SOMALI DEMOCRATIC REPUBLIC
MINISTRY OF AGRICULTURE

MOGAMBO IRRIGATION PROJECT

Design Criteria

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MOGAMBO IRRIGATION PROJECT

DESIGN CRITERIA NOTE

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CHAPTER 1

INTRODUCTION

1.1 General

This note outlines the criteria which have been adopted in the preparation of designs for the Mogambo Irrigation Project. It has been based on the Design Statement issued in January 1982.

Fully detailed supporting calculations are included in a separate volume.

1.2 Background

The Mogambo Project was the subject of a feasibility study prepared by Tippetts-Abbett-McCarthy-Stratton of New York and Financial and Technical Services of Cairo (TAMS/FINTECS), for which the final report was submitted in May 1977. In December 1978 Sir M. MacDonald and Partners were commissioned to carry out a Supplementary Feasibility Study, and the draft report for this supplementary study was submitted in September 1979.

The findings of the supplementary study were subsequently reviewed by the Client (State Planning Commission, now known as Ministry of National Planning) and the Funding Agencies (Kuwait Fund for Arab Economic Development, KFAED, and Kreditanstalt fur Wiederaufbau, KFW). In January 1980 it was decided that an Additional Study should be undertaken by Sir M. MacDonald and Partners. This additional study examined the option of a smaller initial development at Mogambo (some 2 000 ha net instead of the 6 400 ha net originally proposed). The draft report for the additional study was submitted in March 1980.

1.3 Alternative Developments Considered

In the Additional Study four alternative developments were considered. These comprised two alternative irrigation layouts (A and B), each of approximately 2 000 ha net, and two alternative cropping patterns (1 and 2).

Layout A was designed to allow unhindered expansion to the full development (6 400 ha) and, as a result, some of the project components were more expensive than for Layout B.

Layout B was conceived as the least expensive means of developing the proposed 2 000 ha, with no provisions for future expansion (but not precluding such expansion).

Both layouts utilised the basin clay soils nearest to the river and comprised mostly surface irrigation with a relatively small trial area of overhead irrigation. This contrasts with the full development proposals for which the areas of surface and overhead irrigation were approximately equal (3 300 ha and 3 100 ha net respectively).

The alternative cropping proposals examined in the additional study are indicated in Table 1.1.

TABLE 1.1

Alternative Cropping Patterns Considered in the Additional Study

Cropping pattern reference	Crop (irrigation system, net area in ha) gu season	der season
1	Rice (surface, 2 052)	Maize (surface, 1 458) Cotton (overhead, 163)
2	Rice (surface, 1 539)	Rice (surface, 1 539) Cotton (overhead, 163)

Economic analysis of the alternatives showed that cropping pattern Nr 2 (double cropped rice) was the more profitable of the two, but neither of the two alternative layouts showed a significant economic advantage over the other. Since Layout A would permit further expansion up to a total net area of 6 400 ha it was recommended in preference to Layout B. The recommended option was thus A2 (Layout A with Cropping Pattern 2). However, since it was recognised that double cropping of rice might prove difficult to manage, it was also recommended that the irrigation system be designed to enable a changeover to the alternative cropping pattern.

1.4 Brief Description of the Project

The project (referred to as Phase I) comprises a total net irrigable area of 2 215 ha made up of 2 052 ha surface irrigation and 163 ha sprinkler irrigation. Figure 1.1 shows the project layout and indicates the boundary of the Phase II development area (6 400 ha net).

Irrigation water is pumped from the Juba river at the northern end of the project and conveyed to the irrigable area in a main canal. From the main canal water passes into earth reservoirs and thence via distributary canals to the field channels. The sprinkler areas are served by pump stations, one on the main canal and one on a distributary canal.

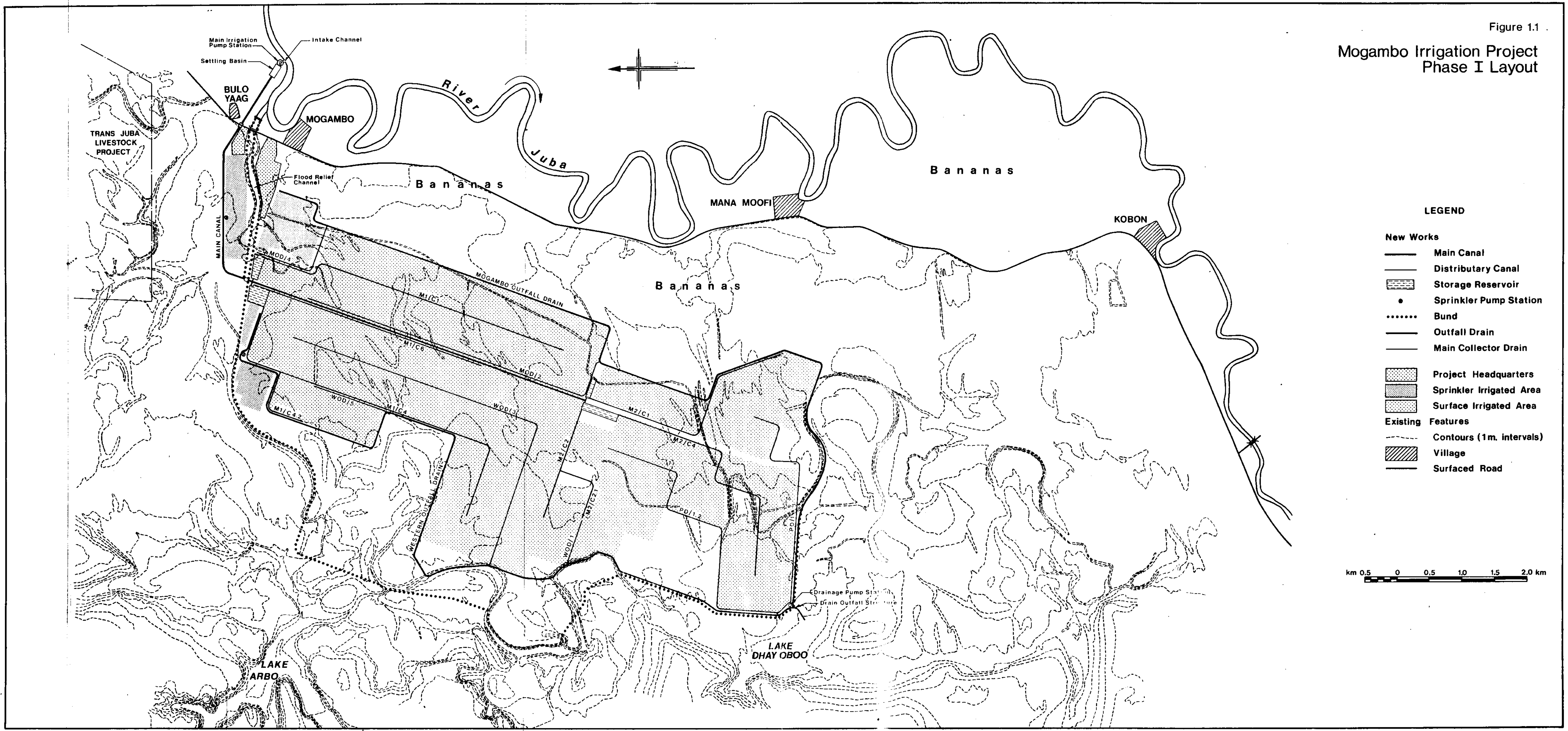
Surface drainage is achieved by a network of open drains flowing southwards. Part of the project area requires pumped drainage because it is too low to allow effective drainage by gravity.

Flood protection works comprise bunds to the north and west and rehabilitation of the existing flood relief channel and regulator. Flood waters will thus be directed around the western project boundary to flow southwards into the Dhesheeg Waamo natural depression.

A project headquarters with the necessary buildings and services has been provided for at the northern end of the project area, adjacent to the main road.

Figure 1.1

Mogambo Irrigation Project Phase I Layout



CHAPTER 2

FLOOD PROTECTION

2.1 Introduction

Flooding of agricultural land along the Lower Juba river has been a problem for many years. Individual projects have constructed their own protection works to alleviate flooding, and this has generally resulted in a worsening of the problem for unprotected areas downstream. Within the Mogambo project area, the riverain banana growers constructed a bund which protects the plantations from overland flow from the west. Such flow emanates from the flood escape structure just to the north of Mogambo village, from run-off, and from flood plain flow further north.

In the last few years, the Juba Sugar Project and the Fanole scheme have constructed bunds on the right and left river banks upstream of Mogambo. These bunds restrict flood plain flow and this results in greater discharges in the main river channel downstream.

2.2 Flood Hydrology

Long-term hydrological records for the Juba river are available for only a few stations and this has made flood flow prediction difficult. In the Supplementary Feasibility Study flood flow return periods for various locations were estimated. These are reproduced in Table 2.1.

TABLE 2.1

Flood Flows and Return Periods

Return period (years)	Flow (m^3/s) Lugh Ganana (1)	Kaitoi (1)	Kaitoi (2)	Kamsuma (1)
10	1 450	1 000	1 120	710
50	1 950	1 190	1 460	780
100	2 160	1 260	1 600	800

Source: (1) Supplementary Feasibility Study, MMP 1979

(2) Juba Sugar Project - Note of Flood Protection Works, MMP 1978

Return periods are based on flows at Lugh Ganana for which records have been kept since 1951. Estimates for Kaitoi and Kamsuma have been based on derived relationships between flows at Lugh and Kaitoi, and flows at Kaitoi and Kamsuma. Such relationships are not well defined and flood estimates are necessarily approximate.

It is now thought that the flows for Kaitoi presented in the Feasibility Study (MMP, 1979) are underestimates and that those put forward in an earlier report on Juba Sugar Project flood protection (MMP, 1978) are more realistic. Both sets of data are reproduced in Table 2.1.

It is clear that there can be no simple relationship between the flow at Kaitoi or Kamsuma and that at Lugh Ganana. Flows at Lugh are dependent on rainfall in the Ethiopian catchment. The extent to which these flows are attenuated between Lugh and Kamsuma will depend on, amongst other things, the shape of the flood hydrograph, antecedent conditions in the flood plain, and in-flow to the river downstream of Lugh. Furthermore the construction of flood bunds along the river will tend to increase the channel flow downstream.

The differences between the 1977 and 1981 floods are indicated by the flood flow figures in Table 2.2. It can be seen that, although at Lugh the 1981 flood flow was only about 75% of the 1977 peak, flood flows in the lower river exceeded the 1977 peak.

TABLE 2.2

Flood Flows at Lugh Ganana, Kaitoi and Kamsuma

Year of flood	Flows (m^3/s) Lugh Ganana	Kaitoi		Kamsuma
1977	2 080(4)	1 240(1)	1 350(5)	790(1)
1981	1 570(4)	1 400(2)	2 000(5)	700(3)

Notes: (1) Estimated using relationships derived in the Supplementary Feasibility Study (1979)

(2) Estimate by MMP from observations made by MMP staff on the Juba Sugar Project (May 1981)

(3) Based on estimated flood levels at Kamsuma and Arara bridges.

(4) Based on revised stage records for Lugh (MMP, March 1982)

(5) Based on Kaitoi rating curve (MMP, March 1982)

2.3 The 1981 Flood

The 1981 gu season rains in the Lower Juba started in March and precipitation was above average. Throughout April river flows were high (500 to 700 m^3/s at Mareri, JSP). River levels continued to rise and reached their peak in mid-May. During the second week in May, the river burst its right bank upstream of Fanole. The flood water flowed southwards through an old channel system and eventually entered the southern end of the Juba Sugar Project. Flood plain flows continued southwards into the Mogambo project area, filling the channel remnants and depressions and finally entering the lake known as Dhesheeg Waamo (see Figure 2.1).

The flood plain flow down the western boundary of the sugar project was estimated at 350 to 400 m^3/s. This was by far the most significant flow outside the main river channel. Several other less significant breaches occurred on the left bank, in the new Fanole project bund, and in the main road between Gelib and Kamsuma and between Kamsuma and Arara bridge. The river also burst its right bank downstream of the sugar project and this flow joined the major overland flow travelling south into the Mogambo project. However, no flow passed over the main road within the vicinity of the Mogambo project. Estimates of maximum river level made during the Flood Damage Assessment Report (MMP September 1981) suggest a flow of 650 to 750 m^3/s in the reach Kamsuma to Jamama. This compares with the estimated 950 m^3/s at Mareri. The difference of 200 to 300 m^3/s must constitute the overland flow on the left and right banks between Mareri and Mogambo.

2.4 Mogambo Flood Protection Works

The construction of the proposed Bardheere dam, in the upper reaches of the Juba river, would significantly reduce the flooding problem in the Mogambo area and the project would require only very limited protection works. However, even if construction of the dam were to start in the near future, Phase I of the Mogambo project would be in operation well before the dam is completed. Substantial flood protection works are therefore necessary.

The proposed works comprise two major elements:

- rehabilitation of the existing flood regulator at Bulo Yaag and reforming and extending the existing flood relief channel to accept a flow of up to 100 m^3/s.

- bunding of the northern and western boundaries of the project.

Thus flood flows diverted into the flood relief channel, or emanating from the river further upstream, will be directed safely around the western boundary of the scheme.

A design flow of 300 m^3/s has been adopted for the western flood defences, but sufficient freeboard has been allowed to accept a peak flow of 400 m^3/s. The figure of 400 m^3/s has been based on the observed flows for the 1977 and 1981 floods, assuming a peak flow of 1 350 m^3/s at Kaitoi. The assumed pattern of flooding is indicated in Figure 2.2.

It is impossible to assign, with confidence, a return period to the flow of 1 350 m^3/s at Kaitoi. The best estimate based on available data is of the order of 30 years. This means that, a 10 year period between the construction of the Mogambo project and the assumed completion of Bardheere Dam, the chances of this flood being equalled or exceeded are less than 1 in 3.

Design criteria for the flood protection works are summarised in Table 2.3.

TABLE 2.3

Design Criteria - Flood Protection Works

Flood relief channel:	
Capacity	100 m^3/s
Bed width	55 m
Maximum depth of flow	2.0 m
Freeboard above design water level	0.6 m
Western flood bund:	
Design discharge	300 m^3/s
Peak discharge	400 m^3/s
Freeboard above design water level	0.6 m
Bund top width	4.0 m minimum
Bund side slopes	1 in 2

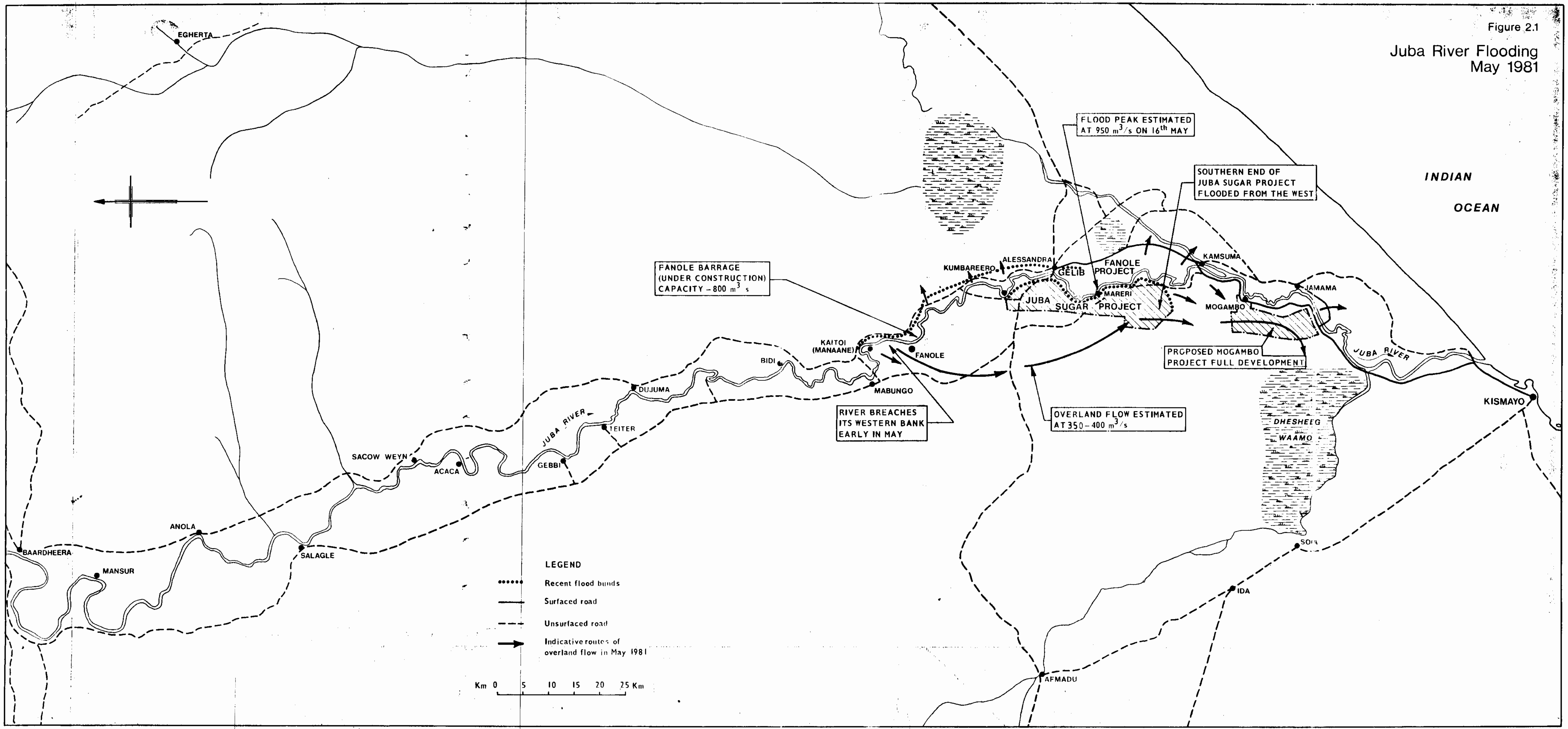
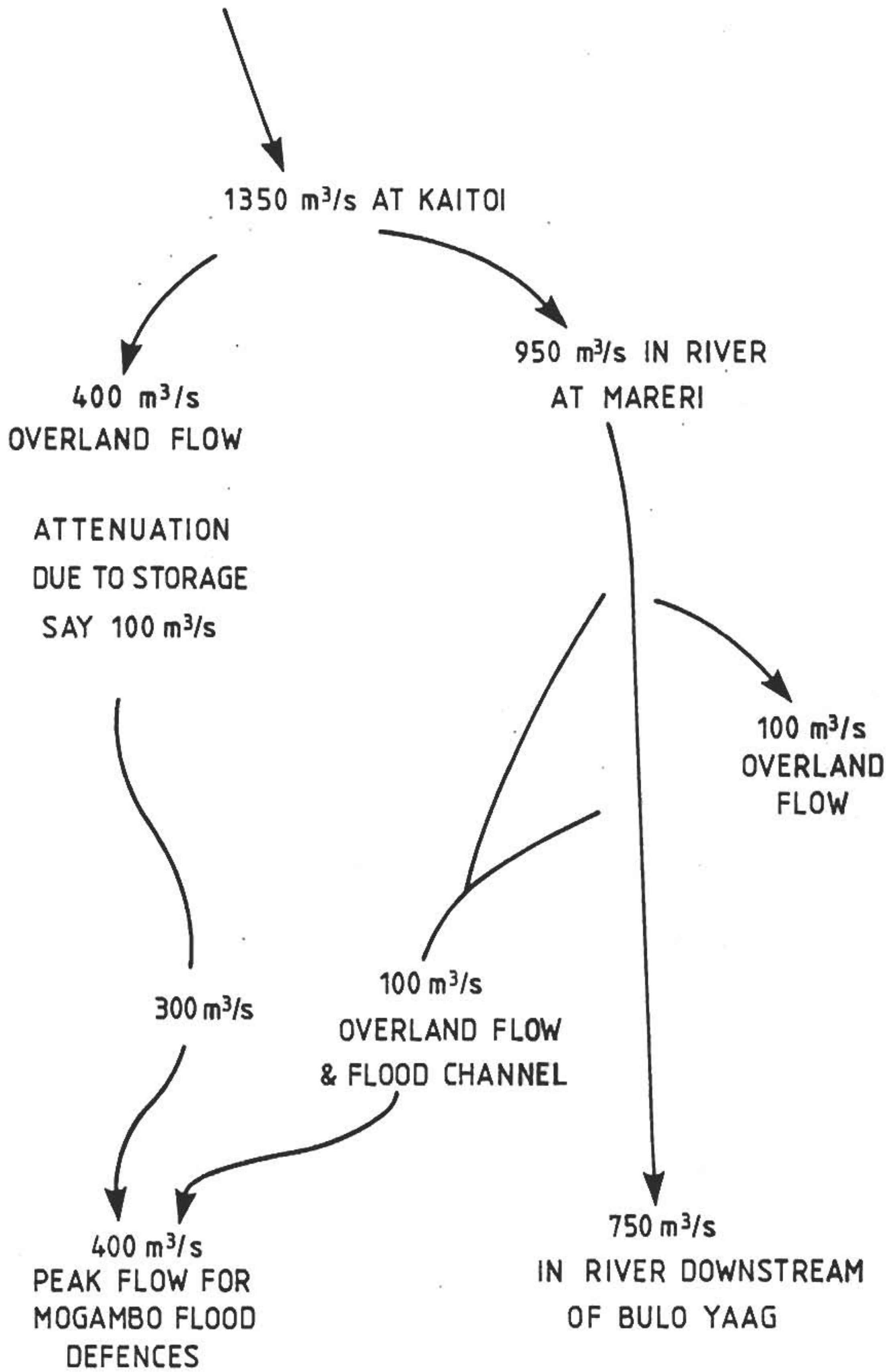


Figure 2.1
Juba River Flooding
May 1981

Figure 2.2

Assumed Flooding Pattern for Mogambo Flood Defences



CHAPTER 3

IRRIGATION AND DRAINAGE SYSTEM

3.1 General Description

Irrigation water for the project will be abstracted by pumping from the Juba river just upstream of Mogambo village. Downstream of the pump station the flow will pass through a settling basin and into the main canal.

From the main canal, the flow will pass into storage reservoirs and thence into distributary canals. The reservoirs provide overnight storage so that the pump station and main canal can operate 24 hours per day whilst surface irrigation takes place during the day only. The two Phase I sprinkler areas will be served by separate pump stations, one on the main canal and one on a distributary canal.

Each distributary canal will serve a number of standard field units (27 ha net) for surface irrigation.

Each irrigation unit will have a drainage channel linking into a system of collector drains which will connect to the outfall drains on the eastern and western boundaries. Drainage water from part of the project area will require pumping into the western outfall drain since the land is too low to be drained effectively by gravity.

3.2 Irrigation Requirements

3.2.1 Introduction

Canal and pump station design capacities have been based on the irrigation water requirements for the 1 in 5 dry year. Two cropping patterns have been examined for the surface irrigation areas to ensure that the system is capable of serving either. These are:

Reference	gu season	der season
1	Rice (100%)	Maize (70%)
2	Rice (70%)	Rice (70%)

Sprinkler system capacities have been based on the peak requirements of the cotton crop, assuming 100% cropping intensity.

3.2.2 Climatic Parameters

Climatic parameters were discussed fully in the Supplementary Feasibility Study. Reference crop evapotranspiration (ETo) and effective rainfall (Re) values were calculated for each month. From these, "10 day" values for ETo and Re have been estimated, and these are reproduced in Table 3.1. It should be noted that effective rainfall values are those with an 80% exceedance probability for each month.

3.2.3 Crop Calendars and Crop Factors

The crop calendars adopted for the purpose of assessing peak requirements are summarised below. It should be noted that the rice is direct drilled and not transplanted. The der season maize crop is not considered as its peak requirement will be substantially less than that for rice.

(a) Rice (gu season)

Sowing	:	10th April - 10th May
Harvesting	:	10th August - 10th September
Prewatering	:	10 days before sowing
Last irrigation	:	20 days (at least) before harvesting

(b) Rice (der season)

Sowing	:	1st September - 10 October
Harvesting	:	1st January - 10 February
Prewatering	:	10 days before sowing
Harvesting	:	20 days (at least) before harvesting

(c) Cotton (der season)

Sowing	:	1 - 31 August
Harvesting	:	1 January onwards
Prewatering	:	10 days before sowing
Last irrigation	:	140 days after sowing

Actual cropping calendars will depend on water availability, crop variety and variations in the climatic parameters. The above listed dates are considered sufficiently representative for the calculation of water requirements.

Crop factors have been based on those presented in the Supplementary Feasibility Study. Peak crop factors are 1.1 for rice (occuring April - July for the gu season crop, September - December for the der season crop) and 1.1 for cotton (occuring October - December).

3.2.4 Prewatering Requirement

The following prewatering requirements have been assumed :

Rice (gu season)	-	150 mm gross
Rice (der season)	-	100 mm gross
Cotton (der season)	-	125 mm gross

The der season rice prewatering requirement is lower than that for the gu season rice since for 50% of the area the der crop follows close behind the gu crop. The cotton crop follows no other crop but is planted after the gu season rains.

TABLE 3.1

Reference Crop Evapotranspiration and Effective Rainfall

Month	Period	Nr of days	ETo (mm)	ETo (mm/day)	Re (mm)
January	1	10	56	5.6	0
	2	10	58	5.8	0
	3	11	65	5.9	0
February	1	10	60	6.0	0
	2	10	61	6.1	0
	3	8	49	6.1	0
March	1	10	65	6.5	0
	2	10	64	6.4	0
	3	11	66	6.0	0
April	1	10	55	5.5	4
	2	10	52	5.2	6
	3	10	50	5.0	7
May	1	10	49	4.9	7
	2	10	47	4.7	7
	3	11	49	4.5	8
June	1	10	44	4.4	12
	2	10	43	4.3	12
	3	10	42	4.2	12
July	1	10	42	4.2	11
	2	10	42	4.2	11
	3	11	48	4.4	10
August	1	10	46	4.6	5
	2	10	47	4.7	3
	3	11	54	4.9	0
September	1	10	51	5.1	0
	2	10	53	5.3	0
	3	10	53	5.3	0
October	1	10	52	5.2	0
	2	10	52	5.2	0
	3	11	56	5.1	0
November	1	10	49	4.9	0
	2	10	48	4.8	0
	3	10	50	5.0	0
December	1	10	51	5.1	0
	2	10	52	5.2	0
	3	11	58	5.3	0
TOTAL	-	365	1 879	(5.15 av)	-

3.2.5 Irrigation Efficiency

In order to derive canal capacities at each level in the system it is necessary to introduce the concept of irrigation efficiency. No irrigation system is able to deliver exactly the quantity of water required to sustain crop growth - inevitably there is wastage. Losses occur throughout the system and are made up of the following components :

Management losses :	water which runs to waste as a result of imperfections in the operation of the system.
Seepage losses :	water seeping from canals and reservoirs and through bunds surrounding the rice fields.
Evaporation losses :	evaporation from free water surfaces in canals and reservoirs and transpiration from plant growth in canals and reservoirs.
Percolation losses :	water which, although applied to the field, passes down through the soil profile beyond the root zone and is therefore lost to the crop.

It is convenient to define two efficiency terms to allow for the above losses. These are :

- conveyance efficiency
- field efficiency.

Conveyance efficiency takes account of all losses between the main pump station and the head of a unit channel. Field efficiency allows for all losses beyond the unit feeder channel head regulator (surface irrigation).

(a) Conveyance Efficiency

Seepage losses from the main canal and distributary canals have been estimated using the Moritz formula with a coefficient appropriate for the heavy clay soils of the project area.

$$S = 4.7\,L\sqrt{A}$$ (see Supplementary Feasibility Study, Annex 5)

where S = seepage loss (l/s)

L = length of canal reach (km)

A = cross sectional area of flow (m^2).

This expression gives an average seepage loss of about 1.5 m^3/s per million square metres of wetted surface. Evaporation losses from canals can be ignored.

For the storage reservoirs an average evaporation rate of 5 mm/d can be assumed, based on the climatic data presented earlier. Seepage losses can be estimated using the formula for canals. With a total reservoir plan area of some 220 000 m^2 these losses are equivalent to a flow rate of about 90 l/s.

To be added to the seepage and evaporation losses are those losses resulting from mismanagement. These should be relatively small because the storage reservoirs are able to absorb differences between supply and demand. A figure of 5% is suggested which will allow for water discharging over the distributary canal tail escapes into the drainage system.

Combining these losses yields an overall conveyance efficiency of 0.85.

(b) Field Efficiency (Surface Irrigation)

For deep percolation losses a rate of 5 mm/d was assumed in the Supplementary Study. In fact, the rate will vary throughout the season. Initially percolation rates will be high since the soil will tend to have cracked in the preceding non-irrigated period. As the soil becomes wetter and the ponded level in the rice fields is raised, percolation rates will reduce significantly. As an average, then, the 5 mm/d is considered to be appropriate. Over a growing season this represents about 50% of the total net crop requirement (including the water required to fill the rice basins).

Losses due to mismanagement, that is water lost through field bunds and that escaping down unit channels into drains, are estimated at about 10% of the total net application.

(c) Field Efficiency (Overhead Irrigation)

Efficiencies for sprinkler irrigation systems have been well documented, field efficiencies varying between 0.60 and 0.85 depending mainly on :

- spacing of sprinklers and laterals
- wind speeds
- management of the system

For Mogambo, in the Supplementary Study, a field efficiency of 0.75 was assumed. This was based on a fairly coarse grid spacing (18 x 18 m) but with irrigation taking place only at night and thus avoiding the high wind speeds associated with daylight hours. For the small trial area we propose that various grid spacings and irrigation regimes are investigated so that the most effective can be found by field trials. We have therefore used a field efficiency of 0.70 to allow for possible daytime irrigation with a variety of grid spacings.

3.2.6 Pump Station Capacity

(a) Cropping Pattern Nr 1 - Gu Season Rice

With 100% rice in the gu season, the peak requirement will occur during May. At this time the process of flooding the rice basin will be under way. It is assumed that the water depth in a basin will be raised gradually from zero to 150 mm over a 30 day period, starting 20 days after the rice is sown. Ponding cannot start before this, as the seedlings will be too small. The rate of ponding should follow the growth of the seedlings and it is assumed that the application which brings the ponded depth to its maximum of 150 mm takes place

50 days after the crop is sown. Assuming that irrigation applications take place at 10 day intervals, ponding water in addition to the plants' consumptive needs is added on days 20, 30, 40 and 50 after sowing. Table 3.2 summarises the water applications required in the 1 in 5 dry year.

TABLE 3.2

Peak Irrigation Requirements for Gu Season Rice

Month	April			May			June
Period	1	2	3	1	2	3	1
ETo (mm)	55	52	50	49	47	49	44
Kc	-	1.10	1.10	1.10	1.10	1.10	1.10
ETcrop (mm)(1)	-	57	55	54	52	54	48
Prewatering (mm)(2)	75	-	-	-	-	-	-
Ponding (mm)	-	-	-	35	35	35	45
Re (mm)	4	6	7	7	7	8	12
Net requirement (mm)	71	51	48	82	80	81	81

Notes : (1) ETcrop = Kc x ETo, Kc = crop factor
(2) Net application. Equivalent to 150 mm gross

From Table 3.2 it can be seen that the peak net 10 day requirement is 82 mm in May. This is made up of 47 mm net crop requirement (ETcrop - Re) plus 35 mm ponding.

To be added to this are 50% for deep percolation and 10% for management losses, giving a gross field requirement of 1.6 x 82 = 131 mm. With a conveyance efficiency of 0.85 the gross daily requirement at the main pump station will be 15.4 mm/d. This is equivalent to a continuous flow of 1.8 l/s/ha, or 3.7 m^3/s for 2 052 ha of gu season rice.

(b) Cropping Pattern Nr 1 - Der Season Maize

The der season maize requirement will inevitably be lower than the gu season rice as maize has a much lower water demand and, in any case, is only planted on 70% of the area.

(c) Cropping Pattern Nr 2 - Gu Season Rice

Irrigation requirements will be as for cropping pattern Nr 1, but with only 1 550 ha planted, the pump station capacity required would be less, at 2.8 m^3/s.

(d) Cropping Pattern Nr 2 - Der Season Rice

Der season rice will be more water demanding than the gu season crop because evapotranspirative conditions are more severe and effective rainfall is zero.

Table 3.3 summarises the calculation of irrigation requirements in the 1 in 5 dry year.

TABLE 3.3

Peak Irrigation Requirements for Der Season Rice

Month	September			October			November
Period	1	2	3	1	2	3	1
ETo (mm)	51	53	53	52	52	56	49
Kc	-	1.10	1.10	1.10	1.10	1.10	1.10
ET crop (mm)(1)	-	58	58	57	57	62	54
Prewatering (mm)(2)	50	-	-	-	-	-	-
Ponding (mm)	-	-	-	35	35	35	45
Re (mm)	0	0	0	0	0	0	0
Net requirement (mm)	50	58	58	92	92	97	99

Notes: (1) ET crop = Kc x ETo, Kc = crop factor
(2) Net application. Equivalent to 100 mm gross

From Table 3.3 it can be seen that the peak net 10 day requirement is 99 mm in November. This is equivalent to a continuous flow of 2.2 l/s/ha, requiring a pump station discharge of 3.4 m^3/s for 1 550 ha of rice. Sprinkler irrigation of cotton will also be taking place during this month, but the additional demand at the main irrigation pump station will be less than 0.2 m^3/s. The total demand would thus be less than for the gu season rice crop.

(e) Design Capacity

The pump station has been designed to meet a peak requirement of 3.7 m^3/s, continuous discharge.

3.2.7 Canal Design Capacities

The main canal and main canal structures have been designed for the full development as envisaged in the Supplementary Feasibility Study (See Section 4.2 herein).

Unit feeder channels must be capable of delivering the peak discharge to the field units. The der season peak requirement of 99 mm in 10 days in November is equivalent to a gross requirement of 1.6 x 99 = 160 mm at the head of the unit channel. This requires a unit channel flow of 85 l/s assuming a maximum of 14 hours irrigation per day. The unit feeder channel section has been designed to accept this flow (see Section 4.5).

For the distributary canals a design capacity of 100 l/s per unit has been adopted. This allows for seepage and management losses in the canals and includes an allowance for silting and weed growth.

3.3 Pump Station Design

3.3.1 General

The pump station is required to deliver a peak discharge of 3.7 m^3/s against a maximum static head of 6.1 m. This should be achieved with a minimum of three pumps so that there is sufficient flexibility to meet lower flow demands.

The pump station has been designed on this basis and allowance has been made for expansion to the full development. For Phase I three pumps are proposed and this would be expanded to five pumps for Phase II, as detailed below.

Pump diameter (m)	Pump discharge (m^3/s)	Number of pumps Phase I (2 215 ha)	Phase II (6 400 ha)
1.0	2.2	2	3
0.7	1.1	1	2

This arrangement gives considerable flexibility of operation and ensures sufficient stand-by capacity.

3.3.2 Type of Pump

The designs in the Supplementary Feasibility Study and in the Additional Study were based on inclined axial flow 'floodlifter' pumps. These pumps were chosen because they need minimal structural works for the pump station. They have the further advantage of being adjustable - the length of casing can be increased/ decreased in order to lower/raise the intake level. This feature might be required should there be a significant change in river regime following the construction of Bardhere dam.

The alternative of centrifugal pumps mounted in a conventional pump station with a sump was discounted because of higher cost and the problems resulting from the wide range of river levels. Inclined axial flow pumps have therefore been adopted.

3.3.3 Layout of Pump Station

The layout of the pump station is generally as suggested in the Supplementary Feasibility Study.

The maximum flood level adopted in the Supplementary Study was 13.1 m and the pump station floor was set at 13.7 m. These two levels correspond to river flows of about 800 m^3/s and 900 m^3/s respectively and should therefore be adequate.

3.4 Settling Basin Design

The settling basin will be as described in the Additional Study. Its dimensions are :

Length	200 m
Bed width	36 m
Side slope	1 in 2
Maximum water depth	2.7 m

3.5 The Surface Irrigation Field Unit

The standard field unit is illustrated in Figure 3.1. Unit shapes and sizes have been kept standard as far as is possible but irregular units are required to fit the plot boundaries in some areas. The aim has been to ensure that all field units are about 27 ha net, comprising 27 one hectare plots.

The standard unit is square so as to permit orientation in either of two directions. This ensures that the unit can be aligned such that the rectangular plots have their short sides parallel to the predominant slope thus reducing land levelling requirements.

Each unit will be served by its own offtake from a distributary canal, discharging into a unit feeder channel. Water will be conveyed onto the plots through flexible syphon pipes. If the peak discharge in the unit channel proves to be too much for one plot to handle, two or more plots can be served simultaneously.

The unit is divided into three sections each of 9 ha and each served by a branch of the unit channel. These unit channel branches must be designed to carry peak flow since the branches will operate in rotation. Each plot within the unit will have a unit drain along one boundary and a unit collector drain connecting these to the main collector drain.

A total of 25 siphon pipes of 64 mm diameter will supply the peak flow of 85 l/s under the design minimum head. Where the actual command is greater than minimum fewer siphon pipes can be used. Field trials will be required to establish the needs of each plot.

Access into the field unit is provided by a 4.5 m wide graded earth road along each unit drain.

3.6 Sprinkler Irrigation System

(a) Introduction

In the Supplementary Feasibility Study, it was demonstrated that, in theory, operating the sprinkler irrigation system during the night time hours only was an economically viable alternative to the traditional 2 x 11 hour sets/day. Irrigating only at night, when wind speeds are very low, results in a higher irrigation efficiency, but requires greater capacity in the pump stations and pipelines. It also allows much greater time for moving the sprinkler laterals, thus reducing the peak labour demand, and all movement can take place during daylight hours which considerably eases the supervision problem.

It is important that the small area of sprinkler irrigation included in the Phase I development is used to determine the optimum system for the 3 000 ha which could be developed in Phase II. For this reason the Phase I system has been designed to permit various options to be tried out, as follows.

(i) both night-time only (12 hours) and day-night (2 x 11 hours) irrigation can be tested under field conditions,

(ii) three sprinkler spacing patterns have been allowed for, namely 18 x 18 m (as in the Feasibility Study), 18 x 12 m and 12 x 12 m, and

(iii) the whole system can ultimately be changed over to one irrigation regime with one sprinkler pattern, the choice depending on the results of the trials.

(b) The Sprinkler Irrigation Area

Two small sprinkler irrigation areas have been included, one served by pumping from the main canal (122 ha net) and one from distributary canal M1/C4 (41 ha net). The standard unit size adopted is 20 ha but non-standard areas have been included to make most use of available land.

The area served by the distributary canal requires a small reservoir off the canal. This is so that, regardless of the time that sprinkler irrigation is taking place, flow in the distributary canal during the day is for surface irrigation only. This will considerably simplify management of the system.

(c) System Details

Details of the five system variations included are presented in Table 3.4.

System 1 is that proposed in the Feasibility Study with an 18 x 18 m grid and irrigation taking place only at night, with a 12 day interval. Irrigation efficiency is relatively high because wind effects are minimal.

System 2 has the same 18 x 18 m grid but with day and night irrigation (2 x 11 hour sets/day). A 9½ day interval has been adopted, the half day ensuring that plots are watered alternatively day and night thus reducing the effects of wind. Irrigation efficiency has been assumed to be lower than that for System 1 because of the wind problem.

System 3 is another variation on the 18 x 18 m grid with day and night irrigation but in a single 22 hour set. Application rates are relatively low and therefore irrigation efficiency is lower than that for Systems 1 and 2.

System 4 has an 18 x 12 m grid (sprinklers at 12 m), 2 x 11 hour sets/day, and a 9½ day irrigation interval. The reduced sprinkler spacing helps to improve irrigation efficiency.

System 5 has a 12 x 12 m grid with 2 x 11 hour sets/day and a 9½ day interval. Irrigation efficiency is higher than the four previous systems because of the close grid of sprinklers.

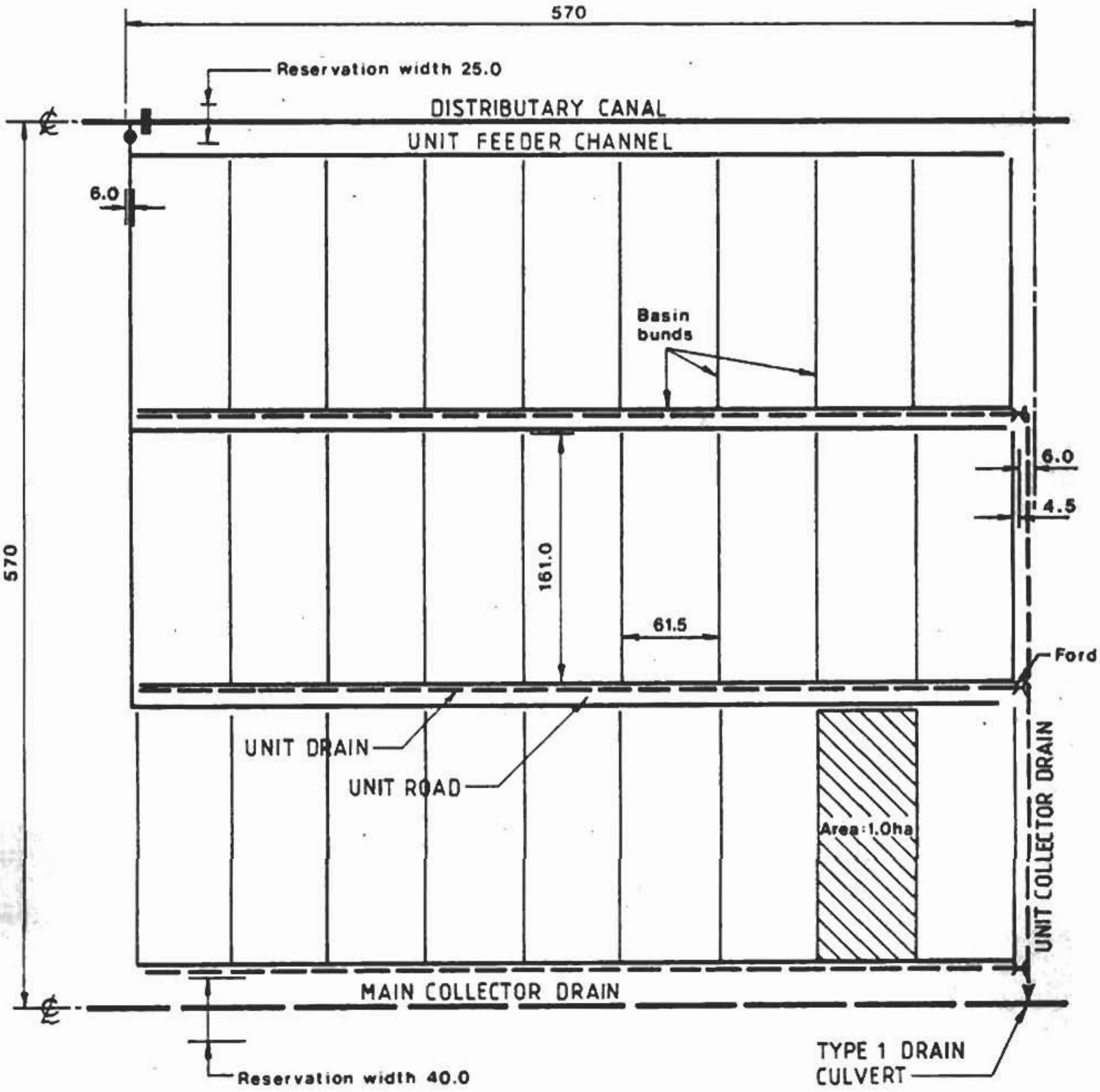
The sprinkler discharge values quoted in Table 3.4 have been based on the peak net irrigation requirement of 5.8 mm/day (cotton crop, late December).

Single nozzle sprinklers are proposed as these give the best all-round performance.

A 54 m hydrant spacing will be adopted giving three positions per hydrant for the 18 m lateral spacing. The 12 m lateral spacing will require a selection of extension pieces to match the 54 m hydrant spacing. If the 12 x 12 m grid proves to be the best option then a 36 or 48 m hydrant spacing would be adopted for any Phase II sprinkler development.

Figure 3.1

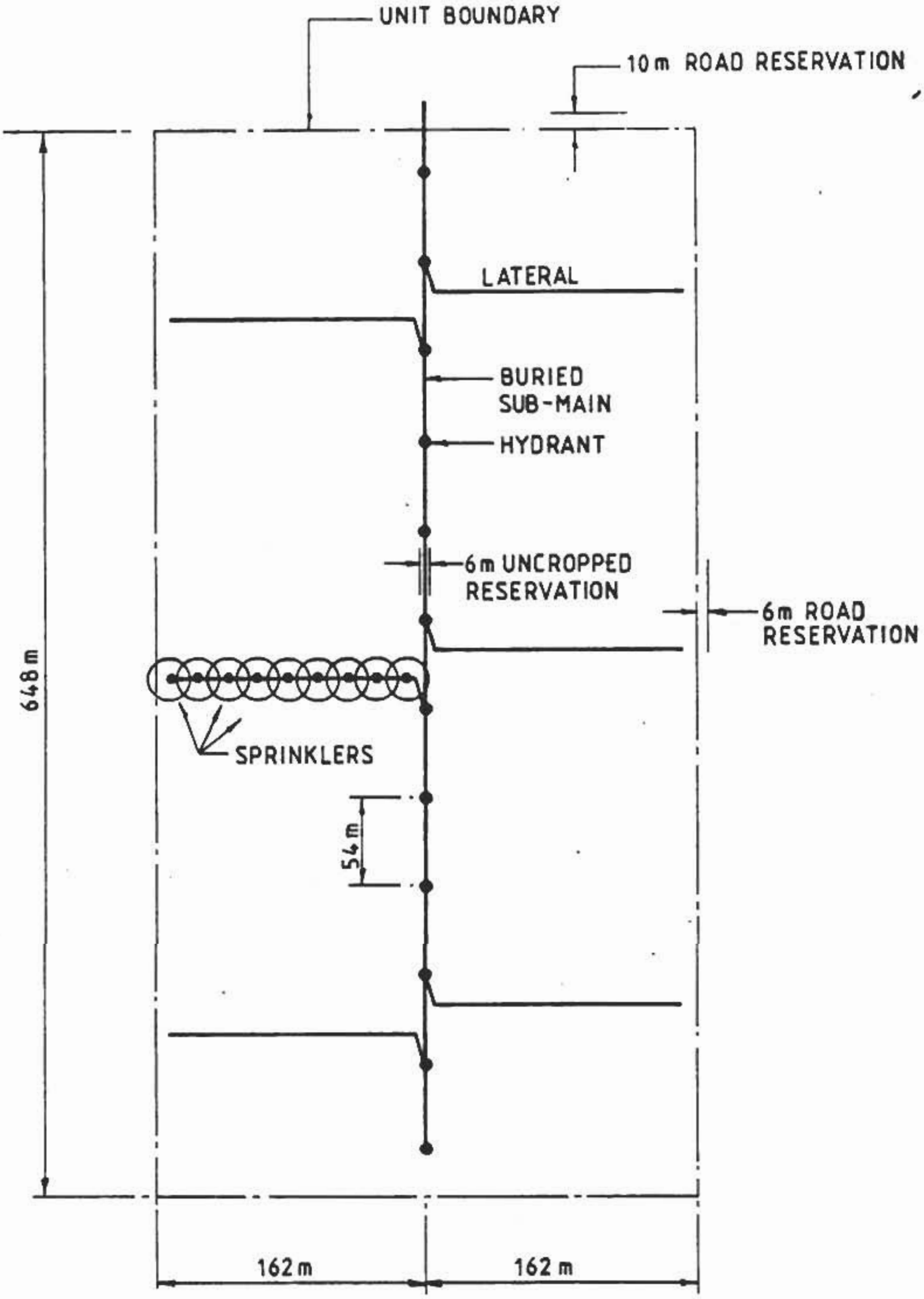
Typical Surface Irrigation Unit



Approximate scale 1:400

Figure 3.2

Standard Sprinkler Unit (Net area 20 ha.)



APPROXIMATE SCALE 1 : 400

For the Phase I sprinkler areas the pump station capacity must be based on the system which requires the greatest flow rate. This is System 1 because irrigation takes place only at night and the flow rate is equivalent to 1.8 l/s/ha at the pump station.

(d) Sprinkler Unit

A standard 20 ha unit has been adopted where possible. The unit is rectangular with one dimension half the other (648 m x 324 m). This arrangement allows twinning of the units to form a square which can be orientated in either of two directions. The buried sub-main, with hydrants at 54 m intervals (12 Nr), runs down the centre of the 20 ha unit parallel to the longer side.

The proposed unit is illustrated in Figure 3.2.

TABLE 3.4

Sprinkler Irrigation System Details

	System reference number				
	1	2	3	4	5
Lateral spacing (m)	18	18	18	18	12
Sprinkler spacing (m)	18	18	18	12	12
Time of set/day (hours)	12(night)	2x11	22	2x11	2x11
Irrigation interval (days)	12	9½	12	9½	9½
Irrigation efficiency	0.75	0.70	0.65	0.75	0.80
Sprinkler discharge (m^3/h)	2.52	2.33	1.58	1.45	0.91
Precipitation rate (mm/h)	7.8	7.2	4.9	6.7	6.3

(e) Sprinkler Pump Stations

Two separate pump stations are required, one on the main canal (P1) and one on distributary canal M1/C4 (P2). The P2 command of 41 ha will all be designed for System 1 and only one pump is necessary. The P1 command will include some areas of all systems and three pumps are required to meet the varying demands.

CHAPTER 4

CANAL DESIGN

4.1 General

The main canal will serve 76 surface irrigation units (total net area 2 052 ha) and a sprinkler irrigation trial area of 163 ha net.

The cotton crop on the sprinkler area will be irrigated from August to January, whereas the gu season rice crop, which determines the main canal capacity, will be irrigated from April to August. The main canal peak flow of 3.7 m^3/s will occur in May. In November, when the der season rice crop would reach its peak demand of 3.4 m^3/s the cotton crop requirements would be only about 0.17 m^3/s, so the total demand would not exceed the main canal capacity.

The net areas served by each canal are listed in Table 4.1.

TABLE 4.1

Irrigable Areas

Offtakes from main canal[(1)]	Net area served (ha)	
	Surface	Sprinkler
P1 (sprinkler pump station)	-	122
M1/C1	459	
M1/C4 + P2 (sprinkler pump station)	459	41
M1/C6	216	
M2/C1	243	
M2/C2	351	
M2/C4	324	
Total	2 052	163

Note : (1) Numbering system based on that for full development.

4.2 Main Canal Design

For the full Phase II development, the capacity of the main canal head reach would be 6.5 m^3/s. This is based on the requirements of 3 321 ha of paddy rice and 1 000 ha of upland rice, in May.
Since much of the canal is in fill (bed level above ground level) there will be little saving achieved if the canal is designed only to accommodate the 3.7 m^3/s required for Phase I. The main canal section has therefore been designed to meet Phase II requirements.

Three main canal structures are required :

- an underpass under the existing main road
- an underpass under the flood relief channel
- a cross regulator (M1)

The second cross regulator (M2) is not required since, for Phase I, the main canal ends here.

These structures could arguably be designed to accommodate only the flow required for Phase I. However, this will make implementation of the full development, assuming it takes place, more difficult. The cost saving involved by building the three structures to the reduced capacity would be relatively small and therefore they have been designed for the full capacity.

There will be an increased risk of sediment deposition in the two underpass structures because the flow velocity for Phase I will be relatively low. However, the amount of sediment carried by the canal should be small (the sediment basin downstream of the pump station will trap most) and manual desilting will be possible at times of canal closure (February/March). Nevertheless stop-log grooves have been provided in the upstream inlet to each structure so that one barrel can be blocked off at times of low discharge.

The effect of the reduced discharge in the main canal will be lower water levels in the reach upstream of the cross regulator. This will only be significant at the sprinkler pump station where water level will be about 0.30 m lower than design level, which can be accommodated in the design pumping head.

The main canal section has been designed for the Phase II flow in accordance with Lacey regime theory with a silt factor of 0.5.

4.3 Storage Reservoir Design

Four storage reservoirs are required. They have been designed to store the peak discharge for a period of 10 hours, assuming 14 hours irrigation per day at peak demand.

As described in Chapter 3, the distributary canal design discharge is 100 l/s per unit, based on 14 hours operation. The figure is derived from the der season rice requirement, and der season rice will only be grown on a maximum of 75% of the irrigable area. However, the 25% uncultivated is not related to a fixed location and therefore all canals and reservoirs must be designed for the peak.

Reservoir volumes are given by :

$$100 \times \frac{14}{24} \times \frac{10 \times 3\,600}{1\,000} \times N = 2\,100\,N \ \ m^3$$

Where N = number of field units served.

The net storage volumes of the four reservoirs have been calculated using this expression. The results are presented in Table 4.2.

TABLE 4.2

Reservoir Volumes

Reservoir	Canals served	Number of units served	Net storage volume (m^3/s)	Approximate plan area of reservoir (1) (m^2)
1	M1/C1	17	35 700	49 000
2	M1/C4 M1/C6	25	52 500	72 000
3	M2/C1	9	18 900	26 000
4	M2/C2 M2/C4	25	52 500	72 000

Note : (1) Based on an operating range of depth of 0.75 m.

TABLE 4.3

Distributary Canal Head Reach Capacities

Canal	Number of units served	Head reach capacity (m^3/s)
M1/C1	17	1.7
M1/C4	17	1.7
M1/C6	8	0.8
M2/C1	9	0.9
M2/C2	13	1.3
M2/C4	12	1.3
Total	76	-

Distributary canal design criteria are summarised in Table 4.4.

TABLE 4.4

Distributary Canal Design Criteria

Q (m^3/s)	B (m)	D(m) S = 0.10 (m/km)	D(m) S = 0.30 (m/km) (1)
0.20	1.5	0.50	0.38
0.30	1.5	0.62	0.46
0.40	2.0	0.64	0.48
0.50	2.0	0.72	0.54
0.60	2.5	0.73	0.54
0.70	2.5	0.79	0.59
0.80	2.5	0.84	0.63
0.90	3.0	0.84	0.62
1.00	3.0	0.89	0.66
1.10	3.0	0.93	0.69
1.20	3.5	0.91	0.68
1.30	3.5	0.95	0.71
1.40	3.5	0.99	0.74
1.50	4.0	0.97	0.72
1.60	4.0	1.01	0.75
1.70	4.0	1.04	0.77

Notes : (1) Canal slope may be increased above 0.30 m/km up to 0.50 m/km for low capacity canals. Slope should be such that tractive force does not exceed 4 N/m^2.

(2) Calculated depths are based on Manning's equation with roughness coefficient n = 0.03.

4.4 Distributary Canal Design

As described previously, the design capacities of the distributary canals have been based on a flow rate of 100 l/s per unit served. The design criteria are summarised in Table 4.4.

Since the Phase I development is being designed so that expansion to the full development can be effected with relative ease, it is necessary to ensure that distributary canal capacities are sufficient for the Phase II sprinkler areas which they may serve in the future.

Surface irrigation will take place during daylight hours only, so night-time flow in the distributary canals can be reserved for sprinkler irrigation. This division between night and day flow will make operation of the system easier. Any future development of sprinkler areas served by distributary canals will thus incorporate reservoirs for each sprinkler pump station, as is the case for the Phase I sprinkler area of canal M1/C4.

The distributary canal capacity is thus determined from the larger of the day-time flow for surface irrigation areas and the night-time flow for sprinkler areas. Based on the peak gross continuous requirement of 1.0 l/s/ha for sprinkler areas, the canal capacity upstream of a sprinkler pump station must be at least 2.0 A l/s where A is the net area of sprinkler irrigation (ha). In fact, the only canal affected is M1/C4.2, which serves only three surface irrigation units but has a substantial potential sprinkler area at its tail.

4.5 Unit Feeder Channel Design

The unit feeder channel has a triangular cross section, which is preferred since this is easier to form in a continuous process. Channel slopes can vary between 0.10 and 2.00 m/km to suit the natural ground slope. At minimum slope the design section can carry the calculated peak flow of 85 l/s with a freeboard of 0.18 m, assuming a Manning roughness coefficient of 0.033.

The minimum slope of 0.10 m/km is necessary because of the generally flat nature of the surface irrigated areas. It is this slope which determines the size of the channel section. A smaller channel section could be used for cases where slopes exceed 0.10 m/km, but it is considered preferable, for ease of construction and maintenance, to adopt a standard section.

The section can be constructed by machine by forming a trapezoidal embankment with a 4.0 m top width set at 0.10 m below required water level. The bank is compacted during its formation and then the channel shape is formed with a V-ditcher.

The top of the embankment is set at 0.15 m minimum above field level. This ensures adequate head for the siphon pipes when the ponded depth in a basin reaches the maximum of 0.15 m. Details are given in Figure 4.1.

4.6 Summary of Canal Design Criteria

A summary of canal design criteria is presented in Table 4.5.

TABLE 4.5

Canal Design Criteria

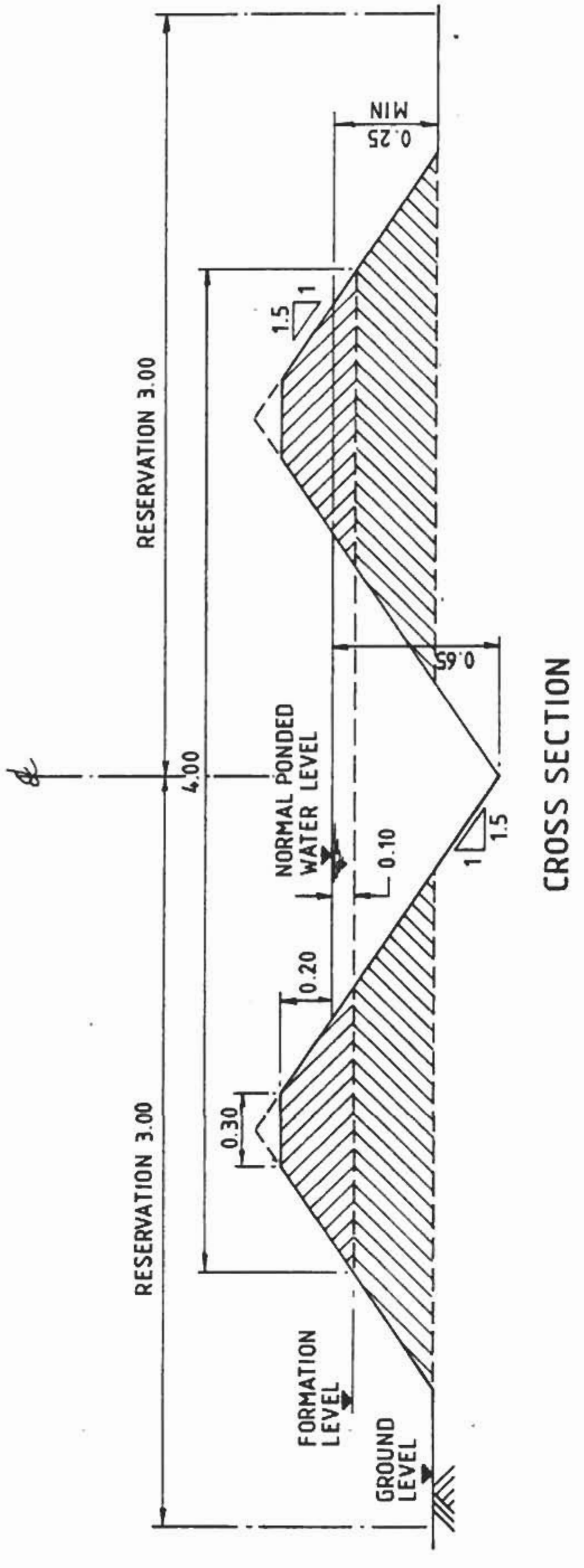
Criteria	Main canal	Distributary canal	Unit channel
Maximum design flow (m^3/s)	6.5 (3.7)(1)	1.7	0.085
Minimum design flow (m^3/s)	4.1 (1.6)(1)	0.2	-
Maximum depth of flow (m)	1.4	1.05	0.65
Minimum depth of flow (m)	1.2	0.35	-
Maximum bed width (m)	8.2	4.0	-
Minimum bed width (m)	6.5	1.5	-
Maximum water surface slope (m/km)	0.10	0.50	2.00
Minimum water surface slope (m/km)	0.08	0.10	0.10
Channel side slopes (vertical : horizontal)	1 : 2	1 : 2	1 : 1.5
Freeboard above design water level (m)	0.50	0.40	0.20
Minimum bend radius at centre (m)	10 x water surface width(2)		5.0
Bank top width (m)	5.0	4.0/1.5	-

Notes: (1) Figures in brackets refer to Phase I development

(2) A minimum canal bend radius of 90 m has been adopted wherever possible.

Figure 4.1

Unit Feeder Channel Design Details



MINIMUM BED SLOPE
=0.10 m/km

FLOW (l/s)	DEPTH OF FLOW AT MINIMUM SLOPE (m)	FREEBOARD (m)
80	0.65	0.20
85	0.67	0.18
100	0.70	0.15

CHAPTER 5

DRAIN DESIGN

5.1 General

In the Supplementary Feasibility Study the requirements for both surface and sub-surface drainage were examined.

With regard to sub-surface drainage it was concluded that this is unlikely to be required in the early stages of the project, but may be required later on for non-rice crops. We have therefore included a watertable monitoring system in the sprinkler area. This will enable a close watch to be kept on groundwater levels so that an early indication of a likely drainage problem can be obtained. The monitoring system will include for regular sampling of groundwater samples to keep a check on salinity levels.

The surface drainage requirements are based on rainfall - run-off relationships developed in the Supplementary Feasibility Study (Annex 1). Pumping of drainage water will be required for a low-lying portion of the irrigable area.

5.2 Surface Drainage Rates

The design run-off rate for surface irrigated areas developed in the Supplementary Feasibility Study is 1.5 l/s/ha (based on gross area). This rate was reduced to 1.4 l/s/ha for main collector drains and 1.2 l/s/ha for outfall drains because of the localised nature of the rainfall. However, in view of the reduced size of the development proposed, an overall rate of 1.5 l/s/ha has been adopted, regardless of drain size.

For the small sprinkler irrigated area a drainage rate of 3.5 l/s/ha has been adopted (see Annex 1 of the Supplementary Study). The significantly lower rate for the surface irrigated area reflects the temporary storage in the rice basins.

5.3 Unit Drains

Unit drains carry away surface run-off from the rice basins. A shallow V-shaped section has been adopted such that access across the drains is possible. This is important since both sowing and harvesting operations will be mechanised. The alternative of a trapezoidal section would require frequent crossing points, in the form of either small culverts or fords, which would be considerably more expensive.

The unit drain section has an nominal depth of 0.30 m in the centre, with 1 in 8 side slopes to ground level. Each unit is served by three separate branches of the unit drain. Each branch of the unit drain serves a gross area of 11 ha and therefore must have a minimum capacity of 16 l/s. The standard drain section can carry up to 40 l/s, even at its flattest slope of 0.05 m/km, assuming a clean section. For the small reach of unit collector drain (the channel into which the three unit drain branches flow) a trapezoidal section has been adopted. Access across this channel is not necessary and the trapezoidal section will be more readily maintained.

The draining off of ponded water in the basins can result in large flows in the drains. This operaton is normally carried out to permit weeding of the rice crop and the application of fertiliser. Since the ponded depth gradually reduces between irrigations at the rate of about 10 mm/d, due to evapotranspiration and deep percolation, and since the weeding operation can be carried out with a ponded depth of 50 mm, it should not be necessary to drain off more than about 50 mm from each basin. Neither will it be necessary to drain a whole unit in one day. Assuming that water levels are dropped by 50 mm over a whole unit in a 3 day period would require a drain capacity of about 50 l/s. This flow corresponds to that required to meet the design run-off of 1.5 l/s/ha on a gross area of 33 ha. The unit collector drain has therefore be designed for this discharge.

Each basin in a field unit will have its own small outlet structure into a unit drain. This structure comprises a concrete inlet box with a simple steel gate, and a pipe into the drain. The steel gate is designed to allow controlled flow from the basin whenever the depth exceeds 0.15 m. Gates can be removed to allow draining of individual basins.

5.4 Outfall and Main Collector Drains

Main collector drains gather the run-off from each unit collector drain and carry it to either of the two outfall drains on the eastern and western project boundaries.

The drains have been designed in accordance with the Manning equation, with a roughness coefficient of 0.033 and a bed width to depth ratio of three. A minimum bed slope of 0.10 m/km has been adopted.

Wherever possible design water level has been maintained at least 0.20 m below ground level. However, where drains pass through localised low areas it has been necessary to relax this requirement and there are some reaches of drain partially in fill.

Main collector drains and outfall drains have been provided with 6.0 m wide access roads on both sides, where necessary. These roads form the main in-scheme access routes. Berms at ground level have been provided between the road the drain for inspection and maintenance traffic and for dumping any sediment excavated from the drains.

5.5 Pumped Drainage

The outflow of drainage water from the project area will generally be achieved by gravity. However, because the natural ground slopes are very flat, a low lying part of the project area requires pumped drainage to ensure adequate evacuation of surface run-off. This is discussed in more detail in Chapter 8.

CHAPTER 6

GENERAL STRUCTURAL DESIGN CRITERIA

6.1 Loading

For all bridges and the underpass structures, traffic loading has been taken as HA to BS 153. Traffic loads on soil surfaces, used for surcharge calculations, have been taken as 10 kN/m^2. All culverts within the scheme have been designed for light road loading, as defined in "Simplified Tables of External Loads on Buried Pipelines" (HMSO 1969).

For footbridge loading 4 kN/m^2 has been adopted, based on the gross plan area.

6.2 Stability

A factor of safety of 1.5 has been adopted in determining the stability of structural elements against sliding or overturning as a result of soil and water pressures and traffic surcharge. For small structures reduced factors of safety have been allowed for the 'sudden drawdown' case where canals or drains are assumed to empty very quickly leaving unbalanced residual hydrostatic forces behind a structure.

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. They reflect the worst conditions likely to be met in the project area.

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 generally been assumed to be at channel design water level or other appropriate level, and the safety of structures has been checked for a rapid drawdown case. Soil below watertable level has been 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 is assumed. Passive resistance is assumed to start at finished ground level or top of pitching, and the coefficient reduced appropriately in instances where the earth surface slopes away from the member under consideration.

6.4 Concrete and Reinforcement

Reinforced concrete has been designed 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 followed.

The following properties of concrete have been assumed:

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	11×10^{-6}/°C
Coefficient of shrinkage	300×10^{-6}

A temperature range of 25°C has been assumed.

Cover to reinforcement is 50 mm except for some small relatively unimportant members, and 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 with the following bar diameters: 8, 10, 12, 16, 20 and 25 mm.

The following minimum reinforcement percentages have been 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 (200 mm and less)
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 cement has been adopted for all major drain structures and will be used elsewhere where a structure is liable to sulphate attack from groundwater or soils.

6.5 Pipes and Pipe Bedding

Non-pressure pipeline designs have been based on the use of spigot and socket concrete pipes with internal diameters of 0.375, 0.45, 0.60, 0.75, 0.90, 1.05 and 1.20 m.

Two classes of pipe have been specified: Class H and Class M; and three classes of bedding: Class A2 (reinforced concrete), Class A1 (mass concrete) and Class B (granular). Details of the bedding are shown in Figure 6.1.

Tables 6.1 and 6.2 give 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)

SINGLE PIPES

Internal diameter (m)	Pipe class	Bedding class A2	Bedding class A1	Bedding class B
0.30	M	-	-	-
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 7.6
0.375	M	0.9 - 7.6	0.9 - 7.6	0.9 - 4.3
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.5
0.45	M	0.9 - 7.6	0.9 - 7.6	0.9 - 4.6
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.6
0.60	M	0.9 - 7.6	0.9 - 7.6	0.9 - 4.4
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.0
0.75	M	0.9 - 7.6	0.9 - 7.6	0.9 - 4.3
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.5
0.90	M	0.9 - 7.6	0.9 - 7.1	0.9 - 4.1
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.0
1.05	M	0.9 - 7.6	0.9 - 7.1	0.9 - 4.2
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.2
1.20	M	0.9 - 7.6	0.9 - 7.0	0.9 - 4.1
	H	0.9 - 7.6	0.9 - 7.6	0.9 - 6.2

Notes: (1) The table gives the minimum and maximum permissible depths of cover based on pipe classes as given in BS 556 and "light road" traffic loading as given in the "Simplified Tables of External Loads on Buried Pipelines" by the Building Research Establishment (published by HMSO).

(2) "Narrow trench" conditions have been assumed and the quoted depths are therefore not suitable for multiple pipes.

TABLE 6.2

Depth of Cover to Pipes (in metres)

MULTIPLE PIPES

Internal diameter (m)	Pipe class	Bedding class		
		A2	A1	B
0.30	M	-	-	-
	H	0.9 - 7.0	0.9 - 5.3	0.9 - 3.7
0.375	M	0.9 - 7.1	0.9 - 5.4	0.9 - 3.7
	H	0.9 - 7.6	0.9 - 6.4	0.9 - 4.6
0.45	M	0.9 - 6.9	0.9 - 5.2	0.9 - 3.6
	H	0.9 - 7.6	0.9 - 6.1	0.9 - 4.3
0.60	M	0.9 - 6.6	0.9 - 5.0	0.9 - 3.4
	H	0.9 - 7.6	0.9 - 6.0	0.9 - 4.3
0.75	M	0.9 - 6.3	0.9 - 4.8	0.9 - 3.2
	H	0.9 - 7.6	0.9 - 5.9	0.9 - 4.2
0.90	M	0.9 - 6.9	0.9 - 5.2	0.9 - 3.6
	H	0.9 - 7.6	0.9 - 6.7	0.9 - 4.8
1.05	M	0.9 - 6.8	0.9 - 5.1	0.9 -3.6
	H	0.9 - 7.6	0.9 - 6.6	0.9 -4.8
1.20	M	0.9 - 6.7	0.9 - 5.1	0.9 - 3.5
	H	0.9 - 7.6	0.9 - 6.5	0.9 - 4.7

Notes : (1) The table gives the minimum and maximum permissible depths of cover based on pipe classes as given in BS 556 and "light road" traffic loading as given in the "Simplified Tables of External Loads on Buried Pipelines" by the Building Research Establishment (published by HMSO).

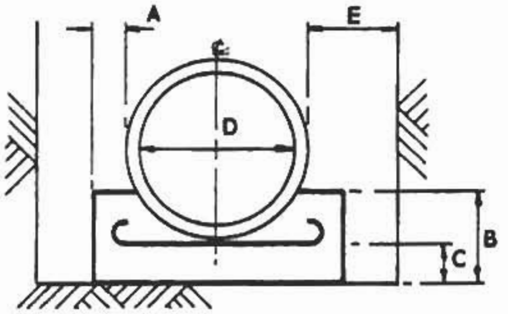
(2) "Wide trench" conditions have been assumed.

Figure 6.1

Pipe Bedding Alternatives

TYPE A2

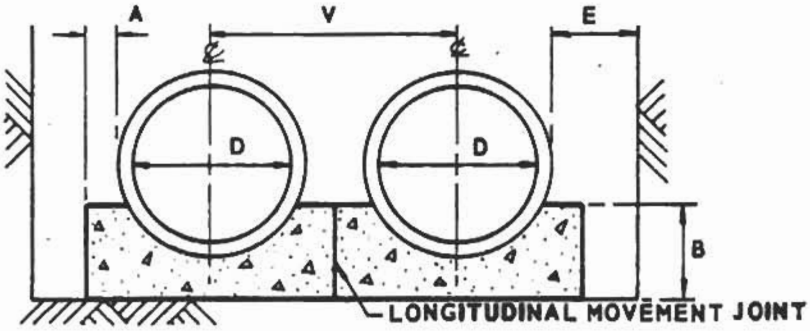
REINFORCED CONCRETE (SINGLE PIPE SHOWN)



MINIMUM TRANSVERSE STEEL AREA TO BE 0.4% OF CONCRETE AREA AT CENTRE

TYPE A1

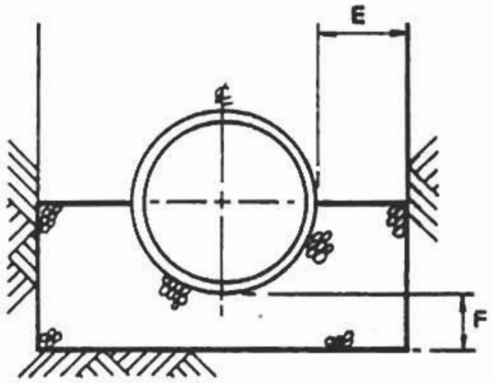
MASS CONCRETE (DOUBLE PIPE SHOWN, TRIPLE PIPE SIMILAR)



TYPE B

GRANULAR

(SINGLE PIPE SHOWN)



D (m)	A	B	C	E	F		V
					UNIFORM SOILS	STONY SOILS	
0.30	0.10	0.20	0.10	0.20	0.10	0.20	—
0.375	0.10	0.25	0.10	0.25	0.10	0.20	—
0.45	0.10	0.30	0.15	0.25	0.15	0.20	0.85
0.60	0.10	0.35	0.15	0.25	0.15	0.20	1.00
0.75	0.15	0.45	0.20	0.25	0.15	0.20	1.20
0.90	0.15	0.55	0.25	0.30	0.15	0.25	1.40
1.05	0.20	0.60	0.25	0.35	0.20	0.30	1.60
1.20	0.20	0.70	0.30	0.40	0.25	0.35	1.90

NOTES

1. TABULATED VALUES ARE ROUNDED UP TO NEAREST 0.05 m
2. FOR NARROW TRENCH CONDITION, E MUST BE NOT MORE THAN THE VALUE GIVEN IN THE TABLE
3. BLINDING CONCRETE IS REQUIRED FOR BEDDING TYPES A2 AND A1

CHAPTER 7

CANAL STRUCTURES

7.1 Underpass Structures

In the Supplementary Feasibility Study the two underpass structures had different cross sections. A slightly larger section was used for the much longer underpass under the flood relief channel, so that the design head loss for each structure was 0.20 m. For the final designs we have used the same larger section for both structures. This will simplify construction and will result in a slightly lower head loss for the main road underpass.

A reinforced concrete twin box structure has been designed with rectangular boxes 1.8 m wide by 1.5 m deep. Stop-log grooves have been provided in the inlet to each structure so that one of the barrels can be closed at times of low flow. This will ensure that sediment deposition is reduced. Each underpass has a trash rack on the inlet.

Head loss through the structure has been estimated from

$$H_L = \frac{1.5v^2}{2g} + H_f$$

where H_L = total head loss (m)

v = velocity of flow through the structure (m/s)

H_f = friction head loss (m)

At the full Phase II flow of 6.5 m^3/s the head losses are thus 0.14 m and 0.23 m for the main road and flood relief channel structures, respectively.

The underpass under the flood relief channel will be protected from damage by scour by a gabion mattress lining in the channel.

7.2 Main Canal Cross Regulator

Only one cross regulator is required on the main canal for the Phase I development. This is located just downstream of the underpass under the flood relief channel. Movable weir structures have been adopted for both the main canal cross regulator and the offtaking reservoir head regulators. These structures permit accurate measurement of flow and the gate can readily be adjusted to suit varying demand situations.

Two 2.5 m wide weirs have been adopted and the cross regulator incorporates a road bridge to permit access across the main canal.

Downstream of the weir a stilling basin has been provided to ensure that the hydraulic jump is contained by the structure. The design of the basin is based on the following opening conditions:

(i) 10% of design discharge with dry downstream bed

(ii) Design discharge

(iii) Sudden increase in flow from 70% to 120% of design discharge, with no change in downstream water level.

7.3 Reservoir Head Regulator

The movable weir regulating structure which controls the flow from the main canal into a reservoir has been called a reservoir head regulator.

Head loss across the structure varies from a minimum of 0.15 m, occurring when the reservoir is full (at the end of the night), to 0.90 m when the reservoir level is at its minimum (at the end of the day). Larger head losses occur where the distributary canal command is non-critical.

The weir will be set to deliver the required discharge into the reservoir and will remain at this setting until such time as the demand changes.

7.4 Distributary Canal Head Regulator

Each distributary canal offtakes from a reservoir through a head regulator structure. The design of the structure must be such that canal flow can be maintained sensibly constant throughout the day as the reservoir water level falls. Automatic gates were considered but these would be complex and expensive, especially since it must be possible to vary the flow throughout the year to meet seasonal demand. A simple lifting gate structure has therefore been adopted. Flow through this structure is relatively insensitive to upstream water level variation but it will be necessary to adjust the gate(s) every 3 to 4 hours throughout the day.

To assist flow measurement a crest tapping and measuring well has been provided downstream of the gate. Gate operators will be provided with simple calibration charts so that they can estimate the canal flow from water level and gate opening measurements.

The structure comprises a concrete inlet box in the reservoir with steel lifting gate(s), a concrete pipeline through the reservoir bank, and a concrete outlet box at the head of the canal.

Figure 7.1 gives the design chart used for selecting the appropriate gate/pipe size for both head regulators and cross regulators. For the head regulators only two pipe sizes have been adopted - 0.90 m and 1.05 m diameter, with twin 1.05 m pipes being used for the larger canals. This results in simplified construction and operation.

7.5 Distributary Canal Cross Regulators

The distributary canal cross regulators are similar structures to the head regulators, with the same lifting gate, but generally have minimum head loss of 0.10 m. Selection of pipe sizes has been based on Figure 7.1, but only four sizes of pipe have been adopted - 0.60, 0.75, 0.90 and 1.05 m diameter.

The cross regulator gates will be adjusted to maintain a constant upstream water level so that the offtaking discharges can be monitored.

7.6 Distributary Outlet

The distributary outlet structure diverts flow from the distributary canal into the unit feeder channel. A similar but smaller structure to the distributary cross regulator has been adopted. It has a peak capacity (gate fully open) of 100 l/s. The required discharge is achieved by setting the outlet gate opening and flow measurement is facilitated by the inclusion of two measuring wells with tappings both upstream and downstream of the gate. The gates can be calibrated on site.

7.7 Other Distributary Canal Structures

Road crossings for distributary canals are provided by the cross regulators. No other road crossings are necessary.

All distributary canals have tail escapes discharging into an adjacent drain. These structures insure against overtopping of the canal banks in the event that outlets are closed when the head regulator is open. The weir of the escape will be set at 0.05 m above design water level and the structure has a capacity of 0.35 m^3/s. An impact type stilling basin is required at the downstream end in order to dissipate the flow energy gained in the drop from canal to drain.

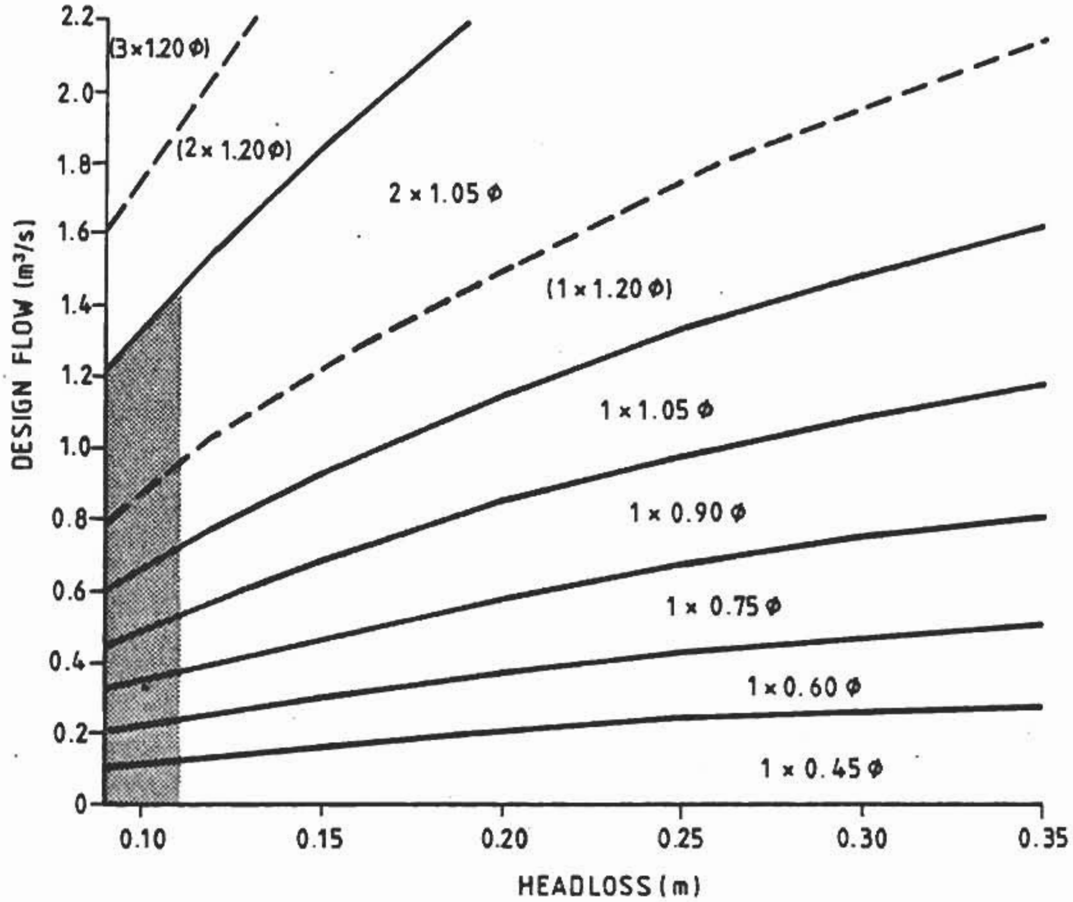
7.8 Unit Feeder Channel Structures

Unit feeder channel drop structures are required downstream of distributary outlet structures where command is high, and elsewhere if natural ground slope exceeds the maximum permissible channel slope. Small concrete straight drop structures have been adopted - these can be either precast or cast in-situ.

Diversion structures are not required on unit channels even though the channel divides into three branches within each unit. It is intended that the whole unit channel flow will be diverted down each branch in turn. This will be achieved by positioning portable check structures to block off the non-operating branches, or by forming temporary earth bunds.

Figure 7.1

Distributary Canal Gated Regulators
Pipe Diameter Selection



NOTES

1. FOR MOGAMBO REGULATORS 1.20 Φ AND 0.45 Φ PIPES HAVE NOT BEEN USED
2. FOR CROSS REGULATORS WITH MINIMUM HEAD LOSS OF 0.1m USE SHADED AREA
3. FOR HEAD REGULATORS ONLY TWO PIPE SIZES HAVE BEEN USED (0.90 Φ AND 1.05 Φ) IN ORDER TO SIMPLIFY OPERATION

CHAPTER 8

DRAIN STRUCTURES

8.1 Culverts

Culverts are generally required where two drains connect so as to maintain road access. Culverts are also required in several other locations in order to improve access within the irrigated area.

Three basic types of culvert have been used, identified as Types 1, 2 and 3. These structures comprise a concrete pipe culvert (single or double) with various inlet and outlet designs to suit the discharge and head loss/drop required.

The Type 1 culvert is used for the junction between a unit collector drain and a main collector drain. There are three variations as follows:

Type 1A - for unit collector drains serving one or two units (50 l/s and 100 l/s capacity)

Type 1B - for unit collector drains serving three units (150 l/s capacity)

Type 1C - for unit collector drains into which a distributary tail escape discharges (350 l/s capacity)

The Type 2 culvert, which has both inlet and outlet formed in drystone pitching, is appropriate for main collector drain road culverts and has a nominal head loss of 0.05 m through it. The pipe size has generally been based on the following criteria:

(a) velocity of flow and design discharge (assuming pipe full) to be not more than 0.7 m/s

(b) soffit level of pipe to be no higher than 0.10 m above drain design water level

(c) invert level of pipe to be no lower than one-third of the pipe diameter below drain bed level

Type 3 culverts are appropriate for junction culverts and road culverts where head loss is greater than 0.05 m but not more than 0.20 m. In addition, for design flows of less than 0.50 m^3/s, head losses of up to 0.50 m and any bed level drop are permissible. The Type 3 culvert has mass concrete inlet and outlet boxes. Selection of the pipe size is generally as for the Type 2 culvert.

The Type 4 culvert, which has an impact type stilling basin downstream, has only been used for canal escapes, where there is a substantial drop in water level through the structure. It has not be necessary to use Type 4 culverts elsewhere on the drains.

8.2 Other Minor Structures

No drop structures are required on the drains because the land slope is so flat.

For the unit road crossing of the unit drain where is joins the unit collector a ford is proposed. This comprises a total of 300 mm depth of aggregate and crushed rock road base, in two layers.

Basin outlet structures, which control drainage flow into the unit drains, have been described in Chapter 5.

8.3 Drain Outfall Structure

Drainage water from the project flows southwards in the drainage system and is discharged through the flood bund at the southern end of the project area. The outfall structure comprises a flap gated reinforced concrete twin box culvert under the bund.

Under normal operating conditions the flap gates will be open and drainage water in the western outfall drain will flow out by gravity. After heavy flooding when the water level outside the flood bund is higher than drain water level, the flap gates will automatically shut preventing flood water entering the irrigable area.

The outfall structure has also be provided with penstock gates which would be closed in the event of failure of a flap gate during flood conditions.

8.4 Drainage Pump Station

Twenty three field units in the southern part of the irrigable area are too low to be drained by gravity. A pumping station is thus required for the main collector drain (PD/1) which serves this area.

A concrete box culvert carries drainage water from a gated intake box at the end of drain PD/1 to a reinforced concrete sump. From here water is pumped into a concrete pipe passing through the flood bund. The maximum static lift is about 5 m and two vertical axial flow pumps (one as standby) will provide the necessary 1.2 m^3/s peak capacity.

After severe flooding the high water level outside the flood defences may prevent drainage outflow by gravity for prolonged periods. For this reason the drainage pump station has also been designed to permit pumping from the gravity outfall drain.