pipe involved. Canal lining membrane and warning netting has been provided for 20 m upstream and downstream of the structure to stop excessive seepage gradients from the canal into the drain.

Although access along the drain access roads is broken by the arrangement, it was not considered necessary to provide an additional canal culvert since there is always an alternative crossing point on the canal, in the form of a cross regulator, within a short distance.

8.8 Surface Water Escapes

Surface water escapes have been provided to carry excess surface water run-off into the drainage system. They have been located along main collector, branch or Lower Outfall drains to serve low lying areas of land within the project area.

The structure comprises an inlet box discharging through a 0.45 m diameter pipe to a Type 4 outlet. In the case of the surface water escape at km 12.63 on drain D2, the pipe size has been increased to 1.20 m diameter to accomodate the potential Main Canal escape flow.

CHAPTER 9

MISCELLANEOUS

9.1 Surfaced Roads

Two types of surfaced road have been provided:

(a) The Spine Road runs along the western edge of the project area connecting the existing Jilib/Kamsuuma main road to the new Jilib/Golweyn road. The surfaced width is 8 m to accommodate two trucks passing and the total length is approximately 23.5 km.

(b) The feeder roads connect the Spine Road to the project villages. They have a surfaced width of 7 m to accommodate two cars passing and a total length of approximately 21 km.

The roads consist of a coral road base, a sub-base and a layer of selected fill overlying a specially compacted earth sub-grade. The thicknesses of the various courses depends on the type of road and the California Bearing Ratio (CBR) of the underlying material as shown on Drawing Nr 51101-46. The surfacing has been taken across the whole width of the road embankment to make maintenance easier.

The minimum radius of curvature of the road centre lines has been taken as 150 and 100 m for the Spine and feeder roads respectively, with a maximum gradient of 1 in 20 in both cases.

The designs have been based on an average traffic intensity of 100 and 20 vehicles per day in both directions for the Spine Road and feeder roads, respectively. The largest vehicle has been taken as a truck-trailer combination of 30 tons with an equivalent standard axle loading of 10.3. This was assumed to constitute about half of the total traffic, with the remainder being cars, Land Rovers and smaller trucks of 5 or 10 tons. This is equivalent to 600 and 120 standard axle loads per day in each direction for the Spine and feeder roads, respectively.

To ensure that the Spine Road does not become a short cut for Kismaayo/Mogadishu traffic, a system of traffic barriers and guards is recommended to prevent unauthorised access.

Cross drainage culverts have been provided at intervals along the Spine Road of sufficient size to take the run-off from the Marine Plain following the 1 in 5 year 24 hour rainfall of about 110 mm.

9.2 Earth Roads

Access roads 6 m wide have been provided along both sides of branch, main collector and Outfall drains, and canal bank tops will be used as inspections roads. These have all been given a 1 in 40 outwards camber to prevent excessive bank erosion. A 4 m minimum reservation has been provided to act as an inspection or field road at ground level adjacent to the non-irrigating side of each watercourse.

The minimum radius of curvature of earth road centre lines has been taken as 10 m, with a maximum gradient of 1 in 10.

9.3 Drainage Pump Stations

Four drainage pump stations have been provided to lift water from main collector drains into the Lower Outfall Drain.

TABLE 9.1

Details of Drainage Pump Stations

Pump station name	Discharge (m^3/s)	Static lift (m)
D4	1.76	1.45
D8	1.04	1.77
D10	0.99	1.91
D14	0.61	1.58

The pumps have been designed as the inclined floodlifter type directly driven by diesel engines through a 90° gear box. The delivery pipes discharge into reinforced concrete outlet boxes which have been provided with non-return flap gates to prevent back flow. A gravity by-pass has been included which will operate when the levels are suitable.

9.4 Buildings

A small building has been provided at each of the four drainage pump stations to house the generator and act as a store and workshop. In addition, operator's quarters have been provided at the pump stations, the Supply Canal Head Regulator, the major regulator groups, the Northern and Eastern Reservoir Outlets and the Lower Outfall Drain Outfall - a total of 16 operators' quarters.

The buildings are similar in design, consisting of reinforced concrete footings, blockwork walls and a flat reinforced concrete roof. Particular attention was paid to the foundation detail in an attempt to prevent the cracking caused by swelling clays such as those found in the project area. This included backfilling with sand, and a layer of polythene sheeting at ground level around the foundation to try and shed most of the rainfall away from the foundations, thus keeping the moisture content of the soil as constant as possible. A large overhang on the roof slab was also provided to assist with this process.

9.5 Flood Bunds and Reservoir Embankments

Details of the criteria used in the design of flood bunds and reservoir embankments are given in Table 9.2.

TABLE 9.2

Details of Flood Bunds and Reservoir Embankments

Bund or reservoir	Slopes inside	Slopes outside	Max seepage gradient	Special compaction	Bank top width (m)	Bank top level (m)	Minimum freeboard (m)
Northern Reservoir flood bund	1:3	1:2 min	1:5	yes	5.0	24.0	1.5
Upper Outfall Drain flood bund	1:2	1:2 min	1:5	yes	5.0	21.0 - 19.5	1.0
Dakadai flood protection embankment	1:2	1:2 min	1:5	yes	5.0	21.0	1.0
Eastern Reservoir flood bund	1:3	1:2 min	1:5	yes	5.0	19.5	1.0
Night storage reservoir embankments	1:3	1:2	1:7	no	5.0 min	varies	0.5

9.6 Shelterbelts

A 5 m wide reservation has been provided along one side of distributary canals and main collector and branch drains for the planting of trees to act as shelterbelts.

9.7 Villages

It is proposed that some 8 850 families will be settled in the project area, each family being allocated 1 ha of irrigated land. The families will be housed in ten villages, one in each irrigation block as shown on the Project Area Map at the end of this note. The villages will all be new, except that for Irrigation Block 5 which will be an extension of Homboy. The villages have been sized on the basis of an average area of 365 m^2 per house which includes an allowance for infrastructure and the criteria used for site selection were:

(a) Walking distance from village to fields generally not to exceed 4 km

(b) Lengths of roads and number of canal crossings to be minimised

(c) Use of irrigated land for roads and villages to be avoided

(d) Villages to be located whenever possible on soils with good foundations conditions - i.e the heavier swelling clays and marine plain soils to be avoided.

Details of the villages are given below:

TABLE 9.3

Details of Villages

Village	Dwellings	Area (ha)	Soil
1	600	21.9	Beach remnant
2	825	30.1	Beach remnant
3	700	25.6	Alluvium
4	925	33.8	Marine plain
5	1 025	37.4 (Homboy extension)	Beach remnant
6	800	29.2	Marine plain
7	1 125	41.1	Alluvium
8	575	21.0	Beach remnant
9	1 525	55.7	Alluvium
10	750	27.4	Beach remnant