JAMHUURIYADDA DIMOQRAADIGA SOMAALIYA
WASAARADDA BEERAHA
SOMALI DEMOCRATIC REPUBLIC
MINISTRY OF AGRICULTURE

# GENALE-BULO MARERTA PROJECT

## ANNEX VII

## Engineering

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## SUMMARY OF REPORT TITLES

## PROJECT AREA AND STUDY AREA

This study contained two elements, a Master Plan covering 67 400 hectares and a feasibility study of 5 000 hectares.

Throughout the reports the term Study Area refers to the area covered by the Master Plan studies and the term Project Area is used for the feasibility study area.

## ABBREVIATIONS USED IN THE REPORTS

| | |
|---|---|
| ADB | African Development Bank |
| ADC | Agricultural Development Corporation |
| CARS | Central Agricultural Research Station - Afgooye |
| DAP | Diammonium phosphate |
| EDF | European Development Fund |
| ENB | National Banana Board |
| FAO | Food and Agriculture Organisation |
| FAO/PP | FAO Pilot Project (Afgooye - Mordiile Project) |
| HASA | Hides and Skins Agency |
| HTS | Hunting Technical Services Limited |
| HV | High volume (crop sprayer) |
| IBRD | International Bank for Reconstruction and Development (the World Bank) |
| ITCZ | Inter-tropical convergence zone |
| ITDG | Intermediate Technology Development Group (London) |
| JOSR | Jowhar Offstream Storage Reservoir |
| LDA | Livestock Development Agency |
| Libsoma | Libya-Somalia Agricultural Development Company |
| LSU | Livestock unit |
| LV | Low volume (crop sprayer) |
| MLFR | Ministry of Livestock, Forestry and Range |
| MMP | Sir M. MacDonald & Partners |
| NCA | Net cultivable area |
| NCB | National Commercial and Savings Bank (formerly National Commercial Bank) |
| ONAT | National Farm Machinery and Agricultural Supply Service |
| PLO | Palestine Liberation Organisation |
| SDB | Somali Development Bank |
| SNAI | Jowhar Sugar Estate |
| TDN | Total digestible nutrients |
| TDP | Total digestible protein |
| ULV | Ultra-low volume (crop sprayer) |
| UNDP | United Nations Development Programme |
| USBR | United States Bureau of Reclamation |
| USDA SCS | United States Department of Agriculture, Soil Conservation Service |
| WHO | World Health Organisation |

## SPELLINGS OF PLACE NAMES

Throughout the report Somali spellings have been used for place names with the exception of Mogadishu where the English spelling has been used. To avoid misunderstanding, we give below a selected list of Somali, English and Italian spellings where these differ.

| Somali | English | Italian |
|---|---|---|
| Afgooye | Afgoi | Afgoi |
| Awdheegle | - | Audegle |
| Balcad | Balad | Balad |
| Baraawe | Brava | Brava |
| Buulo Mareerta | Bulo Marerta | Bulo Mererta |
| Falkeerow | - | Falcheiro |
| Gayweerow | - | Gaivero |
| Golweyn | - | Goluen |
| Hawaay | Avai | Avai |
| Hargeysa | Hargeisa | - |
| Janaale | Genale | Genale |
| Jelib | Gelib | Gelib |
| Jowhar | Johar | Giohar |
| Kismaayo | Kisimaio | Chisimaio |
| Marka | Merca | Merca |
| Muqdisho | Mogadishu | Mogadiscio |
| Qoryooley | - | Coriolei |
| Shabeelle | Shebelli | Scebeli |
| Shalambood | Shalambot | Scialambot |

## GLOSSARY OF SOMALI TERMS

| | | |
|---|---|---|
| Cambuulo | - | Traditional dish of chopped boiled maize with cowpeas or green grams. |
| Chiko | - | Chewing tobacco |
| Der | - | Rainy season from October to December |
| Dharab | - | Five jibals or approximately 0.31 ha |
| Gu | - | Rainy season in April and May |
| Hafir | - | Large reservoir on farms for storing water for use in dry periods |
| Hagai | - | Climatic season June to September characterised by light scattered showers |
| Jibal | - | Area of land approximately 25 m by 25 m or 0.0625 ha |
| Jilal | - | Dry season from January to April |
| Kawawa | - | Two man implement for forming irrigation ditches |
| Moos | - | Measurement of land area equal to a quarter of a jibal |
| Quintal | - | Unit of weight measurement equivalent to 100 kg |
| Uar | - | See hafir |
| Yambo | - | Small short-handled hoe |
| Zareebas | - | Thorn cattle pen |

# PART I

# GENERAL ENGINEERING

# CHAPTER 1

# EXISTING IRRIGATION SYSTEM

## 1.1 The Irrigation Network

Irrigation water in the Study Area currently feeds a net cultivated area of about 21 000 ha, mostly by gravity. This represents 55% of the total irrigated land (excluding uncontrolled flood irrigation) on the Shabeelle Flood Plain.

### 1.1.1 Development at Janaale

Development began in 1926 with the construction of the Janaale barrage together with the Dhamme Yaasiin canal (previously known as the Principale) and Primo Secondario canal on the left bank of the river, and the Asayle canal (previously known as the Riva Destra) on the right bank. These brought into command in 1927 a gross area of some 30 000 ha, the largest single area of about 14 000 ha being served by the Dhamme Yaasiin canal. However, much of this land was abandoned during the 1939-45 war and has never been recultivated under the original perennial crops. Considerable portions, are now in use again for annual cropping by smallholders, co-operatives and government farms, leaving only about 1 300 ha of land completely unused. This figure is derived from the existing land use map (see Annex IV). A study made during 1968 and 1969 (HTS Ltd., 1969) considered the remodelling of this land together with the Dhamme Yaasiin canal, and recently a pre-feasibility study has outlined a 7 200 ha rice project in the area (State Planning Commission, 1977).

Shortly after 1945, the Primo Secondario canal was remodelled and extended to beyond Buulo Mareerta so that it now commands the largest area and takes the greatest discharge (up to 7.0 $m^3$/s was recorded during the study period) of the three main canals offtaking at Janaale.

### 1.1.2 The Buulo Mareerta Project

In 1955, work started on the Buulo Mareerta project which was intended to add 25 000 ha of controlled irrigation, in approximately equal areas on each bank of the river, downstream of the Janaale system. The project was to include three new barrages, at Gayweerow, Qoryooley and Falkeerow, together with three main canals, the Liibaan (previously known as the Qoryooley canal) on the right bank, taking off just upstream of Qoryooley barrage, the Wadajir (previously known as the Fornari canal) on the left bank taking off from below Gayweerow bridge, and the Bokore also on the left bank but taking off from above Falkeerow barrage.

No contour surveys of the Buulo Mareerta project area were carried out and the three main canals were designed on the basis of the ground levels along their arbitrarily chosen lines. Construction of the three canals, together with the Qoryooley and Falkeerow barrages, was carried out between 1955 and 1959. The Gayweerow barrage was never built at the original location, but excavation work has now been started at a new site just downstream of the Gayweerow road bridge. This site is upstream of the Wadajir head regulator and therefore this canal, in its present situation, can never provide reliable irrigation supplies, for as soon as the river discharge drops below the full seasonal rate, the water level drops sufficiently to cut off the supply through the head regulator.

The Buulo Mareerta project was to include a distribution system of 50 km of secondary canals and watercourses, complete with all the necessary water control structures. This work was only partially implemented and a total of just under 10 km of secondary canals were built, all on the Bokore canal. Consequently, the complete project only produced about 1 000 ha of controlled irrigation instead of the intended 25 000 ha.

### 1.1.3 Other Main Canals

The last main canals to be added to the system were the Giddu in 1955 and the Sigaale (known locally as the Cooperativo canal) in 1960. Both of these canals are on the right bank of the river, 1.7 km and 8.4 km upstream of Janaale barrage, respectively.

A new canal has been constructed, although as yet no flow has been admitted, between the site of the Gayweerow barrage and the Primo Secondario, to boost the flow to accommodate the water requirements of the planned grapefruit production scheme financed by the European Development Fund (EDF) and situated close to Buulo Mareerta.

Figure 1.1 shows all the main canals in the Study Area and their secondaries, together with the gross land areas each one serves. The boundaries of irrigated areas have been determined from the 1962/63 aerial photography and the detailed knowledge of the Study Area obtained from field work.

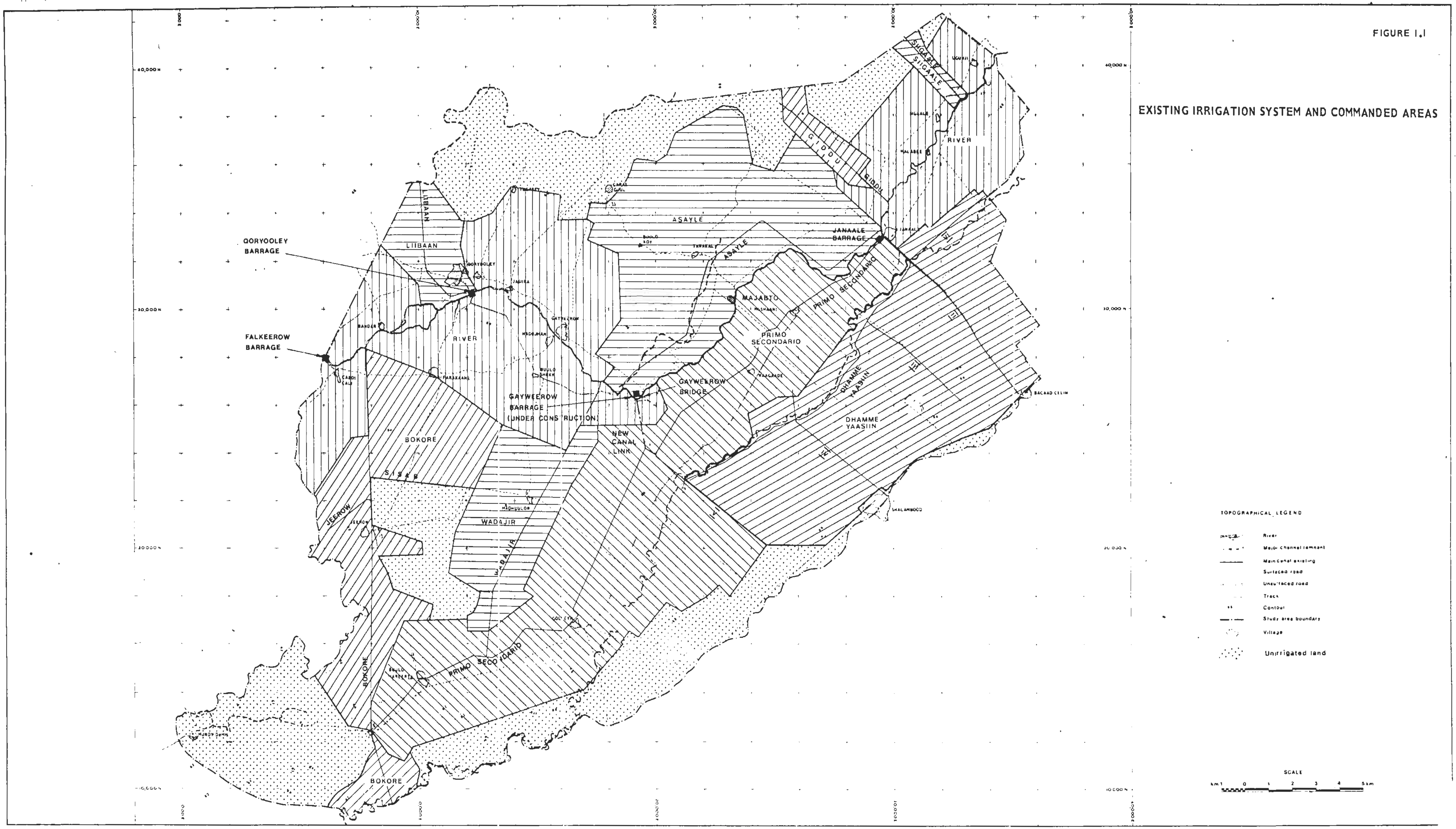
### 1.1.4 Uars

A major feature of the canal system is that the water supply cannot be guaranteed during the jilal (dry season, covering the months of January, February and March). Consequently the banana plantations built storage reservoirs (uars) adjacent to main canals and filled them with the surplus der season flows. This was done during the night when irrigation was impracticable and use could be made of the water that otherwise would go to waste. In 1960, it was reported (HTS Ltd., 1969) that there were 150 reservoirs in use; these covered a total area of 1 300 ha with a capacity of 16 $Mm^3$ and an average water depth of a little over 1 m. If the water was stored for an average period of three months then about 60% of the volume will have been lost due to evaporation and infiltration. Indeed, losses can be far more severe than this; a reservoir on the Dhamme Yaasiin, adjacent to the main Shalambood to Qoryooley road, was filled during December 1977, and only six weeks later the level had fallen by over 1 m. It is thought that the very fast seepage losses here were caused by a direct hydraulic link to groundwater through the old river channel that underlies the reservoir.

The role of the uars has now been mostly taken over by the development of irrigation tubewells to provide supplementary water supplies and a total of 132 wells are now operational. However, the high salinity of groundwater in many areas rules out its use and therefore the uars have been retained in a few areas where perennial crops are still grown and the groundwater is of poor quality. Other than these few, all the uars are now unused.

FIGURE 1.1

## EXISTING IRRIGATION SYSTEM AND COMMANDED AREAS



### 1.1.5 Minor River Offtakes

Apart from the main canals mentioned so far there are many minor offtakes from the river which take advantage of the ponding provided by the barrages and the natural levee elevation to irrigate areas close to the river. Figure 1.1 shows the areas covered by these minor canals, a total of 12 560 ha gross compared with 620 ha served by the eight main canals in the Study Area. As the water is not abstracted through a main head regulator, but nearly always through earth channels cut into the river bank, this must be classified as uncontrolled irrigation. However, for the purposes of water resource planning, these areas have been considered to be the same as the controlled areas, as no differences exist between the methods of in-field water control and application. Furthermore, no significant difference in the level of agricultural production of annual crops in the two types of area exists.

### 1.1.6 River Offtake Survey

During September 1977 a detailed visual survey was made of all the canal offtakes from the river in the Study Area, including pumps. This was done from a boat that was loaned by the Ministry of Agriculture, Department of Hydrology, for the period of the survey, starting at a point 10.4 km upstream of Janaale barrage and finishing 8.2 km downstream of Falkeerow barrage, a total river distance of 53.3 km. The last section beyond Falkeerow barrage is strictly outside the Study Area but the offtakes on the left bank of the left branch of the river (the river bifurcates just downstream of Falkeerow barrage), feed water back into the Study Area and therefore the offtakes are included in the survey results.

Table 1.1 summarises the results of the survey and gives the details of the offtakes from the river broken down into seven sections, working downstream. The exact location of each section can be found from Figure 1.1. The following points are worth mention:-

(i) Many offtakes are simply channels cut through the river bank; the total number of these offtakes, 102, is likely to be an underestimate as many of them are only 300 mm wide and therefore difficult to locate in the reedy or grassy banks of the river. Also the actual numbers will vary during the season as the smaller ones can be blocked off or re-formed at will.

(ii) Practically all the concrete head regulators are congregated upstream of Janaale and Falkeerow barrages, with only two (including the Liibaan canal) upstream of Qoryooley barrage.

(iii) Nearly all the pumps are grouped between Janaale and Gayweerow. This is the section of the river where water levels are only marginally raised above the general ground level, necessitating the use of pumps. The pumps themselves were produced in Shalambood at the National Farm Machinery and Agricultural Service (NFMAS, formerly and still known as ONAT) and are

## TABLE 1.1

### River Offtakes in the Study Area

| River section | Section length (km) | Main canal offtakes in section | Gravity offtakes (number) | | Pumped offtakes (number) | |
|---|---|---|---|---|---|---|
| | | | Concrete head regulators* | Earth breaches | 2 x 0.25 m discharge pipes | 2 x 0.20 m discharge pipes |
| Uguunji to Janaale | 10.4 | Sigaale<br>Giddu<br>Asayle<br>Dhamme Yaasiin<br>Primo Secondario | 12 | 15 | 3 | 0 |
| Janaale to Majabto | 9.4 | - | 0 | 12 | 1 | 2 |
| Majabto to Gayweerow bridge | 6.3 | - | 1 | 10 | 2 | 2 |
| Gayweerow bridge to Gayweerow | 5.3 | Wadajir | 1 | 11 | 1 | 5 |
| Gayweerow to Qoryooley | 5.3 | Liibaan | 2 | 8 | 0 | 0 |
| Qoryooley to Falkeerow | 8.4 | Bokore | 11 | 17 | 0 | 0 |
| Beyond Falkeerow + | 8.2 | - | 0 | 29 | 0 | 0 |
| TOTAL | 53.3 | 8 | 27 | 102 | 7 | 9 |

Notes: * including main canal head regulators
+ left bank only

centrifugal, belt driven pumps with twin parallel discharge pipes of 200 mm or 250 mm diameter. The nominal maximum discharges are 750 $m^3/h$ and 1 000 $m^3/h$, respectively, giving a total pumping capacity on the river of 3.8 $m^3/s$. Unfortunately production of these robust and locally repairable pumps has now ceased.

### 1.1.7 Gross Commanded Areas

Table 1.2 has been prepared from Figure 1.1 to show the relative importance of the gross areas served by each of the main canals and the minor canals. The total irrigated area of 54 180 ha compares with a total net cultivated area (derived from the land use survey, Annex IV) of 20 960 ha, producing an average land use efficiency over the irrigated areas of 39%. The gross areas include those areas which have been abandoned (e.g. Dhamme Yaasiin land and the land fed by the SISAB secondary of the Bokore). Clearly the table shows the importance of the Primo Secondario, commanding 20.3% of the Study Area, which represents 25.3% of the total irrigated area. Indeed, the three main canals at Janaale cover 57.0% of the total irrigated area, making Janaale barrage by far the most important water control structure in the Study Area.

**TABLE 1.2**

**Head Regulator Gated Widths and Gross Areas Irrigated from Each Canal**

| Canal | | Head regulator gated width (m) | Gross area irrigated ha | Percentage of Study Area |
|---|---|---|---|---|
| Sigaale | | 8.00 | 290 | 0.4 |
| Giddu | | 1.00 | 600 | 0.9 |
| Asayle | | 2.60 | 7 550 | 11.2 |
| Dhamme Yaasiin | | 8.75 | 9 630 | 14.3 |
| Primo Secondario | | 3.30 | 13 690 | 20.3 |
| Wadajir | | 5.40 | 2 890 | 4.3 |
| Liibaan | | 5.95 | 1 230 | 1.8 |
| Bokore | | 8.10 | 5 730 | 8.5 |
| Total of: | Main Canals | 43.10 | 41 620 | 61.7 |
| | Minor Canals | 21.30 | 21 560 | 18.6 |
| TOTAL Irrigated Area | | 64.40 | 54 180 | 80.3 |
| Unirrigated Land | | - | 13 230 | 19.7 |
| TOTAL StudyArea | | - | 67 410 | 100 |

## 1.2 Main Canal Survey

During the months of July and August, 1977, a complete ground survey of all the eight main canals in the Study Area was undertaken, including details of all regulating and outlet structures together with a general description of the canal. For each structure on the canal the following items were recorded:-

(i) the type of structure

(ii) the chainage, as a distance from the head regulator

(iii) the type of crossing possible (i.e. vehicular or pedestrian)

(iv) the bank on which the structure is situated (where appropriate)

(v) the number of gates

(vi) the width of each gate

(vii) the number of gates that are operational. Control on all canals is effected by vertical lifting steel sluices operated in some cases by a handwheel through a gearbox or elsewhere by nut and key. Only if this were possible were gates taken as operational. Therefore those gates which are opened and closed by other means (e.g. block and tackle, sledgehammer or hydraulic excavator arm) have been considered as being non-operational.

In addition, frequent estimates were made of the following:-

(i) Canal surface water width

(ii) Canal freeboard. This was done solely as freeboard above the water level at the time of the survey and therefore in a few cases may be unrepresentative

(iii) Canal command above the surrounding land

(iv) The state of weed growth in the canal.

Appendix A contains the full results of the survey and can provide detailed information on any of the main canals. However, for ease of reference, Table 1.3 provides a summary of all the water control equipment on the canals and their state of repair.

The following points are of major interest:-

(i) Nearly all regulating structures are in a poor state of repair. Four of the main canal head regulators have no gates at all in a fully operational state, including the Primo Secondario. Only the Bokore and Giddu head regulators are fully operable by hand. When cross regulators are considered, only 17 gates are fully operational out of the full total of 108. In many cases regulators are broken down and the gates rusted through in places; in the total area there are only two cross regulators where all the equipment is fully operational.

## TABLE 1.3

### Canal Water Control Structures

| | Sigaale | Giddu | Asayle | Dhamme Yaasiin | Primo Secondario | Wadajir | Liibaan | Bokore | Total |
|---|---|---|---|---|---|---|---|---|---|
| Length (km) | 9.5(1) | 6.1 | 15.8 | 15.7 | 32.7 | 12.7 | 8.3(2) | 18.5 | 119.3 |
| **Head Regulators** | | | | | | | | | |
| No. of gates | 4 | 1 | 2 | 5 | 3 | 4 | 7 | 9 | 35 |
| No. fully operational | 1 | 1 | 0 | 2 | 0 | 0 | 0 | 9 | 13 |
| Percentage fully operational | 25 | 100 | 0 | 40 | 0 | 0 | 0 | 100 | 37.2 |
| **Cross Regulators** | | | | | | | | | |
| No. of structures | 1 | 0 | 5 | 4 | 7 | 2 | 2 | 4 | 25 |
| No. of gates | 1 | 0 | 12 | 16 | 25 | 10 | 14 | 30 | 108 |
| No. fully operational | 0 | 0 | 3 | 1 | 4 | 1 | 4 | 4 | 17 |
| Percentage fully operational | 0 | - | 25 | 6.3 | 16 | 10 | 28.6 | 13.3 | 15.7 |
| **Field Outlets** | | | | | | | | | |
| No. of structures | 4 | 1 | 64 | 30 | 98 | 19 | 9 | 29 | 254 |
| No. of gates | 4 | 1 | 64 | 37 | 99 | 19 | 9 | 37 | 270 |
| No. fully operational | 0 | 0 | 8 | 15 | 60 | 5 | 4 | 15 | 107 |
| Percentage fully operational | 0 | 0 | 12.5 | 40.5 | 60.6 | 26.3 | 44.4 | 40.5 | 39.6 |
| No. of pumps | 0 | 0 | 1 | 0(3) | 28 | 0 | 0 | 2 | 31 |
| Earth outlets | 50 | 32 | 13 | 0 | 10 | 7 | 18 | 18 | 148 |

Notes:
(1) Only 3.6 km in the Study Area
(2) Only 6.3 km in the Study Area
(3) Any pumps on the secondary canals were not surveyed

(ii) The field outlet structures tend to reflect the level of irrigated agriculture they feed as the individual farmers are responsible for their upkeep. Consequently the Primo Secondario has by far the best percentage of fully operational outlet gates. The frequency of outlets for the three main canals at Janaale averages out as 3.1 per kilometre of canal. This contrasts very sharply with the 1.4 outlets per kilometre encountered on the three Buulo Mareerta project canals (Wadajir, Liibaan and Bokore), illustrating how these canals have never been fully equipped and therefore cannot adequately serve their purpose.

(iii) The only main canal with a significant number of pumps is the Primo Secondario (28), where the banana plantation farmers have installed them adjacent to normal gravity offtakes to guarantee supplies when the water level drops.

This is partly a reflection of the moderate command available from the canal but, more importantly, a result of the poor state of repair of the cross regulators which are needed to maintain levels when discharges fall. A total of 20 canal pumps were reported in 1969 (HTS Ltd.) feeding from the Dhamme Yaasiin secondary canals, but although the secondary canals were not actually surveyed fully, none was found in operation during field work in 1977.

(iv) On many canals the lack of outlet structures is alleviated by the farmers cutting earth breaches into the canal banks. Mud, twigs, branches and leaves are used to form the bunds and these are broken and re-made at every irrigation. Both the Sigaale and Giddu canals lack field outlet structures and consequently many earth outlets have been formed on these.

### 1.2.1 Sigaale

Only 3.6 km of the total length of 9.5 km lies within the Study Area and in this section there are only six field outlets. Agriculturally, however, these are by far the most important offtakes and take a large portion of the seasonal irrigation water. The many minor earth outlets further along the canal rely on excess flows from high river levels to provide sufficient water for the poorly developed land. At the tail of the canal is a small storage reservoir which is used for livestock watering. At the time of the survey only a small amount of water was in the reservoir and the canal flow failed to reach that far.

The head regulator, with four gates each 2 m wide, is oversized for the required discharge (the maximum recorded discharge was 1.8 $m^3/s$) and hence only one of the gates is maintained for normal use. The other three are silted up and the operating mechanisms non-functional.

The canal was originally excavated to a total width of about 10 m near the head and this has now silted and filled with weed to reduce the water surface width to 2.5 to 3.0 m. This makes canal clearance from the banks difficult and serious consideration should be given to reducing the cross-section of the first 100 m to a size in accordance with the discharge.

In the main reach command is up to 2 m. However, there appears to be no seepage problem even though the canal banks are rather narrow. Surface water is in evidence in the lower reaches where command is minimal (less than 0.5 m), but it is thought to be the result of uncontrolled irrigation water overtopping the poorly constructed banks.

A major drawback to any further development of the Sigaale canal is its location upstream of Janaale barrage (8.4 km) which leaves it sensitive to any changes in the river discharge as the level cannot be maintained. The actual form for development is covered in Chapter 15 of this annex under the Degwariiri development zone.

### 1.2.2 Giddu

The canal is only 6.1 km long and in size (2.5 to 3.0 m wide) similar to many other small canals taking directly from the river upstream of Janaale. The small, 1.0 m wide single gate head regulator is well maintained and the canal operates adequately after weed and silt clearance. Outlets to the fields are entirely through earth breaches, making efficient water control impossible.

Command is high for such a small canal, ranging from 2.5 m to zero at the tail and, because of the very narrow canal banks, intermittent areas of surface water occur where seepage through the canal banks is a problem.

Freeboard appears to be satisfactory except at the inverted siphon which takes the water underneath the Janaale to Awdheegle road where overtopping is prevented by small earth banks placed on top of the concrete abutment walls.

The canal does serve a significant agricultural area (600 ha gross) and therefore widening of the canal banks to reduce seepage problems together with the provision of simple gated outlets to effect better water control are both improvements which should seriously be considered. Like the Sigaale canal, general development of the Giddu is covered in Chapter 15 of this annex under the Degwariiri development zone.

### 1.2.3 Asayle

The canal runs for 15.8 km down the right bank of the river, never being more than 1.7 km from it, and finally drains back into the river just downstream of Gayweerow bridge. At present all the important farmed land lies upstream of the Tawakal cross regulator (Km 8.9) on the right side of the canal, apart from a few areas which are sandwiched between the river and canal, having water supplies from both sources. As with all canals in the Study Area an adequate water supply is only possible after weed and silt clearance; shortly after clearance in October 1977 a discharge of 4.1 $m^3/s$ was measured at the head. The previous months figure (for a similar water level) was only 1.2 $m^3/s$ and at this discharge overtopping problems were arising near the head. Measured water slopes for the canal were:-

| | | | | |
|---|---|---|---|---|
| Km 0.0 | to | 4.8 | - | 250 mm/km |
| Km 4.8 | to | 8.9 | - | 250 mm/km |
| Km 8.9 | to | 15.4 | - | 120 mm/km |

This clearly shows that the first 8.9 km have adequate slope but that the remaining length is only marginal.

The structures on the canal are in a poor state of repair; neither of the head regulator gates operates efficiently and additional lifting gear has to be used to raise them.

Of the five cross regulators only one is functioning (at Km 6.4); all the nine gates on the other four regulators are inoperable. The field outlets are also in poor condition, with only 12.5% operating fully.

The canal banks are adequately wide but freeboard for the first 4 km is very low, the water level often being less than 0.1 m below bank top level; indeed for much of the time strict throttling at the head regulator is necessary to avoid overtopping in this first section. Command is around 1.5 to 2.0 m at the head, steadily falling to less than 0.5 m at the Tawakal regulator. Surface water caused by seepage is quite a serious problem, the worst area being that between the canal and the river. Although seepage rates are thought to be low, this area forms a natural basin from which surface water cannot escape except by slow infiltration. A simple surface drainage system, with the excess water being pumped back into the river could alleviate this problem and allow the redevelopment of the area. In addition the lower half of the canal runs adjacent to an old river channel and very wet areas are found here.

The Asayle canal is of major importance, commanding a gross area of 7 550 ha; therefore the refitting of it with new water control equipment and the remodelling of the earthworks to provide an adequate cross-section is regarded as essential. This has been assessed, together with the rationalisation and restructuring of the minor canals in Section 14.2, the Asayle project. Also the Qoryooley project will have a major influence on the canal, possibly (due to alternative water supply methods) requiring a complete remodelling: this has been dealt with in detail in Chapter 10.

### 1.2.4 Dhamme Yaasiin

This canal is the only one with a fully developed secondary canal system linked to it and the second to sixth secondaries (see Figure 1.1) originally commanded a gross area of some 14 000 ha, mostly for the irrigation of bananas.

Much of this land was abandoned during the 1939-45 war and although much of it has now been recultivated with annual crops, the water control equipment remains unmaintained. The fifth secondary is now partly fed by a small link from the Primo Secondario.

The head regulator has a gated width of 8.75 m but only two of the five gates are fully operational. The maximum discharge recorded at the head was 6.2 $m^3/s$ and, even with only two gates open, only a minimal headloss was required to pass this discharge. Of the 16 gates on the four cross regulators, only one was fully operational. During the survey, overtopping of two cross regulators and the second secondary head regulator was observed, suggesting that these structures are incorrectly constructed to level.

Ponding of seepage water is a serious problem and nearly continuous along both sides from Janaale to the surfaced road at Km 12.3; command in this section falls from over 3 m at the head to about 1 m. The seepage problem appears to be closely related to the excessive command, although the high groundwater table around Janaale is thought to compound the problem by reducing the downward movement of seepage water. Freeboard is generally adequate, apart from the section around the second secondary where negligible freeboard was recorded.

The proposed remodelling of this canal was covered initially by HTS Ltd. (1969) and more recently by the State Planning Commission (1977) to provide irrigation water for a rice production project of 7 200 ha gross. The latter prefeasibility report, covers proposals for refitting and repair of all the structures on the canal together with the provision of stilling basins and downstream protection works. In addition the canal cross-section is to be improved, the existing secondary canals completely refitted, and new secondary canals built ot provide adequate coverage of all the rice project area. This has been reassessed and is covered in Section 14.7 as the Shalambood project.

### 1.2.5 Primo Secondario

This canal is 32.7 km long and maintains, along practically its entire length, a command of between 0.5 and 1.0 m. The only area where seepage is a problem is in the first 4 km. Here a high groundwater table (often less than 1.5 m below ground level) and the proximity of the river make it difficult to avoid the problem of standing surface water adjacent to the canal.

Freeboard is adequate for the entire canal except for two short sections; firstly, the early reaches close to Janaale, where during peak supply only a few centimetres of freeboard are available, and secondly, a short reach around Km 10.0. In this latter section no freeboard is available and during high flows water washes over the banks into the adjacent unused storage reservoirs. Both of these sections require bank heightening and improvement to avoid the loss of water and disruption to road communication caused by overtopping.

The Primo Secondario is the most important canal in the Study Area, with recorded discharges of up to 7.0 $m^3/s$ serving a gross area of 13 690 ha. None of the three head regulator gates is fully operational and only two of them are capable of movement. Consequently it is normal for the head regulator to be operating with a 500 to 600 mm headloss, passing highly turbulent flow into the canal. Of the 25 cross regulator gates on the canal only four were operational and therefore water control along the length of the canal is limited. This means that during periods of low demand for irrigation water, the full peak discharge still has to be passed along the canal to maintain the water level, the excess water being passed into the Bokore canal at the tail of the Primo Secondario and away to an old river canal.

The poor condition of the regulators is at variance with the state of the field outlets which are maintained and operated by the farmers themselves. Of the total number of 99 gates, 60 are fully operational, a much higher proportion than on any other canal.

An important feature of the canal, mentioned previously, is the 28 pumps, almost entirely the twin barrelled discharge pipe type produced in Shalambood, which abstract water for the banana plantations when levels cannot be maintained. They are usually combined with a gravity offtake so that the farmers can use either method to obtain water. If cross regulation was available higher water levels could be maintained and therefore the time needed for pumping reduced.

The Primo Secondario is the only canal in the Study Area, apart from the head reaches of the Bokore, to have sufficient discharge passing along it and a steep enough bed slope effectively to flush the vast majority of the suspended silt load held in the river water along the canal and pass it onto the fields or away to waste (i.e. it is in regime). This means that, once the silt and weeds have been cleared from the canal, it resilts relatively slowly. The complete canal was desilted during August 1977 and by March 1978 the canal was still in a fair condition, with limited weed growth and only a few signs of bad siltation. This compares very favourably with the Dhamme Yaasiin which was cleared during September and October 1977 and by the end of the year was badly overgrown with weeds again (often with 50% of the water surface width lost), or with the Liibaan canal which was completely silted up a little over a month after clearance. The measured water surface slope of the Primo Secondario was 16.2 cm/km over the first 13.3 km, including all structure losses apart from the head regulator. This is far from excessive, indicating that it is the continuously high discharge that is the most important factor in keeping the canal relatively clear of silt and weeds.

The canal is to be modified by the requirements of the Ministry of Agriculture grapefruit production scheme; under this the new barrage at Gayweerow bridge is being built and a new canal (already constructed) will carry water from the barrage to join the Primo Secondario at Km 13.9. This effectively will split the canal into two sections and certain problems are liable to arise in both:-

(i) Upper section Km 0.0 to Km 13.9 : the great risk here is that flows from the new link canal exceed the extra water requirements produced by the grapefruit scheme and the discharge in the upper section is accordingly reduced. This will immediately increase the rate of siltation and weed growth, affecting the best canal in the Study Area. If this is to be the case, then one solution would be to remodel the upper section to be in regime with the new discharges. As this is likely to be prohibitively expensive, it is recommended that the existing discharges in the upper section are maintained, with only the additional requirements of the grapefruit scheme being provided by Gayweerow barrage.

(ii) Lower section (Km 13.9 onwards) : the increased demand in this section is likely to necessitate some remodelling of the canal cross-section, together with the provision of new field outlet structures and cross regulators.

Unfortunately the technical reports for the grapefruit scheme were not available and therefore it was not known whether the above problems had been considered or not. A brief summary of the potential sediment carrying capacities of canals is given in Appendix K.

### 1.2.6 Wadajir

The Wadajir canal passes over uneven terrain; this is clearly shown by the variation in command along the length of the canal. At the head no command is available but it then increases rapidly to 3.5 m and at one point reaches 4.0 m. Finally it decreases in an irregular fashion until the canal joins the Primo Secondario canal at which point the command is about 0.5 m (see Figure 1.1). Because of the excessive command, seepage is a problem along most of the canal but is usually limited to the 25 m beyond the toe of the embankment from where the fill material was borrowed to form the banks.

The canal was originally intended to serve a large area but no secondary canals were built and only 26 direct field outlets exist. Consequently only a small discharge is ever required and the original section has silted up and a dense grass cover established itself, reducing the water surface width to 4 or 5 m at most. Of the recorded discharge near the head (about 0.9 $m^3/s$) a large proportion passes either into the unused storage reservoirs at Madhuulow or into the Primo Secondario.

Freeboard is adequate except for the 3 km downstream of Madhuulow, where only 0.2 m was available at the time of the survey. In fact several months later a breach occurred at this point and approximately 30 ha of land were flooded.

The basic problem of the Wadajir is that its head regulator is 8.2 km upstream of Qoryooley barrage and therefore when the river discharge falls below full capacity, the river level drops sufficiently to prevent water entering the canal. Consequently no refitting of the canal with water control equipment (only one regulator gate out of a total of 14 is operational) can be envisaged unless the canal can be provided with a new offtake point, upstream of the new Gayweerow barrage. This would involve the construction of 1.7 km of new canal and extensive remodelling of the existing canal, together with the provision of water control equipment, secondary canals, and field outlets. This would require full survey and design work to be done and for this reason this possibility has been included in the Faraxaane project described in detail in Chapter 14 rather than as a separate improvement to the existing canal.

### 1.2.7 Liibaan

This canal was originally built to a width of 15 m but, because of the rate at which it silts up, the section is normally limited to a shallow 3 to 4 m width. Silt clearance, although undertaken, has little effect because the very few outlets from the canal (a total of 14) require a small discharge, insufficient to maintain the movement of suspended sediment along the canal. This, therefore, rapidly settles and was observed to block up the canal completely in a little over a month after clearance. The canal has little discharge potential, zero flow being recorded even when river levels were high and the head regulator fully open. Slopes are almost zero with the bed being consistently at a level of 66.0 m plus or minus 200 mm along the entire length of the canal after Km 3.5. The canal occupies an old river channel at Km 6.3, which forms an ideal channel for the disposal of any excess water. At this point a minor extension, 2 km long, feeds water outside the Study Area to a livestock watering reservoir similar to the one at the end of the Sigaale canal. At the time of the survey no flow was reaching the reservoir.

There is scope for remodelling the first 3.5 km of the canal, providing it with correctly designed cross-sections and bed slopes together with new regulating structures and field outlets. Beyond Km 3.5 the land level rises again towards the old river channel and therefore is not suited to refurbishing. If the canal is remodelled it would mean that no water could be passed from the river to the reservoir at the tail and therefore the consequences of this must be carefully considered.

### 1.2.8 Bokore

Although the Bokore system was never completed, it still represents an important contribution to the irrigation network and is generally in a reasonable condition. Water control is available at the head regulator, where all nine of the gates are fully operational, and to a lesser extent at the tail pool entrance (at the junction with the Primo Secondario) where three of the seven gates are operational. Neither of the cross regulators has any gates working but nearly half of the field outlets are fully operational. The last 2.4 km of canal, beyond the tail pool, are in a poorer condition with only earth outlets and very low commands. This section mostly acts as a drain to take the excess canal water gathered from the Bokore, Wadajir and Primo Secondario canals into the old river channel close to the sand dunes.

Because the canal is not running at design discharge, it silts up quite rapidly, but regular clearance appears to be coping with this. Unfortunately the Poclain excavator used to do this was broken down during most of the 1977 der season so that silt and weed clearance was behind schedule. Freeboard is adequate along the entire length of the canal, although a few sections of only 0.2 m require to be increased. The command is reasonably high, starting at around 2.5 m and steadily falling to less than 0.5 m at the tail pool. Seepage problems are minor and limited to the first 7.7 km, as far as Jeerow.

The Bokore appears to have scope for further development and because of this it has been utilised for both the der flood project and Mukoy Dumis project which are described in Chapter 14.

## 1.3 Minor Canalisation

An important feature of the complete irrigation system is the lack of secondary canals; these are limited to the Dhamme Yaasiin and Bokore canals and even here provide limited coverage of the gross irrigated areas. Consequently it is typical to have direct feed from the main canals through field outlets to minor canals which serve the field irrigation requirements. The excessive command of some of the main canal sections means that the field outlets (where they exist) are operated with large headlosses (in some cases over 2 m) and problems of scour and difficult water control result.

The minor canals vary greatly from short field channels only 100 to 200 m long, serving perhaps at most 3 ha, to long channels which cross the field systems for as much as 5 km, feeding a total of up to 100 ha of cultivated land. In this category of canals are included all those that offtake directly from the river, serving a gross area of 12 560 ha within the Study Area. Water feeds directly from the minor canals, almost without exception through breaches cut by hand into the bank, either directly into the large basins used for the pre-planting single irrigation of sesame or into field channels which in turn supply the small basins (jibal) used for the irrigation of maize, bananas and grapefruit. A detailed description of field irrigation practices is given in Annex IV, Chapter 5.

The fundamental problem with the minors is the fast rate at which they silt up and become choked with weeds; a complete clearance is required twice a year, before both the gu and der seasons, often requiring the excavation with simple hand tools of up to 500 mm of solid clay. Only on some of the banana farms are small mechanical excavators in use to assist with this laborious work and within the Study Area the hand clearance of minor canals is a major use of the available labour resources. Clearance is organised on a village basis and one member is nominated to be in charge of the complete operation. Any improvement of the situation can be achieved by either the provision of sedimentation facilities to reduce the silt load entering the minor, or ensuring that more of the silt is flushed directly onto the field and does not deposit in the channel. The latter can only be done by improving slopes along the canals, and as the system tends to extend to the extreme limit of command available, this would involve a reduction in the area served.

The general improvement of the minor canals is considered in Chapter 15 as part of the development of existing irrigation in the Study Area.

## 1.4 The Consumptive Use of Water in the Study Area

During the der season of 1977 regular current meterings were made of all eight main canals. Measurements began in August and continued at monthly intervals until December. On all the canals the metered section was within 900 m of the head, and for the main canals at Janaale, within 300 m of the head, with no significant offtakes upstream of the section. Therefore the measured discharges given in Table 1.4 represent the actual head discharges of the main canals and have been summed for each month to give the total flow entering the eight canals. The sub-total has then been increased on a pro rata basis of the gross irrigated areas to give an estimate of the total consumptive use of irrigation water from the river in the Study Area. This assumes that the average water supply per ha to the areas close to the river and fed by minor canals directly from it, is equal to the average water supply per ha for all the land irrigated by the eight main canals. This is estimated to be a reasonable assumption because of the general uniformity of agricultural production throughout the Study Area where water supplies are available. During the entire period of measurement the river levels were high and therefore the discharge figures should not reflect any features of water shortage, but merely the irrigation demands of the crops in the area.

Regular observation of all the canals, and information provided by the water guards on each canal, make it possible to convert the discharges into approximate monthly consumptions (Table 1.4). The discharges in the two largest consumers of water; the Primo Secondario and Dhamme Yaasiin, were consistently stable and only changed slowly in response to the changes in demand; consequently the monthly consumptions for these canals should accurately represent the actual figures. However, for the other canals, partly due to access difficulties during the abnormally heavy rains, and a lack of central control, it was difficult to follow all the changes in gate opening etc. Even so, it is thought that the monthly consumptions will give a fair estimate of the actual amounts and these have been summed for each month to give a total for the eight main canals. As with the discharges, these totals have been increased on a pro rata basis to give the total consumptive use for the entire Study Area.

Certain interesting features can be noted from Table 1.4:-

(i) The total discharge in October of 31.26 $m^3/s$ represents about 40% of the river capacity as it enters the Study Area.

(ii) The peak demand occurs in October, just before the der season rains start in November. During 1977 the rains were exceptionally heavy and therefore the figures for November and December may well be significantly less than those of a normal year.

(iii) Taking the total net cultivated area of the Study Area in the der season as 20 960 ha (derived from the land use map) the October discharge represents a gross average watering rate of 1.5 l/s/ha. Detailed discussion of watering rates is given in Annex II.

## TABLE 1.4

### 1977 Der Season Water Consumption

| Canal | Gross area irrigated (ha) | Measured discharge ($m^3/s$) | | | | | Approximate monthly consumption ($Mm^3$) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Aug | Sep | Oct | Nov | Dec | Aug | Sep | Oct | Nov | Dec |
| Sigaale | 290 | 0.00 | 0.00 | 1.80 | 1.06 | 0.68 | 0.0* | 0.0* | 4.8 | 2.8 | 1.8 |
| Giddu | 600 | 0.00 | 0.37 | 1.21 | 0.69 | 0.05 | 0.0* | 1.0 | 3.2 | 1.8 | 0.1 |
| Asayle | 7 550 | 1.67 | 1.23 | 4.13 | 2.02 | 1.03 | 2.8) | 2.9+ | 11.1 | 5.2 | 2.8 |
| Dhamme Yaasiin | 9 630 | 3.20 | 2.75 | 6.23 | 6.37 | 3.25 | 8.6 | 7.1 | 16.7 | 16.5 | 8.7 |
| Primo Secondario | 13 690 | 5.13 | 6.66 | 6.98 | 5.73 | 4.64 | 13.8 | 17.3 | 18.7 | 14.8 | 12.4 |
| Wadajir | 2 890 | 0.88 | 0.91 | 0.86 | 0.77 | 0.44 | 2.4 | 2.4 | 2.3 | 2.0 | 1.2 |
| Liibaan | 1 230 | 0.12 | 0.00 | 0.11 | 0.00 | 0.07 | 0.3 | 0.0 | 0.3 | 0.0 | 0.2 |
| Bokore | 5 730 | 1.96 | 1.70 | 2.69 | 2.69 | 1.14 | 5.3 | 4.4 | 7.2 | 7.0 | 3.1 |
| Sub-total of 8 main canals | 41 620 | 12.96 | 13.62 | 24.01 | 19.33 | 11.3 | 33.2 | 35.1 | 64.3 | 50.1 | 30.3 |
| TOTAL for complete irrigated area | 54 180 | 16.87 | 17.73 | 31.26 | 25.16 | 14.71 | 43.2 | 45.7 | 83.7 | 65.2 | 39.4 |

Notes:
* closed all month for weed clearance
) closed 13th to 25th for repair work
+ closed 6th - 9th for seepage test

In addition to the current metering at the heads of the main canals, it was also undertaken at the exit of the tail pool on the Bokore canal to obtain an estimate of the amount of water being wasted from the Bokore, Wadajir and Primo Secondario canals. On 16th October 1977, before the rains arrived and with all the gates of the tail pool supposedly shut, a discharge of 0.70 $m^3/s$ was measured. This is only 6.6% of the total three canals intake. However on 16th November 1977, during the middle of the heavy rains, and with three of the gates open, the discharge was measured at 1.88 $m^3/s$, representing 20.5% of the total three canals intake.

## 1.5 Operation

The irrigation network is operated by the regional irrigation office of the Ministry of Agriculture at Janaale. The staff at Janaale cover both the Lower Shabeelle regional responsibilities and the work for the Marka district. Irrigation control for the Qoryooley district (covering the Bokore and Liibaan canals) is from the district office of the Ministry of Agriculture in Qoryooley. Table 1.5 gives a list of the irrigation staff at both of these offices. Of the total of 45 personnel, 21 are canal guards or barrage operators.

**TABLE 1.5**

**Irrigation Staff**

| | Number |
|---|---|
| Regional Office at Janaale | |
| Regional head of irrigation | 1 |
| Assistant head of irrigation | 1 |
| Head of land registration | 1 |
| Head of canals and equipment | 1 |
| Head of hydrology | 1* |
| Canal guards and barrage operators | 18 |
| Excavator drivers | 8 |
| Bulldozer drivers | 6 |
| Mechanics | 2 |
| Vehicle drivers | 2 |
| TOTAL | 41 |
| District Office at Qoryooley | |
| District agricultural officer | 1 |
| Canal guards and barrage operators | 3 |
| TOTAL | 4 |
| GRAND TOTAL | 45 |

Note: * not present at the time of the study

The flow in the canals is normally continuous although irrigation is limited almost entirely to the day time. At night the water was channelled into the on-farm storage reservoirs for use during water shortages. However these are now mostly abandoned and the over-night flow goes to waste either in the unused reservoirs or at the tail of the canal system. When riverflow is adequate no attempt is made to regulate the canal supplies and gate control is only undertaken to avoid over-topping of the canal banks. Canals are closed either partially or totally at the request of the farmers. Written records of water levels in the canals are not kept nor are discharge observations taken. Major opening dates of canal head regulators after clearance are sometimes recorded but the full history of gate operations is not recorded.

The upstream gauge level at Janaale barrage is recorded and a figure of 4.20 m is regarded as the minimum necessary for satisfactory operation of the three main canals just upstream of the barrage. Regulation is needed when the water level falls below this and at 3.20 m or less a rotation of the canals is imposed, with only one canal at a time taking water.

On some of the main canals the supplies to the farmers during the periods of water shortage are scheduled to achieve a fair distribution of the available water. This is always done on a 10-day rotation for the canals divided into three sections (Dhamme Yaasiin) and on a 9-day rotation for the canals divided into four sections (Primo Secondario and Asayle). Just before a rotation begins, notification is sent to the canal guards (by letter) who in turn notify the farmers; Table 1.6 is a copy of an actual rotation used on the Primo Secondario during June 1977.

## TABLE 1.6

### Irrigation Scheduling on the Primo Secondario

| Date | Section receiving water | Canal chainage (km) |
|---|---|---|
| 15.6.77 | ** outlets closed to build up water level ** | |
| 16/17.6.77 | Shangaani - Antonio | 32.7 - 26.2 |
| 18/19.6.77 | Antonio - Golweyn | 26.2 - 21.7 |
| 20/21.6.77 | Golweyn - Beerdasalax | 21.7 - 16.3 |
| 22/23.6.77 | Beerdasalax - Janaale | 16.3 - 0 |

Two days are available for each section and this is generally inadequate for the water to be rotated around all the farms on each minor canal. Therefore in many places a system of village scheduling has developed so that each farmer can receive water in turn.

This system of scheduling will not work efficiently until the water control equipment on the main canals is repaired or replaced. At present no cross regulators on the Primo Secondario are operational and therefore, to hold the water levels in each section of the canal to provide sufficient command, a large discharge has to be passed, a proportion of which is wasted at the tail of the canal.

## 1.6 Maintenance

Maintenance of the irrigation system is shared between the farmers and the irrigation department at Janaale. The farmers are responsible for all minor canal clearance and the maintenance of the field outlet structures on the main canals and secondaries. The irrigation department is in charge of maintenance of the main canal and secondary canal earthworks and regulating structures. This involves a total of 119.3 km of main canal and 32.8 km of secondary canal (II, III, IV, V and VI secondaries of the Dhamme Yaasiin, and the SISAB and Jeerow secondaries of the Bokore) within the Study Area (but including the full length of the Sigaale and Liibaan canals).

Table 1.7 lists the plant available to the office at Janaale for all the maintenance work in the complete Lower Shabeelle region, but this is almost entirely gathered in the Study Area. In addition a small workshop exists in the grounds of the Janaale Seed Multiplication Centre for the repair of this plant and the six Land Rovers and Toyota Land Cruisers operated from Janaale and Qoryooley. Equipment is available here for mending punctures, welding, lifting of small engines, and general inspection work from a pit together with spanner work. If any heavier or more difficult work is required the plant is transported to the NFMAS workshop at Shalambood where almost any repair work can be undertaken.

TABLE 1.7

Maintenance Plant in Lower Shabeelle Region

| Plant item | Number |
|---|---|
| 70 hp Poclain hydraulic excavators | 2 |
| 80 hp Poclain hydraulic excavators | 3 |
| 100 hp Poclain hydraulic excavators | 1 |
| Fiat Allis BD 14 bulldozers | 5 |
| Fiat Allis BD 20 bulldozers | 6 |
| Fiat AD7C bulldozers | 2 |
| Same tractors | 2 |

The unlined earth canals in the region are being silted up by the heavy sediment load carried by the river waters and this provides an ideal base for rapid weed growth. Consequently much of the year is spent in desilting operations. Clearance of the main canals and secondaries is done by the six Poclain excavators fitted with special desilting buckets. If preliminary clearance of the canal bank is needed, one of the Fiat Allis BD14 bulldozers is used to remove the bushes and excess spoil. At any given time up to three of the BD14s can be engaged in this work.

During clearance operations of the Dhamme Yaasiin canal in September 1977 the rate of working was carefully monitored and a figure of 21 machine-hours per kilometre of bank calculated. Therefore each excavator should be capable of clearing 2 km of canal each week where the entire canal can be cleared from one bank, and 1 km of canal each week where clearance from both banks is needed. This is based on a seven hour working day, six days per week. It is present practice to clear only the Primo Secondario, Dhamme Yaasiin and Bokore canals (a total length of 66.9 km) from both banks and therefore the total machine time required to clear the complete system is 110 machine-weeks. If, at any given time, only four of the excavators are operational (due to breakdowns, servicing etc.) it should take them just over six months to clear the entire system. This rate of working was confirmed during the three months of mid-July to mid-October 1977, a total of 13 weeks, when the entire lengths of the Dhamme Yaasiin, Asayle, Giddu and Liibaan canals and 25 km of the Primo Secondario canal were cleared. The theoretical time for this is only slightly longer at 14 weeks.

Therefore there appears to be adequate plant capacity for one complete canal clearance of weeds and silt each year, with ample spare time for more frequent clearance of the poorer sections of canal, such as the Liibaan, and other earthwork maintenance jobs. However there is a strong case for clearing all canals twice a year, in which case the plant would only just be sufficient.

The maintenance of all regulating equipment on the main and secondary canals is also the responsibility of the Janaale and Qoryooley offices. The office at Janaale compiles a list of the new gates required in the region and passes this to NFMAS in Shalambood. Here the facilities exist for making both simple vertical sliding gates and nut and key operated gates. During a visit in November 1977 a small stockpile of three nut and key operated gates and about ten simple sliding gates was seen. These were to be fitted during the jilal when canal levels were low.

However, when the poor condition of the water control equipment in the Study Area is considered, it is obvious that the rate of replacement of gates represents only a small proportion of what is actually needed.

## 1.7 Recommendations

Section 1.2 has dealt with the detailed descriptions of all the main canals and the specific recommendations for each one, possibly only by referring to the master plan of development described in Chapters 14 and 15. However, there are certain major aspects of improvement that are common throughout the Study Area and these are dealt with in the sub-sections below.

### 1.7.1 Canal Redesigns

Much of the problem caused by siltation and weed growth is aggravated by the poor design of the existing canals. In many cases the size of the canal is far too large for the discharge flowing in it and the cross-section should be reduced in order to improve flow velocities. It should be possible to do this solely by limiting the size of channel excavated during clearance operations and reforming the canal banks with the excess spoil that has gathered over the years of clearance. To effect proper designs, it would be necessary to adjust the slope of the canals and this is regarded as a more difficult operation requiring careful design and supervision.

### 1.7.2 Water Regulation Equipment

At present the water control equipment on regulating structures in the area is in poor condition. This makes the efficient control of water difficult and the amount of wastage high. It should be considered of the utmost importance that a programme of repair work is implemented. With fully operational equipment the time needed for control operations would be reduced, water supplies could be divided more fairly and efficiently during periods of shortage, and the amount of pumping needed reduced.

### 1.7.3 Provision of Field Outlets

On several of the canals very few gated concrete field outlets are available and water supplies to the field are through simple earth breaches. This method is inefficient, requiring the bund to be broken and remade at each irrigation, and provides no opportunity for the close control and fair allocation of water resources. In addition, where no pipes are provided at all, the outlets stop any vehicular movement along the canal bank, impairing the canal clearance operations. Simple gated field outlet pipes should therefore be provided so that close and simple water control can be achieved.

### 1.7.4 Silt and Weed Clearance

The greatest barrier to the successful distribution of water is the rapid siltation of, and weed growth in, the canals. Sufficient plant is available for the clearance operations and it must be ensured that scheduling of the work is prepared so that the canals are cleared before the problem becomes too severe. This is likely to put a severe strain on the existing excavators and therefore provision should be made for an adequate supply of spare parts together with complete machine replacement on a regular basis.

# CHAPTER 2

# IRRIGATION DEVELOPMENT

## 2.1 Introduction

The general improvement of irrigated agriculture in the Study Area, either in the form of specific development projects or more generally as a complete upgrading of the existing irrigation systems, depends upon the correct implementation of a broad spectrum of basic engineering criteria. This covers subjects ranging from the initial clearance of new land for development, through simple canal design, to final field operation and water control. Each subject is discussed in this chapter and the guidelines for development within each group laid down; these apply equally well to both the Qoryooley project feasibility study detailed in Part II and the Master Plan developments described in Part III. The only major subject not included is drainage which, because of its importance, is given separate coverage in Chapter 4.

## 2.2 Land Preparation

### 2.2.1 Bush Clearance

Any new land brought into cultivation or abandoned land to be recultivated after a break of many years, will require the bush vegetation to be cleared away as the first stage of development. The uncultivated land in the Study Area is dominated by light Acacia bushland, with occasional large trees of girth greater than 0.5 m. The number of trees is generally low except for the areas close to the river. Certain pockets of very dense thicket exist, almost entirely associated with the courses of old river channels; the bush in these areas is impenetrable, with practically no open spaces at all.

For either the light bush or dense thicket the recommended procedure for clearance is the same (Caterpillar, 1974) and can be broken down into three distinct phases:-

(i) Cutting: this involves the use of a crawler tractor of 100 hp or larger, preferably with a special cutter blade to remove the vegetation at ground level.

(ii) Root ploughing: special attachments for mounting on tractor tool-bars are available for undercutting vegetation below the soil surface to kill it effectively; any large roots are forced to the surface by special fins. Root ploughing can break up the hard clay soils, improve the moisture intake ability, and obviate any necessity for sub-soiling.

(iii) Raking: to clean all the debris left behind after cutting and root ploughing a simple root raking pass is needed.

Table 2.1 gives the estimated tractor times for the complete clearance of both light bush and dense thicket. In addition to this, special provision must always be made for the removal of some of the large trees. In many cases the trees can be left in place to provide shade or, if in sufficient number, windbreaks. Of those to be removed, the larger ones will require felling and complete excavation of the root stump, which is a time consuming process.

TABLE 2.1

**Tractor Times in Hours per Hectare for Bush Clearance Operations**

| | Light bush | Dense thicket |
|---|---|---|
| Cut and pile | 1.06 | 3.18 |
| Root plough | 1.40 | 2.80 |
| Rake | 1.00 | 2.00 |
| Sub-total | 3.46 | 7.98 |
| Allow for idle time and transport | 1.15 | 2.66 |
| TOTAL | 4.61 | 10.64 |

### 2.2.2 Land Forming

Land forming (also known as land grading and land levelling) is the preparation of the land surface to permit the uniform distribution of irrigation water and provide for drainage of excess surface water. Attempts to irrigate land with an uneven surface generally result in low water use efficiencies, excessive labour requirements and poor crop yields. Running water deep into low points in order to wet the high points causes uneven water distribution and erratic plant germination growth, possibly with failure due to waterlogging. Land forming, therefore, involves the movement of soil to fill in the low spots and provide a uniform plane to the surface of the land.

Assuming that a significant area of land is to be formed, the process becomes a fully mechanised one. Initial movements can be done by scrapers and grader operations but the final corrections to the surface need to be done by a land plane. As far as the Client is aware, no such land planes are currently available in Somalia and therefore the standard of finishing so far has been less than ideal. However, recent work at the Jowhar sugar estate using only a bulldozer, but with a highly experienced operator, has been giving very satisfactory results.

Some settlement of the soil is likely to follow the first irrigations, especially in areas of deep fill, causing irregularities even on soils that have been carefully graded. For this reason it is best to plant an annual crop for the first season and, after harvesting, the low spots which have developed can be refilled. Only then can perennial crops be safely planted.

The complete process of land forming is controlled by an accurate topographic survey which determines the land levels at co-ordinate points established on the field to be graded. Booher (1974) recommends that the maximum spacing between co-ordinate points should be 30 m. During field work for the Qoryooley project three trial areas, each of 6.25 ha, were carefully selected and surveyed on a 25 m co-ordinate grid. The required amount of cut and fill was then calculated using the plane of best fit method as 300, 295 and 195 $m^3$/ha for the three trials (no allowance was made for overcut to compensate for compaction or bulking). These results are quite low, and compares to a figure of 450 $m^3$/ha measured for the Afgoi-Mordile irrigation project (Libsoma), possibly reflecting the effect of existing cultivation. However, the trials were calculated using the optimum orientation for the plane of best fit and this is more likely not to be the case when a new irrigation layout is being superimposed onto the microtopography. The possible increases may be sufficient to double the required amount of fill.

Appendix B gives the results of the three trials together with the calculation of fill requirements. It must be remembered that these trials only apply to the Qoryooley Project Area and can only be interpreted as being more generally representative within the complete Study Area when further surveys have been completed.

## 2.3 Methods of Irrigation

The different methods of irrigation can be broadly assigned to four major categories: namely surface, sprinkler, trickle (or drip) and sub-irrigation. The last group is a method of utilising a deep drainage system to 'top up' the groundwater table, allowing the crop to draw water by capillary rise upwards into the capillary zone. The topsoil needs to be permeable to allow the adequate movement of water horizontally away from the drains, and the water supply free from salts to avoid the rapid build-up of harmful salinity levels. Clearly, therefore, this method is unsuitable for irrigation in the Study Area as the soils are relatively impermeable and the irrigation water supplies moderately saline, and sub-irrigation has not been considered any further.

The trickle systems carry water under pressure along pipes and the water is released onto the field through narow emitters. These are, therefore, susceptible to clogging with any suspended sediment contained in the irrigation water. The Shabeelle suffers during much of the year from extremely high suspended sediment loads; during the der season of 1977, weekly sediment samples produced levels ranging from 2 000 ppm to 7 300 ppm with an average load of 3 300 ppm. Sedimentation of this load is possible and complete settling of the small samples of river water taken for sediment testing was observed to occur in as little as three hours, leaving a clear and unturbid supernatant. However, in an operating system where water is never quiescent, full removal of the sediment will only be possible if filtration is included. This is essential for the operation of a trickle system and this would prove extremely expensive and difficult to operate and maintain. Consequently it is recommended that trickle systems are not considered for any project developments in the Study Area.

If an efficient sedimentation basin could be constructed and maintained, which could remove at least 80 or 90% of the silt load from the river water, then it may be possible to operate a sprinkler irrigation system successfully without the need for an expensive filtration system. However it is thought to be sensible to await for both:-

(i) the successful operation of a sedimentation basin to settle the Shabeelle silt load for a more traditional surface irrigation project; at present there is none but one is proposed for the Qoryooley project and also the Ministry of Agriculture grapefruit scheme, and

(ii) the successful implementation and operation of the sprinkler irrigation system designed for the Juba sugar project

before any sprinkler systems are considered for use in the Study Area. A study of the comparative costs of surface irrigation and sprinkler irrigation has been made and the results are given in Appendix J. This clearly shows that surface irrigation is more economic in the Study Area. Therefore only surface irrigation methods can be immediately recommended and only these have been considered any further for irrigation development.

## 2.4 Surface Irrigation

The methods of surface irrigation are varied and many individual types can be identified. However, for the purposes of this study they can be grouped into the following categories.

### 2.4.1 Wild Flooding

During flood periods river banks are breached and the adjacent land inundated. This method is practised in the higher sections of the river above Mahaddaay Weyn but because of the very poor efficiency of water use and its non-uniform distribution it is thought to be unsuitable for further development on the Shabeelle Flood Plain. Limited water supplies, and the possible storage of flood flows in offstream storage reservoirs, demand an efficient use of irrigation water, greater than that possible by this method.

### 2.4.2 Large Rectangular Basins

During the der season many of these large basins, covering anything between 1 and 6 ha, are formed in the Study Area by bulldozers pushing up earth banks of about 1 m height. In some cases the banks are, in fact, permanent. The basins are then filled, either by canals directly from the river or by channels feeding from the existing main canal network, to a depth of about 300 mm, although this will vary greatly because of the poor land levels within the basin. This slowly infiltrates, taking as much as two weeks completely to soak into the ground; after this the traditional sesame crop is planted and has to grow by drawing solely on the available moisture in the soil at planting, assisted slightly by the der rains in November.

The net irrigation requirements of the crop (i.e. the requirement after allowing for effective rainfall) are 270 to 312 mm for the complete season, depending upon the planting date. Taking a rooting depth of 800 mm and an available soil moisture content of 20% (Annex VI, Chapter 2) provides a total supply of water for the crop from the soil of 160 mm. This represents at best only 60% of the net irrigation requirement. However, if some additional supply by capillary rise from the soil moisture below 800 mm depth is included (this is unlikely to be capillary rise from the groundwater as it is almost always at too great a depth), then sufficient moisture is available to grow sesame successfully using this method of simple pre-irrigation with only minor problems of water stress.

The extension of this method to other crops does, however, face major problems. One crop that has been grown successfully with this method is cotton. In the Gash area of the Sudan (Thorne and Peterson, 1954) the soil is flooded for about three weeks. This wets the soil to a depth of at least 4.5 m. After flooding, the cotton is planted and the roots follow the receding water table and bring the crop to maturity without further irrigation. Also on the Mbuga black cotton soils fringing Lake Victoria, cotton is grown without irrigation, relying solely upon the soil moisture due to the previous wet season flooding of the land. However, with the Study Area soils, it is considered that rooting depths will be severely limited, around 1 m being a maximum. In addition large quantities of capillary movement are not possible (indeed this could be dangerous because of the high salinity of groundwater). Consequently it is not possible physically for high requirement crops such as cotton (net irrigation requirement 638 mm) to be grown under this system.

Only sesame has therefore been considered for single pre-irrigation large basin flooding and this has been dealt with specifically in the der flood project described in Chapter 14.

### 2.4.3 Small Rectangular Basins

This is the most common method of controlled irrigation used in the Study Area, all the maize and established banana irrigation being done this way. It is suited ideally to the traditional techniques of cultivation as no sophisticated control is needed and water movement can be simply controlled by making breaches in the small basin bunds (about 150 mm high). The low soil intake rates allow a good water application efficiency and by making the basins small the problem of uneven land surfaces can be, at least partly, overcome, allowing a reasonably uniform water distribution. The normal basin size is the jibal which is 25 x 25 paces, but where the land surface is very uneven a further sub-division into four is made.

As this is the traditional irrigation method and local expertise is associated with it, the majority of new developments are expected, at least initially, to use a certain proportion of small basin irrigation.

### 2.4.4 Border Strips

This method, which directs the water between two long, parallel levees, is ideal for the use of farm machinery. However, the land levelling requirements are extremely stringent and the degree of water control needed very high. As this method is untried in Somalia, it is not recommended for any developments.

### 2.4.5 Furrows

Short furrows have been in use in Somalia for a long time, both at the Jowhar sugar estate (MMP, 1976) where 25 m furrows are used, and in the Study Area, where approximately 50 to 100 m long furrows are utilised for the first few months after planting of bananas. These, however, are little different to the normal small basins and in the latter example the fields quickly revert to basins (see Annex IV, Section 11.2.6).

Long furrows (more than 100 m), where the full benefits of reduced labour requirements and easy machine operations can be felt, are almost unknown in Somalia and appear to have been used at only three places:-

(i) The Afgooye-Mordiile irrigation project (Libsoma) where 138 m furrows have been used. Major problems have occurred because hand weeding has obliterated the furrows.

(ii) The Palestinian Liberation Organisation farm in the Study Area. Using furrows of more than 100 m, tomatoes have been successfully irrigated.

(iii) The Jowhar sugar estate drainage trials (MMP, 1978). Here 265 m long furrows have been used in two fields to irrigate the cane fields in the drainage trials.

With the correct level of mechanisation it is thought that long furrow irrigation will be highly successful. It can give a high water application efficiency and the land forming requirements are far more flexible than for border strip irrigation. Therefore long furrow irrigation is considered as the method holding most potential for improved agricultural development in the Study Area.

Table 2.2 gives a summary of the methods of surface irrigation, together with the constraints on use, advantages and disadvantages of each system, and the crops they are most suited to.

## 2.5 Operation and Water Control: the Watercourse Unit

### 2.5.1 System Control

For new projects in the Study Area, the method of water supply and control, together with the scheduling system, have to be selected. Water supply can either be achieved by a 'demand' system where water can be drawn by the farmer whenever he wants, the correct discharges and water levels being maintained by automatic water control equipment, or by a 'forced scheduling' system where water supply is controlled by the project management. The former method requires larger canals, more expensive water control equipment structures at more frequent intervals and is less efficient in the use of water. The structures themselves are sensitive and require a very high level of maintenance. Therefore, considering the possibility of water shortages and the poor standard of water control equipment maintenance in the Study Area, it is recommended that only forced scheduling systems are used.

With such a system the discharges in the main canals (assuming flow on a continuous basis) remain constant for long periods and therefore manual control can be achieved easily. For the Qoryooley project, a vertical sluice gate head

## TABLE 2.2

### Methods of Surface Irrigation

| Irrigation method | Suitabilities and conditions of use | | | | |
|---|---|---|---|---|---|
| | Crops | Topography | Water supply | Soils | Remarks |
| Wild flooding | Pasture, grain | Irregular surfaces with slopes up to 20% | Can utilise small continuous flows on steeper land or large flows on flatter land | Soils of medium to fine texture with stable aggregates which do not crack on drying | Little land grading required. Low initial cost for system. Best adapted to shallow soils since percolation losses may be high on deep permeable soils |
| Large rectangular basins | Grain, field crops, rice | Flat land must be graded to uniform plane | Large flows of water | Soils of fine texture with low intake rates | Lower installation costs and less labour required for irrigation than with small basins. Substantial levees needed |
| Small rectangular basins | Grain, field crops, orchards, rice | Relatively flat land; area within each basin should be levelled | Can be adapted to streams of various sizes | Suitable for soils of high or low intake rates; should not be used on soils that puddle | High installation costs, considerable labour required for irrigation. When used for close spaced crops, a high percentage of land is used for levees and distribution ditches. High efficiencies of water use possible |

**TABLE 2.2 (cont.)**

| Irrigation method | Suitabilities and conditions of use | | | | |
|---|---|---|---|---|---|
| | Crops | Topography | Water supply | Soils | Remarks |
| Narrow borders up to 5 m wide | Pasture, grain, vineyards, orchards, lucerne | Uniform slopes less than 7% | Moderately large flows | Soils of medium to heavy texture | Borders should be in direction of maximum slope. Accurate cross-levelling required |
| Wide borders up to 30 m wide | Grain, lucerne, orchards | Land graded to uniform plane with maximum slope less than 0.5% | Large flows up to 600 l/s | Deep soils of medium to fine texture | Very careful land grading necessary. Minimum of labour required for irrigation. Little interference with use of farm machinery |
| Furrows | Vegetables, row crops, orchards, vineyards | Uniform slope not exceeding 2% for cultivated crops | Flows up to 350 l/s | Can be used on all soils if furrow length is adjusted to type of soil | Best suited to crops which cannot be flooded. High irrigation efficiency possible. Well adapted to mechanised farming. |

Source: L.J. Booher, Surface Irrigation (1974) regulator and movable weir cross regulators at intervals of 3 to 4 km have been used for water control on the branch canals. The great advantage of a weir is that, as long as it is not drowned out, the discharge over a given weir is solely a function of the upstream head. Therefore, given accurate control at the head, a system can be easily controlled.

### 2.5.2 The Watercourse Unit

The most important requirement of a water distribution system is that it can provide adequate supplies, allowing the correct scheduling and rotation of irrigation water to meet the demands of the irrigation interval. It is advised that all rotation of water supplies is limited to within a simple field or watercourse unit. The unit would be provided with a continuous discharge for the length of every irrigation day and water fed along a permanent watercourse to the section of land to be irrigated. If the irrigation interval is say, ten days, then each day one-tenth of the land within the unit is watered. This simple system has the advantages mentioned below:-

(i) in main canals the discharge can remain almost constant, only changing slowly during a season in response to the variation in crop water requirements. This simplifies water control and provides the canals with sufficient discharge to keep them relatively free from silt.

(ii) the rotation of water supplies is a simple problem which can be dealt with by the individuals farming the watercourse unit. With a system of many small farmers the equitable sharing of water therefore should present no major problems.

(iii) the hydraulic design of canals, and the efficiency of water use, are both as close as is practicable to the optimum system of continuous flow to all land

(iv) access to the field can be easily arranged.

(v) the unit is flexible and can be adapted to any form of surface irrigation.

The maximum area of a watercourse unit can be found from the simple equation:-

$$A\ max = \frac{q}{I_g} \cdot \frac{h}{24}$$

where q (litres per second) is the maximum stream size a farmer can handle. A figure of 40 to 60 l/s was recommended for the Lower Khalis project in Iraq (MMP, 1971) but more recent information, both from the Sudan and the Afgooye-Mordiile irrigation project (Libsoma), indicates that streams up to 100 l/s can successfully be handled.

Ig is the continuous irrigation requirement in litres per second per hectare, including losses for deep percolation, seepage from watercourses and surface run-off.

h is the length of the irrigation day in hours.

## 2.6 Seepage Losses and Canal Lining

The seepage of water through the banks and beds of canals is a problem much in evidence in the Study Area. In areas of high command and/or water tables close to the surface, the borrow trench either side of the main canals is often filled with standing water presenting a health hazard, a barrier to access and a loss of land and water.

Consequently, consideration has been given to the question of canal lining. The main factors which favour lining are as follows:-

- (i) saving of the water lost by seepage for other use
- (ii) reduction in canal earthworks costs
- (iii) reduction in maintenance costs
- (iv) elimination of a possible health hazard.

Against these points must be balanced the cost of lining and the greater technical skill needed in lined canal repair work. A detailed comparison made for the Lower Khalis project feasibility study (MMP, 1971) showed that buried membrane lining of major canals would increase the total cost over 30 years for the canalisation by 11%. The similar increase for hard surface lining was 20%. These results were based on a seepage rate of 2.50 $m^3/s$ per million square metres of wetted area. The use of clay lining was not considered necessary in the clay soils of the Study Area.

A tank seepage test was executed between the 6th and 9th September 1977 on a 450 m long section of the Asayle canal with no offtake points, just downstream of the Tawakal cross regulator. A large bund was put across the downstream end of the section using a Poclain hydraulic excavator and then the top end was sealed in the same way. Water level measurements started immediately and were taken with the aid of reference spikes to the nearest millimetre, at four points along the tanked section. Detailed cross-sections were taken at five points and the total wetted area of 2 790.6 $m^2$ and water surface area of 2 433.4 $m^2$ were calculated from these.

Apart from an initial rapid drop, caused by the seepage of water through cracks in the newly wetted section of banks (between completing the lower bund and sealing the top bund the level rose by approximately 300 mm) the rate of fall was entirely constant over the full 68 hours of the test at 1.958 mm/h. It was even possible to detect the minor variation caused by the different day and night time evaporation rates.

The figures are equivalent to a seepage rate of 0.474 $m^3/s$ per million square metres of wetted area. The site was carefully chosen to be, as much as possible, representative of all the major canals in the Study Area. Consequently, as the seepage rate was only one-fifth of the rate used for the Lower Khalis cost comparison, there can be no doubt that canal lining would be an uneconomic proposition for the Study Area. However, if new unlined canals are to be built the following points must be taken into consideration to avoid the problems of the existing canals:-

(i) The total command should be the minimum possible consistent with maintaining field command.

(ii) To avoid seepage water reaching the surface the maximum gradient between the peak water level and the toe of the embankment should be limited. A suggested maximum is one in seven although, with the impermeable soils in the Study Area, it may be possible to increase this gradient.

(iii) At present any ponded water in the region of canals has no chance to drain away anywhere as infiltration rates are low and no surface drains exist. On new canals it should be made possible to link any areas where surface water does gather, to the surface drainage system necessary for any new project.

(iv) Borrow trenches along the edges of canals should be avoided if possible and the fill required to form the canal embankments borrowed from elsewhere.

(v) There are certain areas where seepage rates are far in excess of the measured value on the Asayle canal. This has been observed in the storage reservoir on the Dhamme Yaasiin next to the Shalambood road where the water level fell by over 1 m in about six weeks (see Section 1.4) and is also implied by the bulge of better quality groundwater in the region of Golweyn. This phenomenon is thought to occur because of the occasional direct linkage of surface waters in canals or reservoirs to the main groundwater aquifer, both through sand lenses that are encountered and the coarse alluvial deposits in sub-surface river channel remnants. Therefore, any new canals and indeed reservoirs, must be preceded by detailed augering and infiltration testing to find out if any problems are likely to arise. If this proves to be the case, it may be necessary to line the canal or reservoir over the section involved with the readily available clay soils of the Study Area.

The clays used for linings should be selected so that they are within the limits recommended by the USBR (USBR 1976). These recommendations cover various types of soils but the one most appropriate to the area is the CH group which consists of inorganic clays of high plasticity or fat clays. Most of the Vertisols in the Study Area should come within this classification. The problem associated with these soils is high volume changes between the wet and dry states. In the Study Area these soils generally form impermeable canals and the high volume change should not be a major obstacle to their use in the very few places where lining may be required. It is recommended that the linings are not less than 0.6 m thick.

## 2.7 Canal Hydraulic Design

The hydraulic design of earth channels should be based upon the Lacey regime equations as long as adequate slope is available. These expressions should ensure that neither deposition of silt nor scour occur in the canals when operating under design conditions. The symbols used in the equations are as follows:-

| | | |
|---|---|---|
| $f$ | - | Lacey silt factors |
| $e$ | - | Lacey width factor |
| $B$ | - | bed width (m) |
| $A$ | - | cross-section area of flow ($m^2$) |
| $p$ | - | wetted perimeter (m) |
| $W_s$ | - | water surface width (m) |
| $D$ | - | depth to bed (m) |
| $D_m$ | - | mean depth of flow = $A/W_s$ (m) |
| $Q$ | - | discharge ($m^3/s$) |
| $Q_L$ | - | seepage loss ($m^3/s$) |
| $Qp$ | - | peak (maximum) discharge ($m^3/s$) |
| $E$ | - | shape factor = $P/W_s$ |
| $S$ | - | water surface slope |
| $L$ | - | length of reach (km) |
| $F_b$ | - | freeboard (canal bank top level above normal water level) (m) |
| $R_m$ | - | minimum channel radius (m) |
| $V$ | - | mean velocity (m) |

The relevant regime equations and associated design equations are:-

$$W_s = 4.83 \,.\, e \,.\, Q^{0.5}$$

$$D_m = 2.46 \,.\, V^2/I$$

$$S = 0.00030 \,.\, e^{0.333} \,.\, I^{1.667} \,.\, D/Q^{0.167}$$

$$B = 0.8 \,.\, W_s \qquad (B \nless 1.0)$$

$$F_b = 0.20 + 0.227 \,.\, Q^{0.333} \; (F_b \nless 0.4)$$

$$R_m = 128 \,.\, e \,.\, Q^{0.5} \quad (R_m \nless 90)$$

$$Q_L = 0.012 \,.\, L \,.\, Q$$

The following notes are applicable to the equations:-

(i) The width factor, e, depends upon the strength of the canal banks to resist erosion. Its value ranges from 0.75 for stiff clay-loam soils to 1.00 for fine friable soils.

(ii) The silt factor, f, can vary between certain limits without changing the effectiveness of the regime equations. Experience has shown that a range from 0.6 (tending towards siltation) to 1.0 (tending towards scour) is acceptable. The correct value of the silt factor is vital with the heavy silt load of the river water, to ensure that the silt is flushed through the canal and not deposited. Analysis of the Primo Secondario discharges (Table 1.4), which range from 4.64 to 6.98 $m^3/s$, and the measured water slope of 16.2 cm/km for the upper 13.3 km of canal, produces a silt factor which varies between 0.74 and 0.82 depending on the discharge and whether a width factor of 0.83 or 1.00 is assumed (based on canal cross-section on Drawing No. 45701 - 13). The success of this canal to keep itself free from silt indicates that a silt factor of 0.8 should be sufficient in the Study Area.

(iii) The channels have a trapezoidal section with a maximum side slope of one in two. However, to limit canal seepage, a maximum gradient of one in seven is allowed between the canal normal water level and the outer toe of the embankments.

(iv) The peak discharge in a canal allows for short period demands in excess of the normal design discharge. This corresponds to a peak depth of flow equal to 1.087 times the normal depth.

(v) The losses due to seepage ($Q_L$) are significantly greater than if the seepage loss of 0.474 $m^3/s$ per million square metres of wetted area calculated from the tank test on the Asayle canal is used. This can be converted to produce an equation for the seepage loss of:-

$$Q_L = 0.0022 \, . \, L \, . \, Q^{\frac{1}{2}}$$

The larger coefficient of 0.012 has been used to compensate for areas of more rapid losses that will occur.

(vi) Canal water levels can be found by applying a minimum command necessary at all field outlets. Table 2.3 lists this against the distance upstream of the next cross regulator.

TABLE 2.3

**Minimum Commands of Field Outlet**

| Distance upstream of cross regulator (m) | Minimum command (m) |
|---|---|
| 0 - 50 | 0.40 |
| 50 - 350 | 0.45 |
| 350 - 700 | 0.50 |
| 700 - 1000 | 0.55 |

Figures 2.1 and 2.2 are canal design charts based on the Lacey equations for e = 0.83 and e = 1.00.

## 2.8 Night Storage

To achieve the optimum design of any main canalisation system, and to permit the maximum abstraction from the available water in the river, the design should proceed on a continuous flow basis (i.e. flow for 24 hours a day). However, irrigation during the night is not generally practised in Somalia and considerable social pressures would be imposed by its action. Therefore daytime irrigation only has been adopted for the Qoryooley project and is thought to be applicable to any new surface irrigation projects in the Study Area except for those using large basins.

A method is required therefore to store the over-night flow from the main canal so that it can be released, together with the normal continuous flow, the next day. Two methods are considered below.

### 2.8.1 Farm Offstream Storage Reservoirs

Figure 2.3 is an outline diagram of how this system operates. During the night, control gate 2 is closed, together with all the distributary outlets. The continuous flow from the main canal, regulated by the movable weir, passes into the reservoir to be stored. In the morning the distributary outlets and control gate 2 are opened and the continuous flow from the main canal together with a controlled release from the reservoir pass down the secondary canal and into the watercourses. As the level in the reservoir falls during the day the head across control gate 1 also drops and therefore it may be necessary to adjust the gate once or twice to maintain a nearly constant discharge from the reservoir.

The required area of night storage reservoir ($A_r m_2$) is then given by the expressions:-

$$Ar = \frac{Iq \;.\; A \;.\; (24-h) \;.\; 3.6}{x}$$

where Ig = the continuous gross irrigation requirement in litres per second per hectare including losses for deep percolation, seepage from watercourses and surface run-off. Distributary losses can be included if significant.

A = the net cultivated area served by the reservoir, in hectares.

h = the length of the irrigation day, in hours.

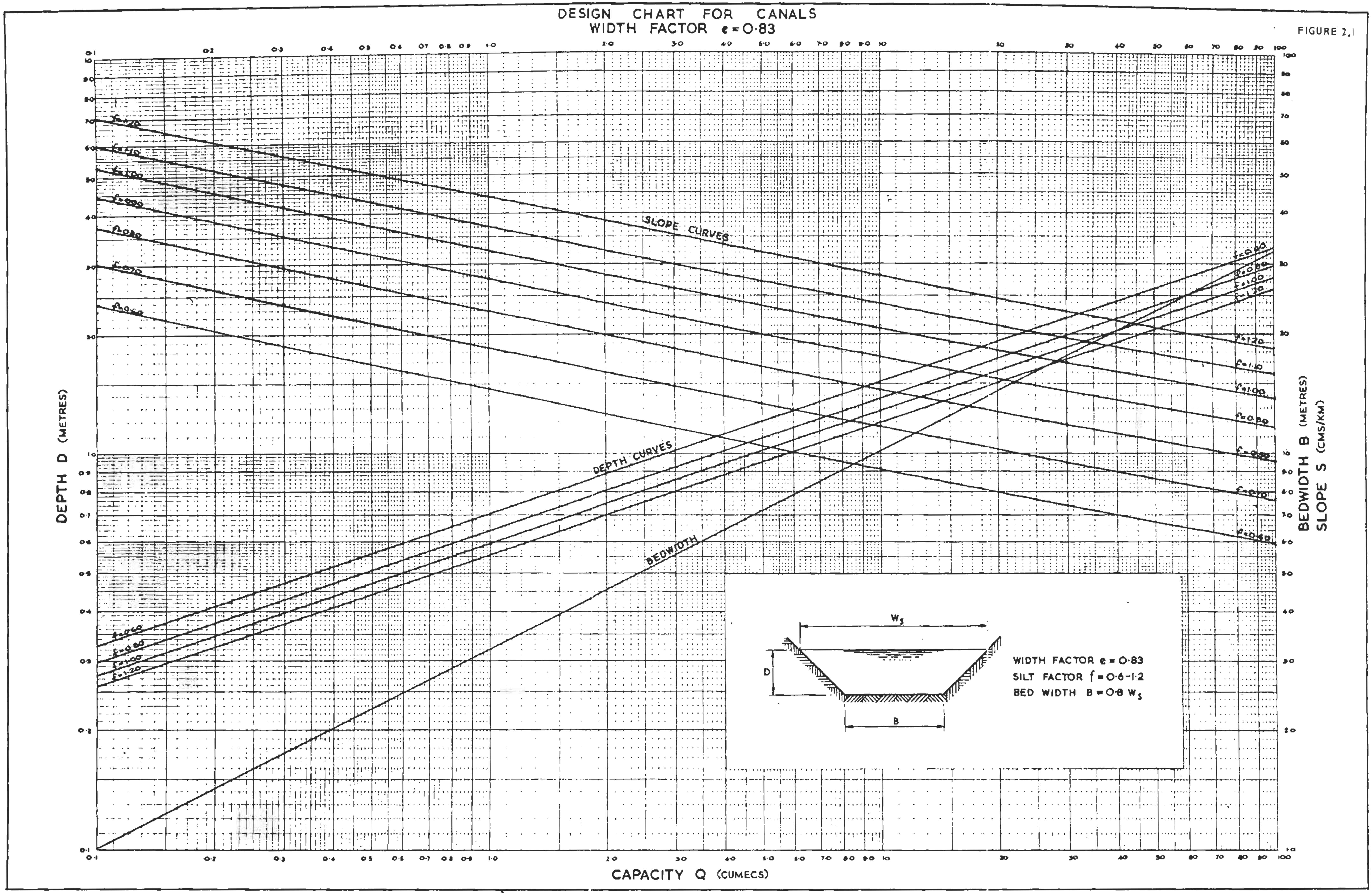
x = the difference in height (m) between the maximum storage level and the minimum storage level in the reservoir (i.e. the live depth). Because of the difficulties of operating control gate 1 to maintain a constant discharge if x is large, its value is limited to an upper ceiling of about 0.50 m. This also reduces the pumping head. This means that the reservoir area will be at least 1% of the net cultivated area it serves.

### 2.8.2 Onstream Distributary Canal Storage

This simple method eliminates the reservoir and both the control gates. It relies solely upon the extra width and water depth available in an oversize distributary canal to store the night time flow from the main canal. The only operation required is to close the distributary outlets at dusk and to open them again at dawn, the stored volume being accommodated in a rise in water level.

For a distributary canal having no cross regulators, with a trapezoidal cross-section of one in two side slopes, and assuming that the night time water surface is horizontal, the governing storage equation is:-

FIGURE 2.1



DESIGN CHART FOR CANALS
WIDTH FACTOR e = 0·83
DEPTH D (METRES)
BEDWIDTH B (METRES)
SLOPE S (CMS/KM)
CAPACITY Q (CUMECS)
SLOPE CURVES
DEPTH CURVES
BEDWIDTH
f = 1·20
f = 1·10
f = 1·00
f = 0·90
f = 0·80
f = 0·70
f = 0·60
$W_S$
D
B
WIDTH FACTOR e = 0·83
SILT FACTOR f = 0·6–1·2
BED WIDTH B = 0·8 $W_S$

FIGURE 2.2

DESIGN CHART FOR CANALS
WIDTH FACTOR $e = 1{\cdot}00$

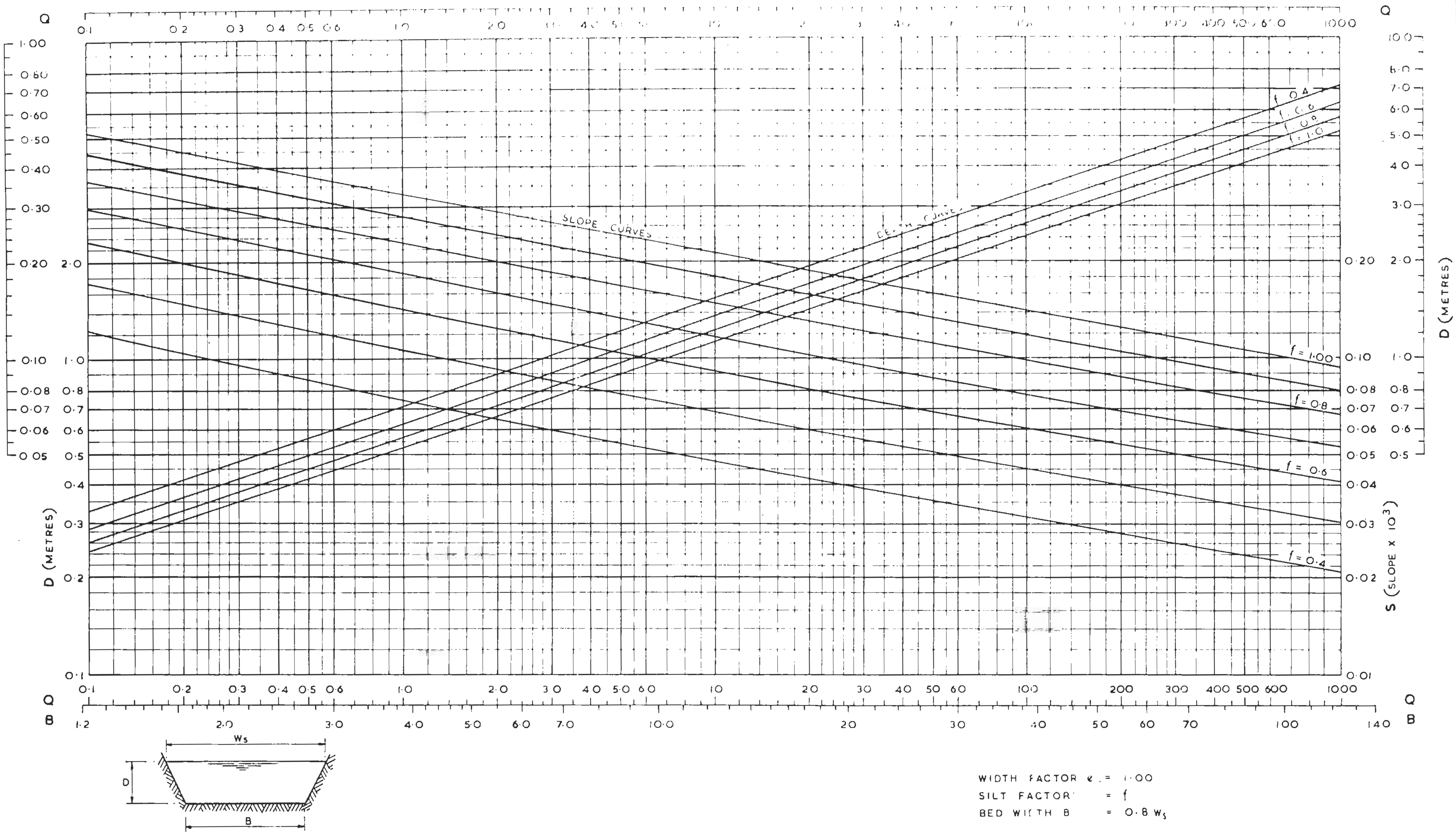
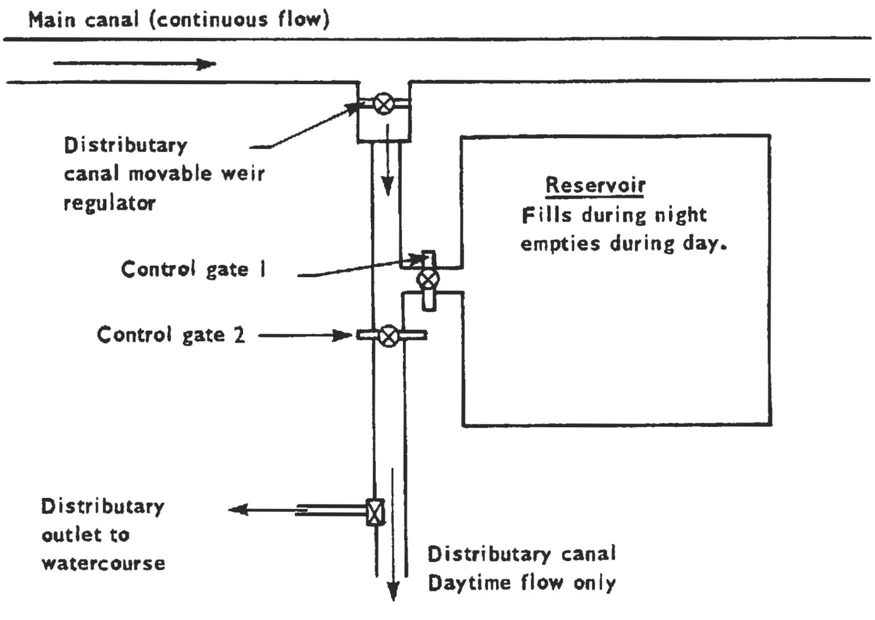


FIGURE 2.3

SCHEMATIC DIAGRAM OF A FARM OFFSTREAM NIGHT STORAGE RESERVOIR



$$I_g \;.\; A \;.\; (24-h) \;.\; 3.6 \;=\; \quad . \; L \; + \; . \; L^2 \; + \; \frac{2S^2L^3}{3}$$

where $\quad = \quad Bd + Bx + 2d^2 + 2x^2 + 4dx$

$\quad = \quad \frac{BS}{2} + 2xS + 2dS$

B = the distributary bed width in metres, assumed to be constant.

d = the required depth of flow (m) in the distributary to pass the continuous discharge (1 000 . $I_g$.A $m^3/s$) fed from the main canal down it.

S = the bed slope of the distributary, assumed to be constant.

L = the length in metres of the distributary.

x = the rise in water level (m) allowed at the head during the night. At the tail, therefore, the increase in water level will be ( x + S.L) metres. As with the night storage reservoir, x will have a definite upper limit.

The governing equation can be solved by establishing x at the maximum acceptable value and solving by iteration for B and d. This requires the use of Manning's equation to provide the additional link between B and d.

Although this method is simpler to operate and less expensive to construct and maintain than storage reservoirs, there are certain limitations that must be observed:-

(i) The land slopes must not be excessive, otherwise the bed slopes of the canals will be large. This will make the variation in water level at the tail (x + S.L) unacceptably large, causing major problems of water control through the distributary outlets at the tail. Another solution is the introduction of special cross regulators.

(ii) Because of the oversize channel section, flow velocities will be low, allowing much of the silt load in the water to settle. Any silt deposited will immediately have a detrimental effect as the distributary is one of the main components of the water distribution system. Therefore it is essential that very effective sedimentation of the silty river water is provided before the use of distributary night storage can be considered.

## 2.9 Sedimentation

### 2.9.1 The Problem

Throughout the Study Area the heavy silt load in the irrigation water from the river causes major problems; all the existing main canals require clearance at least once every year. With some of the poorer, oversized canals, clearance should really be carried out about every two months, a stringent requirement which cannot be met by the equipment and manpower available. Consequently these canals are in poor condition with their cross-sections and water control equipment heavily silted up. This enables weed growth to establish a permanent base which is difficult to remove and does not allow the canal to function properly.

With any new development projects that involve the use of night storage facilities where water velocities are very low, any significant silt load in the water supplies will settle, quickly silting up the channel or reservoir. It is essential, therefore, that silt exclusion facilities are provided to prevent as much as possible of the load reaching the night storage areas. The suspended nature of the heavy silt load is such that silt exclusion structures are not suited to this type of problem and therefore sedimentation in channels or reservoirs is necessary. The silt load dropped in these should be very heavy, requiring continuous, or nearly continuous, removal. To facilitate this the use of sedimentation channels or basins is recommended, rather than reservoirs, where the complete clearance process can be achieved by a rope operated dragline standing on the channel banks.

The actual level of suspended sediment in the river is variable and changes not only with discharge and the seasons, but also with localised heavy rainfall. In addition the deterioration of the grazing lands in the upper catchment of the Shabeelle has led to increased rates of erosion and a build-up in sediment loads in recent years. There is no evidence to suggest whether or not the deterioration will continue and levels of sediment get even higher.

Apart from one sample taken on 11th April 1968 (HTS Ltd. 1969) at Qoryooley, the Consultant is unaware of any other sediment samples having been taken in the Study Area before the present study period. During this period, weekly samples were taken between 19th July 1977 and 5th February 1978 at a point 9.4 km downstream of Janaale barrage. The full results are given in Annex II. These show a reasonably stable level through most of the der season of 1977 and an average level of 3 000 parts per million by weight is thought to be representative of that der season.

The particle size distribution of the suspended load is summarised for the Study Area in Figure 2.4.

### 2.9.2 Sedimentation Basin Design

A simple analysis of an ideal settling tank, where each discrete particle settles at its terminal velocity as determined by Stokes Law, whilst being passed along the tank at the average flow velocity, can immediately show that the amount of sediment settled for a given discharge is solely dependent upon the surface area of the basin. Several methods are available (Fair and Geyer, 1954; ASCE, 1975) to make use of this fact to produce a method of basin design. The simple equation below is a rearrangement of the expression used in American practise:-

$$\frac{Q}{A} = \frac{w}{\ln (Wo/W)}$$

where Q = the basin discharge ($m^3/s$)
A = the basin surface area ($m^2$)
w = the settling velocity of a particle (m/s)
W = the weight of sediment not trapped
Wo = the weight of sediment entering the basin

FIGURE 2.4

## PARTICLE SIZE DISTRIBUTIONS OF SUSPENDED SEDIMENT

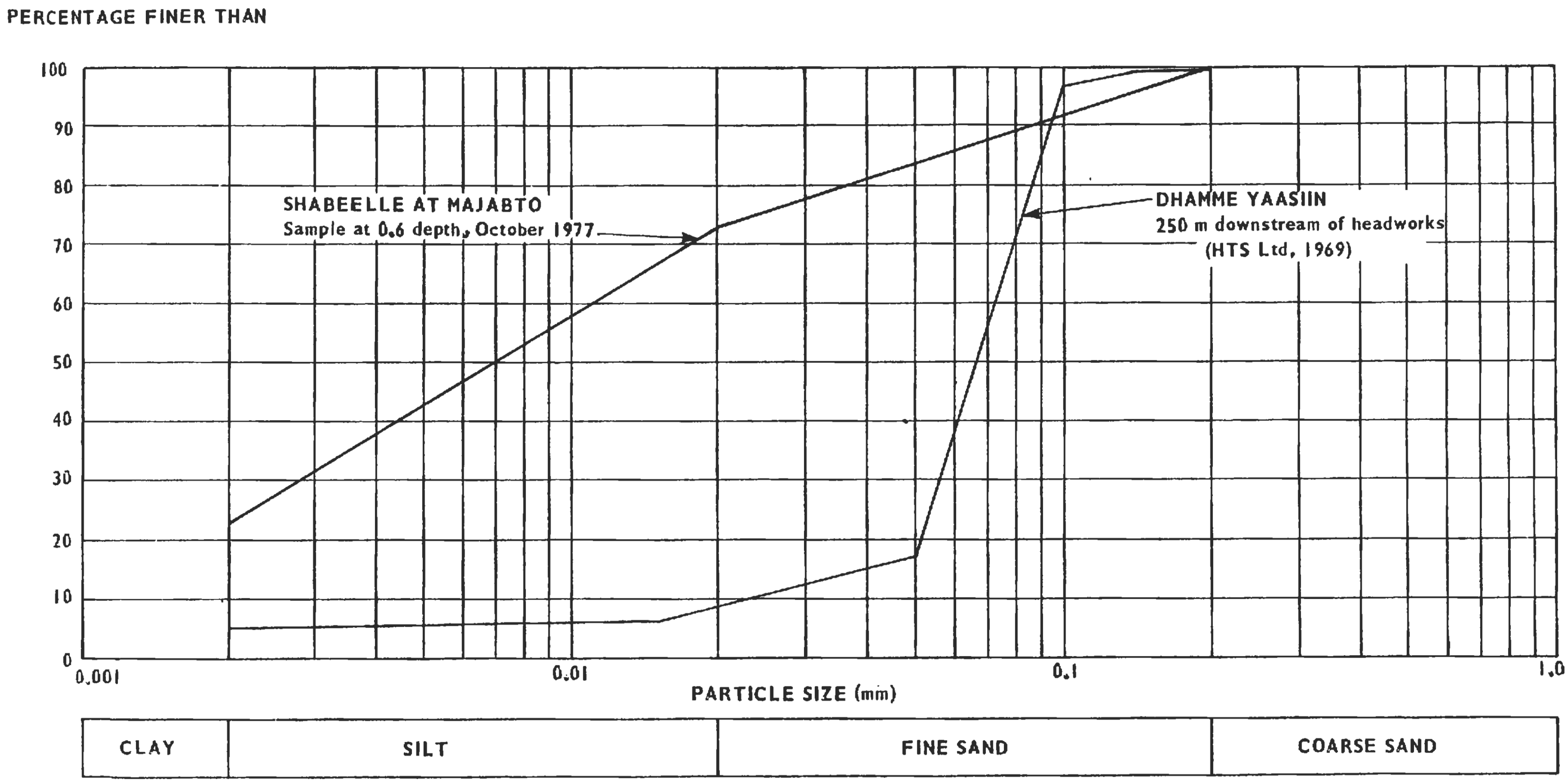


Table 2.4 has been developed from the equation to give an indication of the required areas of sedimentation basins needed in the Study Area, based on a settling velocity of 2.29 mm/s; this is equivalent to a discrete particle diameter of 0.05 mm, which is approximately the mean particle size indicated on Figure 2.4 for the canal sample. The table depicts the ideal situation and in reality short circuiting, non-uniform stream velocities and lower effective settling velocities caused by turbulence will reduce the efficiency of the basin. However, the exact extent of the reduction is difficult to predict and therefore the actual efficiency of a basin can only be estimated at the design stage.

## TABLE 2.4

**Required Surface Area ($m^2$) of Sedimentation Basins**

| Percentage removal | Discharge ($m^3/s$) | | | | | |
|---|---|---|---|---|---|---|
| | 2.0 | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 |
| 60 | 800 | 1 600 | 2 400 | 3 200 | 4 000 | 4 800 |
| 70 | 1 052 | 2 104 | 3 156 | 4 208 | 5 260 | 6 312 |
| 80 | 1 406 | 2 812 | 4 218 | 5 624 | 7 030 | 8 436 |
| 90 | 2 010 | 4 020 | 6 030 | 8 040 | 10 050 | 12 060 |
| 95 | 2 616 | 5 232 | 7 848 | 10 464 | 13 080 | 15 696 |
| 99 | 4 022 | 8 044 | 12 066 | 16 088 | 20 110 | 24 132 |

Note: Based on ASCE method with a settling velocity of 2.29 mm/s

In addition to the basin surface area, two other design criteria should be met. Firstly, the depth of flow should be great enough to keep the flow velocities below that at which scour of the settled sediment occurs. An expression for this, based upon the method of tractive force, is:-

$$V_{max} = \frac{R}{n}^{1/6} \left(0.05 \, . \, (SG - 1) \, . \, d\right)^{1/2}$$

where R = the hydraulic radius of the section (m)
n = Manning's roughness coefficient
SG = the specific gravity of the sediment particles
d = the particle size (m)

Secondly, sufficient storage depth must be available below the design bed level for the collection of the settled sediment before it can be removed by dragline. The average volume of sediment brought into a basin each day may be estimated as:-

$$Vol = \frac{Q . X . 10^{-6} . 86\,400}{SG . (1-e)}$$

where X = the sediment concentration (ppm)
e = the porosity of the settled sediment

Assuming that SG is 2.4 and e is 0.33 this equation is simplified to:-

$$Vol = 0.054 \; . \; Q \; . \; X$$

As an example the inlet channel to the Qoryooley project may be considered. In this case $Q = 5.8\ m^3/s$ (see Section 6.3 below) and if X is taken as 3 000 ppm, then $V = 940\ m^3/d$. However, not all the sediment will be settled in a basin and if an efficiency of 80% is as taken and a length of 800 m and bed width of 20 m the average depth of sediment deposited will be approximately 50 mm/d.

# CHAPTER 3

# BARRAGES

## 3.1 General Description

At the moment there are three barrages on the river within the Study Area and a fourth is being constructed near Gayweerow bridge as part of the Ministry of Agriculture grapefruit project. The basic data concerning the effect of these four barrages on the river are given in Table 3.1.

**TABLE 3.1**

**Retention Levels at River Barrages**

| | Name of barrage | | | |
|---|---|---|---|---|
| | Janaale | Gayweerow | Qoryooley | Falkeerow |
| Distance downstream of Janaale (km) | 0 | 15.7 | 26.3 | 39.7 |
| Normal retention level (ASL) | 71.17 | 69.11 | 67.16 | 66.11 |
| Level of gauge zero (upstream) | 66.87 | - | 62.96 | 62.11 |
| Gauge height (m) | 5 | - | 5 | 4 |
| Level of gauge zero (downstream) | - | - | - | 61.54 |
| Gauge height (m) | 5 | - | - | 5 |
| Maximum normal fall through structure (m) | 2.06 | 1.95 | 1.05 | - |

Notes:- 1. All levels are subject to minor corrections.

2. Existing gauges at Qoryooley and Falkeerow have sections missing.

3. Before the completion of Gayweerow barrage the maxiumum normal fall through Janaale barrage will be 4.01 m.

## 3.2 Janaale Barrage

Janaale barrage was commissioned in 1927 and regulates the river level at Janaale so that it is possible to divert water into the Dhamme Yaasiin and Primo Secondario canals on the left bank and the Asayle canal on the right bank. The barrage also has a major influence on the Giddu and Sigaale canals although both are some way upstream on the right bank (1.6 and 8.4 km respectively).

Water for almost 60% of the area currently irrigated within the Study Area is abstracted from the river at Janaale. The barrage is also important for the reduction of flooding downstream since, at times of flood, as much water as possible is diverted into the Dhamme Yaasiin and Primo Secondario canals whence it eventually flows to Shingaane basin and from there into the Gofca channel.

No drawings of the barrage were available and all descriptions hereafter are based on observations taken during the study period.

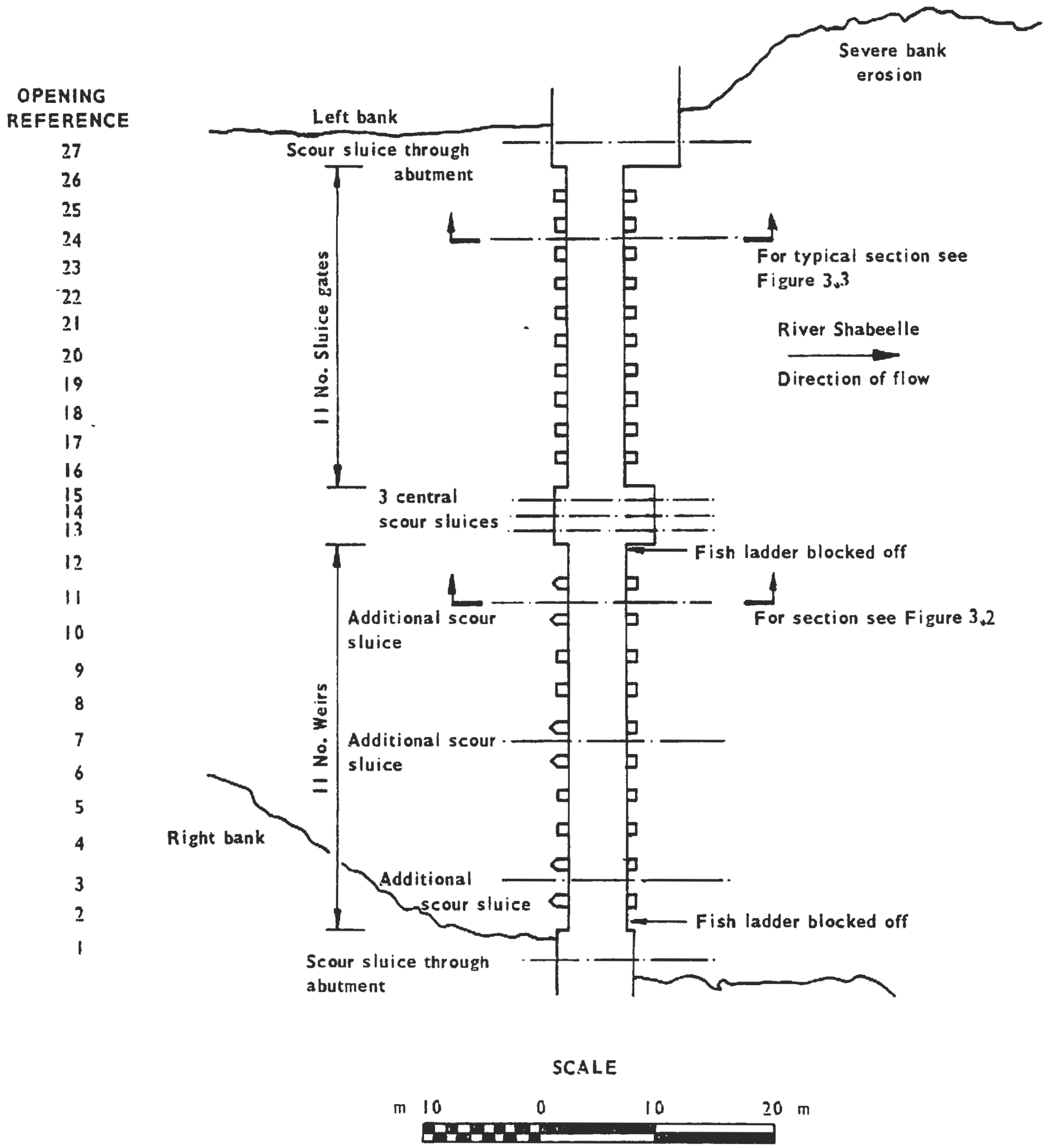
The barrage consists of three main parts and a sketch plan is shown in Figure 3.1. There are 11 weirs on the right hand side, each about 2.65 m wide and 11 sluice gates on the left hand side, each 2.07 m wide. There are also eight low level scour sluices. A summary of all openings is given in Table 3.2.

The right bank section consists of 11 weirs (reference 2 to 12) but weirs reference 3, 7 and 11 have scour sluices beneath them and these should be controlled by gates upstream. A sketch of a cross-section through weir 11 is shown on Figure 3.2. There is a single carriageway bridge deck spanning across the openings.

This carriageway is almost 5 m wide overall but there is a long continuous opening along the bridgedeck which is to permit stop logs to be lowered onto the top of each weir crest. This groove is partly filled by lengths of old rail lines which are dropped into position when stop logs are not being raised or lowered. However, the presence of this continuous groove effectively reduces the width of the carriageway to about 3.2 m for vehicles.

FIGURE 3.1

## LAYOUT OF JANAALE BARRAGE



CROSS SECTION THROUGH WEIR II AT JANAALE BARRAGE

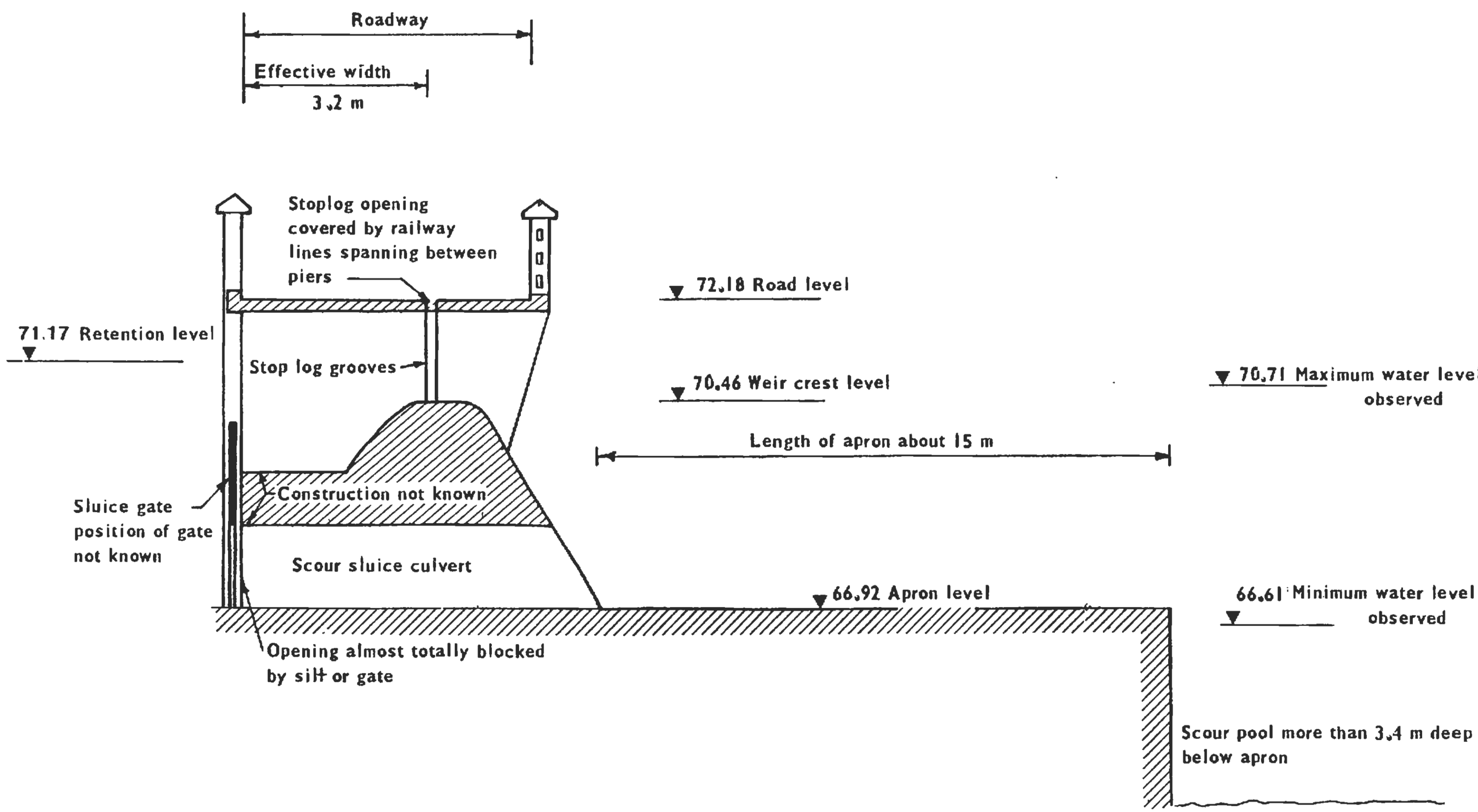



FIGURE 3.2

TABLE 3.2

**Janaale Barrage - Summary of Weirs and Sluices**

| Opening reference | Type of structure | Clear span (m) | Type of gearing | Remarks |
|---|---|---|---|---|
| 1 | Scour sluice | - | Bevel | Pipe through right hand abutment completely blocked |
| 2 | Weir | 2.65 | | Contains fish ladder |
| 3 | Weir plus scour sluice | 2.65 | Double bevel | |
| 4 | Weir | 2.63 | | |
| 5 | Weir | 2.66 | | |
| 6 | Weir | 2.65 | | |
| 7 | Weir plus scour sluice | 2.65 | Double bevel | |
| 8 | Weir | 2.66 | | |
| 9 | Weir | 2.63 | | |
| 10 | Weir | 2.56 | | |
| 11 | Weir plus scour sluice | 2.65 | | Gearing missing |
| 12 | Weir | 2.65 | | Contains fish ladder |
| 13 | Scour sluice | | | |
| 14 | Scour sluice | | | |
| 15 | Scour sluice | | | |
| 16 | Sluice | 1.86 | Worm | |
| 17 | Sluice | 2.07 | Worm | |
| 18 | Sluice | 2.07 | Worm | Gate missing but gear in situ |
| 19 | Sluice | 2.07 | Worm | Gate and gear missing |
| 20 | Sluice | 2.07 | Worm | Gate and gear missing |
| 21 | Sluice | 2.07 | Worm | Gate and gear missing |
| 22 | Sluice | 2.07 | Worm | Gate and gear missing, concrete breast beam damaged |
| 23 | Sluice | 2.07 | Worm | |
| 24 | Sluice | 2.07 | Worm | Gate and gear missing |
| 25 | Sluice | 2.07 | Worm | |
| 26 | Sluice | 2.07 | Worm | Gate and gear missing |
| 27 | Scour sluice | | | Pipe through left hand abutment completely blocked |

The central section of the barrage projects some 2.5 m downstream of the section on either side. In plan, it is a box-like structure with three gates in the upstream face and three outlet pipes in the downstream face. At the moment the centre is filled with rubbish and it is not possible to say whether there were *continuous* pipes from the sluice gate to the outlet pipe or whether the whole box was intended as an energy dissipation chamber. At the moment there is little flow through this structure even at times when the headloss across the barrage is near the maximum of about 4.0 m.

The left bank section consists of 11 vertical sluice gates with rising spindles which are controlled by worm and pinion gearing but only four gates are complete and able to function. The gates are all 2.07 m wide (clear span) with the exception of gate 16 which is slightly narrower. A section through one of these gates is shown in Figure 3.3.

There is no evidence of any major works immediately upstream of the barrage but the river has silted up and it is probable that all the scour sluices are blocked. Downstream an almost horizontal apron extends for about 15 m from the base of the broad-crested weirs.

Apart from the stop log grooves above the broad-crested weirs there are no other stop log grooves or any provisions for temporary works. At the moment, at periods of very low flow, it is possible to gain full access to the downstream side of most gates and weirs, excluding the scour gates, but this will change once Gayweerow barrage has been commissioned. The upstream sides of gates are about 3.5 m below upstream water level unless all the scour sluices are opened and passing the river flow without considerable headloss. This means that any work on the gates and seals requires a coffer dam upstream. In order to minimise the temporary work, maintenance should be done in February and early March to coincide with the period of lowest riverflow.

In order to assist in the operation of the barrage, openings 2, 5 and 12 have had blockwork walls built in the position of the stop log grooves; the top of these walls are just below normal retention level so that generally water passes over them in reduced quantity. Gates 3 and 7 have walls built across them upstream of the stop log position but again water is allowed to pass over these walls when the maximum retention level is being maintained.

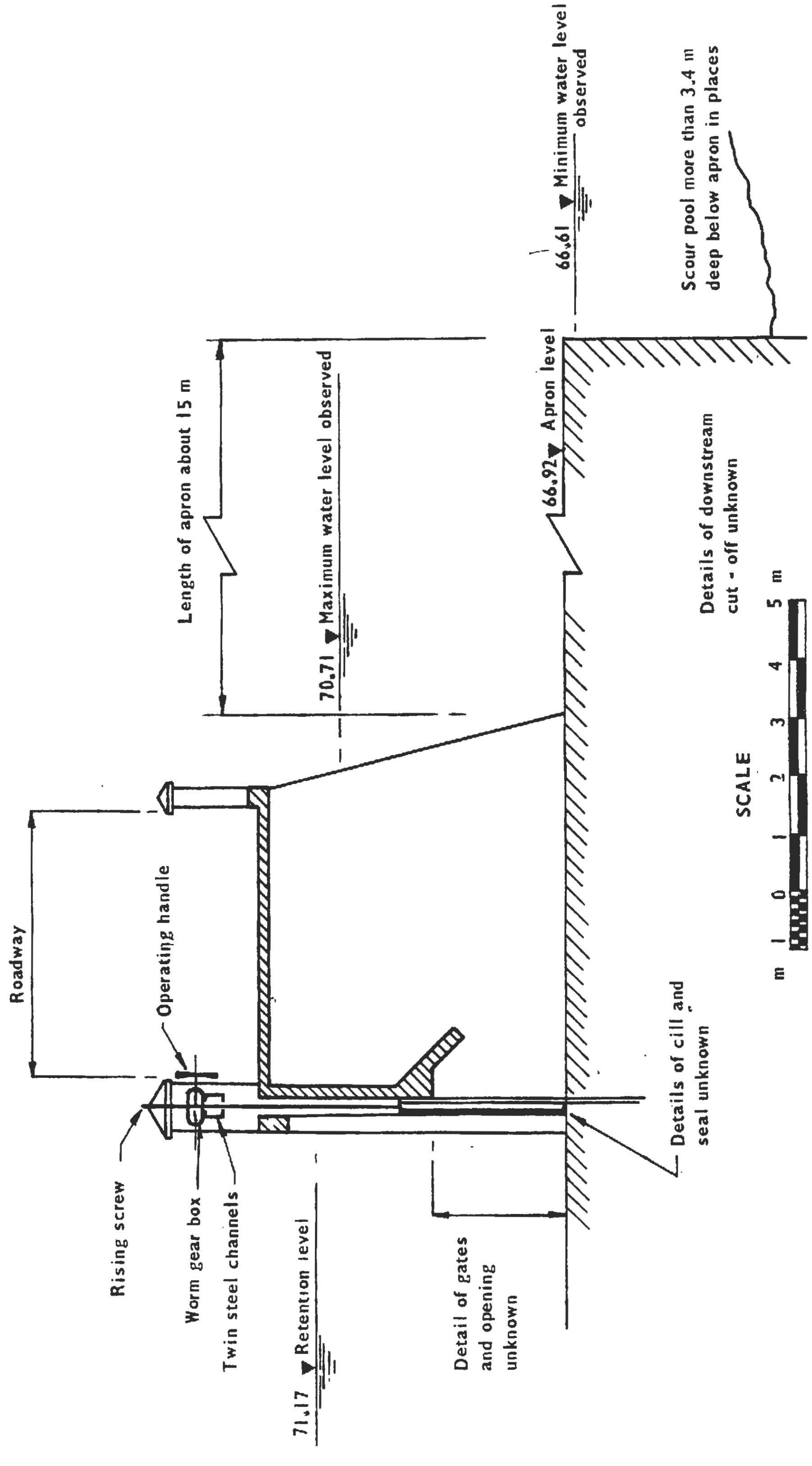
Janaale barrage is so complex that flow gauging by measuring upstream levels and gate openings is impractical. A downstream gauge has been installed as part of the project and this could be calibrated by current metering although the characteristic will change when Gayweerow barrage is built.

### 3.2.1 Condition of Structure

The concrete on the left hand section of the barrage is in better condition than that on the right hand or centre sections, so that it appears that it could have been built much later than the other sections. Inspection was difficult because of the large amount of dust and rubbish present, much of which had been cemented in position by natural agents. Before a thorough examination is made, cleaning work needs to be carried out. Nonetheless it can be said that the state of the structural concrete of the barrage itself does not give cause for immediate concern although renovation of downstream balustrades and provision of entirely new upstream balustrades would be worthwhile, the upstream balustrades are required to protect the gate operating gear.

FIGURE 3.3

# CROSS SECTION THROUGH LEFT HAND SLUICE GATES AT JANAALE BARRAGE



The main cause for concern at Janaale is the depth of the scour hole immediately downstream of the apron. The full depth was not measured but a 3.4 m long levelling staff was pushed into the water by men standing on the edge of the apron and the bottom of the staff had still not reached the bottom of the hole. This situation must be examined further, urgently.

Downstream of the abutments there has been considerable erosion on each bank. On the right bank the 'temporary' works have been moderately successful and the situation does not appear to be deteriorating but on the left bank the protection that has been provided has been almost washed away. The bank needs to be backfilled and large stone beaching laid over a sound gravel base. Some of the reinforced concrete breast beams upstream of the sluice gates have been broken and need to be replaced.

### 3.2.2 Condition of Operating Equipment

The condition of the operating gates is bad. None of the scour sluices appears to work although there is a slight trickle through scour sluices 13, 14 and 15. However, of the eleven main sluices on the left bank section, seven are out of action for various reasons. To maintain the command level for canals, inoperative sluice openings have been deliberately blocked by branches of trees, banana leaves, palm trunks and the like. In addition it is not always possible to operate the other gates efficiently because of various defects and they are often raised and lowered by disconnecting the manually operated gearing and forcing the gate up or down by means of the bucket arm of a hydraulic excavator. In normal times such a system cannot be considered satisfactory but in flood times the present method of operation is hazardous and may lead to severe flooding upstream. All gates and gear should be made good as quickly as possible. In the course of the study period the rate of deterioration was rapid, and works which were planned to be executed in January 1978 were not completed successfully.

## 3.3 Qoryooley Barrage

The Qoryooley barrage was constructed between 1955 and 1959 and the purpose was to control the level of the river to enable water to be diverted to the Liibaan canal.

The barrage consists of nine vertical sluice gates each of which is approximately 2.6 m wide. The gate depths vary; originally they were all 4.5 m deep but over the course of years one or two sections have been taken out of the gates so some are 4.0 m or even 3.5 m deep.

The gates have long rising spindles which are controlled by hand operated worm gears, operated from a platform which is about 4 m above abutment level. This operating platform is high enough to allow the gates to be almost lifted clear of the water.

The gates are of mild steel construction running in guides formed by mild steel angles. The rising spindle is attached to the gate by a 1.8 m long square hollow section steel tube and when this is placed in compression, the top, which is not restrained, tends to move laterally. Immediately behind the gates is a single carriage roadway 2.85 m wide. This bridge is part of the tarmac road system which connects the town of Qoryooley to Shalambood and thence to Marka and Mogadishu; it also carries the local traffic from Qoryooley town to some of the chief villages in Qoryooley district, so the bridge forms an important link. Normally the bridge is adequate for its purpose and, although on occasions vehicles have had to wait two hours for herds of camels to be led across, its replacement would be a low priority.

Unfortunately, the carriageway is close to the gate support structure and there are signs of damage to the supports caused by passing vehicles.

### 3.3.1 Condition of Structure

Generally speaking the concrete elements of the barrage appear to be in good condition and no extraordinary repair or maintenance work is required.

An exception to this is the kerb on the downstream side of the carriageway. There the balustrading has been damaged by passing traffic.

In the past there have been river level gaugeboards bolted to the concrete both upstream and downstream. Some of these boards are missing and should be replaced.

The flood banks upstream appear low in places but this occurs on a number of reaches of the river, details of which are given in Annex II. Downstream there is little evidence of erosion although a detailed inspection of the downstream apron and river bed was not possible. Erosion of the downstream banks is minimal and little bank protection works are required.

### 3.3.2 Condition of the Sluice Gates

The main cause for concern at Qoryooley barrage is the condition of the gates and gate operating gear. There are many places where the gates are corroded and holes allow water to pour through. In addition, the side seals are poor and water leaks round in large quantities. The light lattice work structure supporting the gates and operating platform has been damaged by vehicles and requires reinforcement or replacement.

A description of the state of repair of individual gates is given in Table 3.3. In each case the gates run in guides formed by angles 70 mm x 70 mm in section, set so that there is 140 mm clearance between them. In most cases the left hand top section of these angles on the downstream side has been removed. This is to allow the gate to be swung into or out of the guides without dismantlino the gearing and lifting the gate clear of the structure.

TABLE 3.3

**Details of Gates at Qoryooley Barrage**

| Gate No. | Overall width of opening (m) | Remarks |
|---|---|---|
| 1 | 2.67 | Downstream balustrade missing from road |
| 2 | 2.63 | Top of left hand downstream guide missing. Gate still usable. |
| 3 | 2.66 | Top two left hand downstream guides missing therefore gate not usable and gate kept in closed or almost closed position. Upstream balustrade missing from road. |
| 4 | 2.65 | As gate No. 2 |
| 5 | 2.64 | All in order |
| 6 | 2.63 | As gate No. 2 |
| 7 | 2.66 | As gate No. 2 |
| 8 | 2.64 | As gate No. 2 |
| 9 | 2.63 | Left hand guide severely damaged by lorries, therefore the gate is kept in the almost closed position. The gearing has been removed. Gate corroded. |

Note: Gates are numbered in sequence starting on right bank.

## 3.4 Falkeerow Barrage

Of the three existing barrages in the Study Area, that near Falkeerow is in the best condition. Like Qoryooley barrage this was built between 1955 and 1959. There are slight differences from Qoryooley barrage. At Falkeerow eight of the nine gates are nominally 2.5 m effective width but the ninth, gate number 4 from the right bank, is only 0.85 m wide. In addition the gate support guides at Falkeerow are embedded in concrete. It appears that they were originally lattice steel but were concreted in at some date after the construction period. One effect of this is that, to remove the gates, they must be dismantled in situ or lifted clear of the operating platform.

The major defect at Falkeerow is the erosion on the left bank downstream of the barrage. This looks serious and the danger exists of the river causing a break in the bank, so remedial work should be done immediately. The problem of bank erosion appears to have been minimised by keeping the two extreme left hand gates permanently closed but this cannot be considered a satisfactory solution.

The other point worthy of note at Falkeerow is that there appears to have been a problem with the extreme right hand gate at some stage since the screw on the rising spindle is different from the remaining eight. The spindle is now bent, indicating that the gate cannot be moved by the operating gear. Otherwise the gates and gears appear to be adequate at Falkeerow.

A full inspection of the downstream apron was not possible but a cursory inspection indicated that major repairs had been carried out at some stage. However, a detailed inspection by a diver is still needed.

### 3.5 Repairs to Janaale Barrage

No drawings of Janaale barrage were located in Somalia and a brief attempt to locate drawings in Rome was unsuccessful. The situation is serious for two reasons. Firstly, most of the main sluice gates are inoperative and, if a sudden flood occurs, it may not be possible to pass it through the barrage and the river banks may be overtopped upstream. Secondly, the extent of the scour pool downstream must be ascertained since, if it extends beneath the apron, there is a danger that the barrage itself will collapse.

The most urgent attention should be given to the gates and all those gates on the left bank section should be restored as quickly as possible, using the facilities of ONAT at Shalambood. At the same time, an attempt should be made to check the depth of the downstream cut-off wall by probing from a boat. If the probing appears to indicate that the cut-off is deeper than the hole, then the matter may be left until early in 1980. Finally, every attempt should be made to locate the drawings of the barrage by an exhaustive search in Italy.

The main work on the barrage should be carried out early in 1980. By that time the Jowhar offstream storage reservoir should be operating and it will be possible to control most of the flows downstream of Jowhar. It is proposed that the barrage is dewatered upstream and downstream so that a thorough inspection may be made of the whole structure and the operating gear. This can be done by building an earth dam upstream between the barrage and the offtake to the Dhamme Yaasiin canal. A low bank can also be built downstream of the barrage. By positioning the first barrage downstream of the Dhamme Yaasiin offtake, it should be possible to maintain supplies to the Dhamme Yaasiin, Primo Secondario and Asayle canals. The effect on the river at Janaale would be to deprive downstream villages of their water supply for domestic purposes and for cattle. It will be possible to pass a limited quantity of water into the river from the tail of the Asayle canal but this will be negligible compared with the quantity required. There are very few banana farms which rely on the river downstream of Janaale so the effect of damming the river will not be too serious for perennial crops since most banana plantations will still be irrigated from the canals offtaking from Janaale. The control of flow in the river will be difficult and the releases from the Jowhar offstream storage reservoir should not be greater than requirements or flooding will occur since there is no emergency escape route. It will also be difficult to estimate requirements since this is not done at present and any predictions of demand will be highly theoretical. It will therefore be important to be conservative and give slightly less water than required, particularly in the early stages. It is estimated that the effect of gate opening in the outlet of the Jowhar offstream storage reservoir will take three or four days to reach Janaale so, with communication between the two points, it should be possible to avoid releasing too much water from the reservoir. Even so, the overall effect must be to reduce the volume of water available for irrigating perennial crops and it may be necessary to resort to pumping from tubewells to supplement the supply. It is clear that there will be no water available south of Janaale for annual crops during this period and it is important to warn farmers of this in advance so that they may sow early to avoid the need for irrigation in January.

Another problem may occur from an early gu flood. The river sometimes rises quickly in February and subsides before the gu flood but such floods should be controlled by the Jowhar offstream storage reservoir. On 6th March 1958 a flood of 85 $m^3/s$ was recorded at Mahaddaay Weyn, and a flood of 95 $m^3/s$ was recorded on 16th March 1968. It is doubtful whether the Jowhar offstream storage reservoir could divert sufficient water from these floods to prevent overtopping upstream of the Janaale bund. Therefore any work on Janaale barrage should be substantially completed by 6th March so that the upstream bund can be removed quickly if necessary. There will also be pressure to complete quickly from farmers downstream.

It is considered unlikely that it will be possible to redesign, fabricate and refit new gates in the time available between the formation of the bund upstream and the removal of that bund. Therefore provision should be made to close off gate openings, one at a time, to refit gates later. Temporary controls can be fitted either in the form of locally fabricated gates or some form of screen upstream.

The procedure for the repair of the barrage will therefore be as shown below:-

(a) August 1979. Inform all people dependent on river water downstream of Janaale that there will be no flow from mid-January to mid-March 1980.

(b) September 1979. Plan the operation of Jowhar offstream storage reservoir so that there will be ample capacity to take all river flows in January, February and early March 1980. Appoint hydraulic structure specialist consultants and hydraulic gate contractors and make preliminary surveys.

(c) In January 1980. Operate Jowhar offstream storage reservoir so that the discharge downstream is just sufficient to meet that required at Janaale and intermediate points. Start constructing the temporary embankment immediately upstream of Janaale barrage.

(d) When the upstream embankment has been completed, build the smaller bank downstream and dewater the structure and the downstream scour pool using pumps and discharging the water downstream.

(e) Clear all debris from the barrage and excavate all silt immediately upstream so that a thorough inspection may be made.

(f) Make a detailed survey of all gates and scour sluices, redesign all gates and cills and side seals.

(g) Design temporary measures to permit access to gate openings later.

(h) Fit as many gates and ancillary equipment as possible; where this cannot be done make temporary gates.

(j) Remove temporary embankments before the gu flood.

(k) Fit remaining gates when possible.

This programme will require to be planned carefully because of the short time available for the work of replacing the gates but there does not seem to be any alternative method. The employment of the gate suppliers should be on a cost-plus basis since it will be difficult to forecast the extent of the work involved and the contractor will be required to be capable of responding quickly to unpredictable situations. Fabrication of gates often takes nine months and this work will have to be programmed so that delivery of the permanent gates is at a time when it is possible to fit them.

After the survey of the gates and weirs has been completed, it is desirable that the operation of the gates is studied carefully for two reasons. Firstly, the gates should be operated to minimise problems caused by the settlement of sediment upstream. By controlling the settlement of sediment it may be possible to keep the entrances to the gates and weirs free from silt and thus reduce problems of blocking of the gates. The second reason for studying the operation of the gates is to measure the discharge through the barrage. The discharge through each gate will depend on the upstream level, the downstream level and the gate opening. The discharge over the weirs can be calculated by use of the appropriate formula.

Finally, consideration should be given to providing a screen to protect the gates from floating and submerged debris, since the gate openings are so small that there is a chance that they will be blocked.

## 3.6 Repairs to Qoryooley Barrage

All the gates for Qoryooley barrage should be replaced. To make a permanent replacement, including re-design of gates, gearing and gate grooves would cost some So. Shs. 2 million plus engineering costs and expenses. However, if the Faraxaane project is developed in about ten year's time the barrage will only be required to control levels to feed the Haduuman development zone which has a net cultivated area of 823 ha. The expenditure of some So. Shs. 2.5 million is difficult to justify in these circumstances. Therefore it is proposed that temporary repairs are made to the existing gates and the work has been costed accordingly.

## 3.7 Repairs to Falkeerow Barrage

Falkeerow barrage controls the discharge to the Buulo Bokore canal, amongst others, and is therefore considerably more important than Qoryooley barrage. Therefore full allowance has been made for replacing the gates and this should be done within the next five years. However, repairs are urgently required to the river banks downstream of the barrage where erosion is sufficiently severe for there to be a danger of the river breaking through and flowing down the secondary channel to Furuqley. In these circumstances it is imperative to repair the banks downstream as soon as possible.

## TABLE 3.4

### Estimated Costs of Repairs to Barrages

| | So. Shs. '000 |
|---|---|
| **Janaale** | |
| Form banks upstream and downstream | 600 |
| Survey gates | 200 |
| Survey scour pool | 100 |
| Fill scour pool | 300 |
| Supply and fit temporary gates | 200 |
| Remedial work on left bank | 100 |
| Supply and fit permanent gates | 2 500 |
| Fit screen | 500 |
| Remove temporary banks | 200 |
| Sub-total | 4 700 |
| Add 15% engineering costs | 705 |
| | 5 405 |
| Add 10% contingencies | 541 |
| TOTAL cost of repairs to Janaale barrage | 5 946 |
| **Qoryooley** | |
| Supply and fit temporary gates | 400 |
| Add 15% engineering costs | 60 |
| | 460 |
| Add 10% contingencies | 46 |
| TOTAL cost of repairs to Qoryooley barrage | 506 |
| **Falkeerow** | |
| Supply and fit gates | 2 000 |
| Remedial work to left bank downstream | 400 |
| Sub-total | 2 400 |
| Add 15% engineering costs | 360 |
| | 2 760 |
| Add 10% contingencies | 276 |
| TOTAL cost of repairs to Falkeerow | 3 036 |

## 3.8 Costing of Repairs to Barrages

The cost of the remedial works to the barrages is difficult to estimate because of the lack of information and the nature of the flows in the Shabeelle river which make it difficult to predict with confidence when periods of low flows are likely to occur. In such circumstances it is customary for work to be undertaken departmentally since labour can be employed on routine maintenance and called in to make essential repairs when conditions permit. This is obviously not convenient in the case of Janaale barrage due to the complex nature of the works, but because of the unpredictable nature of the river neither will it be possible to employ contractors on a fixed price contract. Furthermore the extent of repair work required on the scour hole is unknown. For these reasons, the estimates given in Table 3.4 should be regarded as indicative only. These estimates do not include for the simple routine maintenance jobs, except that the estimated costs of all new gates have been given even though there is a programme to replace these at the moment.

# CHAPTER 4

# DRAINAGE

## 4.1 The Need for Drainage

Field drainage can be divided into two groups:-

Surface drainage

Sub-surface or groundwater drainage.

Surface drainage provides for the disposal of excessive irrigation applications and storm run-off from the fields into the main collector system. At the present time no complete system operates in the Study Area and excess water simply gathers in low spots. These areas consequently become waterlogged, affecting the yields. A common practice, where possible, is to drain surplus water from the field into adjacent borrow pits that have been dug for road or canal construction. Semi-permanent ponds of stagnant water therefore exist in many areas, aggravating the problem of seepage from canals and presenting a serious health hazard. Certain attempts have been made by the banana plantations to provide surface drainage around the perimeter of a field so that irrigation water can be drawn off after about six hours of infiltration. However no collection system is provided and the water can only be disposed of by pumping it back into the canal system.

The severe consequences of having no surface drainage system can be appreciated from an investigation of the 1977 gu season maize crop; earlier than expected April rains meant that the complete season was started badly and by late May, due to poor irrigation supplies despite the high level of the river, the available soil moisture was severely depleted. However overcast skies hinted at heavy rains and the farmers had to delay vital irrigations because, if it had rained shortly after irrigating, the fields would have become waterlogged and the crops been lost.

Unfortunately the rain never materialised and by the time irrigation water was applied the maize had suffered severely from water stress. A comprehensive but simple surface drainage system would have eliminated the problem, as any excess water could have been drained away.

Sub-surface drainage becomes necessary wherever high groundwater tables occur. The effect of the water surface being as little as 1 or 2 m below the ground level is two fold; firstly the root zone is easily waterlogged and after each irrigation the complete depth becomes saturated. Most crops can only withstand the anaerobic conditions produced by waterlogging for a short period, after which crop failure can follow rapidly; secondly the proximity of the water table to the surface vastly reduces the rate of natural leaching (the downward movement of water through the root zone).

This results, if even only moderately saline irrigation water is being used, in the build-up of salts in the root zone. Indeed the complete process of leaching can be reversed and capillary rise of water from the groundwater occur. In some irrigated areas of the world this makes a significant contribution to the crop water requirements. However, this is only possible where fresh groundwater is available; this is not the case in the Study Area. Here any capillary rise of the saline groundwater would cause the precipitation of salts in the root zone, a situation which is to be avoided.

The use of sub-surface drainage provides the opportunity to control the level of the groundwater table thereby eliminating the risk of waterlogging and guaranteeing the downward movement of water through the root zone to control the build-up of salts.

No experience in the use of sub-surface drainage had been gained in Somalia until 1977 when two fields at the Jowhar sugar estate had field drains installed at spacings of 12.5 and 25 and 50 m, specifically to test the effectiveness of buried field drains under the soil conditions found on the estate (MMP, 1978). The major problem is the closeness of groundwater to the surface (generally less than 2 m) inhibiting natural drainage and causing waterlogging of the root zone. This has led to the abandonment of approximately 15% of the cropped area. Although the tests are, as yet, incomplete, the observations made so far are encouraging, indicating that it will be possible successfully to drain the estate.

In the Study Area no sub-surface drainage has been undertaken and as yet no major land areas have been abandoned due to waterlogging or salinity. However, under much of the intensively irrigated banana plantations, the groundwater table has risen to less than 3 m below ground level, and over significant areas to less than 2 m below ground level. The situation is likely to get progressively worse and therefore sub-surface drainage is going to become imperative for the maintenance of banana yields.

The exact rate of rise of the groundwater table is difficult to predict because of unknown amounts of horizontal outflow from the intensively irrigated areas. However, if this effect, together with the results of groundwater pumping are ignored a maximum rate of rise can be found by simply dividing the rate of deep percolation (i.e. leaching water) by the average porosity of the soil profile. Table 4.1 summarises the rate of rise by assuming that deep percolation is 10% of the gross water application. These figures compare with a maximum measured rate over 14 years of 0.84 m per year in the Study Area.

**TABLE 4.1**

**The Maximum Rate of Rise of Groundwater (m/year)**

| Average porosity % | Average gross annual water application (mm) | | | |
|---|---|---|---|---|
| | 500 | 1 000 | 1 500 | 2 000 |
| 5 | 1.00 | 2.00 | 3.00 | 4.00 |
| 10 | 0.50 | 1.00 | 1.50 | 2.00 |
| 15 | 0.33 | 0.67 | 1.00 | 1.33 |
| 20 | 0.25 | 0.50 | 0.75 | 1.00 |

Note: 10% deep percolation assumed.

## 4.2 Maximum Groundwater Levels

The maximum static level that the groundwater table can be allowed to rise to is essentially determined by the rooting depths required for good growth by mature crops (Table 4.2). Shortly after irrigation higher levels can be tolerated as long as the table quickly drops back down to the static level. The root zone has been shown to be confined between the surface and a depth approximately 0.3 m above the water table (Elliot, 1951; Luthin and Bianchi, 1954) and ideally the minimum depth to water table should be taken as the mature rooting depth plus 0.3 m. However, this requirement is considered to be rather conservative and in economic terms a depth equivalent to the mature rooting depth is likely to produce the most efficient results.

**TABLE 4.2**

**Mature Crop Rooting Depths and Minimum Depth to Groundwater**

| Crop | Mature rooting depth (m) | Minimum depth to groundwater (m) |
|---|---|---|
| Banana | 1.00 | ) |
| Maize | 0.80 | ) |
| Sesame | 0.80 | ) |
| Cotton | 1.00 | ) 1.00 |
| Rice | 0.45 | ) |
| Groundnuts | 0.60 | ) |
| Citrus | 1.50 | 1.50 |

A further constraint has to be applied because of the risk of capillary rise. With annual crops long fallow periods occur during the jilal plus, sometimes, the gu and der seasons.

If the water table is too high in these periods capillary rise will be at unacceptably high rates, depositing quantities of salt on the surface of the soil. The rate increases rapidly as the distance to groundwater decreases (Table 4.3) and by limiting it to the value of 0.5 mm/d recommended by Kessler (1973) the minimum depth to groundwater becomes 1.0 m. The actual rate of capillary rise will be less than the figures given in Table 4.3 because of the drying out of the soil surface mulch which inhibits the movement of moisture.

TABLE 4.3

Maximum Capillary Rise Rates for Clay Soils
(FAO/UNESCO 1973)

| Distance to groundwater (m) | Maxiumum rate of capillary rise (m) |
|---|---|
| 0.25 | 10 |
| 0.40 | 4 |
| 0.50 | 2.5 |
| 0.75 | 1.0 |
| 1.00 | 0.5 |
| 1.50 | 0.2 |
| 2.00 | - |

The minimum depth to groundwater for any crop will therefore be 1.00 m unless the mature rooting depth is greater than this (Table 4.2). For the crops considered in the Study Area only citrus has a rooting depth greater than 1.00 m.

## 4.3 Sub-surface Drainage Methods

Two methods are in use for draining the subsoil:-

Tubewell drainage

Buried field drains.

Tubewell drainage consists of sinking a series of deep boreholes in the area to be drained and pumping the water out in order to lower the general water table (Ridder, 1973). Drainage water would then be disposed of in a small open channel system. The feasibility of tubewell drainage depends upon certain favourable aquifer characteristics:-

(i) the transmissivity must be fairly high to allow the wells to be widely spaced

(ii) with a semiconfined aquifer with an upper layer of clay overlying the main aquifer (as is the case in the Study Area) the hydraulic conductivity of the clay must be high enough to ensure effective dewatering of the subsoil

(iii) where the main aquifer is confined and only a piezometric surface rather than a free water table exists, pumping, unless done in excessive quantities, may only reduce the pressure in the aquifer and not drain the overlying stratum at all.

The transmissivity of the main aquifer is good. Where an aquifer is covered by shallow clays, tubewell drainage may be considered. This method is only used in parts of the world where extensive investigations have been made and considerable experience gained in the local conditions. Before the possibility of tubewell drainage could be considered, detailed geophysical investigations would be required but present indications are that the clay layer is often 30 m deep with occasional sandy lenses (see Annex II, Part II). In these conditions tubewell drainage cannot be recommended.

A buried field drainage system consists of a network of underground pipes placed at an appropriate depth and spacing to control water table levels. Water enters the pipe through gaps or slots in the pipe wall, is collected in a deep channel or buried pipe collector system and conveyed to some convenient point for disposal. The field drainage normally operates by gravity and should not require pumping before reaching the main disposal point. This method of drainage is relatively simple to construct and maintain, having proved successful on many different soil types and is therefore recommended, given the successful completion of trials at Jowhar, for use on drainage schemes in the Study Area.

A choice has to be made for any buried field drain system between buried pipe collectors and deep open channel collectors. Buried pipes have a considerable advantage where land is at a premium as they take up much less space than open channels and hence provide more land for cultivation. This advantage, however, is offset by the cost of providing an inspection manhole at each field drain junction. This is necessary in order to see that the field drain is operating properly and to be able to clean it out should a blockage occur. Experience has shown that this type of system can easily fall into disuse through blockage from material entering the network through the inspection manholes and it is not always immediately apparent when and where the blockage occurs.

Open channel collector drains are generally easier to construct and the connection between the field drain and the collector is quite simple. The field drain is usually visible from the channel bank at all times and it is immediately obvious when either the field drain or collector is not functioning properly. If further field drains are required in a field they can be installed easily without the need for pipe connections and manhole construction.

As water supply rather than land is the limiting resource on agricultural development, and in view of the easier construction and maintenance of open collector drains, they are recommended for use in the Study Area. Figure 4.1 shows a typical buried field drain and open collector drain arrangement in section.

## 4.4 Open Drainage System Design

Suitably designed channels are required for a drainage system to carry the water away to the disposal point. Of prime interest, therefore is the quantity of water to be removed as the drainable surplus. Assuming deep field drains have been installed, this comprises:-

(i) seepage from main and secondary canals

(ii) field distribution losses

(iii) field irrigation losses through surface run-off and deep percolation

(iv) surplus rainfall.

These elements are described in more detail below and the results summarised in Table 4.6.

### 4.4.1 Canal Seepage

In areas of purely clay soils very low seepage rates are likely, as indicated by the result of the canal tank test on the Asayle canal of 0.474 $m^3/s$ per million square metres of wetted area (Section 2.6). However to accommodate higher rates of seepage through sandy areas the more general expression for seepage losses has been adopted:-

$$Q_L = 0.012 . L . Q^{\frac{1}{2}}$$

where $Q_L$ = seepage loss in section ($m^3/s$)
$Q$ = discharge in the section ($m^3/s$)
$L$ = length of the section (km)

Some of these seepage losses will enter the drainage system, the remainder percolating directly to groundwater.

### 4.4.2 Field Distribution Losses

The losses in the field distribution channels have been estimated to be 5 to 10% of the gross field water requirement for the type of soil encountered in the Study Area (Annex VI, Chapter 2).

### 4.4.3 Field Irrigation Losses

The major component of field losses of water occurs because of over-application and mis-management of the irrigation supplies. This wasted water has to be drawn off as surface run-off and passed into the drainage system. For design purposes this can be taken as 15 to 25% of the gross field water requirement (Annex II, Chapter 2).

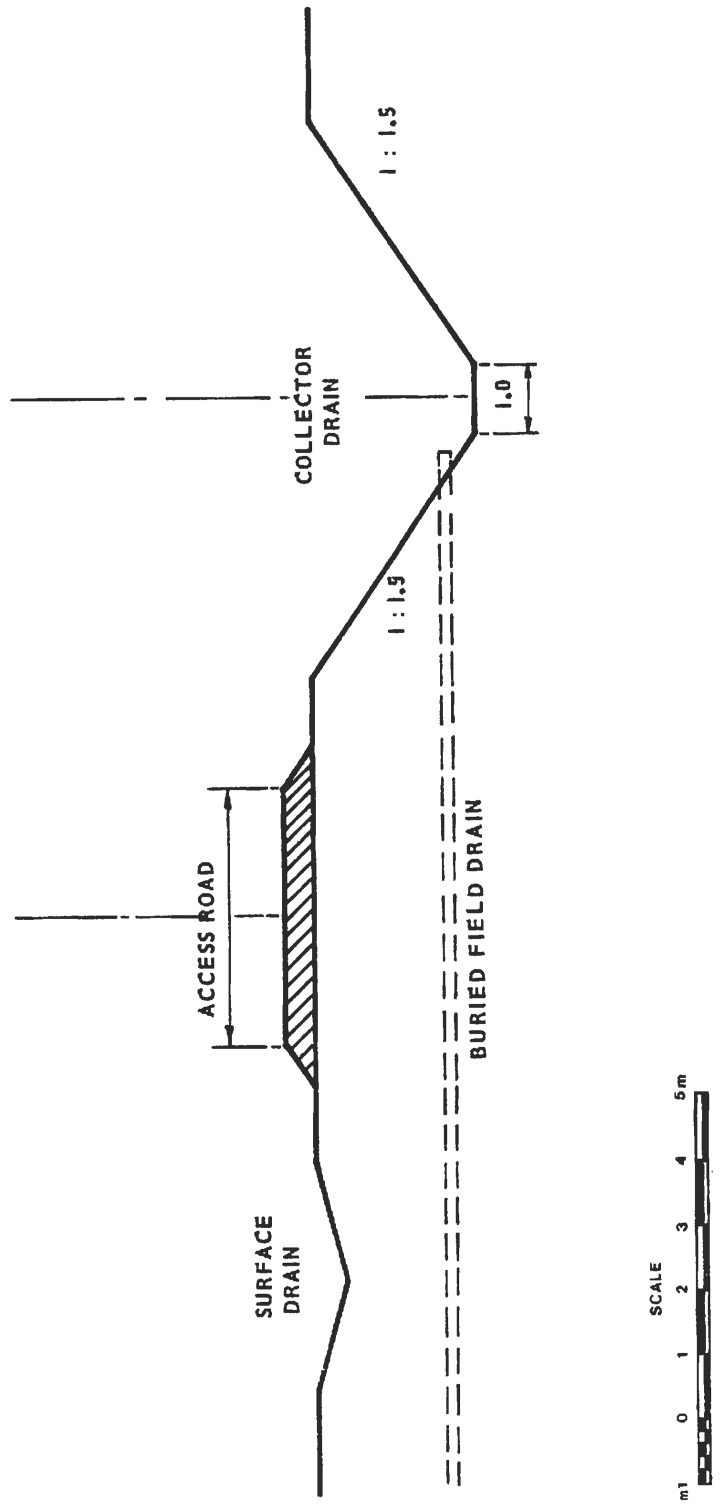
In addition the percolation losses will contribute to the drainage discharge. Because of the low hydraulic conductivity and infiltration rates of the Study Area soils, the amount of downward movement of moisture in the soil is estimated to be, at most, 10% of the gross field water requirement (Annex VI, Chapter 2).

### 4.4.4 Surplus Rainfall

The rainfall to be disposed of by the drainage system under normal conditions (storm run-off is dealt with separately in Section 4.5) is solely that portion that is not used by the crop and can be extracted from Table 2.5 in Annex VI. Total amounts over an area should be based on the gross area, unlike the field losses which apply only to the net cultivated area.

FIGURE 4.1

BURIED FIELD DRAIN AND OPEN COLLECTOR DRAIN ARRANGEMENT



### 4.4.5 Design

An open drainage system can, unlike an irrigation system, easily accommodate overloads without being damaged and will thus tend to smooth out the peaks in the discharge. Therefore certain reductions in the design discharges of outfall drains and disposal pump stations can be permitted, but each case must be assessed individually taking the local topography and crop requirements into consideration.

Open drainage channel design is based upon a trapezoidal cross-section with side slopes of 1 in 1.5 (vertical to horizontal), and generally a ratio of bed width to water depth of three to one; utilising Manning's equation gives:-

$$V = \frac{R^{2/3} \cdot S^{1/2}}{n}$$

where
- $V$ = mean velocity (m/s)
- $n$ = Manning's roughness coefficient. A value of 0.025 is normally adequate for drains
- $R$ = hydraulic radius (m)
- $S$ = water surface slope.

and by using the Lacey equation:-

$$F = 2.46 \frac{V^2}{D_m}$$

where
- $F$ = modified Froude factor
- $D_m$ = mean depth (m).

Values of F from 0.4 to 1.2 represent the permissible range of design velocities within which any bed slope can be chosen so that natural ground slopes can be followed as much as possible to avoid excessive excavation. Figure 4.2 is a design chart for open drains, based on the above expressions.

Collector drains will have a fixed bed width of 1 m, the minimum practical size, and the bed will be a minimum of 0.3 m below the field drain outlets, thus ensuring that the outlets are not normally submerged. A minimum bed slope of 0.25 m/km is required in collector drains. At the junction between collectors and main collectors or outfall drains a headloss of about 0.06 is required to pass the discharge through the junction structure; this can be achieved by setting the design water level in the main collector or outfall drain equal to the bed level of the collector.

## 4.5 Surface Drainage Design

A shallow open channel is required at the bottom of any field to collect surface water (see Figure 4.1) and convey it to the end of the field where a control culvert can pass the water into the main collector system. The channel need not be designed for a particular discharge but must form a depression with gentle enough side slopes to allow farm machinery access across it. The structure, however, requires careful consideration because it must not only throttle the release of storm run-off into the main system so that the channel and pumping capacities are not exceeded but also allow water to be drained from the field fast enough to avoid crop damage due to waterlogging.

The design discharge of a surface system may be divided into three elements i.e. drainable surplus, surface run-off of excess irrigation water and storm run-off. The drainable surplus has been considered in Section 4.4. The excess irrigation water obviously depends on the skill of the farmer but for this report a figure of 15 to 25% of the gross application at the field outlet has been taken. Storm run-off requires a special study and is considered in Section 4.5.2.

### 4.5.1 Sensitivity to Waterlogging

The effect a period of waterlogging will have on a particular crop depends largely upon its physiological response to anaerobic conditions in the soil. The detrimental effects can be shown to be caused by four factors namely oxygen deficit, carbon dioxide excess, anaerobic micro-organisms affecting nutrient availability, and the production of toxic ethylenes. The sensitivity of different crops to each of these factors will vary, making practical analysis of the problem extremely difficult. This is reflected in the nearly complete absence of quantitative data on crop sensitivity to waterlogging.

**TABLE 4.4**

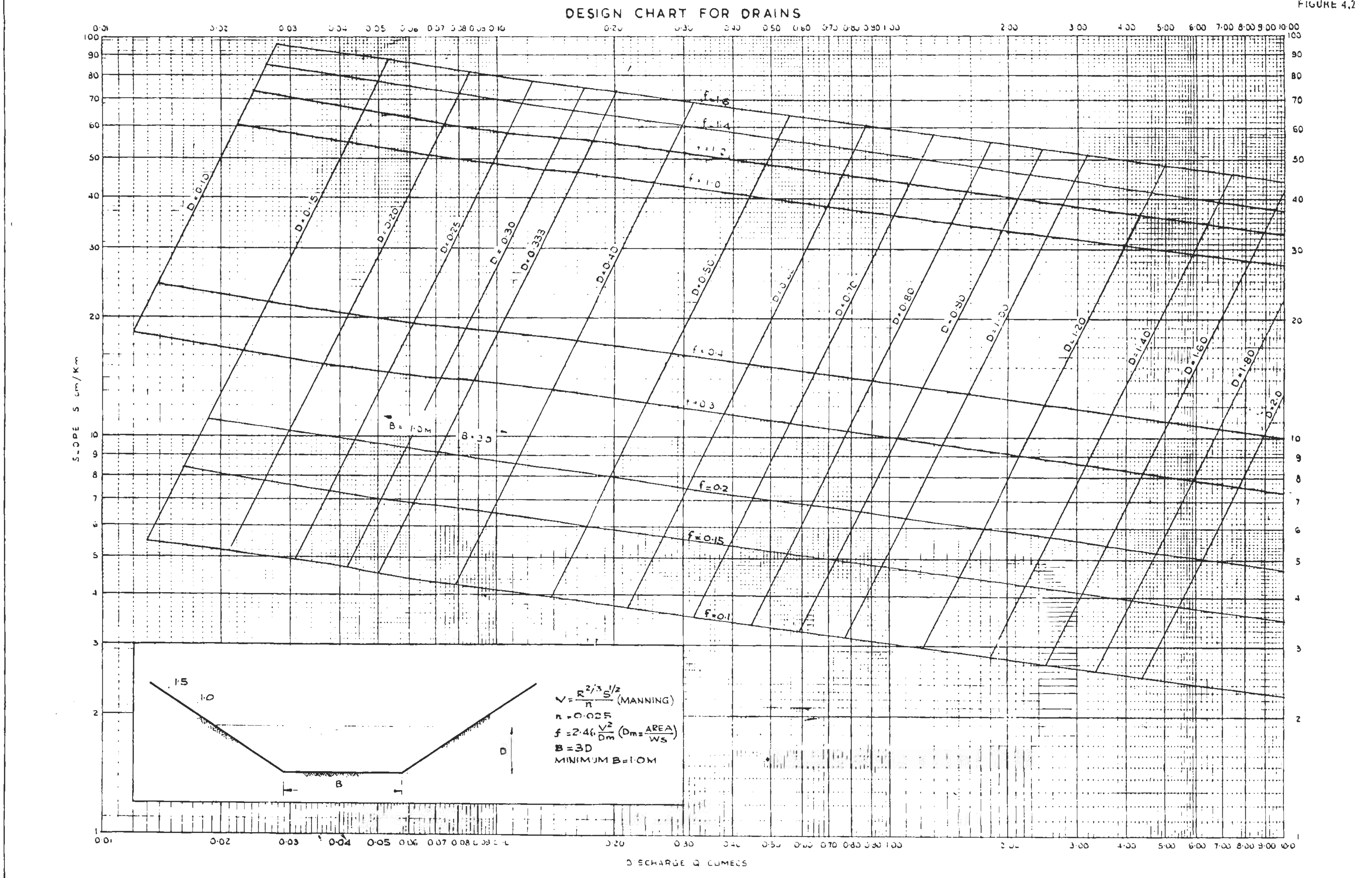
**Crop Sensitivity to Waterlogging (FAO/UNESCO 1973)**

| Sensitive | Moderately tolerant | Highly tolerant |
|---|---|---|
| Maize | Cotton | Sugar-cane |
| Beans | Citrus | Rice |
| Tobacco | Tomato | |

Table 4.4 gives a qualitative listing of crops divided into those that are sensitive, moderately tolerant and highly tolerant to the effects of waterlogging, based on their laboratory response to oxygen deficiency (FAO/UNESCO 1973). No information was available for bananas. Maize, the most important food crop in the Study Area, is classified as sensitive, meaning that adequate provision of surface drainage should be considered of importance.

From actual observations of waterlogged maize during the heavy der rains of 1977 it is possible to estimate a tentative value of four days for the waterlogged period that maize can withstand before complete failure occurs. However significant yield reductions will occur if the crop is waterlogged for as little as 24 hours, the exact reduction depending greatly upon the actual stage of crop growth at the time of waterlogging. Even this figure is quite high, but is thought to be applicable because of the heavy clay soil's ability to remain aerated at quite shallow depths even when the surface is completely saturated.

FIGURE 4.2


DESIGN CHART FOR DRAINS
SLOPE S cm/Km
DISCHARGE Q CUMECS
f=1.6
f=1.4
f=1.2
f=1.0
f=0.4
f=0.3
f=0.2
f=0.15
f=0.1
D=0.10
D=0.15
D=0.20
D=0.25
D=0.30
D=0.333
D=0.40
D=0.50
D=0.70
D=0.80
D=0.90
D=1.00
D=1.20
D=1.40
D=1.60
D=1.80
D=2.0
B=1.0M
B=3D
1.5
1.0
B
D
$V = \frac{R^{2/3}S^{1/2}}{n}$ (MANNING)
$n = 0.025$
$f = 2.46\frac{V^2}{Dm}$ ($Dm = \frac{AREA}{WS}$)
B = 3D
MINIMUM B = 1.0M

### 4.5.2 Storm Run-off

The daily records for rainfall at Janaale collected by A. Fantoli (1965) have not been preserved and only the monthly totals are available. Therefore for the prediction of design storms the daily records of rainfall at Mogadishu have been used. A total of 38 complete years of records were analysed and plotted both on a normal and extreme probability basis to produce the 24 hour design storms for a given return period (Table 4.5). The maximum recorded rainfall at Mogadishu in 24 hours is 150 mm.

After allowance has been made for infiltration and evaporation, the remaining rainfall from a storm of known return period represents the storm run-off that has to be passed through the control culvert of a field into the main collector drain system. The storm run-off must be removed from the field fast enough to avoid crop losses due to waterlogging, but at the same time slowly enough not to overload the drain channel capacities and pumping capacity.

TABLE 4.5

**24-hour Design Storms**

| Return period (years) | Storm intensity (mm) |
|---|---|
| 2 | 66 |
| 5 | 99 |
| 10 | 120 |
| 25 | 147 |

When the disposal of the gross quantity of storm run-off from a large area is being considered, an overall reduction can be made in the intensity because of the localised nature of storm centres. The following formula can be used to do this:-

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

where $Q$ = total discharge to be drained ($m^3/s$)
$C$ = discharge per square kilometre, assuming the full storm run-off applies over this area ($m^3/s$)
$A$ = area in $km^2$

The effects of the various factors in the design of drains are summarised in Table 4.6.

## TABLE 4.6

### Components of Drainage Water

| Component | Quantity % of Ig[1] | Quantity mm/d[2] | Quantity l/s/km²[2] |
|---|---|---|---|
| **A. Drainage surplus for deep infield drains** | | | |
| (i) Deep percolation | 10 | 1 | 11 |
| (ii) Field distribution losses | 5 | 0.5 | 6 |
| (iii) Main and secondary canal seepage losses | 5 [3] | 0.5 | 6 |
| TOTAL | 20 | 2 | 23 |
| **B. Design discharge for open collector systems [4] [6]** | | | |
| (i) Drainage surplus[5] | 20 | 2 | 23 |
| (ii) Surface run-off of excess irrigation water | 15 - 25 | 1.5 - 2.5 | 17 - 29 |
| TOTAL | 35 - 45 | 3.5 - 4.5 | 40 - 52 |

Notes:
(1) Ig is the gross irrigation application at field outlet
(2) Taking Ig to be 320 mm/month
(3) Dependant upon layout
(4) Storm run-off has to be considered independently and may demand a higher design discharge
(5) Total from A : only applies if deep field drains have been installed
(6) To determine the diameter of field drain pipes a peak drainable surplus of 71 $l/s/km^2$ is used.

### 4.6 Field Drainage Design

A system of buried field drains should be designed to optimise the spacing and depth of the drain lines. With large spacings a great depth is needed to control the water table to permissible levels, making excavations for the open drains excessive. Conversely a close spacing can minimise open drain excavation but the total length of buried field drains becomes excessive. The final choice of depth and spacing relies upon the selection of the minimum cost combination for the complete drainage system. However, a suitable design relationship is required to link the field drain spacing to the depth of the drain. The former is a complex function of :-

(i) the drainable surplus of the field

(ii) soil permeability

(iii) depth of drain

(iv) effective depth to an impermeable layer

(v) effective radius of the drain.

#### 4.6.1 Drainable Surplus of the Field

The amount of water to be drained from the soil for any given irrigation application (I) is the sum of the deep percolation losses of the field, seepage from the field distribution channels and an allowance for seepage from the main and distributary canals. By taking the maximum net irrigation requirement of the crops under consideration for one month as 160 mm (Table 2.7, Annex VI) and an overall irrigation efficiency of 50%, the total irrigation water allowed at field outlet is equivalent to 320 mm or just over 10 mm per day. Assume that deep percolation is 10% of this, field distribution losses 5% and an additional 5% is contributed towards the drainable surplus from main and secondary canals. This gives an expected drainable surplus of 2 mm per day. This compares with a maximum figure of 2.4 mm per day calculated for the Jowhar sugar estate (MMP, 1976).

#### 4.6.2 Soil Permeability

All the soils in the Study Area are either formed directly of, or are underlain at shallow depth by, the uniform dark/greyish brown clays of the Meander Flood Plain alluvium and it is the hydraulic conductivity of these clays which will affect most significantly the drainage characteristics and drainability of the area.

Table 4.7 is a summary of the auger hole pump-out test results for hydraulic conductivity undertaken in the Study Area, showing the large difference between the mean values for the clay soils and the clays with coarser textured lenses within the test depth. Although three different methods of hydraulic conductivity testing were performed, the pump-out test was regarded as the most suitable for assessing drainage parameters (Annex I).

TABLE 4.7

**Hydraulic Conductivity (K): Mean Values (m/d)**

| Soil texture | Sample number | Mean | Standard error |
|---|---|---|---|
| Clays | 12 | 0.23 | 0.04 |
| Clays with silt/sand | 6 | 0.76 | 0.14 |

Note: Results of auger hole pump-out tests.

The results closely correspond with the conductivity values reported for soils elsewhere on the Lower Shabeelle flood plain and therefore the value of 0.23 has been adopted as a representative figure for the Study Area. However, more detailed soil studies may reveal considerable variations on a local scale. It is interesting to note that the regression equation for the decrease in conductivity (K) with depth (d) for clay soils:-

$$K \simeq 0.52 - 0.11 \cdot d$$

produced for the soils (Annex I), gives a depth of 2.6 m for a value of 0.23 m/d for K. This corresponds well to the expected depth of a buried field drain system of 2.0 to 2.5 m.

The soils are relatively isotropic and the effect of cracking upon the drainability of an intensively irrigated area is expected to be low. This is because of the very low aggregate stability of the clay soil with wetting, which means that at the soil moisture levels encountered under irrigated crops the cracks will have disintegrated into an amorphous body of soil.

### 4.6.3 Effective Depth to Impermeable Layer

Within the Study Area no clearly defined impermeable boundary exists at depth, a permeable aquifer in fact underlying the surface clays at a depth of about 30 m. However, an expression for the effective depth to an impermeable layer (d,m) was derived by Hooghoudt (MMP, 1976) as:

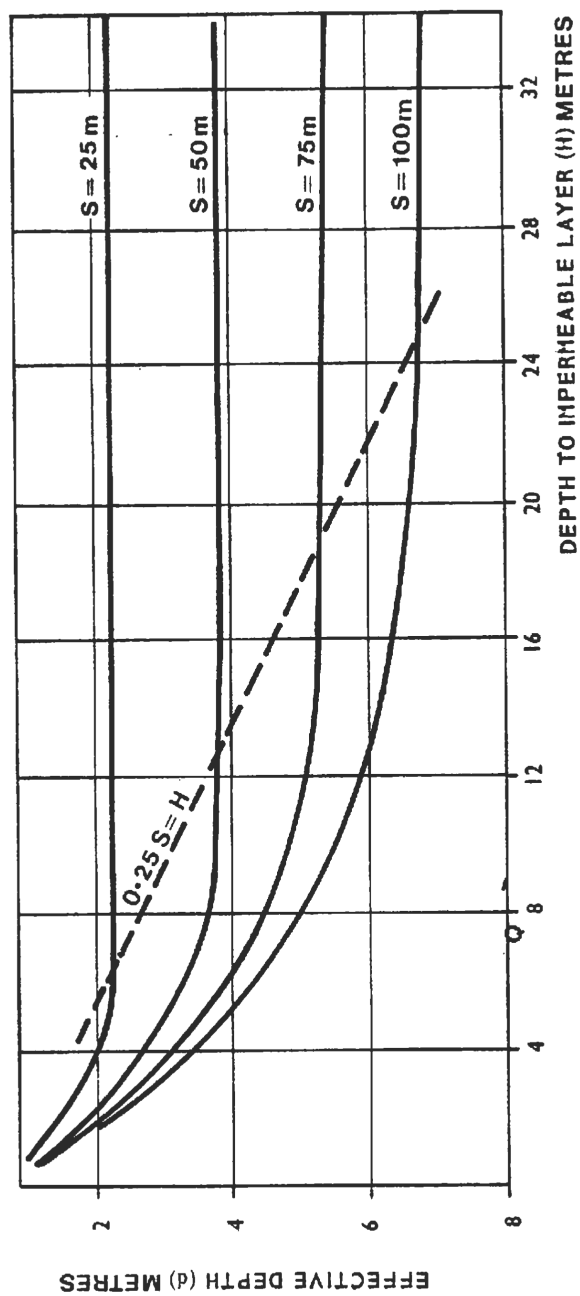
$$d = \frac{S}{8\left[\frac{1}{\pi} \cdot \ln \frac{H}{r\,2} + \frac{(1 - \sqrt{2}\ H/S)^2}{8 \cdot H/S}\right]}$$

where S = the drain spacing (m)
H = the depth to the impermeable layer (m)
r = the drain radius (m)

Figure 4.3 is a plot of the above expression for various spacings and r = 0.1 m. This shows that as long as H is greater than 0.25.S virtually no change in d occurs and an effective depth can be found by setting H equal to 0.25.S. As drain spacings are unlikely to be more than 100 m it is sufficient to assume that H = 25 m.

FIGURE 43

## EFFECTIVE DEPTH TO IMPERMEABLE LAYER



### 4.6.4 Drain Spacing Calculation

Various analytical methods are available for calculating the required drain spacings given known conditions of permeability, drainable surplus etc. These either assume that the drainable surplus is applied at a continuous rate and the water table is in static equilibrium with it (steady state method) or the more complex approach is taken where the drainable portion of the irrigation water is assumed to cause an instantaneous rise in the water table at the time of application and this then slowly falls until the next irrigation (non-steady state method). Table 4.8 summarises the salient features of five different methods of analysis. All the methods rely heavily upon the adopted value of hydraulic conductivity of the soil mass.

**TABLE 4.8**

**Methods of Drain Spacing Analysis**

| Method | Form of water application | Nature of soil |
|---|---|---|
| Steady state method for single layered soils | Continuous | Homogeneous and isotropic |
| Steady state method for two layered soils | Continuous | Two soil layers of different permeability but both homogeneous and isotropic |
| Steady state method for two layered soils (Ernst) | Continuous | Two soil layers, the upper being less permeable but both homogeneous and isotropic |
| Steady state method for two layered anisotropic soils (Lindenburgh) | Continuous | Two soil layers of different permeability but both homogeneous and anisotropic |
| Non-steady state method (Glover) | Instantaneous | Homogeneous and isotropic |

The basic problem of all the methods is that none of them was derived for use on the low permeability, cracking, montmorillonitic expanding lattice clay soils found in the Study Area. The errors due to soil cracking are likely to be much less than might at first be suspected; because of their inability to remain stable at high moisture contents the cracks are expected to have only a minor influence upon drainage of intensively irrigated land. However, the low permeability and expanding nature of the clays will introduce unknown errors into the analysis and it is for this reason that final drain spacings to be used on any projects in the Study Area must be based upon the results of detailed field trials.

For this purpose the trials at Jowhar sugar estate (MMP, 1976 and 1978) should, when complete, provide a valuable source of information.

However, to obtain a preliminary estimate of drain spacings for planning and costing purposes it is sufficient to adopt the steady state method for single layered soils. This assumes that the drainable surplus acts as a continuous supply (q mm/d) to the soil and that the groundwater table is static (Figure 4.4). If K (mm/d) is the hydraulic conductivity of the soil mass then Hooghoudt derived the following expression:

$$S^2 = \frac{8\ Kdh + 4\ Kh^2}{q}$$

This cannot be solved directly because d, the effective depth to an impermeable layer, is a function of S, and consequently an iterative solution is required. This problem was overcome by a nomograph drawn by Boumans from data supplied by Ernst (ILRI, Wageningen, 1973); Figures 4.5 and 4.6 show the nomograph and where K, H, h, r and q are known, a value of S/h and so of S, can be found.

The nomograph has been used to assess the likely range of drain spacings that may be needed for the variety of agronomic, irrigation and soil conditions to be encountered in the Study Area. Each parameter has been assessed for a most likely value and the expected range; these are summarised in Table 4.9. The spacing is relatively insensitive to both the effective drain radius (r) and the depth to the impermeable layer (as long as it is more than 25% of the spacing) and so only most likely figures are given for these.

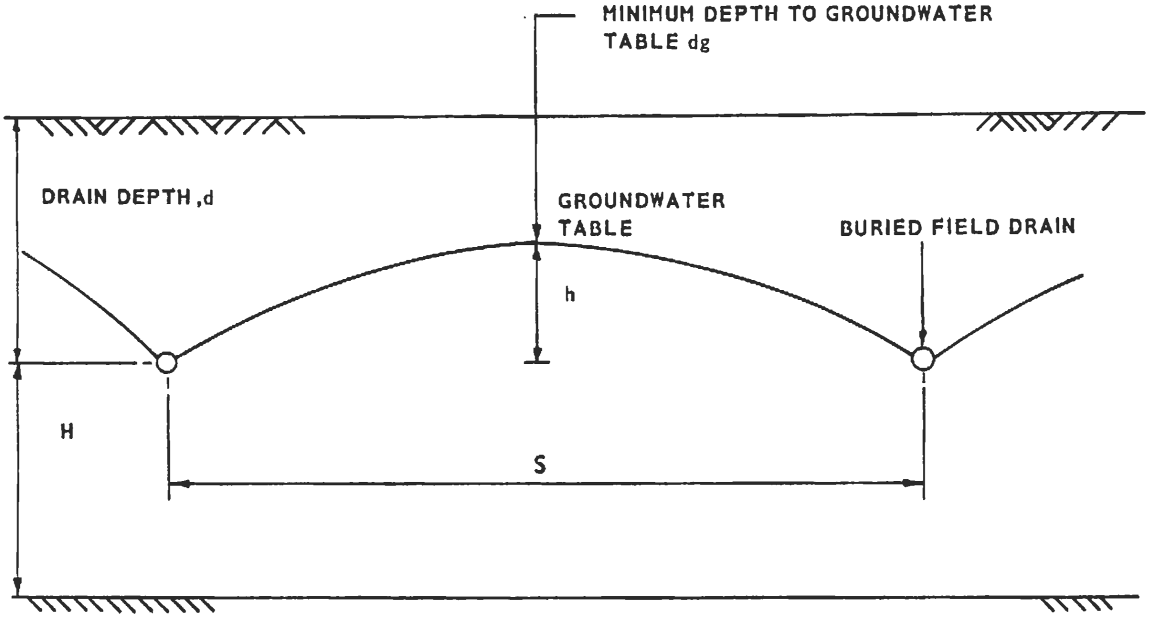
TABLE 4.9

**Most Likely Values and Ranges of Drain Design Parameters**

| Parameter | Unit | Most likely value | Range of values |
|---|---|---|---|
| Hydraulic conductivity | m/d | 0.23 | 0.05 - 0.40 |
| Drainable surplus | mm/d | 2.0 | 1.0 - 4.0 |
| Minimum depth to groundwater | m | 1.00 | 1.5 for citrus |
| Minimum depth of drain | m | 2.00 | 1.50 - 2.50 |
| Effective drain radius | m | 0.10 | - |
| Depth to impermeable layer | m | 25 | - |

FIGURE 4.4

## SCHEMATIC BURIED FIELD DRAIN LAYOUT




WHERE

S DRAIN SPACING

H DEPTH TO IMPERMEABLE LAYER BELOW DRAIN

h HYDRAULIC HEAD OF THE GROUNDWATER ABOVE THE DRAIN

FIGURE 4.9

# BOUMANS NOMOGRAPH FOR DRAIN SPACINGS

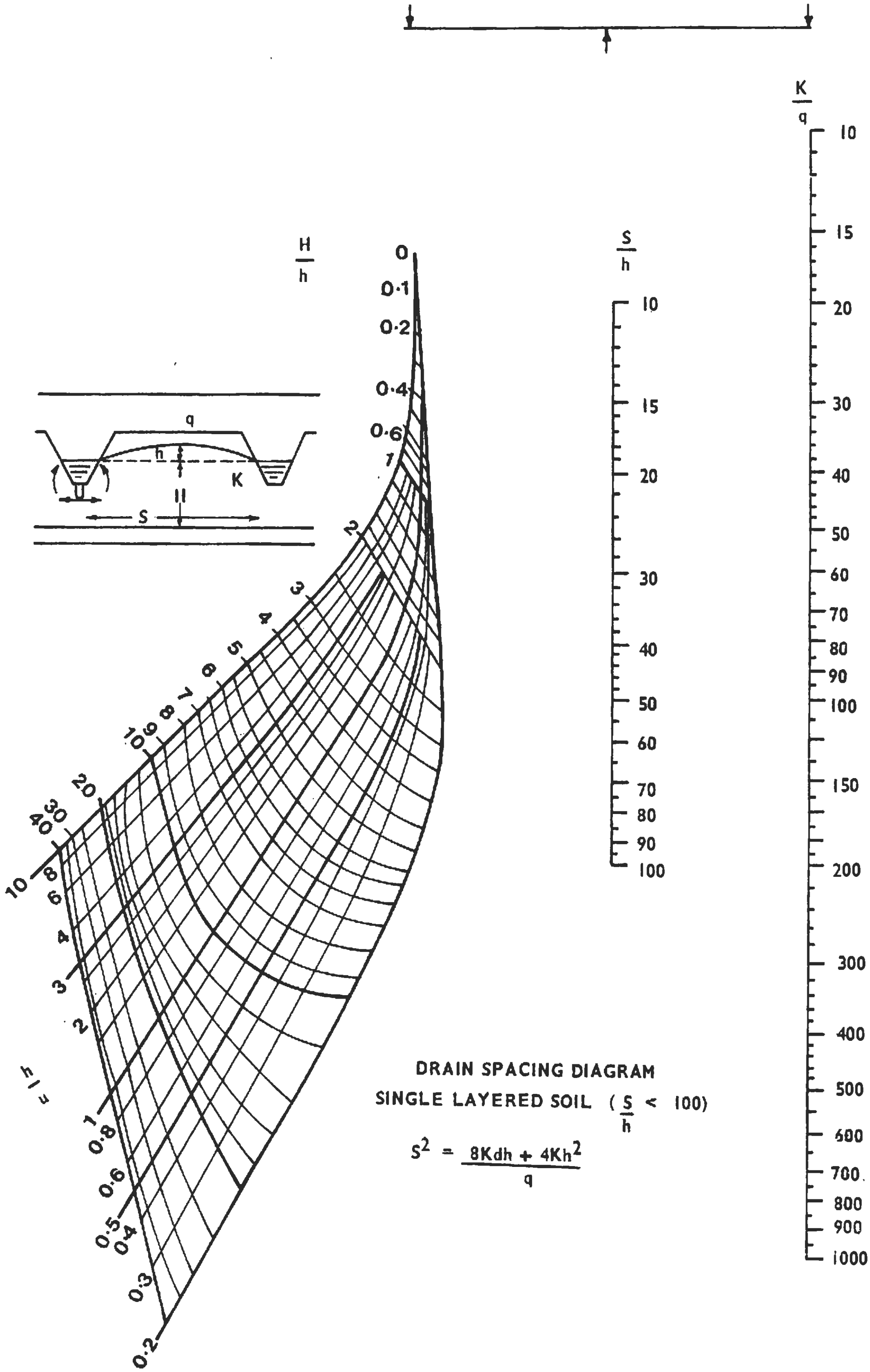
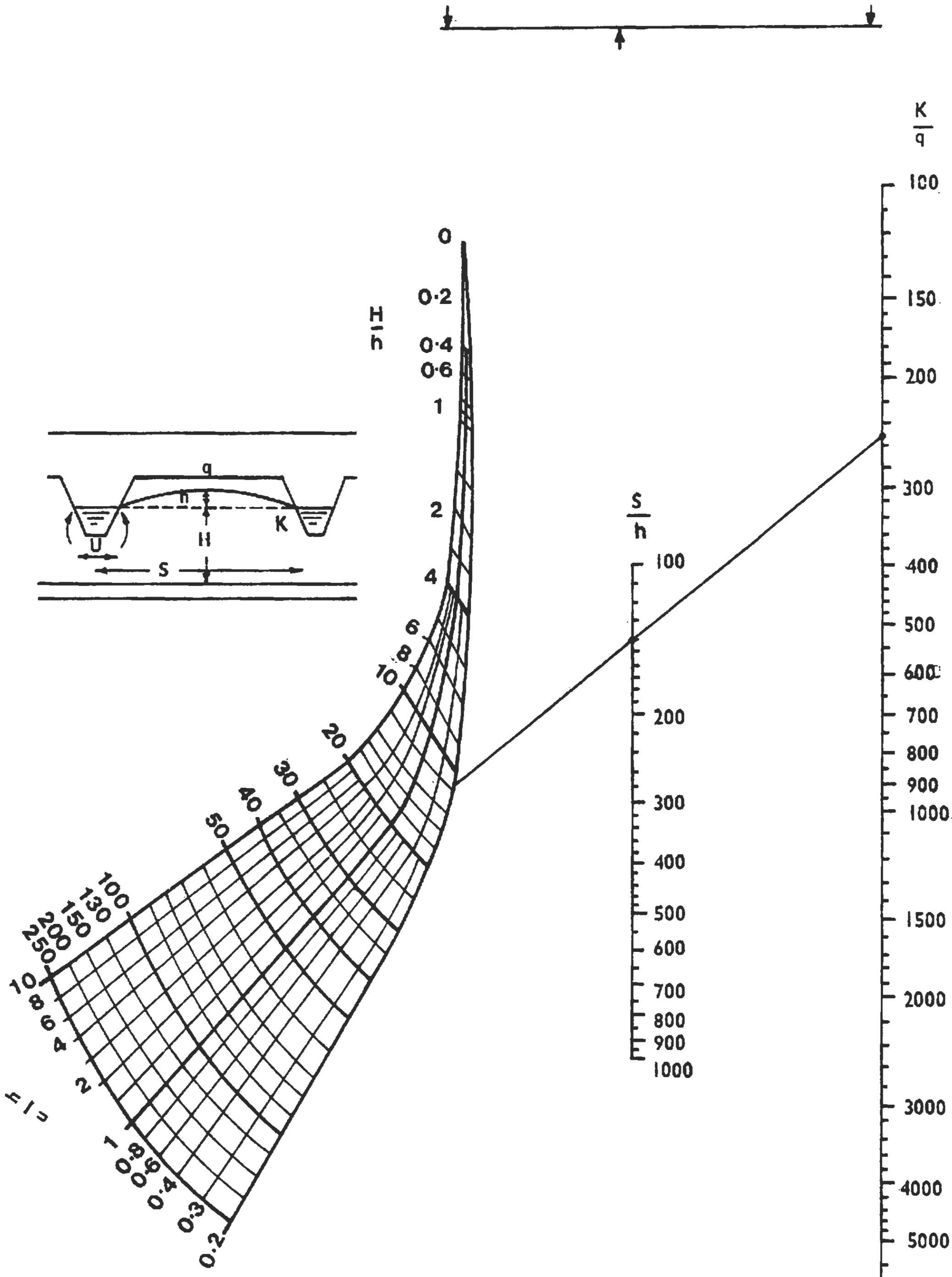


FIGURE 4.6

## BOUMANS NOMOGRAPH FOR DRAIN SPACINGS




DRAIN SPACING DIAGRAM

SINGLE LAYERED SOIL ($\frac{S}{h} > 100$)

$$S^2 = \frac{8\ Kdh + 4Kh^2}{q}$$

Hydraulic conductivity is the most basic and variable parameter so the three derived graphs (Figure 4.7) use this as the key variable for predicting the required drain spacing. For each graph a second parameter is then selected and, whilst holding the remaining factors constant at their most likely values, the influence of the second parameter is illustrated.

By taking all of the most likely values a drain spacing of 61 m is required, although by allowing the conductivity to vary, the spacing ranges from 22 to 100 m. The same wide variation occurs if the conductivity is set at 0.23 m/d and the drainable surplus is varied between 1 and 4 mm/d. The range in this case is 40 to 110 m. A dramatic reduction in spacings of 45% occurs if the minimum depth to groundwater is increased from 1.0 to 1.5 m; this means that any subsurface drainage of citrus will prove very costly. The effect of increasing the drain depth from 2.0 to 2.5 m is to increase the required spacing by 40%.

### 4.6.5 Pipe Sizes and Length

Flow in field drain pipes gradually increases with distance down the pipe. Therefore, pipe sizes have to be designed upon spatially varied flow equations. Cavelaars (1973) gives the following equations, based on the average hydraulic gradient along a pipe with spatially varied flow:-

smooth pipes: $$Q = 89 \cdot d^{2.714} \cdot \bar{i}^{0.572}$$

corrugated pipes: $$Q = 38 \cdot d^{2.667} \cdot \bar{i}^{0.5}$$

where

$Q$ = the pipe capacity ($m^3/s$)

$d$ = the inside pipe diameter (m)

$\bar{i}$ = the average hydraulic gradient

It is important to limit the value of i to the actual slope of the field drain (i) because otherwise hydrostatic head will be required to pass the discharge, causing the soil water to submerge the drain. Also, due to the effects of silt in the pipes, to joints and to poor alignment when installed, the pipe capacity has to be reduced. This reduction has been taken as 25% for plastic pipes and 40% for clay and concrete pipes. The final equations are:

smooth plastic: $$Q = 67 \cdot d^{2.714} \cdot i^{0.572}$$

corrugated plastic: $$Q = 29 \cdot d^{2.667} \cdot i^{0.5}$$

clay and concrete: $$Q = 53 \cdot d^{2.714} \cdot i^{0.572}$$

Work by Yarnell and Woodward (1920) indicates that a design drainable discharge of 71 $l/s/km^2$ (which is equivalent to a drainage rate of 6.1 mm/d) is sufficient for field drain design. This is likely to be at least double the actual average field drainable surplus, but this higher value is necessary because of the rapid initial drainage immediately after irrigation.

Using the above drainage discharge and pipe flow equations Figure 4.8 has been derived so that the maximum pipe length of a given size for a known drain spacing can be found. For the example shown, an 80 mm inside diameter corrugated plastic pipe is being laid to a slope of 0.5 m/km, and a spacing of 60 m. The pipe discharge capacity is 0.76 l/s, and a spacing of 60 m. The pipe discharge capacity is 0.76 l/s and the maximum pipe length possible is 180 m. If a longer length than this is needed the 100 mm corrugated plastic pipe would have to be used, up to a maximum total length of 330 m.

A normal limit of 300 m should be applied to the length of any field drain as this is the practical limit to which maintenance can be successfully carried out without the provision of intermediate inspection manholes. A recommended minimum pipe slope is 1.5 m/km; this should be steep enough to avoid drain blockage due to accumulated silt deposits.

### 4.6.6 Pipe Surround

The purpose in providing a surround to the field drains is to improve the inflow characteristics to the drain. Graded coral is the most suitable material available for this purpose in Somalia. In designing the surround it is important to ensure that its permeability is much greater than that of the base material; thus 100% of the material should pass the 25 mm sieve, at least 90% should pass the 19 mm sieve, and not more than 10% should pass the 0.25 mm sieve. Within these restraints, the size graduation of the material is not vitally important, but the thickness of the surround should not be less than 75 mm.

In view of the stability of the Study Area soils, the pipe surround should not have to act as a filter medium. However, in order to keep the amount of coral required to a minimum and to prevent the ingress of soil particles from the disturbed and loosely compacted backfill above the pipes, it may prove necessary to cover the top of the pipe surround with a layer of pervious material to act as a filter membrane. Information on the need for filter membranes should be available from the Jowhar sugar estate trials.

## 4.7 Disposal of Drainage Water

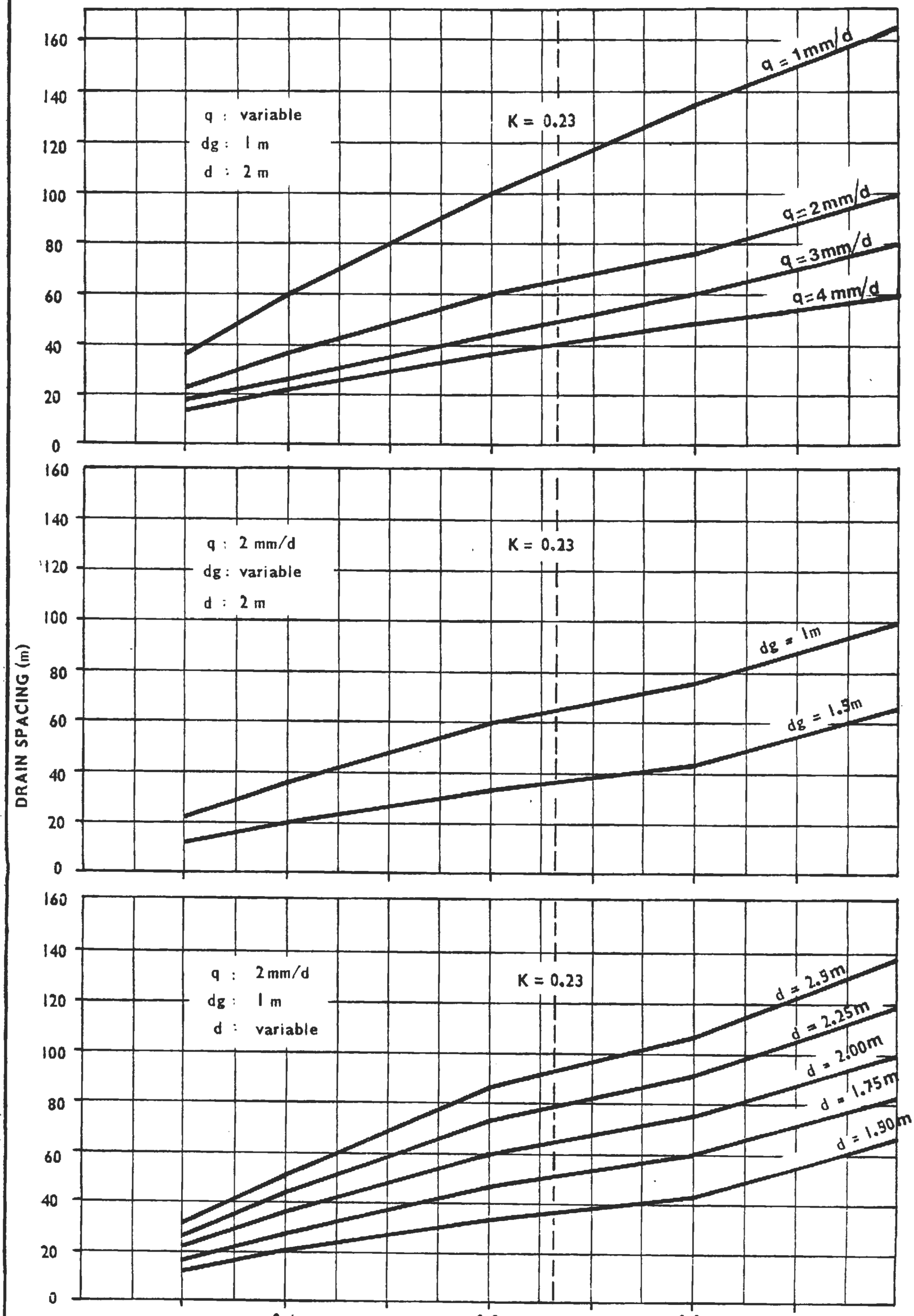
The suitable means of disposal of drainage waters depends upon its nature and quality; if it is solely run-off and excess irrigation water, the quality should only be marginally poorer than before it washed across the field. Therefore, it should be possible to pass the water back into the river for re-use further downstream, although checks for the presence of any toxic solutions washed off the land should be carried out.

However, if the drainage water is from buried field drains it is likely to be saline; preliminary results from the field drains at Jowhar have given electrical conductivities ranging from 3.7 to 11.8 mmho/cm (MMP, 1978) and are totally unsuitable for re-use. The water must be disposed of somewhere safely. Fortunately there is a number of old river channels which run out of the Study Area towards the west and south-west, on both sides of the river, and these can provide disposal channels to take the water safely away and into the swamp lands.

In possibly every case, because of the levee effect of the river itself and the old river channels, it will be necessary to provide pumping facilities to dispose of the water from the main drains.

## DRAIN SPACINGS

FIGURE 4.7




NOTE: In all cases r = 0.1m H = 25 m

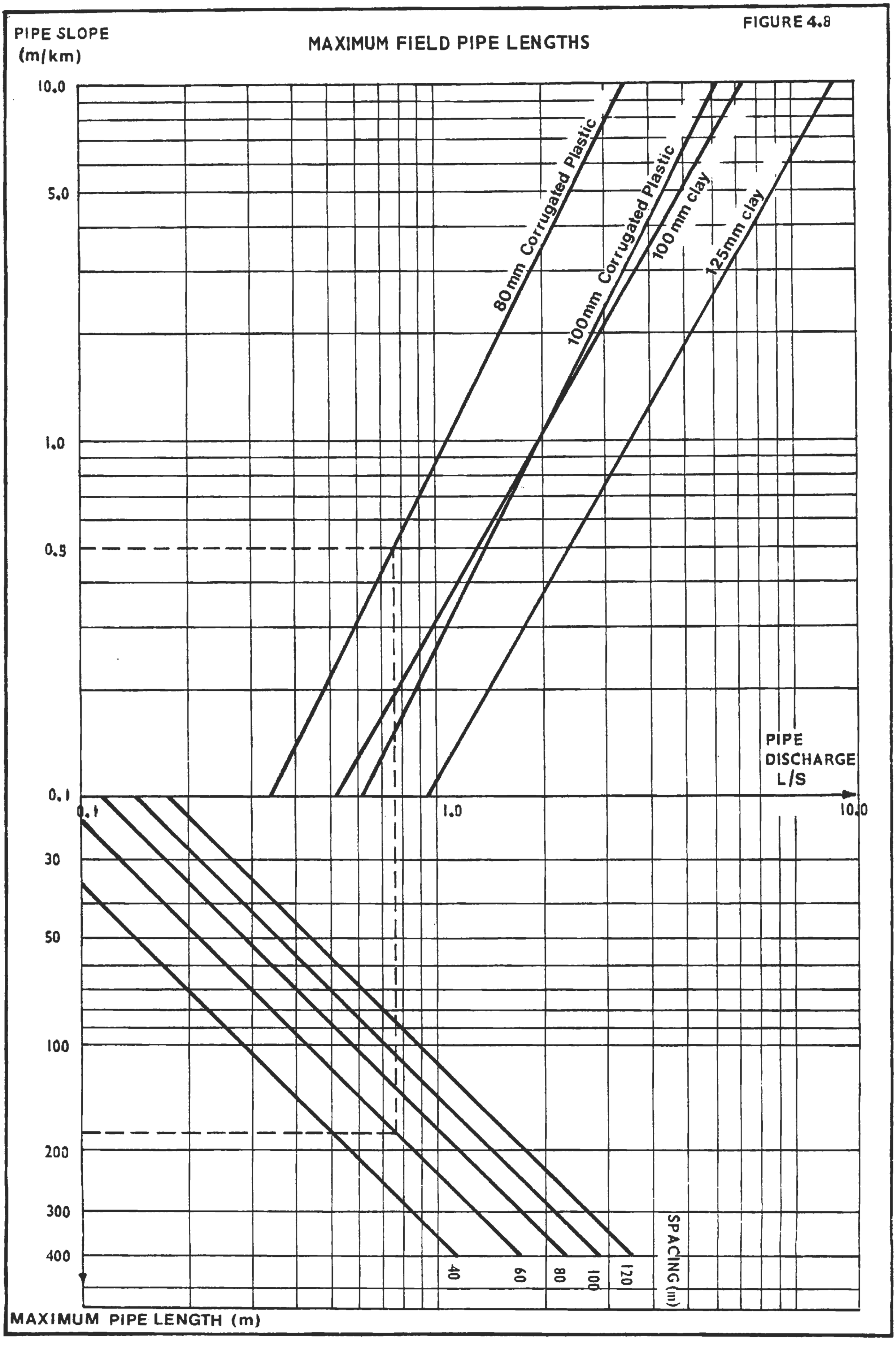


FIGURE 4.8
MAXIMUM FIELD PIPE LENGTHS
PIPE SLOPE (m/km)
10.0
5.0
1.0
0.5
0.1
80 mm Corrugated Plastic
100mm Corrugated Plastic
100 mm clay
125mm clay
PIPE DISCHARGE L/S
0.1
1.0
10.0
30
50
100
200
300
400
40
60
80
100
120
SPACING (m)
MAXIMUM PIPE LENGTH (m)

## 4.8 Field Drainage Construction and Costs

No suitable engineering rates are available for the installation of buried field drains in Somalia and therefore their costs have to be assessed from the various inputs needed. These can be grouped into machinery costs, pipe costs, pipe surround costs, and labour costs.

### 4.8.1 Pipe Laying Machinery

The method of installation of drains ranges greatly from hand dug trenches and hydraulic excavators to using purpose built machines. However, both hand digging and hydraulic excavation are prohibitively slow in relation to the required installationr ate for any major project; the rate of 320 km per year proposed for the Jowhar sugar estate (MMP, 1976) would require 775 men or eight hydraulic excavators, both of which are impracticable when it is considered that one purpose built machine can easily achieve this rate. Therefore, only purpose built buried field drain machinery has been investigated.

Trenching machines can be divided into three categories:-

Bucket wheel trenchers

Bucket chain trenchers

Bucket ladder trenchers.

Bucket wheel trenchers are capable of digging trenches 1.2 to 2.1 m deep but, because of the heavy weight of the bucket wheel, ground pressures are large and it is possible that the trench may collapse; because of this they cannot be recommended. Bucket ladder trenchers are used for laying drains at depths greater than 2.5 m and need only be considered for use in the Study Area if drains at this depth can be shown to be necessary.

Chain trenching machines are the most popular type available, having the advantage over wheel trenchers that their capacity and performance are many times higher. Typical operating depths are from 1.5 to 2.5 m and their proven reliability makes them the obvious choice of trenching machine. Clay, concrete or plastic pipes can be fed continuously to the machine which automatically lays them in the trench bottom. The cost CIF Mogadishu, of a 250 hp chain trencher which can cut a 2.5 m deep trench at a rate of 300 m/h is taken as So. Shs. 742 500.

Together with the trenching machine, certain support services are required; the gravel surround has to be placed by a tractor mounted hopper and a second tractor is needed with a blade to place the backfill in the trench. The cost, CIF Mogadishu for a 25 hp tractor with the associated attachments is taken as So. Shs. 77 700.

An alternative to using a trenching machine is to use a trenchless pipe-laying machine which draws continuous plastic pipe into the ground without the need to dig a trench first.

This type of machine has fewer moving parts than a trenching machine and therefore is cheaper to maintain. In addition, fewer support services are required. However, in recent studies comparing the effectiveness of drainage from field drains laid by trenchless and trenching machines in heavy clay soils, the drains laid by trenchless machines were significantly less effective. Trenchless machines are therefore not recommended for use.

Table 4.10 lists the annual cost of pipe-laying machinery: the annual capital costs have been calculated assuming a five year machine life and amortising at an interest rate of 8%. Annual maintenance and service charges are taken as 10% of the total capital cost. Fuel costs are based on consumptions of 50 litres per hour for the trenching machine and 30 litres per hour for each supporting tactor, at a cost of So. Shs. 1.58 per litre. The working year has been taken as 2 000 hours, the low figure being adopted to allow for breakdowns and maintenance; at a rate of 300 m per hour, and allowing a 25% reduction for turnaround time and stoppage, the total length of drain installed in a year is 450 km.

### 4.8.2 Labour Requirements

For the trenching machine laying clay or concrete pipes, one man is required to feed the pipes from the trailer to the machine, one man to feed the pipes into the machine and one man to ensure that the pipes are butted together correctly, a total of three men. With continuous plastic pipe, jointing is only required infrequently and only one general labourer is needed.

Experience has shown that compaction of the backfill, especially close to the collector drain, is essential and three compacting labourers have been allowed for. This is the same whether clay, concrete or plastic pipe is being used. In addition the trenching machine driver and a surveyor are needed.

Table 4.10 gives the annual labour cost for laying clay pipes and plastic pipes, based on a 2 000 hour working year, So. Shs. 1.0/h for labourers, So. Shs. 2.5/h for tractor drivers, and So. Shs. 4.0/h for machine operators and surveyors. The total labour cost is then combined with the machinery and supporting services cost to produce a total annual cost. This is then converted into a cost per linear metre of drain, knowing that during a year 450 km of drain would be laid.

### 4.8.3 Pipes

There are three types of material suitable for the manufacture of field drains:-

Clay

Concrete

Plastic

Clay pipes are the traditional material for use as buried field drains and have been in use in Great Britain for 150 years. No clay pipes are currently produced in Somalia but good quality clay bricks are made at Afgooye and it is thought that the clay is also suitable for use in pipe making. An addendum to the Jowhar sugar estate, Drainage and Reclamation Study (MMP, 1976), entitled 'Manufacture of Field Drains' (1.6.76), gave the cost of clay pipes (including modification

TABLE 4.10

Annual Cost of Pipe-laying Machinery and Labour (So. Shs.)

| Item | Clay pipe | Plastic pipe |
|---|---|---|
| Trenching machine | | |
| Amortised annual cost | 185 625 | 185 625 |
| Maintenance | 74 250 | 74 250 |
| Fuel | 158 000 | 158 000 |
| Sub-total | 417 875 | 417 875 |
| Supporting services | | |
| Amortised annual cost | 38 850 | 38 850 |
| Maintenance | 15 540 | 15 540 |
| Fuel | 189 600 | 189 600 |
| Sub-total | 243 990 | 243 990 |
| Labour | | |
| Labourers | 12 000 | 8 000 |
| Machine operator | 8 000 | 8 000 |
| Tractor drivers | 10 000 | 10 000 |
| Surveyors | 8 000 | 8 000 |
| | 38 000 | 34 000 |
| TOTAL COST per annum | 699 865 | 695 865 |
| TOTAL COST per metre based on 450 km per annum | 1.56 | 1.55 |

costs to the brickworks at Afgooye and transport costs) as So. Shs. 2.54/m for a 100 mm pipe. Allowing an increase of 15% for inflation gives a 1977 price of So. Shs. 2.92/m. By interpolation from the original report the equivalent cost for a 125 mm pipe is So. Shs. 3.65/m.

Concrete pipes are presently manufactured in Shalambood by NFMAS (ONAT) but only in small numbers and sizes too large for field drains. However, it is possible to install and operate pipe-making machinery on any large project and the Jowhar sugar estate addendum calculated a cost for 100 mm concrete pipe, on just such a basis, as So. Shs. 2.90/m. Allowing for inflation, and interpolating for the 125 mm pipe, gives a final cost of So. Shs. 3.34/m for the 100 mm pipe and So. Shs. 4.07/m for the 125 mm pipe. These figures are higher than those for clay pipes. If, however, concrete pipes could be produced less expensively, for example, by the construction of a national concrete drainage pipe factory, then they may well prove to be more economic than clay pipes. If such an instance arises then it should be noted that the high sulphate concentrations in the soils could attack concrete pipes and cause deterioration. In order to avoid this problem, sulphate-resisting cement in fully compacted concrete of low permeability must be used.

Slotted PVC pipe has been used in Europe for the last 15 years and is popular because of its light weight and easy handling. Originally smooth pipe was used, but this has been superseded by corrugated pipe which is lighter, cheaper, has greater mechanical strength and yet is flexible enough to be coiled on large drums. Plastic pipe has the advantage of providing a continuous drainage line not subject to displacement, is resistant to sulphate attack and can be used with all methods of pipe-laying. Plastic pipe is not at present made in Somalia and costs per metre used in the addendum to the Jowhar sugar estate study were calculated on the basis of installing a pipe manufacturing plant. The cost for an 80 mm corrugated plastic pipe was So. Shs. 4.46/m. Allowing for inflation gives So. Shs. 5.13/m for 80 mm pipe and, by interpolation, So. Shs. 8.36/m for 100 mm pipe.

### 4.8.4 Pipe Surround

For the standard trenching machine, it is assumed that the trench is filled with crushed coral to about 50 mm above the pipe, allowing 75 mm below the pipe and on each side. Also, for costing purposes, a filter membrane has been allowed for to cover the width of the trench, although this may not be necessary.

Costs of So. Shs. 150/$m^3$ for graded coral and So. Shs. 6.5/$m^2$ for filter membrane have been used. The coral can be obtained from the coastal quarries near Marka although special facilities may have to be provided for correct grading.

### 4.8.5 Total Costs

All the components of the cost per linear metre of installing field drains are drawn together in Table 4.11 to produce the total cost. An allowance of 30% has been made for management and overhead costs.

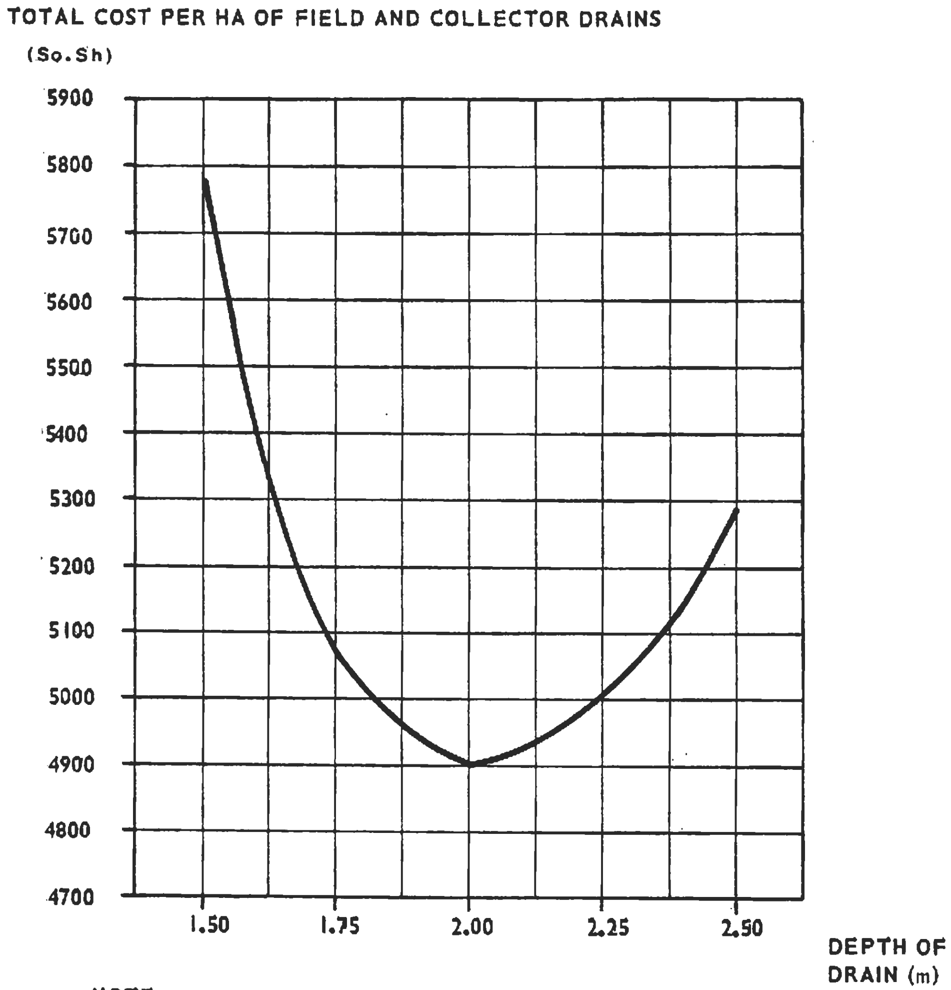
## TABLE 4.11

**Total Cost of Pipe Laying per Linear Metre (So. Shs.)**

| Item | Clay pipes 100 mm diameter | Clay pipes 125 mm diameter | Plastic pipes 80 mm diameter | Plastic pipes 100 mm diameter |
|---|---|---|---|---|
| Machinery and labour (from Table 4.10) | 1.56 | 1.56 | 1.55 | 1.55 |
| Pipe manufacture and transport | 2.92 | 3.65 | 5.13 | 8.36 |
| Pipe surround | 5.25 | 6.30 | 4.50 | 5.25 |
| Filter material | 1.63 | 1.78 | 1.50 | 1.63 |
| Sub-total | 11.36 | 13.29 | 12.68 | 16.79 |
| Management and overheads | 3.41 | 3.99 | 3.80 | 5.04 |
| TOTAL cost | 14.77 | 17.28 | 16.48 | 21.83 |

The table shows that clay pipes may prove to be less expensive than corrugated plastic ones. The cost for one metre of 100 m diameter clay pipe of So. Shs. 14.77 and a cost for the excavation of collector drains of So. Shs. 7.60/$m^3$, together with all the most likely design parameters (see Table 4.8) have been used to produce Figure 4.9. This shows the total cost per ha of the required field drains and collector drains for various depths of field drain, but does not include the cost of either main collector and disposal drains or the pumping facilities. The figure shows a rapid increase of costs for depths less than 1.75 m and a more gentle increase for depths more than 2 m. As the cost of the other components not included in the figure will increase rapidly with increased depth, the optimum depth for field drains must lie in the region of 1.75 to 2.00 m.

## TOTAL COST PER HA. OF FIELD AND COLLECTOR DRAINS




NOTE:

| | |
|---|---|
| K | 0.23 m/d |
| dg | 1.0 m |
| q | 2 mm/d |
| r | 0.1 m |
| H | 25 m |

# PART II

# QORYOOLEY PROJECT FEASIBILITY STUDY

# CHAPTER 5

## FEASIBILITY STUDY INTRODUCTION

### 5.1 Surveyed Area and Datum

The initial area of land to be selected for the feasibility study was outlined in the Janaale-Buulo Mareerta project Inception Report (Annex X). In this, two areas of approximately 5 000 ha each were proposed, one on the left bank of the river in the region of Faraxaane and the other on the right bank of the river between the villages of Qoryooley, Gayweerow and Tawakal. At the time of the presentation of the Inception Report (June 1977), insufficient information was available to identify the advantages and disadvantages between these two areas on technical criteria. Therefore the final selection, the Qoryooley Project Area, was made by the Client who could consider social and political factors.

The boundaries shown in the Inception Report were determined from the 1962/63 aerial photographs. When the 1 : 25 000 contour maps of the Study Area became available it was immediately apparent that large amounts of scale distortion occurred in the region of Qoryooley. Consequently the boundaries of the Project Area had to be modified and the final surveyed area, shown on Figure 5.1, was 5 820 ha.

The detailed survey information was used to plot 1 : 5 000 topographical maps with contours at 250 mm intervals. These have been presented in the form of orthophotographs where the contours are superimposed upon the background detail available from the photographs, which have been corrected to be true to scale (Annex X). Certain minor discrepancies have been discovered between the original FAO benchmarks located in the Project Area and therefore all levels dealing with the feasibility study have been based upon one single benchmark, A 39, located on Gayweerow bridge. All references to levels in Part II of this annex, unless otherwise noted, are made with respect to A 39. The detailed descriptions of benchmarks, the survey work undertaken and the discussion of discrepancies are located in Annex X.

The Project Area roughly forms a dish, with high ground in the region of 66.00 to 68.00 m almost entirely ringing the lower lands in the centre of the area. The central depression does not form a single low-lying area but isolated areas of land falling to as low as 64.00 m, with gentle ridges (65.00 to 65.50 m mostly) separating them. The only low spot in the encircling higher land occurs in the west, just north of Qoryooley where the land falls to 65.25 m; this point also links closely to the depressions in the centre and therefore is the obvious place through which the line of the disposal drain should pass.

### 5.2 Present Irrigation

The surveyed area is currently irrigated from two sources, the Asayle canal nominally commanding a gross area of 2 390 ha, and direct irrigation from the river which covers a total area of 2 870 ha. The remaining 560 ha are either villages or bushland. The gross irrigated areas represent the actual limits to which irrigation has been attempted (and often abandoned); this is normally far beyond the practical limits for the successful supply of adequate water for correct irrigation practice and therefore much of the irrigated agriculture is of a marginal nature. The actual net cultivated area, including a small area that is just outside the Project Area but will lose its water supply because of the new development, is estimated to be 2 110 ha.

There are many engineering problems involved in the successful operation of the existing irrigated areas and the major ones are listed below.

(i) The minor canals served by the Asayle canal have their offtake points downstream of the Tawakal cross regulator. Consequently command is rather limited and the water supplies unreliable. The latter arises because the canal capacity is normally, due to silting and weeds, too small to satisfy the full demand and the larger farms nearer the head of the canal take their complete share, leaving only a minimal flow for other users. The Asayle canal beyond the Tawakal regulator can be regarded as little more than an outfall drain to dispose of excess flows in the upper reaches of the canal.

(ii) The small canals that offtake directly from the river can be as much as 8.5 km upstream of Qoryooley barrage. Consequently they rely on large discharges in the river to maintain levels high enough for sufficient water to be abstracted. When river discharges fall the barrages are closed and their upstream water levels maintained as far as possible; however, with an almost flat backwater curve, the levels any distance from the barrage fall considerably and the supply to the minor canals is denied.

(iii) In all the area, no operational water control equipment or field outlet gates exist. All water control is either achieved by breaking and remaking the earth banks and bunds or, occasionally, by opening and closing with earth small concrete pipes (less than 200 mm diameter) in the canal bank. This makes the fair distribution of water difficult and the overall efficiency of water use low.

(iv) The minor canals, either offtaking from the canal or the river, run tortuously across the land, making poor use of the ground contours in many cases and often providing only minimal command over the field level. Because of this much duplication of channels occurs.

(v) The minor channel slopes are inadequate (for the small discharges they carry) to keep the sediment load in motion. This results not from the land slopes being insufficient but from the attempts to extend the channels as far as possible beyond the central low point up the other side of the Project Area 'basin'. This has been carried to the extreme in at least two cases where quite large channels have been extended right across the Project Area; these canals rarely receive any water at their tails.

(vi) The lack of adequate slope on the minor canals means that they require silt and weed clearance before the start of every season and, as their banks are almost everywhere too small to accommodate machinery, this work is done by hand.

(vii) No land forming has been undertaken and the microtopography is uneven. Therefore only small basin irrigation is possible, the actual size of the basin varying with the unevenness. Some basins are as small as 4 $m^2$. However, poor water distribution still occurs with both waterlogging and water shortage arising within some basins.

It has not been possible, therefore, to make much use of the existing irrigation system within the Project Area for intensive development and the preliminary layouts that have been designed for the feasibility study are largely new.

## 5.3 Sources of Irrigation Water Supply

The first major point concerning water supply in the Project Area is that the groundwater is, without exception, of poor quality; from the few hand dug wells existing in the area electrical conductivities (EC) of from 1.46 to 19.48 mmho/cm were measured, with an average value of 7.93 mmho/cm and from the single accessible tubewell a value of 7.81 mmho/cm was obtained. These figures are beyond the acceptable limits for irrigation water.

Irrigation supplies must therefore be obtained solely from the river and it was originally hoped that, with the construction of the new barrage at Gayweerow bridge (see Figure 5.1) for the Ministry of Agriculture grapefruit production scheme, it would be possible to gravity feed the entire system from there. A design holding level behind the barrage of 15.10 m, based upon an arbitrary datum, was provided by the Consultants, and when transferred to the Somalia national datum it produces a level of 69.11 m (based upon A 39 benchmark). An inlet channel would carry the water to the other side of the Asayle canal, where it would split into two branch canals: the left branch to run alongside the river, passing the villages of Gayweerow and Jasiira; the right branch to run parallel with the Asayle canal as far as Tawakal and then to swing across the higher ground in the north of the Project Area almost as far as Tugarey (see Figure 5.1).

The preliminary designs for these canals showed that if gravity supply were to be possible the required level at the barrage would be 70.41 m. The only ways to provide this are either to raise the holding level of the barrage by 1.3 m or to abandon the attempts to provide full gravity supply and to pump instead.

### 5.3.1 Raising Gayweerow Barrage

The extra command required for full gravity supply cannot be provided by raising the holding level of the barrage and the associated flood banks because, unfortunately, during periods of high river discharge, this would have the effect of flooding out Janaale barrage. A design flood water level of 69.21 m has been calculated for upstream of Gayweerow barrage and at the flood flow of 65 $m^3/s$ assumed for this section of the river, this is equivalent to a level on the downstream side of Janaale barrage of 71.34 m; allowing 200 mm headloss through the barrage gives an upstream level of 71.54 m which is equivalent to a gauge reading of 4.67 and is only 0.6 m below the existing road level across the barrage. Consequently any raising of the river levels beyond this point would have very serious consequences for Janaale barrage and the associated flood protection works upstream of it. All levels given above are based on a complete set of benchmarks, not solely A 39.

### 5.3.2 Pumped Supply

As full gravity supply cannot be provided from upstream of Gayweerow barrage, pumping of the irrigation water is necessary. Two different methods of achieving this have been considered in detail and are outlined below.

(i) Pumping at the offtake point from the river to a level of 70.41 m so that from there onwards the complete system could be run by gravity flow.

The advantage of this system is that the pumping facility is situated in one place, but it has the major drawback that the earthwork requirements in fill are very large (some 900 000 $m^3$ greater than the second alternative) and on the northern branch canal just downstream of Tawakal a saddle has to be crossed, giving a maximum total command of 3.0 m at this point.

(ii) Allowing the inlet channel and branch canals to operate under gravity feed in a cut and fill balanced canal section. The earthworks for this type of section are efficient, with only minimal amounts of haul being necessary. Pumping would then be provided from the branch canals into the head works of the distributary canals; by grouping of the 15 distributary canal heads at convenient points it has been possible to reduce the number of pump stations to seven small units. The main advantage of this system, in addition to the efficient earthwork quantities, is that a total of 573 ha of cultivated land can be gravity fed, hence saving pumping costs.

The detailed comparison between the two alternatives took into account all the major earthwork and pump costs together with all the detailed differences required in water control equipment and structures; this was done for capital costs, maintenance costs, machinery replacement costs and running costs over an assumed project life of 30 years and discounted at 8% to produce the present cost of each case.

The results were conclusive, with alternative (i) costing a total of So.Shs. 22.7 million more than alternative (ii) over the complete life span of the project. Therefore only pumping at the head of distributary canals has been included in the preliminary designs. Another alternative has been investigated in the next section and full costs are given for this.

## 5.4 Sources of Supply other than Gayweerow Barrage

The opportunity exists to divide the supply of irrigation water to the project into two parts, with the northern branch canal being supplied by gravity from the Asayle canal (see Figure 5.1). This has the effect of increasing the net cultivated area fed by gravity from 573 ha to 1 662.5 ha, which is equivalent to nearly 50% of the total intensively irrigated area. This alternative would require greater earthwork quantities and extensive remodelling of the Asayle canal; in addition the problem of excessive canal commands occurs. The project, modified for this alternative, has been assessed in Chapter 10, and is presented with a full economic and financial analysis for direct comparison with the situation where water supply is solely from Gayweerow barrage.

Also in Chapter 10 the consequences upon the project of the new barrage not being commissioned are discussed.

## 5.5 Project Area Villages

Only two existing villages lies within the irrigated areas of the project, as shown by the layouts on Map 3A. These are:-

Buulo Koy
Tugarey.

In addition the area is ringed by a total of six villages and the irrigation layouts have been designed to cause no disruption of the housing in these. They are:-

Qoryooley (district centre)
Jasiira
Gayweerow
Murale
Tawakal
Garas Guul.

For the successful implementation of the project a certain amount of movement of people away from the larger villages (Qoryooley and Gayweerow) to the smaller outlying ones is necessary. In two cases new villages will be required namely:-

Nimcooley
Shamaan.

## 5.6 Soil Suitability and Land Classification

Over the entire area of 5 820 ha a detailed soil survey, with a sampling density of five sites per square kilometre, was undertaken. This provided information for each soil horizon giving capacity, pH, and the electrical conductivity of the soil saturation extract. This latter parameter enabled a soil salinity map to be produced (Map 2C), showing that throughout the area salinity represents, at worst, only a moderate hazard. Over approximately 95% of the land the salinity hazard is low or negligible.

The basic information on Map 2C has been combined with detailed survey data of topography, soil permeability and surface water problems to produce the Land Suitability Map (2B). This divides the land into four classes for surface irrigation of annual crops excluding paddy rice namely highly suitable, suitable, moderately suitable and unsuitable. Ninety-three per cent of the feasibility study area falls into the suitable and moderately suitable classifications and no major restraints on successful irrigation of these areas are expected.

The remaining 7% has been classified as unsuitable. Apart from one small area between Tawakal and Buulo Koy the complete area of unsuitable land is associated with the alluvial deposits adjacent to the only old river channel in the Project Area. This runs approximately parallel to the Asayle canal but the unsuitable land only has significant width in the section downstream of Murale.

## 5.7 Final Project Area

Table 5.1 gives a listing of the final irrigated and associated areas as determined by the layouts shown on Map 3A. New villages have been included as a part of the Project Area, but existing ones, unless they lie within the irrigated areas, have been excluded. The total figure of 5 154 ha represents the final area for the Qoryooley project feasibility study.

Apart from the areas lost to villages, roads and reservations etc. all the land encircled by the two branch canals is to be irrigated intensively. The only exception to this is the bulk of the 'unsuitable' land adjacent to the lower 3.5 km of the Asayle canal. This area is contained within the branch canals and, therefore, is regarded as part of the project. However, it is classified as unsuitable and it is not to be irrigated intensively. Despite this the opportunity to provide a more reliable water supply and some drainage for the existing agriculture has been taken. The total area involved is 230 ha.

Some small areas of unsuitable land do occur within the intensively irrigated areas but these are small enough for manipulation of field cropping patterns and irrigation practices to render their detrimental effect negligible.

Taking the final net cultivated area (NCA) of the intensively irrigated lands (3 963.5 ha) gives an overall project land use efficiency of 77%.

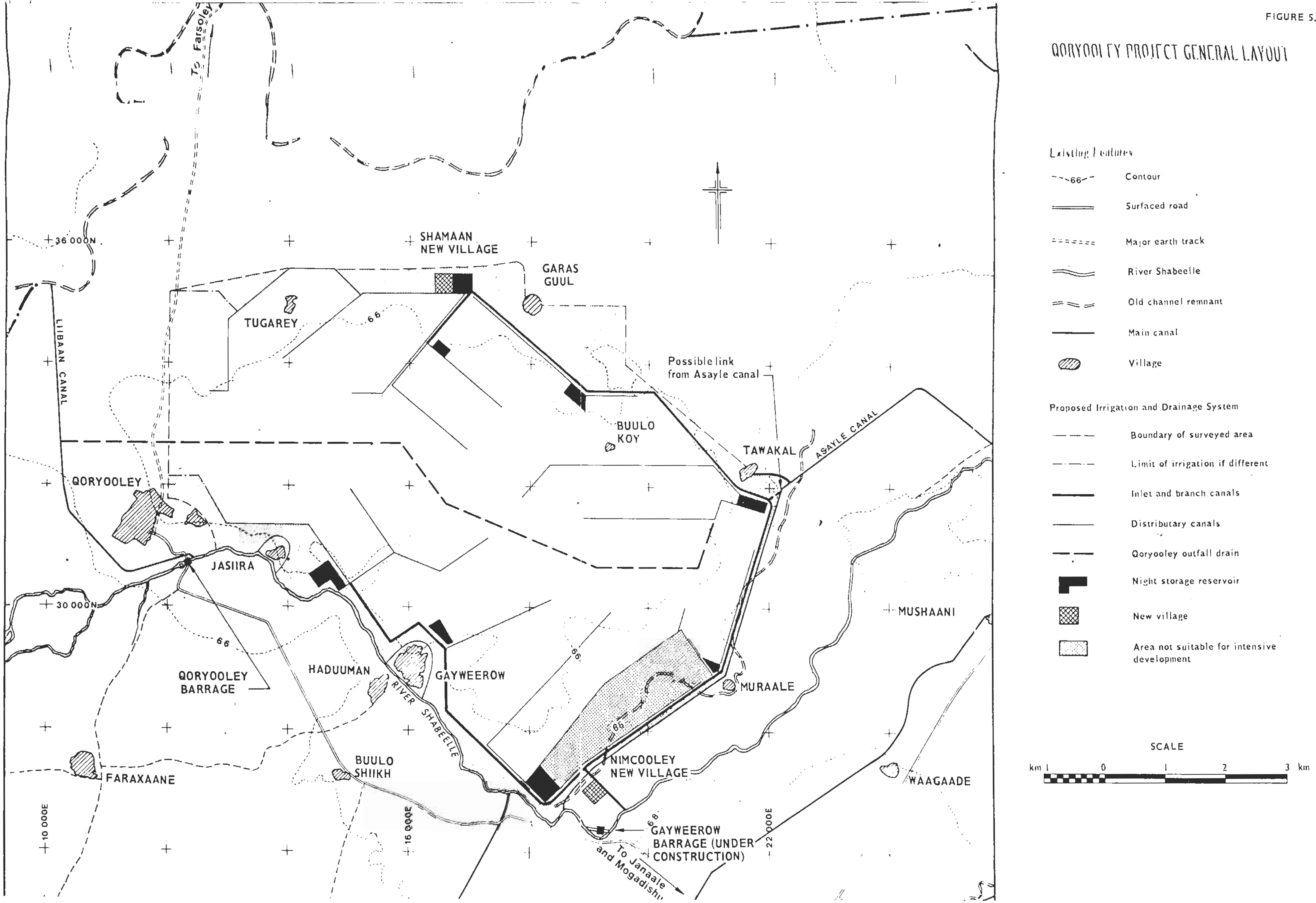
**TABLE 5.1**

**Final Project Area**

| | ha |
|---|---|
| Net cultivated area of 3 963.5 ha at 90% land use efficiency* | 4 404 |
| Other non-included reservations | 126 |
| Branch canals | 116 |
| Inlet channel | 10 |
| Outfall drain | 99 |
| Storage reservoirs | 69 |
| Road (surfaced) | 40 |
| Included villages (Buulo Koy, Tugarey) | 20 |
| New villages (Nimcooley, Shamaan) | 20 |
| Project headquarters | 3 |
| Area of non-intensive irrigation on unsuitable lands | 230 |
| Unirrigated land within the system | 17 |
| TOTAL Project Area | 5 154 |

Note: * this includes the basic field unit with its watercourse, surface drain, and the relevant half section of distributary and main collector drain.

FIGURE 5.1




QORYOOLEY PROJECT GENERAL LAYOUT

## 5.8 Bush and Tree Clearance

The majority of the Project Area has been cleared of bush already for the purposes of the existing cultivation, even in areas where no irrigation water has been able to reach and the agriculture is solely rainfed. However, small patches of light bush do exist and the total area is estimated to be 150 ha. In addition a few pockets of dense thicket, especially close to waterlogged land and roads, still exist and will have to be cleared; the area involved is approximately 30 ha.

The removal of large trees will only be necessary where they cannot conveniently be accommodated into the new irrigation layout. Indeed as many trees as possible should be left to provide shade and act as windbreaks. The number of trees that will require removal has been estimated from ground observation and photo interpretation to be 500.

## 5.9 Farm Management

The project land is to be cultivated on the basis of co-operative group farms; the total NCA of 3 963.5 ha has been divided into eight farms of sizes ranging from 263 to 623 ha, with an additional 137 ha reserved for a pilot farm.

Within each farm the land is divided up into field units (see Chapter 7) of an average size of 22 ha; the typical field unit is then cultivated communally by 10 to 12 families, with technical assistance both from the farm management and the project management. The normal field is reserved for the production of basic food crops (maize and sesame) and cash crops (cotton and upland rice). However, each family is to be provided with an additional one-eighth of a hectare for the production of miscellaneous vegetable and legume crops. These 'houseplots' are to be grouped together and be as close as possible to the villages. The detailed organisation of each farm unit is discussed in Annex VI.

## 5.10 Cropping Pattern and Water Requirements

Full cropping of the NCA in the der season is expected but, to comply with the restraints imposed by the critical water supply situation in June and July, the gu season cropping must be limited to the currently cropped area of 1 977 ha. (This excludes the existing cultivation on the unsuitable lands that are not to be intensively irrigated). However, the houseplots will need to be cultivated fully in both the gu and der seasons, a total of 253 ha for each, leaving only 1 724 ha of NCA as the maximum area of gu season field crops. This represents 46% of the total NCA available for normal fields of 3 710.5 ha (i.e. 3 963.5 - 253).

Table 5.2 gives the proposed cropping pattern for the project. The 60% cropping intensity for the gu season does not in fact infringe the 46% limit because the forage is only a short season crop with no water requirements in the month of July. For a full explanation of the choice of cropping pattern refer to Annex VI.

TABLE 5.2

**Project Field Cropping Pattern: Percentages of NCA**

| Crop | Gu season | Der season |
|---|---|---|
| Maize | 20 | 20 |
| Forage (fodder) | 20 | |
| Upland rice | 20 | 20 |
| Cotton | | 35 |
| Sesame | | 25 |
| TOTAL | 60 | 100 |

The net irrigation requirements for the proposed cropping pattern have been calculated in Annex VI, Chapter 19, but for ease of reference the complete build up of requirements has been given again in tabular form. Table 5.3 shows the monthly and annual requirements for an NCA of 10 ha of normal field crops. Table 5.4 repeats this but for an NCA of 10 ha of houseplots.

Table 5.5 combines the net requirements for both field crops and houseplots in the correct proportion (i.e. 6.37 ha NCA of houseplots and 93.63 ha NCA of field crops) to form the net requirements per 100 ha NCA of project land. The net demands are then converted into gross requirements at the distributary outlet to the field by allowing an overall field water distribution and application efficiency of 60% (Annex VI, Chapter 2). The field losses incurred are made up of the following, all expressed as a percentage of the gross requirement:-

| | |
|---|---|
| Watercourse seepage losses | 5% |
| Deep percolation losses | 10% |
| Surface run-off | 15% |
| Mis-management and over supply | 10% |

## 5.11 Engineering Properties of Soils

The soils of the Shabeelle Flood Plain are mainly alluvial clays and are very fine textured in the Project Area. Nevertheless sand lenses were encountered quite frequently. Cohesion appears to be quite high and the river often overtops its banks without eroding the clay soils. In the Project Area the hydraulic gradient away from the river was often incredibly steep and seepage losses from the river bed are negligible. This was confirmed by the seepage test on a portion of the Asayle canal. The clays are therefore suitable for forming unlined canals but precautions must be taken wherever sand lenses are located.

Atterberg limit tests were carried out on several soils in the Project Area and the results of these tests are given in Table 5.6. This indicates that a safe bearing pressure of 100 $kN/m^2$ (1.0 $ton/ft^2$) may be used for all irrigation structures. However, it is important that simple exploratory boreholes are drilled at the site of major structures to test that no sand lenses or soft clays occur.

TABLE 5.3

Net Irrigation Requirements (Mm$^3$) per 10 ha NCA of Field Crops

| Season/ Total Crop | Culti- vated area (ha) | Planting date | Culti- vated area per planting date (ha) | M | A | M | J | J | A | S | O | N | D | J | F | requi- rement |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Gu | | | | | | | | | | | | | | | | |
| Maize | 2.0 | Apr 15 | 1.00 | | 0 | 0.81 | 1.04 | 1.00 | | | | | | | | 2.85 |
| | | Apr 30 | 1.00 | | | 0.66 | 0.91 | 1.21 | 0.38 | | | | | | | 3.16 |
| Forage | 2.0 | Apr 15 | 1.00 | | 0 | 0.81 | 0.32 | | | | | | | | | 1.13 |
| | | Apr 30 | 1.00 | | | 0.66 | 0.91 | | | | | | | | | 1.57 |
| Upland rice | 2.0 | May 1 | 1.00 | | | 1.10 | 1.11 | 1.28 | 0.52 | | | | | | | 4.01 |
| | | May 15 | 1.00 | | | 0.36 | 1.04 | 1.28 | 1.32 | | | | | | | 4.00 |
| Sub- total | 6.0 | | 6.00 | | | 4.40 | 5.33 | 4.77 | 2.22 | | | | | | | 16.72 |
| Der | | | | | | | | | | | | | | | | |
| Maize | 2.0 | Sep 1 | 0.67 | | | | | | | 0.27 | 0.85 | 0.79 | 0.33 | | | 2.24 |
| | | Sep 15 | 0.67 | | | | | | | 0.11 | 0.49 | 0.79 | 0.83 | | | 2.22 |
| | | Sep 30 | 0.67 | | | | | | | | 0.23 | 0.66 | 0.98 | 0.45 | | 2.32 |
| Upland rice | 2.0 | Aug 15 | 0.67 | | | | | | 0.30 | 1.09 | 1.06 | 0.79 | | | | 3.24 |
| | | Aug 30 | 0.67 | | | | | | | 0.98 | 1.06 | 0.84 | 0.43 | | | 3.31 |
| | | Sep 15 | 0.67 | | | | | | | 0.43 | 1.01 | 0.84 | 0.98 | | | 3.26 |
| Cotton | 3.5 | Aug 1 | 1.17 | | | | | | 0.20 | 0.96 | 1.68 | 1.39 | 1.63 | 1.59 | | 7.45 |
| | | Aug 15 | 1.17 | | | | | | 0.03 | 0.57 | 1.40 | 1.39 | 1.71 | 1.78 | 0.69 | 7.57 |
| | | Aug 30 | 1.17 | | | | | | | 0.38 | 0.86 | 1.32 | 1.71 | 1.98 | 1.56 | 7.81 |
| Sesame | 2.5 | Oct 15 | 0.83 | | | | | | | | 0.08 | 0.38 | 1.23 | 0.57 | | 2.26 |
| | | Oct 30 | 0.83 | | | | | | | | | 0.10 | 1.04 | 1.28 | | 2.42 |
| | | Nov 15 | 0.83 | | | | | | | | | 0.00 | 0.53 | 1.49 | 0.58 | 2.60 |
| Sub- total | 10.0 | | 10.00 | - | - | - | - | - | 0.53 | 4.79 | 8.72 | 9.29 | 11.40 | 9.14 | 2.83 | 46.70 |
| TOTAL | | | | - | - | 4.40 | 5.33 | 4.77 | 2.75 | 4.79 | 8.72 | 9.29 | 11.40 | 9.14 | 2.83 | 63.42 |

## TABLE 5.4

### Net Irrigation Requirements ($Mm^3$) per 10 ha NCA of Houseplots

| Cropping | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Jan | Feb | Total requirement |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Misc. gu & der | - | - | 8.90 | 7.20 | 8.60 | 9.90 | 12.40 | 11.30 | 8.60 | 11.00 | 6.80 | - | 84.70 |

Note: Planting May 1st - Dec 1st

## TABLE 5.5

### Net and Gross Irrigation Requirements ($Mm^3$) per 100 ha NCA of Combined Field Crops and Houseplots

| Cropping | NCA (ha) | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Jan | Feb | Total requirement |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Net Field Requirements** | | | | | | | | | | | | | | |
| Field crops | 93.63 | - | - | 41.20 | 49.90 | 44.66 | 25.75 | 44.85 | 81.65 | 86.98 | 106.74 | 85.78 | 26.50 | 594.01 |
| House-plots | 6.37 | - | - | 5.67 | 4.59 | 5.48 | 6.31 | 7.90 | 7.20 | 5.48 | 7.01 | 4.33 | - | 53.97 |
| TOTAL | 100.00 | - | - | 46.87 | 54.59 | 50.14 | 32.06 | 52.75 | 88.85 | 92.46 | 113.75 | 90.11 | 26.50 | 647.98 |
| **Gross Field Requirements*** | | | | | | | | | | | | | | |
| Field crops | 93.63 | - | - | 68.67 | 83.17 | 74.43 | 42.92 | 74.75 | 136.08 | 144.97 | 177.90 | 142.97 | 44.17 | 990.03 |
| House-plots | 6.37 | - | - | 9.45 | 7.65 | 9.13 | 10.52 | 13.17 | 12.00 | 9.13 | 11.68 | 7.22 | - | 89.95 |
| TOTAL | 100.00 | - | - | 78.12 | 90.82 | 83.56 | 53.44 | 87.92 | 148.08 | 154.10 | 189.58 | 150.19 | 44.17 | 1 079.98 |

Note: * Gross field requirement at distributary outlet assuming a 60% field water distribution and application efficiency

## TABLE 5.6

### Results of Atterberg Limit Tests on Project Soils

| Location of sample, and description | Mean moisture content (%) | Liquid limit (%) | Plastic limit (%) | Plasticity index (%) | Shrinkage limit (%) |
|---|---|---|---|---|---|
| Site of Intake | | | | | |
| 1. Medium-dark brown fissured clay - 0.9 m deep | 31.80 | 54.00 | 28.92 | 32.56 | 20.08 |
| 2. Black-brown clay with shells - 1.7 m deep | 29.72 | 48.00 | 20.08 | 27.92 | 11.48 |
| New Benchmark E41 | | | | | |
| 3. Black fissured clay - 0.9 m deep | 23.55 | 78.00 | 43.82 | 34.18 | 15.54 |
| 4. Light brown silty clay - 1.7 m deep | 30.16 | 65.50 | 32.56 | 32.94 | 17.27 |

Vertisols are characterised by the formation of deep shrinkage cracks when they dry out and subsequent expansion when they are wetted again. Foundation problems with buildings are common and great care must be taken with the design of the buildings for the project since uneven settlement may occur if the foundations are not adequate.

Tests were carried out on the gypsum content of various soils and the results are given in Annex I, Table C.4. The highest percentage of gypsum recorded was 1.17 but only two other samples contained more than 0.1% gypsum. There is no risk of gypsum being leached out of the soil and changing the engineering properties.

## 5.12 Sulphate Attack on Concrete

In all deep structures, there is a risk of attack on the concrete by soluble sulphates. Various measures may be taken to overcome this problem and these are summarised in Table 5.7. This table is based on experience obtained in the United Kingdom and some experts consider that in the Middle East each class should be downgraded by one step to allow for more extreme conditions.

## TABLE 5.7 - Requirements for Concrete Exposed to Sulphate Attack

| Class | Concentration of sulphates expressed as $SO_3$ | | | Type of cement | Requirements for dense, fully compacted concrete made with aggregates complying with the requirements of BS 882 or BS 1047 | | | |
|---|---|---|---|---|---|---|---|---|
| | In soil | | In ground water | | Minimum cement content | | | Maximum free water/cement ratio |
| | Total $SO_3$ | $SO_3$ in 2 : 1 water extract | | | Nominal maximum size of aggregate (mm) | | | |
| | | | | | 40 | 20 | 10 | |
| | % | g/l | Parts per 100 000 | | $kg/m^3$ | $kg/m^3$ | $kg/m^3$ | |
| 1 | Less than 0.2 | — | Less than 30 | Ordinary Portland or Portland-blastfurnace | 240 | 280 | 330 | 0.55 |
| 2 | 0.2 to 0.5 | — | 30 to 120 | Ordinary Portland or Portland-blastfurnace | 290 | 330 | 380 | 0.50 |
| | | | | Sulphate-resisting Portland | 240 | 280 | 330 | 0.55 |
| | | | | Supersulphated | 270 | 310 | 360 | 0.50 |
| 3 | 0.5 to 1.0 | 1.9 to 3.1 | 120 to 250 | Sulphate-resisting Portland or supersulphated | 290 | 330 | 380 | 0.50 |
| | | | | High alumina | 290 | 330 | 380 | 0.45 |
| 4 | 1.0 to 2.0 | 3.1 to 5.6 | 250 to 500 | Sulphate-resisting Portland or supersulphated | 330 | 370 | 420 | 0.45 |
| | | | | High alumina | 300 | 340 | 410 | 0.40 |
| 5 | Over 2 | Over 5.6 | Over 500 | Sulphate-resisting Portland or supersulphated plus adequate protective coatings | 330 | 370 | 420 | 0.45 |
| | | | | High alumina | 330 | 370 | 420 | 0.35 |

NOTE 1. This table applies only to concrete made with aggregates complying with the requirements of BS 882 or BS 1047 placed in near-neutral groundwaters of pH 6 to pH 9, containing naturally occurring sulphates but not contaminants such as ammonium salts. Concrete prepared from ordinary or sulphate-resisting Portland cement would not be recommended in acidic conditions (pH 6 or less). High alumina cement can be used down to pH 4.0 and supersulphated cement has given an acceptable life provided that the concrete is dense and prepared with a free water/cement ratio of 0.40 or less, in mineral acids down to pH 3.5.

NOTE 2. The cement contents given in Class 2 are the minima recommended by the manufacturers. For $SO_3$ contents near the upper limit of Class 2 cement contents above these minima are advised.

NOTE 3. Where the total $SO_3$ in column 2 exceeds 0.5 % then a 2 : 1 water extract may result in a lower site classification if much of the sulphate is present as low solubility calcium sulphate. Reference should be made to BRS Digest 90 : 1970.

NOTE 4. For severe conditions, e.g. thin sections, sections under hydrostatic pressure on one side only and sections partly immersed, consideration should be given to a further reduction of water/cement ratio and, if necessary, an increase in cement content to ensure the degree of workability needed for full compaction and thus minimum permeability.

NOTE 5. The prescribed mixes given in Table 50 may be used where appropriate to meet the requirements of this table and for this purpose the maximum free water/cement ratios of grades 20, 25 and 30 shall be taken as 0.55, 0.50 and 0.45 respectively.

Source: British Standards Institution CP 110 : Part I, 1972.

The results of the chemical analyses of soils are given in Table C.2, Annex I. These show that the highest concentration of $SO_4$ ions is 113.8 milliequivalents per litre (meq/l) and the maximum in the Qoryooley series was 36.9 meq/l. The conversion factors are as follows:-

(a) to convert from meq/l to mg/l, multiply by 48.03.

(b) to convert from mg/l to percentage multiply by 0.0001.

(c) to convert from $SO_4$ concentrations to equivalent $SO_3$ concentrations, multiply by 0.833.

Therefore, the combined conversion factor is 48.03 x 0.0001 x 0.833 i.e. 0.004.

Therefore the highest concentrations of soluble sulphates in the various soil series vary from 0.15 to 0.46% expressed as $SO_3$. This places the soils mainly in Class 3. The chemical analyses of groundwater samples are given in Annex II, Appendix G. The worst sample had an $SO_4$ concentration of 62.50 meq/l with the median concentration being 20 meq/l. The conversion factors are similar to those mentioned above except that it is required to convert the concentration into parts per 100 000. The conversion factor then becomes 4.00. Therefore the median and maximum concentration of soluble sulphates in groundwater become 250 and 80 parts per 100 000 expressed as $SO_3$. Comparison with Table 5.7 shows that the groundwater concentrations are in Class 2 and Class 3.

The worst concentration recorded for Shabeelle river water is 11.1 meq/l of $SO_4$ (see Table 4.4, Annex II). Therefore the salinity of groundwater becomes the overriding factor. It can be seen that the worst case of soils and water is on the limit between Class 3 and Class 4. There is no guarantee that the samples selected for analysis are the worst which will be encountered and, in view of the recommendations of increased precautions for the Middle East area, the levels of protection should be equivalent to those recommended for Class 4 or Class 5.

For the purpose of costing this report, the precautions recommended in Table 5.7 for Class 5 concentrations have been used for all drain structures. These recommendations should be adhered to until there is proof that they are not required in particular situations.

# CHAPTER 6

# CANALISATION

## 6.1 Definitions

The canal system proposed for the project, in the case of all water supply being from Gayweerow barrage, can be broken down into the following stages:-

Inlet Channel

This runs from the offtake point on the river (550 m upstream of the barrage site) to the heads of the branch canals, a total distance of 900 m. The section is designed oversized so that it will act as a sedimentation basin.

Branch Canals

Two branch canals take off from the tail of the inlet channel. These are the Tawakal branch canal, which runs parallel to the Asayle canal as far as Tawakal and then sweeps across the northern boundary of the Project Area almost as far as Tugarey, and the Gayweerow branch canal which follows the line of the river, passing the villages of Gayweerow and Jasiira.

Distributary Canals

The branch canals provide water to the distributary canals, in almost all cases with the assistance of small irrigation pump stations situated at the head of the distributaries. Flow in the distributaries is restricted to day-time only so that irrigation during the night-time is not necessary (Section 2.8).

Night Storage Reservoirs

At the heads of distributaries, reservoirs have been provided to store the continuous flow of the branch canal during the night, when the distributary canals are closed. The stored water is then released the next day into the distributary canal, together with the normal flow from the branch canal. The reservoirs also play an important role in village and livestock water supplies.

Watercourses

Water is fed from the distributary canals, through distributary outlets, into permanent watercourses. These are the basic water distribution channels within the field units, one field unit being fed by one watercourse.

## 6.2 Design Watering Rate and Irrigation Day

Although the maximum gross irrigation requirement of the mixed cropping pattern at a distributary outlet is only 0.71 l/s/ha (i.e. from Table 5.5 the requirement per 100 ha NCA for December is 189.58 $Mm^3$), to be fully flexible the system must be able to accommodate the requirement of the largest single crop demand. This is the January gross irrigation requirement of sesame (planted November 15th) of 1.11 l/s/ha (equivalent to a net irrigation requirement for the complete month of 179 mm; see Annex VI, Table 2.7).

This figure has been increased by 15% to allow for a margin of safety, producing the final continuous design watering rate at a distributary outlet of 1.275 l/s/ha.

Because of the social pressures and practical difficulties caused by night-time irrigation, only day-time watering is recommended. It is essential, however, that full use of the daylight hours is made so that scheduling can be organised correctly. Therefore a 12 hour irrigation day has been adopted. The actual design watering rate at a distributary outlet during the irrigation day is therefore 2.55 l/s/ha.

## 6.3 Inlet Channel

Continuous free flow of water from the river is allowed into the channel and no head regulator structure needs to be provided. Water levels can be controlled by the correct adjustment of the new barrage, and the discharge in the channel can be governed by the setting of the branch canal head regulators. The only structure along the length of the channel is the inverted siphon which passes the tail waters of the Asayle canal underneath the inlet channel.

The design discharge of the channel is 5.8 $m^3/s$: this allows for the normal continuous watering rate over the complete project, together with 3% losses in the distributary canals and 6% losses in the branch canals.

The main function of the inlet channel is to act as a sedimentation basin to exclude much of the heavy suspended sediment load of the river from the main irrigation system. This will reduce the amount of weed and silt clearance needed from the branch canals and distributary canals to keep them operating satisfactorily. Indeed, without adequate sedimentation facilities it would be difficult to install and operate night storage reservoirs as the quiescent reservoir water would deposit the sediment load onto the bed, rapidly silting the complete basin. The requirements for, and the design of, sedimentation basins are dealt with in Section 2.9.

Of the total channel length of 900 m, only 800 m can be used for sedimentation purposes as the inverted siphon needs to be as short as possible to reduce costs; therefore the channel width is constricted at this point to the normal Lacey width of about 9 m. The width of the sedimentation basin itself is limited by the maximum reach of the machine used to remove the deposited sediment. The use of a rope operated dragline is recommended and with a 15 m boom it is feasible to clear a channel of 20 m bed width (with operation from both banks) to a depth of 4 m below the bank level. This is based on channel side slopes of 1 : 2 and the assumption that the dragline operator is skilled enough to throw the bucket 5 m beyond the boom point.

The sedimentation basin, therefore, has an effective surface area of at least 16 000 $m^2$. Using Table 2.4, this would indicate that the basin is large enough to settle 99% of particles with settling velocities of 2.29 mm/s. This is equivalent to a particle size of the sediment of 0.05 mm. If, however, a particle size of only 0.01 mm is considered, the settling velocity falls dramatically to 0.092 mm/s and the percentage removal drops to only 22%. Taking the particle size distribution shown in Figure 2.4, 80% of the canal headworks sample particles are coarser than 0.05 mm and these will all settle fully. If 22% average removal of the remainder is assumed, then the total percentage of silt removed is 84. Allowing for minor irregularities and short circuiting of flow, it is expected that 80% removal should be attainable, if particle size distributions are similar to the canal headworks curve shown in Figure 2.4.

The required depth of flow to restrict water velocities to below the critical value at which scour of the settled sediment will occur is calculated to be 2.25 m (see Section 2.9). Taking water level to be 69.11 m (holding level of the barrage) this gives a maximum bed level (i.e. just before clearance) of 66.85 m. The bank top level is determined by the river flood as there is no head regulator to the channel: this is 70.02 m at this point, making the bank top to bed level difference in height 3.16 m. If the working depth of the dragline is limited to 4.0 m then the maximum depth of overexcavation possible during silt clearance is 0.84 m.

During the peak flow period (December) the average silt load is expected to be around 3 000 ppm, contained in a flow of about 280 000 $m^3$/d. Assuming a porosity of the settled sediment of 0.4, 423 $m^3$ of sediment will be deposited every day if 80% removal is achieved in the basin. The available capacity of the basin is 13 440 $m^3$ (i.e. 0.84 x 800 x 20) and therefore the nominal maximum clearing rate is once every 31 days.

## 6.4 Asayle Canal

With the proposed system, although some of the land irrigated from the Asayle canal is encompassed, the canal itself is not affected at all, apart from the provision of an inverted siphon to take the canal tail waters underneath the inlet channel. This will allow the continued disposal of excess irrigation water in the Asayle canal back into the river. During the present investigations only minor tail water discharges in the canal were observed and a design flow of 0.5 $m^3$/s for the inverted siphon is regarded as ample.

## 6.5 Branch Canals

The two branch canals had several constraints imposed upon them which gave little choice in the selection of their exact lines. It was necessary to keep both of them, as far as possible, on the outside edge of the Project Area so that they remained on the highest ground. In addition, for the Gayweerow branch the village of Gayweerow proved to be a major obstacle that had to be avoided; the final length of this branch was 6 550 m. With the Tawakal branch the Asayle canal determined the line of the first 5 500 m; the two canals run parallel with a gap of 90 m between centre lines. This gives sufficient space for the construction of a primary road if it were to prove necessary at some time in the future. The remainder of the Tawakal branch, up to a total length of 13 360 m, was mostly fixed by maintaining the highest ground levels, although the village of Tawakal also influenced the line.

The complete lengths of both branch canals have been surveyed at 100 m intervals and the long sections have been drawn (Annex X), showing details of ground levels, water levels, bed levels and cross regulation points. Also all the intersection points of the canals have been located and benchmarks emplaced.

The cross-sections are designed to be in cut and fill, and trapezoidal with one in two side slopes. Figure 6.1 shows a typical cut and fill section of a branch canal.

The hydraulic design is based entirely upon the Lacey regime equations as described in Section 2.7, using a width factor of 1.00. Table 6.1 shows the output results from an example computer design of the Gayweerow branch canal; all canals have been designed in this way. The important feature of the canal as designed is the balance between the total volume of cut and the total volume of fill. This ensures that only minimal haul is necessary for construction and the total earthworks required are minimal.

The same balance has been achieved for the Tawakal branch canal where the total of cut or fill is about 89 500 $m^3$. The use of a balanced section, together with a controlled drop in water level at chainage 5 500 of 0.85 m has made it possible to overcome the awkward ground level profile of the canal downstream of Tawakal to produce an efficient solution.

Water control along the canals is achieved with the use of movable weir cross regulators at chainages 1 020 and 4 100 on the Gayweerow branch and 2 970, 5 500, 9 810 and 12 100 on the Tawakal branch. These are situated at the grouped offtakes for the distributary canals so that close water control can be maintained at these points. Figure 6.2 shows a section through a typical movable weir regulator. Because of the possibility of large variations in the inlet channel water level, weir regulators are not suitable for branch canal regulators and simple vertical lift sluice gates have been used instead.

There is a slight risk that, due to pump failures, the level in the branch canals would rise enough to cause some overtopping of the banks before the head regulator could be closed down. Consequently water escape structures, formed from simple fixed concrete weirs and buried pipes, have been included to pass excess water into the surface drainage system. Because the water levels are often close to ground level and distributary canals block possible escape routes, the choice of location for water escapes is very limited. A total of five has been included, at chainages 2 920 and 5 480 on the Gayweerow branch and at chainages 2 620, 8 200 and 13 050 on the Tawakal branch. The surface drains that receive escape water have been enlarged to accommodate the larger flow.

During the survey work of the branch canal lines, 2.5 m deep auger holes were made at approximately 1 km intervals to test for the presence of sandy deposits at shallow depths. Table 6.2 gives the depths at which the very first signs of any sandy deposits were noted. The minimum depth at which they were encountered was 1.00 m and as the bed on the branch canals is nearly always less than 1.0 m below ground level the deposits should not present any major problems. Indeed the clay content of the sandy horizons was often high and, therefore, seepage losses through these should not be excessive.

However, there are three sections of canal where the bed level is significantly deeper, namely downstream of the Gayweerow cross regulator at chainage 1 020 (1.53 m deep), downstream of the Tawakal cross regulator at chainage 5 550 (1.62 m deep) and around chainage 12 100 on the Tawakal branch (1.72 m deep). There is a chance, therefore, that a more permeable sand lens will be encountered. If this proves to be the case then clay lining of short sections of canal may have to be undertaken.

FIGURE 6.1

## CANAL CROSS SECTIONS

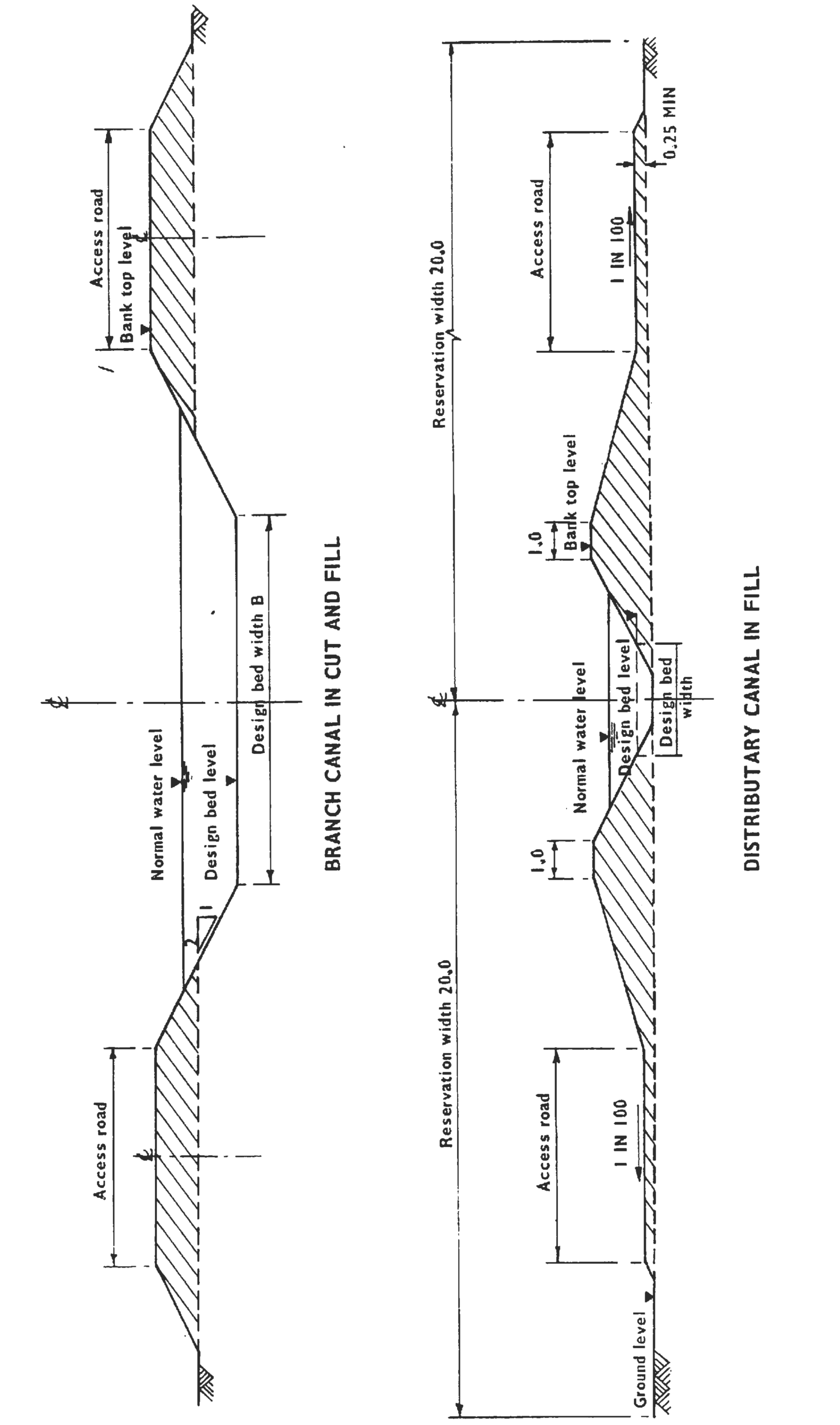
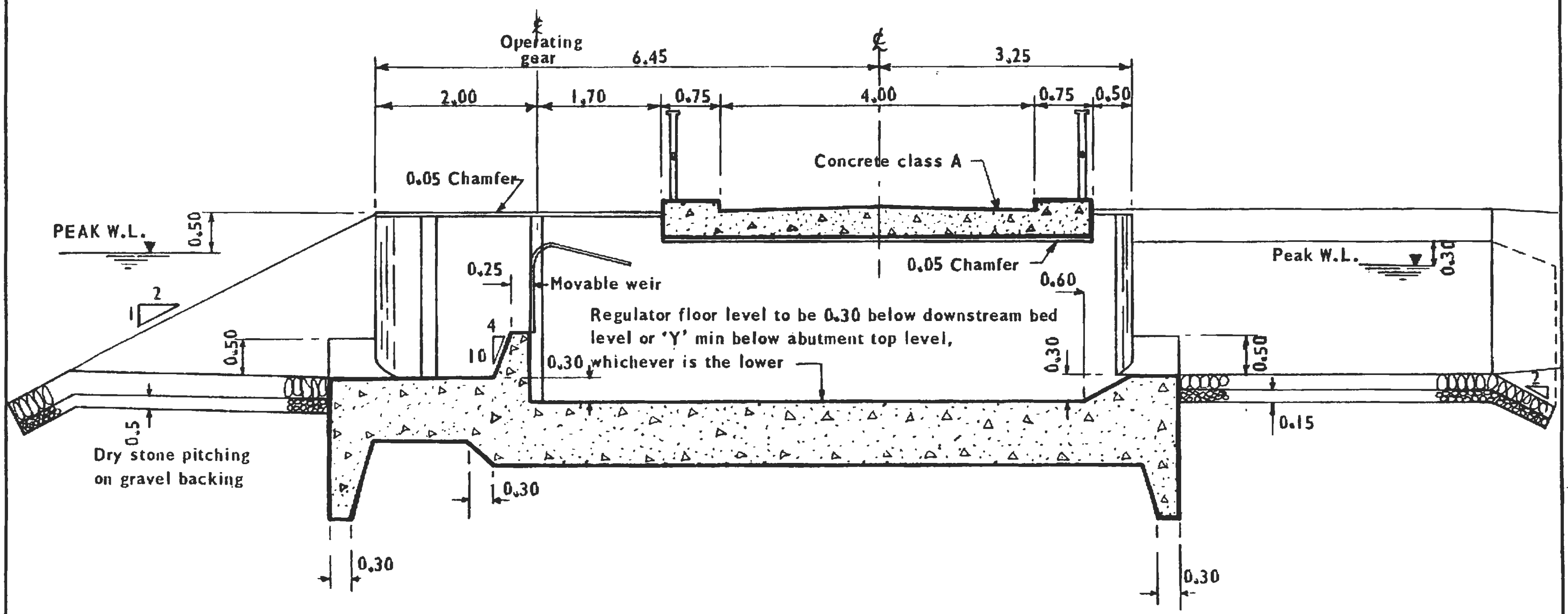


FIGURE 6.2




MOVABLE WEIR REGULATOR

TABLE 6.1

## Branch Canal Design Example

INPUT DATA FOR CANAL DESIGN

| DEVELOPMENT STAGE | CANAL NAME | WIDTH FACTOR |
|---|---|---|
| FEASIBILITY | GAYWEROW | 1.00 |

| STRUC NAME | CHAINAGE | OFFTAKE DISCHARGE | MINIMUM UPSTREAM WL | HEAD LOSS |
|---|---|---|---|---|
| 1 G10/G12 | 6120 | 1.004 | 65.75 | 0.00 |
| 2 G8 | 4100 | 0.438 | 0.00 | 0.48 |
| 3 G2/G4/G6 | 1020 | 1.011 | 0.00 | 1.13 |
| 4 HEAD | 0 | 0.000 | 0.00 | 0.00 |

| REACH | SILT FACTOR |
|---|---|
| 1 | 0.6 |
| 2 | 0.6 |
| 3 | 0.6 |

OUTPUT CANAL DESIGN

| DEVELOPMENT STAGE | CANAL NAME | WIDTH FACTOR |
|---|---|---|
| FEASIBILITY | GAYWEROW B.C. | 1.00 |

F = LACEY SILT FACTOR B = DESIGN BED WIDTH

| CHAINAGE | DESIGN W.L. U/S | DESIGN W.L. D/S | NORMAL DISCHARGE U/S | NORMAL DISCHARGE D/S | F | B | BED LEVEL U/S | BED LEVEL D/S | BANK TOP LEVEL U/S | BANK TOP LEVEL D/S |
|---|---|---|---|---|---|---|---|---|---|---|
| 6120 | 65.75 | 0.00 | 1.004 | 0.000 | 0.6 | 3.92 | 65.13 | 0.00 | 66.18 | 0.00 |
| 4100 | 66.52 | 66.04 | 1.466 | 1.028 | 0.6 | 4.75 | 65.81 | 65.41 | 66.98 | 66.47 |
| 1020 | 68.06 | 66.93 | 2.522 | 1.511 | 0.6 | 6.16 | 67.21 | 66.22 | 68.57 | 67.39 |
| 0 | 68.18 | 68.18 | 2.541 | 2.541 | 0.0 | 0.00 | 0.00 | 67.33 | 0.00 | 68.69 |

OUTPUT EARTHWORK QUANTITIES

| DEVELOPMENT STAGE | CANAL NAME |
|---|---|
| FEASIBILITY | GAYWEROW B.C. |

| CHAINAGE FROM | CHAINAGE TO | BED WIDTH | VOLUME OF CUT | VOLUME OF FILL | VOLUME (CUT-FILL) |
|---|---|---|---|---|---|
| 6120 | 4100 | 3.92 | 8123 | 8307 | -184 |
| 4100 | 1020 | 4.75 | 15802 | 15881 | -79 |
| 1020 | 0 | 6.16 | 6739 | 6480 | +260 |
| | TOTALS: | | 30665 | 30668 | -3 |

TOTAL OF POSITIVE VALUES OF CUT MINUS FILL = +13832
TOTAL OF NEGATIVE VALUES OF CUT MINUS FILL = -13835

## TABLE 6.2

### Depth to Sandy Deposits

| Gayweerow branch canal chainage (m) | Depth to sandy deposits* (m) | |
|---|---|---|
| 0 | Greater than | 2.50 |
| 845 | Greater than | 2.50 |
| 1 840 | | 1.50 |
| 2 840 | Greater than | 2.50 |
| 3 290 | | 1.25 |
| 4 240 | | 2.00 |
| 4 833 | | 1.00 |
| 5 419 | | 1.00 |
| 6 452 | | 2.00 |
| **Tawakal branch canal chainage (m)** | | |
| 0 | Greater than | 2.50 |
| 1 805 | | 2.25 |
| 2 805 | Greater than | 2.50 |
| 3.805 | | 1.50 |
| 4 785 | | 1.00 |
| 5 785 | | 1.50 |
| 6 380 | Greater than | 2.50 |
| 7 380 | Greater than | 2.50 |
| 8 475 | Greater than | 2.50 |
| 13 365 | Greater than | 2.50 |

Note: * The depth is that at which the very first signs of sandy deposits, usually fine sands and clay, were encountered. Above this depth the soil is entirely clay.

## 6.6 Distributary Canals

A total number of 15 distributary canals are provided, three of which subdivide into two sections. The name of each canal is formed from a letter, either T, for distributaries on the Tawakal branch, or G, for those on the Gayweerow branch, and a number. The numbers start at the head of the branch with 1 and are ordered consecutively, with odd numbers reserved for distributaries on the left bank of the branch canals and even numbers on the right bank. Where a distributary splits, the left branch is given an additional number 1, and the right bank a number 2. The broken, pocketed nature of the low lying areas and the occurrence of gentle but well defined ridges between them, makes the choice of distributary canal line relatively straightforward. Only in a few areas were the lines difficult to position and here optimum field layouts were used to assist the selection. Table 6.3 gives the details of all the distributary canals together with the NCA that each one serves.

## TABLE 6.3

### Distributary Canals

| Branch canal | Chainage of offtake point (m) | Distributary canal name | Length (m) | NCA (ha) |
|---|---|---|---|---|
| Tawakal | 2 970 | T 1 | 1 860 | 185 |
| | 5 500 | T 3 | 2 680 | 184.5 |
| | 5 500 | T 5 | 800 | 31.5 |
| | - | T 5.1 | 3 450 | 317 |
| | - | T 5.2 | 900 | 40 |
| | 9 810 | T 7 | 1 240 | 145 |
| | 9 810 | T 9 | 1 850 | 236 |
| | 12 100 | T 2 | 1 850 | 21 |
| | - | T 2.1 | 1 520 | 177 |
| | - | T 2.2 | 4 100 | 482 |
| | 12 100 | T 4 | 2 550 | 217 |
| | 13 200 | T 11 | 1 550 | 168 |
| | 13 200 | T 13 | 2 460 | 125.5 |
| Sub-total | | | 26 810 | 2 329.5 |
| Gayweerow | 1 020 | G 2 | 3 400 | -* |
| | 1 020 | G 4 | 3 850 | 263 |
| | 1 020 | G 6 | 4 000 | 284 |
| | 4 100 | G 8 | 1 850 | 327 |
| | 6 120 | G 10 | 880 | 128 |
| | - | G 10.1 | 1 250 | 157 |
| | - | G 10.2 | 1 750 | 111 |
| | 6 120 | G 12 | 3 140 | 364+ |
| Sub-total | | | 20 120 | 1 634 |
| TOTAL | | | 46 930 | 3 963.5 |

Notes: * G2 serves the existing agriculture in the land areas classified as unsuitable close to the Asayle canal.
+ including pilot farm (137 ha).

G2 is rather different from all the other distributaries because it provides water solely to the lands classified as unsuitable for intensive irrigation. However, significant existing agriculture occurs in this area to which water supplies must be maintained. G2 will not only provide the supply, but give the farmers a much more reliable source of water than the existing offtakes from the Asayle canal. G2 will be given sufficient distributary outlets adequately to supply the existing minor canals.

Hydraulic design of the distributaries has, as outlined in Section 2.7, been based on the Lacey regime equations with a width factor in this case of 0.83. The calculations have been carried out by a computer and Table 6.4 is the output sheet for the design of G8 together with the corresponding earthworks requirements. Figure 6.1 shows a typical distributary canal cross-section; they are almost entirely in fill with only very small quantities of cut occurring.

Water regulation at the distributary heads is dealt with in Section 6.7 as an integral part of the night storage reservoir operation. Cross regulation down the distributary is achieved by means of simple pipe regulators with vertical screw rod operated gates (Figure 6.3). The regulators have to be closely spaced so that accurate water levels and discharges through the distributary outlets to the watercourse can be achieved; the maximum distance a distributary outlet can be upstream of the controlling cross regulator has been set at 1 000 m. The distributary outlets themselves are also simple piped structures with vertical sliding gates. The diameter of the pipe is determined by the stream size of the watercourse. Up to 60 l/s, a 0.30 m diameter is sufficient, but for greater discharges up to a maximum of 100 l/s, a 0.375 m diameter pipe is necessary.

There is always the risk that a heavy rainstorm will force the farmers to stop irrigating suddenly and close the distributary outlets. Before the flow in the distributary canal can be reduced, the excess flow must be passed into the surface drainage system and each distributary is provided with a tail water escape structure to permit this.

## 6.7 Night Storage

Night-time irrigation is generally not practised in Somalia and because of the technical difficulties and social pressures it would impose, irrigation of the Qoryooley project land during the night has not been recommended. However, continuous flow in the branch canal is necessary as they are too long to open and close each day, and this makes the provision of night storage facilities essential. In Section 2.8, alternative methods of night storage are considered, namely:-

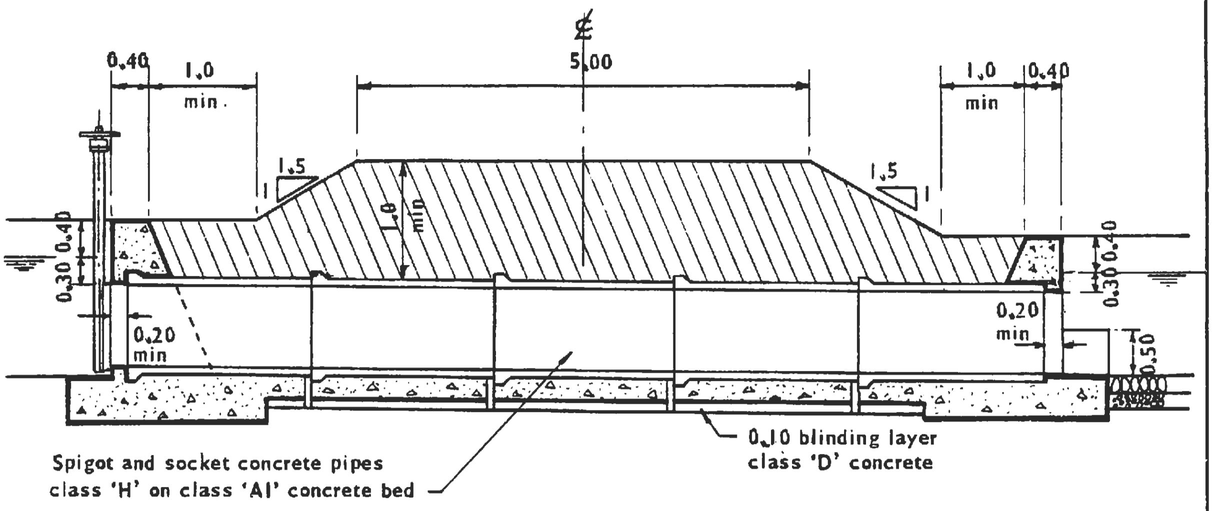
- onstream distributary canal storage
- offstream distributary head reservoir storage.

The former method can only be undertaken where the total fall along the distributary is not greater than about 0.8 m (unless special cross regulators are employed) because of the storage problems imposed by large differences in level. However, in the Project Area, level differences of over 1.5 m occur and distributary canals up to 4.0 km long are planned; therefore the option of onstream distributary canal storage was rejected because it would require special cross regulators and distributary head reservoir storage was selected.

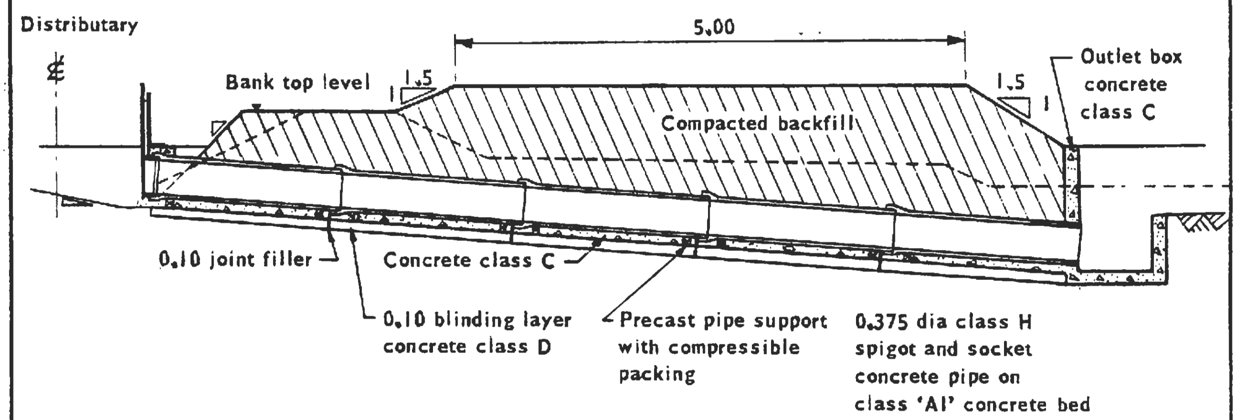
By careful selection of distributary offtake points, and grouping as far as possible, the number of night storage reservoirs has been limited to eight. Figure 6.4 shows the typical layout of the reservoir regulator group. Water is either fed by gravity over a movable weir regulator, or pumped from the branch canal into the diversion pool. During the night, the pipe regulator at the head of the distributary canal is closed and the continuous flow from the branch canal is passed into the reservoir. During the day the canal head pipe regulator

FIGURE 6.3

## DISTRIBUTARY CANAL STRUCTURES

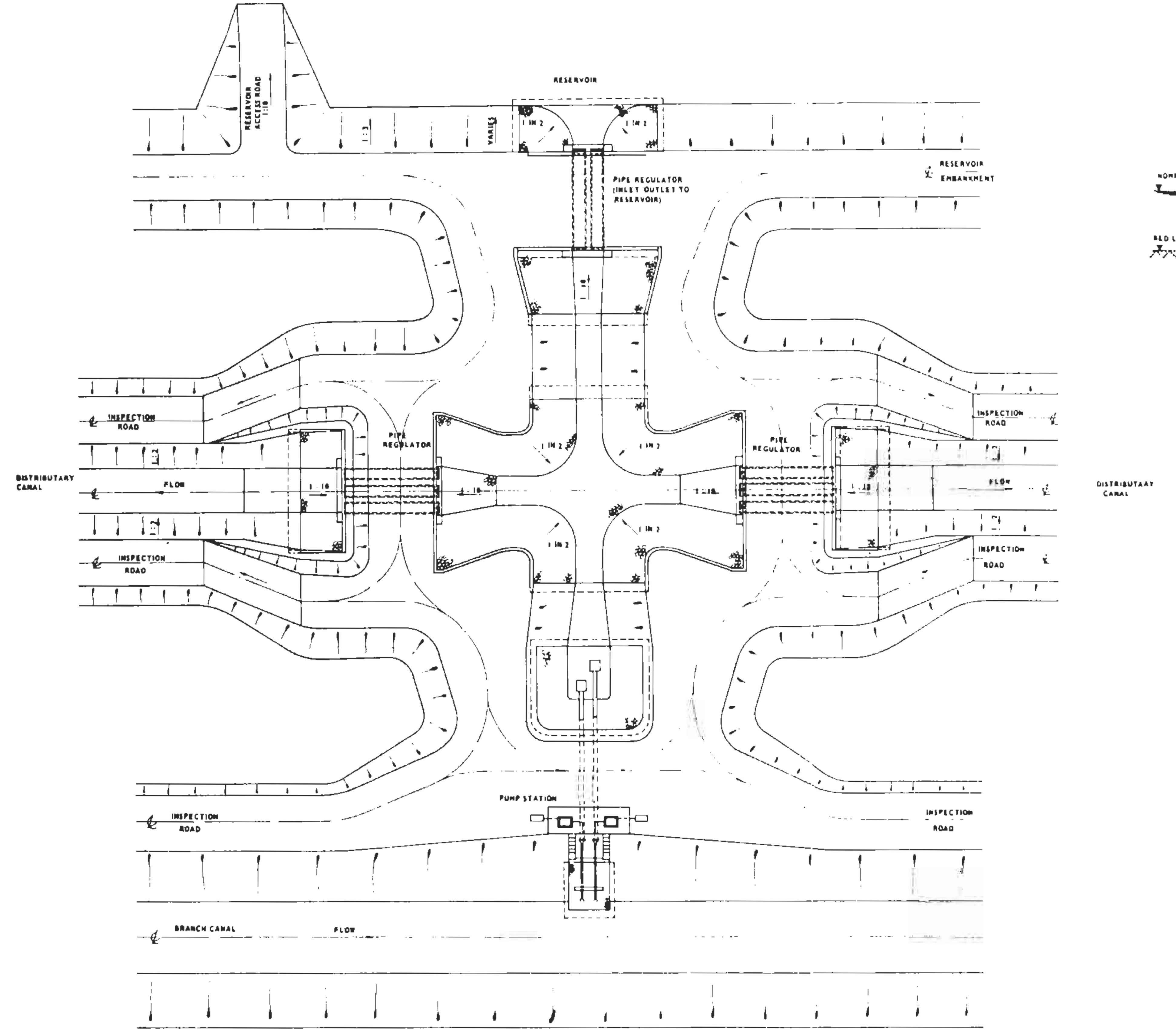



PIPE REGULATOR



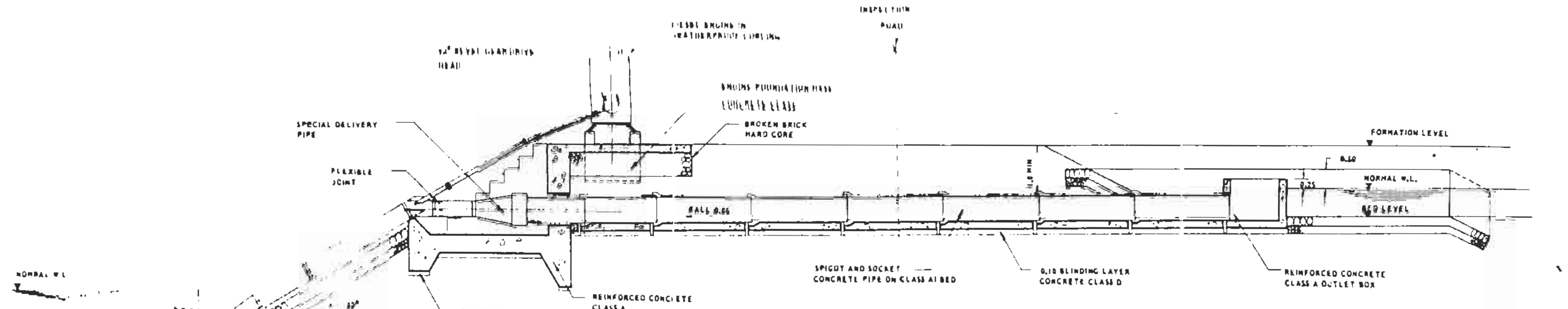

DISTRIBUTARY OUTLET

FIGURE 6.4



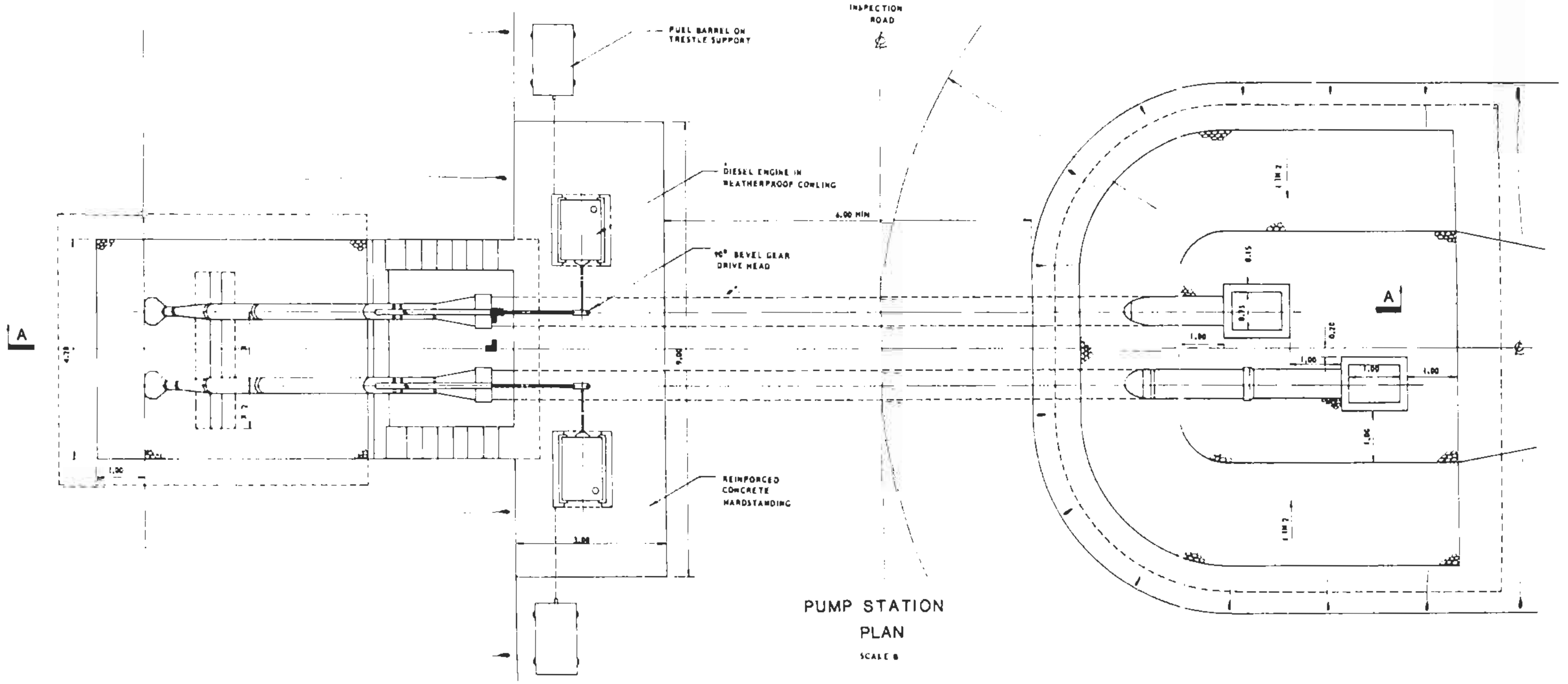

TYPICAL RESERVOIR REGULATOR GROUP & PUMP STATION LAYOUT

SCALE A




SECTION A - A

SCALE B




PUMP STATION PLAN

SCALE B

SCALES

A

B

## TABLE 6.4

### Distributary Canal Design Example

INPUT DATA FOR CANAL DESIGN

| DEVELOPMENT STAGE | CANAL NAME | WIDTH FACTOR |
|---|---|---|
| FEASIBILITY | G8 | 0.83 |

| STRUC NAME | | CHAINAGE | OFFTAKE DISCHARGE | MINIMUM UPSTREAM WL | HEAD LOSS |
|---|---|---|---|---|---|
| 1 | TAIL | 1850 | 0.120 | 66.14 | 0.00 |
| 2 | OFFTAKE | 1530 | 0.095 | 66.25 | 0.00 |
| 3 | OFFTAKE | 1230 | 0.054 | 66.30 | 0.00 |
| 4 | X REG | 910 | 0.089 | 66.25 | 0.14 |
| 5 | OFFTAKE | 600 | 0.100 | 66.65 | 0.00 |
| 6 | X REG | 300 | 0.110 | 67.15 | 0.40 |
| 7 | HEAD | 0 | 0.000 | 0.00 | 0.00 |

| REACH | SILT FACTOR |
|---|---|
| 1 | 0.8 |
| 2 | 0.8 |
| 3 | 0.8 |
| 4 | 0.8 |
| 5 | 0.8 |
| 6 | 0.6 |

OUTPUT CANAL DESIGN

| DEVELOPMENT STAGE | CANAL NAME | WIDTH FACTOR |
|---|---|---|
| FEASIBILITY | G8 | 0.83 |

F = LACEY SILT FACTOR B = DESIGN BED WIDTH

| CHAINAGE | DESIGN W.L. U/S | DESIGN W.L. D/S | NORMAL DISCHARGE U/S | NORMAL DISCHARGE D/S | F | B | BED LEVEL U/S | BED LEVEL D/S | BANK TOP LEVEL U/S | BANK TOP LEVEL D/S |
|---|---|---|---|---|---|---|---|---|---|---|
| 1850 | 66.14 | 0.00 | 0.120 | 0.000 | 0.8 | 1.55 | 65.79 | 0.00 | 66.54 | 0.00 |
| 1530 | 66.25 | 66.25 | 0.216 | 0.121 | 0.8 | 1.55 | 65.87 | 65.90 | 66.65 | 66.65 |
| 1230 | 66.35 | 66.35 | 0.272 | 0.218 | 0.8 | 1.68 | 65.93 | 65.96 | 66.75 | 66.75 |
| 910 | 66.59 | 66.45 | 0.363 | 0.274 | 0.8 | 1.94 | 66.13 | 66.03 | 66.99 | 66.85 |
| 600 | 66.67 | 66.67 | 0.465 | 0.365 | 0.8 | 2.19 | 66.18 | 66.22 | 67.07 | 67.07 |
| 300 | 67.15 | 66.75 | 0.578 | 0.468 | 0.6 | 2.44 | 66.57 | 66.26 | 67.55 | 67.15 |
| 0 | 67.20 | 67.20 | 0.580 | 0.580 | 0.0 | 0.00 | 0.00 | 66.61 | 0.00 | 67.60 |

| STRUCTURE NAME | CHAINAGE | WATER SURFACE SLOPE |
|---|---|---|
| TAIL | 1850 | 0.000358 |
| OFFTAKE | 1530 | 0.000315 |
| OFFTAKE | 1230 | 0.000300 |
| X REG | 910 | 0.000282 |
| OFFTAKE | 600 | 0.000268 |
| X REG | 300 | 0.000163 |
| HEAD | 0 | 0.000000 |

## TABLE 6.4 (cont.)

OUTPUT EARTHWORK QUANTITIES
------ --------- ----------

DEVELOPMENT STAGE — FEASIBILITY

CANAL NAME — G8

| CHAINAGE FROM | CHAINAGE TO | BED WIDTH | CUT AV. DEPTH | VOLUME OF CUT | BANK AV HEIGHT | VOLUME OF FILL | VOLUME (CUT-FILL) |
|---|---|---|---|---|---|---|---|
| 1850 | 1690 | 1.55 | 0.00 | 0 | 0.94 | 866 | -866 |
| 1690 | 1530 | 1.55 | 0.00 | 0 | 1.00 | 967 | -967 |
| 1530 | 1500 | 1.55 | 0.00 | 0 | 1.00 | 181 | -181 |
| 1500 | 1350 | 1.55 | 0.00 | 0 | 0.92 | 789 | -789 |
| 1350 | 1230 | 1.55 | 0.00 | 0 | 1.00 | 719 | -719 |
| 1230 | 1215 | 1.68 | 0.00 | 0 | 1.00 | 90 | -90 |
| 1215 | 1055 | 1.68 | 0.00 | 0 | 1.01 | 973 | -973 |
| 1055 | 910 | 1.68 | 0.00 | 0 | 1.00 | 863 | -863 |
| 910 | 755 | 1.94 | 0.00 | 0 | 1.14 | 1151 | -1151 |
| 755 | 600 | 1.94 | 0.43 | 189 | 0.42 | 242 | -53 |
| 600 | 565 | 2.19 | 0.47 | 52 | 0.42 | 55 | -3 |
| 565 | 485 | 2.19 | 0.05 | 10 | 0.84 | 361 | -351 |
| 485 | 370 | 2.19 | 0.28 | 89 | 0.62 | 316 | -227 |
| 370 | 300 | 2.19 | 0.34 | 69 | 0.55 | 163 | -94 |
| 300 | 150 | 2.44 | 0.03 | 13 | 0.95 | 831 | -818 |
| 150 | 0 | 2.44 | 0.00 | 0 | 1.00 | 903 | -903 |
| | | TOTALS: | | 423 | | 9471 | -9049 |

TOTAL OF POSITIVE VALUES OF CUT MINUS FILL = +0

TOTAL OF NEGATIVE VALUES OF CUT MINUS FILL = -9049 is opened and flow both from the branch canal and the reservoir passes down the distributary. Figure 6.4 shows two distributary canals taking off from the diversion pool but in fact as many as three can be abstracting from the same pool.

Table 6.5 gives the basic requirements for the eight reservoirs, together with the pump duties. Only distributaries T3 and T5, commanding a total NCA of 573 ha, are fed by gravity. The locations of the reservoirs, shown on the layouts (Map 3A), are naturally determined by the positions of the distributary canal heads. However, there is a risk that a 'leaky' spot may be encountered with high infiltration rates wasting the reservoir water directly to groundwater. Although the chances of this happening are thought to be relatively small, it is essential that preconstruction detailed augering and infiltration testing at each site is undertaken to reveal any extensive sandy deposits that may cause leakage so that clay lining to the reservoir may be provided.

## TABLE 6.5

### Night Storage Reservoirs and Pump Duty Details

| Reservoir | Storage capacity required ($m^3$) | Live depth (m) | Plan area (ha) | Pump duty Q ($m^3$/s) | H(approx.) (m) |
|---|---|---|---|---|---|
| G 2 ) G 4 ) G 6 ) | 43 675 | 0.3 | 14.5 | 1.011 | 1.91 |
| G 8 | 18 921 | 0.5 | 4.0 | 0.438 | 1.48 |
| G10 ) G12 ) | 43 373 | 0.3 | 14.5 | 1.004 | 1.90 |
| T 1 | 10 411 | 0.3 | 4.0 | 0.241 | 1.44 |
| T 3 ) T 5 ) | 32 227 | 0.3 | 11.0 | Gravity | feed |
| T 7 ) T 9 ) | 21 161 | 0.3 | 7.5 | 0.513 | 1.80 |
| T11 ) T13 ) | 16 676 | 0.5 | 3.5 | 0.386 | 1.66 |
| T 2 ) T 4 ) | 30 153 | 0.5 | 10.5 | 1.191 | 2.33 |
| T2.2 Distributary canal | - | - | - | 1.275 | 0.91 |

The project is to be used solely for the production of annual crops during the gu and der seasons and therefore the canals can close down during the dry season. However, this leaves the villages a long way away from the river without a water supply for domestic purposes or livestock. Under the existing irrigation system, when the canals run dry the people have to walk long distances to fetch water and the livestock have to be herded similar distances, with the associated waste of energy and the risk of picking up diseases. The night storage reservoirs provide an opportunity to overcome this problem by storing sufficient water at the end of the der season to last through the dry season.

Taking free surface water evaporation during the dry season to be 7 mm/d, reservoir losses due to deep percolation to be 2 mm/d and the total consumptive abstraction 3 mm/d (equivalent to 300 $m^3$/d for a 10 ha reservoir), the complete rate of depletion becomes 12 mm/d. If the dry season is taken to last 90 days, then the total depth of storage required is 1.08 m; this must be provided below the minimum operation level of the reservoir. Details of livestock watering are given in Annex V and the provisions for basic village supplies are outlined in Section 11.7 of this annex.

## 6.8 Earthworks Balance

Table 6.6 gives a summary of the main earthwork components of the project, excluding any landforming quantities and the formation of the flood bund. The table shows a net shortfall of cut against fill of 804 000 $m^3$ and the best way of borrowing this extra fill must be sought. The prime aim is to minimise the amount of haul involved.

The inlet channel deficit of 9 000 $m^3$ can be easily found from over-excavation of the channel, and both the branch canals and the infield canal/drain earthworks are in individual balance.

The cut obtained from main drain excavation is almost entirely utilised in forming their associated roads, leaving only 61 000 $m^3$ to assist, where possible, in distributary canal and reservoir formation. Naturally, reservoir embankments can be formed from the over-excavation of the reservoir bed; this is in fact desirable to allow sufficient depth for the storage of water over the dry season to provide village and livestock water supplies.

### TABLE 6.6

### Approximate Project Earthworks Balance

| Item | Required fill ($m^3 \times 10^3$) | Required cut ($m^3 \times 10^3$) |
|---|---|---|
| Formation of inlet channel | 43 | 34 |
| Formation of branch canals | 121 | 121 |
| Formation of night storage reservoirs | 307 | - |
| Formation of distributary canals | 397 | 6 |
| Formation of associated roads to distributary canals | 158 | - |
| Excavation of infield surface drains and formation of watercourses | 156 | 156 |
| Excavation of main drains | - | 243 |
| Formation of associated roads to main drains | 182 | - |
| TOTAL | 1 364 | 560 |

There remains, therefore, the net figure of 549 000 $m^3$ of fill to form distributary canals and associated roads. Some of this can be taken from the excess of 61 000 $m^3$ from drain excavation but the rest must be found from borrow or over-excavation elsewhere. There are four possible sources:-

(i) Over-deepening of the night storage reservoir beds. This is the preferred source but because of the haul limitations it cannot provide all the requirements.

(ii) Over-excavation of main drains. This is only practical where drains lie adjacent to distributary canals.

(iii) Scraping of surface soil from the high ground in fields. The large depth of soil available and careful selection of areas to be scraped should ensure that this alternative has no harmful effects on the later operation of the project.

(iv) Excavation of deep borrow pits. This method is not recommended as it leads to the production of many stagnant pools of water which present a major health hazard.

Utilising the first three methods it should be possible to develop an efficient haulage programme to keep earthworks costs to a minimum.

## 6.9 Irrigation Pump Stations

A total of seven irrigation pump stations at the storage reservoirs and an additional pump station to feed the branch distributary T2.2 are required. Table 6.5 gives the design discharge and static lift required for each pump station and Figure 6.4 shows a plan and section through a typical station.

Each pump station is provided with three pumps, two to accommodate the design discharge and one to act as a standby. This gives a single pump discharge range of from 0.12 to 0.64 $m^3/s$. The standby is necessary because in all cases, except for the supply to T2.2, the pumps will have to operate for 24 hours a day.

Problems have been encountered on the Afgooye-Mordiile irrigation project (Libsoma) with pumping directly from the river. The pumps reach right to the bed of the river so that water can be abstracted even during very low flow periods. This has meant that large quantities of silt have been sucked up into the pumps and discharge pipes, blocking them up. If a blocked pipe is allowed to dry out, the silt becomes extremely hard and difficult to remove. This problem is unlikely to arise with the project irrigation pump stations for two reasons:-

(i) the pumping stations abstract from the branch canals. This is downstream of the inlet channel in which an expected 80% of the suspended sediment load will be deposited. In addition no river bed load will be drawn into the pumps as this can be completely excluded from the branch canals.

(ii) water levels in the branch canals can be closely controlled so that the inlet orifices to the pumps does not have to be right on the bed of the canal.

The pump stations are of very simple design to ensure easy operation and maintenance. Much experience has been gained in the Study Area of the operation of tubewell pumps with diesel engines and for this reason direct diesel drive is regarded as superior to the alternative of diesel/electric drive.

## 6.10 Farm Units and Implementation

The project land is to be cultivated on the basis of co-operative group farms; the total NCA of 3 963.5 ha has been divided into eight farms of sizes ranging from 263 to 623 ha, with an additional 137 ha reserved for a pilot farm. Each farm has been given the name of the dominant village serving its area. The farm units are integrally linked to the distributary canal system because any given canal is limited to serving only one farm unit. This is essential so that water control along the distributary can be the sole responsibility of the single farm, with no interference from other farms and that the main project operation staff need only provide the required discharge at the head of the distributary. Table 6.7 shows how the distributaries have been assigned to the eight farms and the resulting cultivated areas.

### TABLE 6.7

### Farm Units (in Correct Order for Implementation)

| Phase of development | Farm unit | Serving distributaries | NCA (ha) |
|---|---|---|---|
| 1. Pilot farm | Pilot farm | G12* | 137 |
| 2. Tawakal | Muraale | T1, T3 | 369.5 |
| | Tawakal | T5, T5.1, T5.2, T7 | 533.5 |
| | Garas Guul | T9, T11, T13 | 529.5 |
| | Shamaan | T2, T2.1, T4 | 415 |
| | Tugarey | T2.2 | 482 |
| 3. Gayweerow | Nimcooley | G4 | 263 |
| | Gayweerow | G6, G8 | 611 |
| | Jasiira | G10, G10.1, G10.2, G12 | 623 |

Note: * pilot farm initially fed by its own canal, later by G12.

The disruption to crop production caused by the introduction of a new irrigation system to the Project Area, where significant areas of irrigated agriculture already exist, must be kept to a minimum. Much thought has been given to this problem and the implementation programme is described in detail in Annex IX. The phasing of development has been broken into three parts:-

(i) The pilot farm. This is to provide research and training facilities from the earliest possible time. Consequently, for the first five years of the project it will use its own small canal offtaking from the river just upstream of Qoryooley barrage.

(ii) Tawakal contract. This involves the development of all the areas associated with the Tawakal branch canal. It is generally less intensively cultivated than the rest of the project and should provide the simplest development. Within this phase the implementation is a natural progression along the branch canal, with first the Muraale farm unit incorporating the construction of T1 and T3, followed by the Tawakal, Garas Guul, Shamaan and Tugarey units (Table 6.7).

(iii) Gayweerow contract. Likewise this phase follows the development of the Gayweerow branch canal along its length, with the formation, in sequence, of the Nimcooley, Gayweerow and Jasiira farm units.

For all design, construction and costing purposes the project has been broken down into these three component parts.

## 6.11 Maintenance

The provision for the upkeep of structures and pump stations is relatively straightforward and relies upon regular inspection work, minor repair work and periodic replacement of certain major items such as pump engines. The services to deal with this are dealt with in Chapter 11 and the associated staff requirements are listed in Annex IX.

However, earthworks maintenance needs special mention because of the large sediment clearance operations needed. All infield silt and weed clearance is to be the responsibility of the individual farmers who communually cultivate a particular field. However, the inlet channel, branch canals, distributary canals and all associated earth roads are the responsibility of the project authority; the special provisions for their upkeep are given below.

### 6.11.1 Rope Operated Dragline

Almost continuous clearance of the inlet channel will be necessary. With the removal of up to 423 $m^3/d$, at depths of up to 4.0 m, below more than 3.0 m of water, being necessary, a rope operated dragline is the obvious choice for the clearance of silt from the inlet channel. Table 6.8 gives the hourly requirements and the total operation times for the dragline based upon a rate of 1 $m^3$ cleared per minute. Certain dry season clearance will be necessary (because the inlet channel has no head regulator) although sediment loads are very low in this period anyway. In addition other demands on the dragline time are likely to arise and therefore the total annual operation time of the machine has been estimated as 1 560 hours.

TABLE 6.8

Dragline Operating Hours: Inlet Channel Only

| | | J | F | M | A | M | J | J | A | S | O | N | D | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Required rate of clearance* | ($m^3/d$) | 335 | 99 | - | - | 174 | 203 | 186 | 119 | 196 | 330 | 344 | 423 | - |
| Daily hours of operation | (h) | 5.6 | 1.7 | - | - | 2.9 | 3.4 | 3.1 | 2.0 | 3.3 | 5.5 | 5.7 | 7.1 | |
| TOTAL operation time | (h) | 174 | 48 | - | - | 90 | 102 | 96 | 62 | 99 | 171 | 171 | 220 | 1 233 |

Note: * assuming 3 000 ppm of sediment; 80% removal; porosity 0.4

### 6.11.2 Hydraulic Excavators

With the sedimentation of 80% of the silt load in the inlet channel, and the operation of canals under regime conditions, one clearance per year of silt and weeds from the branch and distributary canals should be sufficient to keep them in a good condition. This type of operation is currently undertaken in the Study Area using hydraulic excavators fitted with special desilting buckets and this is the recommended method for the project. However, the use of a mechanical link on the lower arm to operate the bucket rather than a normal hydraulic ram is preferred as this can get over the problem of failing oil seals due to the abrasive and penetrating nature of the silt.

Branch canals and distributaries will require clearance from both sides and the expected rate of progress has been estimated, from current operations in the Study Area, to be 25 m/h for both banks to be cleared. This means that the time needed to clear the complete length of 66.73 km of canals is 2 669 hours, and allowing 10% transit time the total becomes 2 936 hours.

The excavator will also be needed to provide occasional clearance of the night storage reservoris. The average depth of sediment deposited each year in a reservoir, assuming 10% of the total silt load falls here, is only 19 to 35 mm, depending upon the live depth of the reservoir. However, most of the sediment will deposit itself in an alluvial cone spreading from the inlet/outlet regulator, tending to block the structure. It is considered therefore that every four years the area aroung the inlet/outlet regulator will have to be cleared (i.e. two reservoirs each year must be cleared). To do this the excavator must first build its own bund to operate from and then clean the area around, moving steadily backwards, removing the bund again as it goes. This operation will require the assistance of at least one tractor-drawn tipping trailer but, as long as the timing of clearance is carefully chosen, these should be freely available from the farms.

The total annual time required to clean two reservoirs is estimated as 425 hours.

The total annual excavator time is therefore 3 361 hours and this is beyond the capabilities of one machine. Therefore two hydraulic excavators will be necessary for the project.

### 6.11.3 Tractor-mounted Mower-bars

In addition to the clearance of silt and weeds from the canal itself, extensive weed clearance from the outside banks and reservation areas of canals, reservoirs and drains will be necessary. Each farm is to be provided with a tractor-mounted mower-bar and this provides ample capacity for all weed clearance operations.

### 6.11.4 Road Graders

An extensive system of earth roads exists in the Project Area, on both sides of all canals and main drains. These will require dressing and grading twice a year after the seasonal rains. The calculated time required for grading the roads is only 228 hours per annum. This figure is too low to consider buying in a machine especially for the project and therefore hiring will be necessary.

# CHAPTER 7

# FIELD UNIT

## 7.1 Introduction

This chapter covers the detailed layouts of the individual fields in the intensively irrigated areas of the project (NCA 3 963.5 ha). These layouts do not apply to the land (classed as unsuitable) served by distributary G2, where no infield works are to be provided. Here only the existing water supplies are to be maintained (by building G2) and a main collector drain (Q11) provided.

The cultivated area of land irrigated from one watercourse is known as a field unit. Each one is to be farmed communally for field crops by a group of families. For the average field size of 22 ha a group of 12 families will be necessary. Some field units, or part field units, have to be reserved for the houseplots of one-eighth of a hectare per family.

To field unit will be under the control of the grouped families and they will be responsible for operation and maintenance of all the infield works including minor canals and drains. Most importantly, all irrigation scheduling is to be arranged within the field unit, the steady day-time flow through the distributary outlet being shared out to the field by the grouped families.

## 7.2 Field Numbering System

Each field unit is given a unique code, made up of three parts, from which it can be identified. An example is:-

T13/12/24

where T13 is the number of the distributary that feeds the unit, 12 is the field number on the distributary (fields are numbered consecutively from the head with odd numbers reserved for fields on the left bank of the distributary and even numbers on the right bank), and 24 is the NCA of the field in hectares.

If a letter A is included with the distributary canal number (e.g. T9A/3/31) this shows that the distributary outlet offtakes from above the head regulator, in other words directly from the diversion pool (see Figure 6.4). For watercourses that irrigate land on both sides, a letter A is included with the field number for the land on the left bank of the watercourse (e.g. T5.1/5A/8.5) and a letter B for the land on the right bank (e.g. T5.1/5B/15.5).

## 7.3 Watercourses

The watercourse is a permanent, vee-sectioned channel that carries the water supplies from the distributary outlet to the section of the field unit being irrigated (Figure 7.1). The normal command should lie between 0.20 and 0.50 m and the channel slope be not less than 10 cm/km nor more than 100 cm/km. Where steep ground slopes are encountered a simple fall structure can be provided to limit the watercourse slope to 100 cm/km. The size of the watercourse depends upon the streamflow required, which in turn is determined by the NCA of the field. For the project, the normal watercourse discharge has been set at 60 l/s (see Section 2.5) but to accommodate the larger field units a second watercourse size is needed, to pass flows up to 100 l/s.

The construction of watercourses can be achieved by the laying of a ribbon of fill, excavated from the adjacent surface drain, up to formation level. The section is then formed by a vee-ditcher.

## 7.4 Furrow Irrigation

The method of surface irrigation recommended for the normal field crop production is long furrow (see Section 2.4). Table 7.1 gives the recommended maximum furrow lengths for clay soils for different water applications and furrow slopes (Booher, 1974). The total water application is limited by the very low terminal infiltration rates of the clay soils to about 70 mm, excluding surface run-off. Adding this in gives a total average application of 85 mm at the furrow head. Also the land slopes in the Project Area range from almost flat to 0.5%. Taking these factors into account a standard furrow length of 300 m has been adopted.

**TABLE 7.1**

**Suggested Maximum Furrow Lengths for Clay Soils and Maximum Non-erosive Flow Rates (Booher, 1974)**

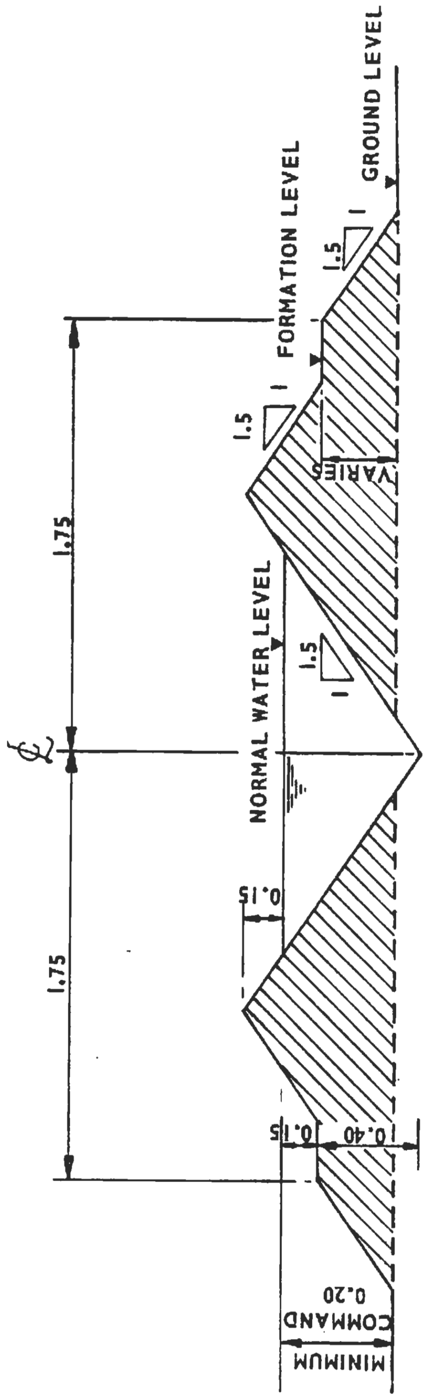
| Furrow slope (%) | Average depth of water applied (mm) | | | | Maximum flow rate per furrow (l/s) |
|---|---|---|---|---|---|
| | 75 | 150 | 225 | 300 | |
| | maximum furrow length (m) | | | | |
| 0.05 | 300 | 400 | 400 | 400 | - |
| 0.1 | 340 | 440 | 470 | 500 | 6.0 |
| 0.2 | 370 | 470 | 530 | 620 | 3.3 * |
| 0.3 | 400 | 500 | 620 | 800 | 2.0 |
| 0.5 | 400 | 500 | 560 | 750 | 1.2 |
| 1.0 | 280 | 400 | 500 | 600 | - |
| 1.5 | 250 | 340 | 430 | 500 | - |
| 2.0 | 220 | 270 | 340 | 400 | 0.3 |

Note: * by graphical interpolation

The maximum continuous rate of gross water application at distributary outlet is 1.11 l/s/ha (derived from Table 19.7, Annex VI). Taking a typical irrigation interval of 15 days, an irrigation period of six hours (i.e. two sections of field are irrigated per 12 h irrigation day) and a furrow width of 800 mm, then the average discharge in a single furrow over the complete six hour period has a maximum value of 1.6 l/s. However, for even water distribution along the length

FIGURE 7.1

## WATERCOURSE SECTION (FOR 60 LITRES/S)



of a furrow, the initial watering rate needs to be much greater than the average rate. This ensures that equal opportunity for infiltration at all points along the furrow can be achieved. The maximum rate has been set at 3.2 l/s. This indicates that the maximum furrow slope should be limited to 0.2% (Table 7.1) to avoid scour and erosion of the soil. However, slopes of up to 0.5% occur in the Project Area and therefore it will be necessary to run the furrows down the minor slope of a field, the watercourse running down the major slope (see Figure 7.2).

The use of plastic tubing to siphon the irrigation water directly from the watercourse into the furrows is recommended. A 40 mm diameter tube will provide a nominal discharge, assuming a headloss of 100 mm, of 1.06 l/s. Therefore the initial furrow wetting at a high discharge rate can be achieved using three tubes; this can then be reduced to two as soon as the water front reaches the tail of the furrow, and eventually, as the infiltration rate approaches its terminal value, to a single tube.

Research work on the Rahad irrigation project in the Sudan suggests that the use of 'level' furrows might also be considered (MMP, 1978). The Rahad soils are heavily cracking montmorillonitic type clays similar to those of the Study Area but with a clay content of just over 70% compared with the Study Area soils, where the clay content for the majority of the area is between 50 and 55%.

The furrows used for the trials were 280 m long with slopes of 0.025% and 0.065%. Irrigation applications were similar to those required for the Project Area and an average of about 2.5 l/s was applied to each furrow for about four hours using a single 50 mm diameter siphon.

The siphons were cut off as soon as the water reached the end of the furrow but it is now considered that water should be cut off earlier to avoid water standing in the furrow. The use of 50 mm diameter siphons is under review and experiments may be made with 40 mm diameter siphons.

## 7.5 Small Basin Irrigation

The row spacing for upland rice, of about 170 mm, is too close for successful furrow irrigation and therefore this crop is to be irrigated using the traditional method of small basins. In addition the houseplots of one-eighth of a hectare will be irrigated in this manner. The existing small basin irrigation in the Study Area is based, for reasonably even land surfaces, upon the jibal of 25 large paces square. Because of the familiarity with this basin size it is recommended, at least initially, for use on the project.

To provide water to the basins from the watercourse a farm channel is necessary. This is a semi-permanent channel that is re-formed at the start of each season. The offtake point from the watercourse can be controlled using a portable canvas check structure. Each farm channel will serve 3 ha of cultivated land (i.e. 48 jibals). A string of four basins will be aligned along the major slope and water from the farm channel passes into the uppermost of the four. When sufficient water has been applied the continuing inflow can be passed into the second basin and so on. At the end of irrigation any surplus water can be drawn off into the minor surface drain (see Figure 7.2).

## 7.6 Infiltration and Irrigation Intervals

The infiltration rates, both cumulative and terminal, are very limited on the Project Area soils. After six hours of infiltration the terminal rate has been reached and, in the case of the Saruda soils, is extremely low at only 2mm/h. This means that irrigations for periods of longer than six hours are of little benefit and, consequently, this has been taken as the standard length of time for water application. This has the advantage that two complete periods can be fitted into the normal irrigation day of 12 hours. Over the standard six hour irrigation period, the total infiltration has been taken as 70 mm.

For the crops in the proposed pattern, Table 7.2 gives the required irrigation intervals at the time of maximum consumptive use of water. These have been taken directly from Table 19.8 in Annex VI; here a full discussion of the irrigation interval for each crop is given. In all cases apart from upland rice, the interval is determined from the total infiltration limitation of 70 mm rather than the readily available moisture in the soil. Rice is the exception because of its very shallow mature rooting depth of only 450 mm which limits drastically the available moisture.

TABLE 7.2

**Irrigation Intervals**

| Crop | Interval (days) |
|---|---|
| Maize | 15 |
| Upland rice | 7 |
| Sesame | 12 |
| Cotton | 12 |
| Forage | 23 |

## 7.7 Standard Field Layout and Scheduling

The cultivated width of all fields has been set at 300 m. Consequently, for a normal watercourse capacity of 60 l/s and the design watering rate of 2.55 l/s/ha, the cultivated length of the standard field is 800 m. This gives the standard field an NCA of 24 ha.

However, the strong topography of the area, with easily definable ridges and depressions, makes the distances between the distributary canal and the main collector drain a function of the relief rather than standard field lengths. Therefore, variable field sizes have been permitted and only 28% of the fields are standard size.

For fields longer than 800 m net, a larger distributary outlet and watercourse are necessary, and provision has been made for a 100 l/s structure and channel where necessary. This increases the maximum possible field unit size from 24 ha to 39 ha. The average field size is 22 ha, with 148 standard distributary outlets (including those on G2) and only 40 large ones.

Figure 7.2 gives the standard field layout for both long furrow and small basin irrigation. For clarity, the access roads have not been included and are dealt with separately in Section 11.6. A reservation of 15 m has been allowed in the minor slope direction to cover the watercourse, surface drain and access road. In the major slope direction, half-reservations of 20 m for the distributary canal and 25 m for the main collector drain have been included. This gives an overall land use efficiency for the unit as shown of 90%.

### 7.7.1 Idealised Furrow Irrigation Scheduling

The watercourse is divided into a number of equal sections, equivalent to the number of irrigation periods contained within the irrigation interval. For example, taking the irrigation interval of maize, 15 days, there will be 30 groups of six hour irrigation periods within this. Therefore, the field should be broken into 30 equal areas and each one irrigated in turn.

Starting at the distributary outlet a portable watercourse check is placed in the channel one section away and all the discharge siphoned into the furrows above this. Initially, the full discharge can be taken by the furrows but as the number of siphon tubes operating is reduced, some of the water will overflow the check. By placing a second watercourse check a further section distance down the watercourse, the second section can be prepared for irrigation. The water level will rise and siphoning can begin in the second section before it is complete in the first, thus ensuring a correctly phased changeover, maintaining as far as possible a constant siphoning rate. At the end of the day, the system is shut off and the same process performed the next day but with sections three and four. This is repeated along the watercourse.

### 7.7.2 Idealised Basin Irrigation Scheduling

This is undertaken in exactly the same way as with furrows, the only difference being that the number of sections in a field is determined by the number of field channels. This will not necessarily agree with the number of irrigation periods within the irrigation interval. For example, the irrigation interval for upland rice is seven days (equivalent to 14 irrigation periods) but there are eight farm channels in a standard field unit. Therefore, farm channels will have to operate for lengths of time different from the irrigation period - in the case of upland rice about 10.5 hours. A sub-rotation within the jibals, fed by a farm channel, must therefore be imposed to ensure that each piece of land receives its fair share of water - again in the case of upland rice this could be achieved by irrigating just two strings of four jibals with the complete discharge for 1 hour 45 minutes and then pass on to the next two strings, leaving the ponded water to infiltrate slowly.

Water is diverted into the farm channel being watered, simply by removing the portable watercourse check from the head of the farm channel and placing it in the watercourse.

The methods of scheduling outlined above are idealised and the actual scheduling within a field unit will be dependent upon many factors. Perhaps the most basic feature is that many fields will contain mixed cropping and therefore both furrow and basin irrigation may occur in the same field. Also, for much of a season, the crop water requirements are less than the design capacity. Therefore, a choice is open either to irrigate smaller areas at the same discharge rate per unit area for a shorter time or to maintain the sections and duration of irrigation and irrigate at a lower discharge per unit area. In either case, it is recommended that the irrigation interval is kept constant. The ultimate success of a schedule will be the sole responsibility of the grouped farmers whose livelihoods depend upon the satisfactory irrigation of their crops.

## 7.8 Land Forming

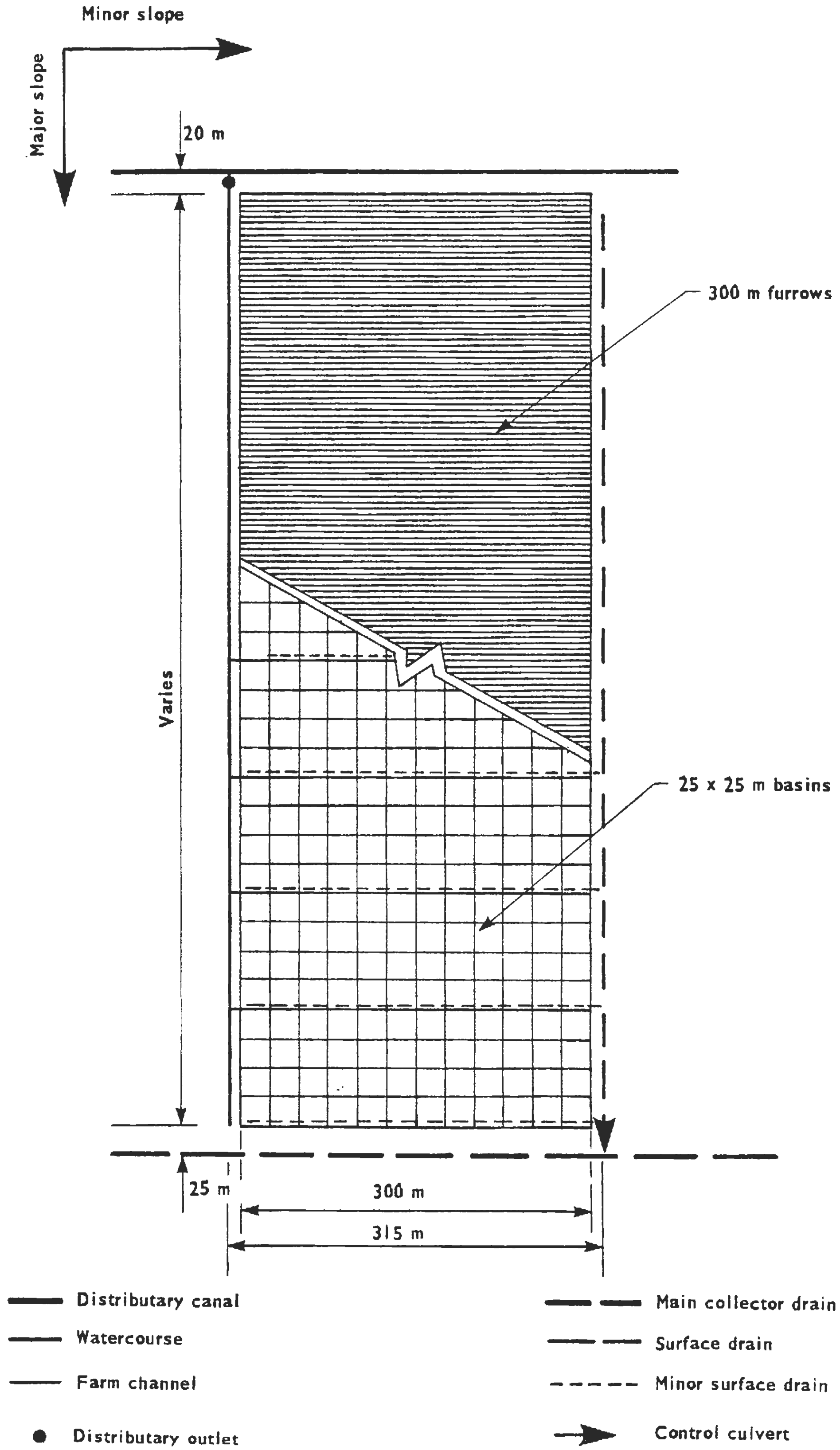
The uniform distribution of water along a furrow or within a basin depends greatly upon correct land forming operations being completed before the land is irrigated.

Three areas inside the Project Area, each of 6.25 ha, were surveyed on a 25 m grid and the results used to calculate the required amounts of fill for correct land forming (Section 2.2). The volumes were 300, 295 and 195 $m^3$/ha. However, these have been calculated for the optimum orientation of the slopes and therefore some increase beyond these figures is necessary. In addition the extensive minor canal system in the Project Area will have to be included in the land forming operation. It is considered that a gross figure of 500 $m^3$/ha of fill is appropriate for the project.

This figure has been applied to all of the intensively irrigated areas but not to the 'unsuitable' land fed by G2. No land forming is to be done in this area.

FIGURE 7.2

## STANDARD FIELD LAYOUT FOR FURROW AND BASIN IRRIGATION



# CHAPTER 8

# DRAINAGE SYSTEM

## 8.1 Definitions

Detailed discussion of the need for adequate drainage is presented in Chapter 4; a complete system is proposed and has been designed for the project. The various elements are defined below:-

Surface drain

These are shallow drains in the field unit which dispose of surface run-off and excess irrigation water from the cultivated land.

Main collector

Water from the surface drains passes through a control culvert directly into the main collector drain.

Qoryooley outfall drain

All the main collector drains discharge through junction structures directly into the outfall drain. In addition many surface drains pass water directly into it. The outfall drain terminates at the drainage pump station, where the drainage water is lifted into the existing Liibaan canal for disposal in the old river channel system.

## 8.2 Surface Drains

The surface drains do not have to be designed to any particular discharge, but must merely provide an adequate section and uniform bed slope to pass the excess irrigation water and storm run-off from the field into the main collector system. Water enters the surface drain either directly from the tails of furrows or, in the case of basin irrigation, from the minor surface drain.

The surface drain must be shallow enough to allow access for farm machinery across it. The surface drain, by necessity, must be between the access road and the bulk of the field (see Figure 11.2) and therefore every machine entering or leaving the field must cross it. The nominal section proposed therefore is trapezoidal with a flat bed width of 1.0 m, side slopes of one in four, and a minimum, depth of 0.4 m. Even this open section presents a considerable obstacle and, during wet periods, difficulties may arise on the slippery up-slope. Therefore the provision of occasional crossing points (exactly comparable to a ford) is recommended. These can be either of crushed coral or concrete; by providing two of them in a standard field and three for larger fields it should be possible to maintain adequate access.

The control culvert at the end of the surface drain is required to throttle the discharge of storm run-off from the field so that the main drainage system is not overloaded. This is dealt with in Section 8.8.

## 8.3 Buried Field Drains

Buried field drains are only needed where control of the groundwater table, to alleviate waterlogging problems and stabilise salinity levels, has become necessary. At present the groundwater table under the Project Area is generally encountered at depths of between 5 and 20 m, therefore, presenting no major waterlogging hazard. Salinity levels are almost entirely medium to negligible. Observation of the perennially irrigated bananas in the Study Area and the sugar crops at Jowhar, has shown that the build-up of groundwater levels has been relatively slow. Only after 30 to 50 years of intensive irrigation has waterlogging become a major problem and, in the case of Jowhar sugar estate, led to the abandoning of large areas.

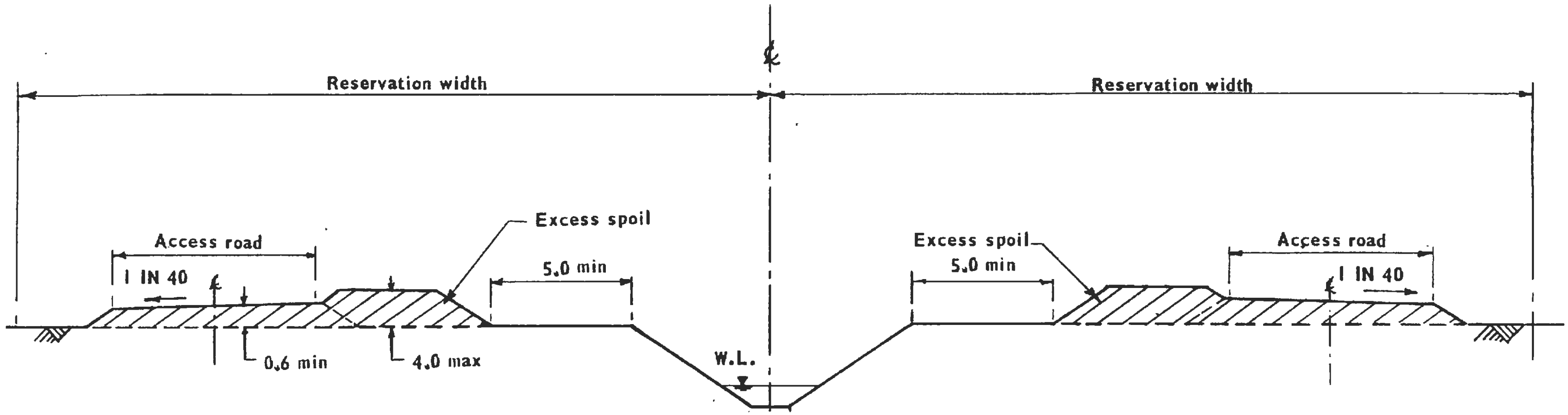
It is considered therefore that within the project horizon of 30 years, no major infield buried drainage works will be necessary. However, minor perched aquifers at shallow depth do occur and in one or two fields waterlogging may prove to be a problem. The solution will be to install buried field drains if and where any problems arise. It would be uneconomic to deepen the entire main collector and outfall system solely to be able freely to drain one or two fields fitted with buried drains. Therefore it is recommended that for any such fields the deep collector drain is constructed (see Section 4.3) and the drainage water is pumped from the collector drain into the existing shallow main collector drainage system.

## 8.4 Main Collector Drains

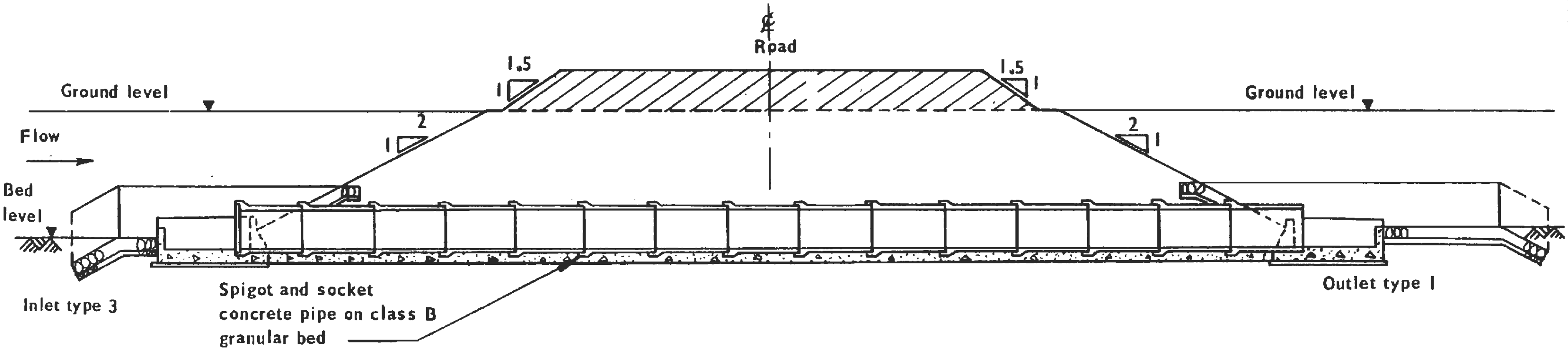
The main collectors have been identified by the use of the letter Q followed by a number. Starting from the drainage pump station and working up the Qoryooley outfall drain the main collectors have been numbered consecutively with odd numbers reserved for the left bank of the outfall drain and even numbers for the right bank (as always, the left bank is defined as the bank on the left hand side when facing downstream). A total of 10 main collector drains are shown on the layouts, with two of them, Q4 and Q6, having multiple branches feeding into them. The branches are given a second number, determined in the same way as the main number with odd numbers reserved for those feeding in on the left bank of the main colector, even numbers on the right bank. The strong topography of the Project Area, with well defined internal ridges and depressions, has made the choice of drain line, like the distributary canal lines, relatively straightforward. Q11 has been extended to provide surface drainage of the existing agricultural land in the 'unsuitable' areas fed by distributary G2.

Design of the drains has been done using the combined Manning and Lacey expressions described in Section 4.4. In addition a minimum bed width of 1.0 m has been imposed. The design discharge has been taken as the full element of surface run-off contained within the day-time design watering rate. This is equal to 15% of 2.55 l/s/ha, equivalent to 38 l/s/km$^2$ of NCA. The detailed calculations have been undertaken by computer and Table 8.1 shows the output for the main collector drain Q7.

The drain cross-section is trapezoidal with 1 : 1.5 side slopes and Figure 8.1 shows a typical cross-section of a main collector drain.




OUTFALL AND MAIN COLLECTOR DRAINS




MAIN COLLECTOR DRAIN & OUTFALL DRAIN JUNCTION

FIGURE 8.1

# TABLE 8.1

## Main Collector Drain Design

DESIGN DATA FOR DRAIN Q7

| STRUCTURE NAME | CHAINAGE (METRES) | DESIGN DISCHARGE U/S | DESIGN DISCHARGE D/S | U/S BED WIDTH | DESIGN WATER LEVEL U/S | DESIGN WATER LEVEL D/S | BED LEVEL U/S | BED LEVEL D/S |
|---|---|---|---|---|---|---|---|---|
| G6/1 | 3000 | **** | 0.01 | *** | ***** | 65.38 | ***** | 65.17 |
| G6/2 | 2970 | 0.01 | 0.02 | 1.0 | 65.37 | 65.37 | 65.16 | 65.16 |
| G6/3 | 2690 | 0.02 | 0.03 | 1.0 | 65.27 | 65.27 | 65.05 | 65.05 |
| G8A'4 | 2650 | 0.03 | 0.04 | 1.0 | 65.26 | 65.26 | 65.04 | 65.04 |
| G6/5&G6/7A | 2370 | 0.04 | 0.05 | 1.0 | 65.17 | 65.17 | 64.93 | 64.93 |
| G8/2 | 2310 | 0.05 | 0.06 | 1.0 | 65.15 | 65.15 | 64.91 | 64.91 |
| G6/9&G6/7B | 2050 | 0.06 | 0.07 | 1.0 | 65.07 | 65.07 | 64.81 | 64.81 |
| G8/4 | 1970 | 0.07 | 0.08 | 1.0 | 65.05 | 65.05 | 64.78 | 64.78 |
| G8/6&G8/8A | 1630 | 0.08 | 0.09 | 1.0 | 64.95 | 64.95 | 64.65 | 64.65 |
| G6/13&G6/11 | 1430 | 0.09 | 0.11 | 1.0 | 64.89 | 64.89 | 64.57 | 64.57 |
| G6/15 | 1110 | 0.11 | 0.12 | 1.0 | 64.80 | 64.80 | 64.45 | 64.45 |
| G8/8B&G8/10A | 950 | 0.12 | 0.12 | 1.0 | 64.75 | 64.75 | 64.39 | 64.39 |
| G6/17 | 490 | 0.12 | 0.13 | 1.0 | 64.63 | 64.63 | 64.22 | 64.22 |
| G6/19 | 180 | 0.13 | 0.14 | 1.0 | 64.54 | 64.54 | 64.10 | 64.10 |
| R.O/F.DRAIN | 0 | 0.14 | **** | 1.1 | 64.49 | ***** | 64.03 | ***** |

## Earthworks Quantities for Drain Q7

| CHAINAGE | BED WIDTH | CUT AV. DEPTH | VOLUME OF CUT NO BERMS | VOLUME OF CUT BELOW BERM | VOLUME OF CUT ABOVE BERM | VOLUME OF CUT TOTAL |
|---|---|---|---|---|---|---|
| 3000 TO 2970 | 1.0 | 1.15 | 94 | 0 | 0 | 94 |
| 2970 TO 2920 | 1.0 | 1.12 | 151 | 0 | 0 | 151 |
| 2920 TO 2880 | 1.0 | 0.99 | 100 | 0 | 0 | 100 |
| 2880 TO 2690 | 1.0 | 0.81 | 344 | 0 | 0 | 344 |
| 2690 TO 2650 | 1.0 | 0.76 | 66 | 0 | 0 | 66 |
| 2650 TO 2590 | 1.0 | 0.76 | 97 | 0 | 0 | 97 |
| 2590 TO 2430 | 1.0 | 0.64 | 203 | 0 | 0 | 203 |
| 2430 TO 2370 | 1.0 | 0.50 | 52 | 0 | 0 | 52 |
| 2370 TO 2310 | 1.0 | 0.44 | 45 | 0 | 0 | 45 |
| 2310 TO 2050 | 1.0 | 0.54 | 258 | 0 | 0 | 258 |
| 2050 TO 2030 | 1.0 | 0.67 | 27 | 0 | 0 | 27 |
| 2030 TO 1970 | 1.0 | 0.78 | 103 | 0 | 0 | 103 |
| 1970 TO 1630 | 1.0 | 0.92 | 752 | 0 | 0 | 752 |
| 1630 TO 1520 | 1.0 | 0.94 | 248 | 0 | 0 | 248 |
| 1520 TO 1430 | 1.0 | 0.86 | 178 | 0 | 0 | 178 |
| 1430 TO 1170 | 1.0 | 0.93 | 582 | 0 | 0 | 582 |
| 1170 TO 1110 | 1.0 | 1.06 | 166 | 0 | 0 | 166 |
| 1110 TO 950 | 1.0 | 1.19 | 525 | 0 | 0 | 525 |
| 950 TO 490 | 1.0 | 1.28 | 1720 | 0 | 0 | 1720 |
| 490 TO 460 | 1.0 | 1.30 | 116 | 0 | 0 | 116 |
| 460 TO 360 | 1.0 | 1.19 | 337 | 0 | 0 | 337 |
| 360 TO 280 | 1.0 | 0.97 | 196 | 0 | 0 | 196 |
| 280 TO 180 | 1.0 | 0.91 | 182 | 0 | 0 | 182 |
| 180 TO 100 | 1.1 | 0.72 | 123 | 0 | 0 | 123 |
| 100 TO 0 | 1.1 | 0.80 | 185 | 0 | 0 | 185 |
| | | TOTAL | 6849 | 0 | 0 | 6849 |

The only structure required on the drain is a junction structure into the outfall drain to prevent scour in this area and permit road access along both banks of the outfall drain. Figure 8.1 also shows a longitudinal section through a typical junction structure. Because of the high sulphate content of the soils the use of sulphate resisting cement is necessary for all drainage structures; sulphate resisting cement would also be required if tile drains were ever installed.

## 8.5 Qoryooley Outfall Drain

The Qoryooley outfall drain takes water from the main collectors and directly from the surface drains and passes it down to the drainage pump station. The line has been chosen to link together as many as possible of the low depressions in the centre of the Project Area. The exit from the Project Area is through the lowest point in the encircling high ground; this occurs just north of Qoryooley, in a line running due west. Because the drain runs for 1.8 km outside the area covered by the detailed topographic survey, additional surveying has been undertaken and the general ground level either side of the proposed line assessed. The results of this survey work are given in Annex X. The complete length of the outfall drain is 12 230 m.

The drain cross-section (Figure 8.1) is the same as for main collector drains and design of the channel has, likewise, been done with the assistance of a computer.

The outfall drain forms a major barrier to the movement of traffic in a north-south direction, both within the project itself and also along the road linking Qoryooley to the village of Farsooley (see Figure 8.2). Consequently, a total of five road culverts has been provided, at distances of 1 500 m (Farsooley road), 4 250 m, 7 290 m, 8 500 m and 10 950 m upstream of the pump station to allow the free movement of traffic.

## 8.6 Drainage Pump Station

The pump station, situated on the bank of the Liibaan canal (see Figure 8.2) is required to lift the drainage water from the outfall drain (water level 62.6 m) into the canal. The existing bed level of the canal is 66.0 m and bank top level 67.6 m. The maximum possible level in the canal is therefore 67.6 m, making the pump station maximum static lift equal to 5.0 m.

The design discharge is simply the total surface run-off from the entire Project Area of 1.6 $m^3/s$ calculated from a total NCA of 3 963.5 ha plus allowance for the G2 undeveloped area and a surface run-off rate of 38 $l/s/km^2$. The pump station has been designed to be capable of handling this discharge with two pumps operating and a third pump acting as standby in case of breakdown. This spare capacity will be necessary to remove the large quantities of storm run-off that can occur.

Figure 6.5 gives the simple layout of an irrigation pump station and the drainage pump station will be similar to this. As with the irrigation pumps, direct drive diesel engines are preferred.

## 8.7 Liibaan Canal

The Liibaan canal is the poorest of the eight main canals already existing in the Study Area. Negligible slopes, very low demand for water, and an oversized section means that the head reaches of the canal silt up very rapidly. At no time during the present studies was any flow observed at the point where the proposed project outfall drain meets the canal and the largest discharge recorded at the head was only 0.12 $m^3/s$. The total canal serves only an estimated 247 ha NCA of marginal land. Of the irrigated land only 50 ha NCA relies upon offtake points beyond the point where the outfall drain meets the canal.

The canal forms an ideal channel to carry the drainage waters away (see Figure 8.2). It is large and can therefore accommodate the occasional large discharges produced by storm run-off. It is also relatively silt free as most of the sediment load is dropped in the head reaches, and the inside banks and bed of the canal are well grassed. The canal ends by sweeping around into an old river channel; this in itself forms an extension of the channel and no earthworks would be required to form this into the disposal channel for drainage water. To verify the suitability of this canal and old river channel, extensive survey work was undertaken and the resulting cross-sections and longitudinal levels are recorded in Annex X.

The drainage water pumped into the canal will be of only marginally poorer quality than the original irrigation water, as it will be only collected surface run-off and contain no element of saline sub-surface drainage waters. It is considered, therefore, that the pumping will not interfere substantially with the existing irrigation and will allow the continued cultivation of all the present areas.

At some stage in the future the remodelling of the Liibaan canal into an effective small canal may be considered (see Section 15.8). This, however, will not cause any difficulties with the pump station because, logically, it lies at the lowest ground point along the line of the canal, with ground levels rising again downstream of the pump station to the old river channel levee. Therefore, any properly designed canal will have its tail at the approximate position of the pump station and the two systems will not interfere.

## 8.8 Storm Run-off

A design storm of 125 mm in 24 hours has been adopted when considering storm run-off. This is equivalent to a storm, based on the daily rainfall records of Mogadishu, with a return period of ten years. After allowances for infiltration and evaporation have been made, the remaining 50 mm is all surface run-off and must be disposed of by the surface drainage system before the ponded water has any harmful effects upon the crops standing in the field.

The rate at which the water is released from the field into the main collector system is controlled by the size of pipe in the outlet culvert. For a 225 mm diameter pipe operating with a 100 mm head loss across it the discharge would be 34 l/s. Taking a standard 24 ha field, the total volume of run-off would be 12 000 $m^3$ and therefore it would take four days to remove this. This means that at the bottom of the field some crops may be standing in water for up to four days; this will have serious effects upon maize but, because of the clay soil's ability to retain some air at shallow depths even when the surface is fully saturated, the loss is not expected to be total (see Section 4.5). Other crops will not be so badly affected and the upland rice (unless very young) will be totally unaffected by a four day waterlogging.

By avoiding the planting of maize in the lower parts of the fields near the main collector drains (see Figure 7.2) it is possible to allow the drainage of storm run-off to be spread out over four days without any significant loss of crop yields.

The design storm run-off of 50 mm refers to the intensity of a point event and can be reduced when the average intensity of the storm event over the complete Project Area is being considered. Using the formula given in Section 4.5, the average project intensity of the design storm run-off is 14.0 mm. If this is to be disposed of by the drainage system in four days, the pump station must work at a rate of 2.09 $m^3/s$. This is greater than the normal design capacity of the pumping station of 1.6 $m^3/s$ and the standby pump must be used to handle this discharge. This is regarded as acceptable because of the infrequency of such events, but it does mean that care must be taken to ensure that, as far as possible, all three pumps are always in working order.

## 8.9 Surface Water Escapes

Within any large area certain low spots will occur that cannot be drained by the normal infield surface drainage system and a few special surface water escapes must be provided. These can be formed from simple buried pipe structures with special inlet and outlet boxes to prevent scour. Ponded water can then be removed into the surface drainage system.

In the Project Area a total of ten escape structures has been positioned in low spots within the field systems. Of these, a total of seven feed directly into the Qoryooley outfall drain. In addition, six more escapes have been provided, three of these are to release water trapped in between the Tawakal branch canal and the Asayle canal. The other three are to allow the water trapped between the river flood banks and the Gayweerow branch canal to be passed into the drainage system. All six of these pass under a branch canal and therefore special provision will be made to avoid excessive seepage losses from the canal entering the pipe.

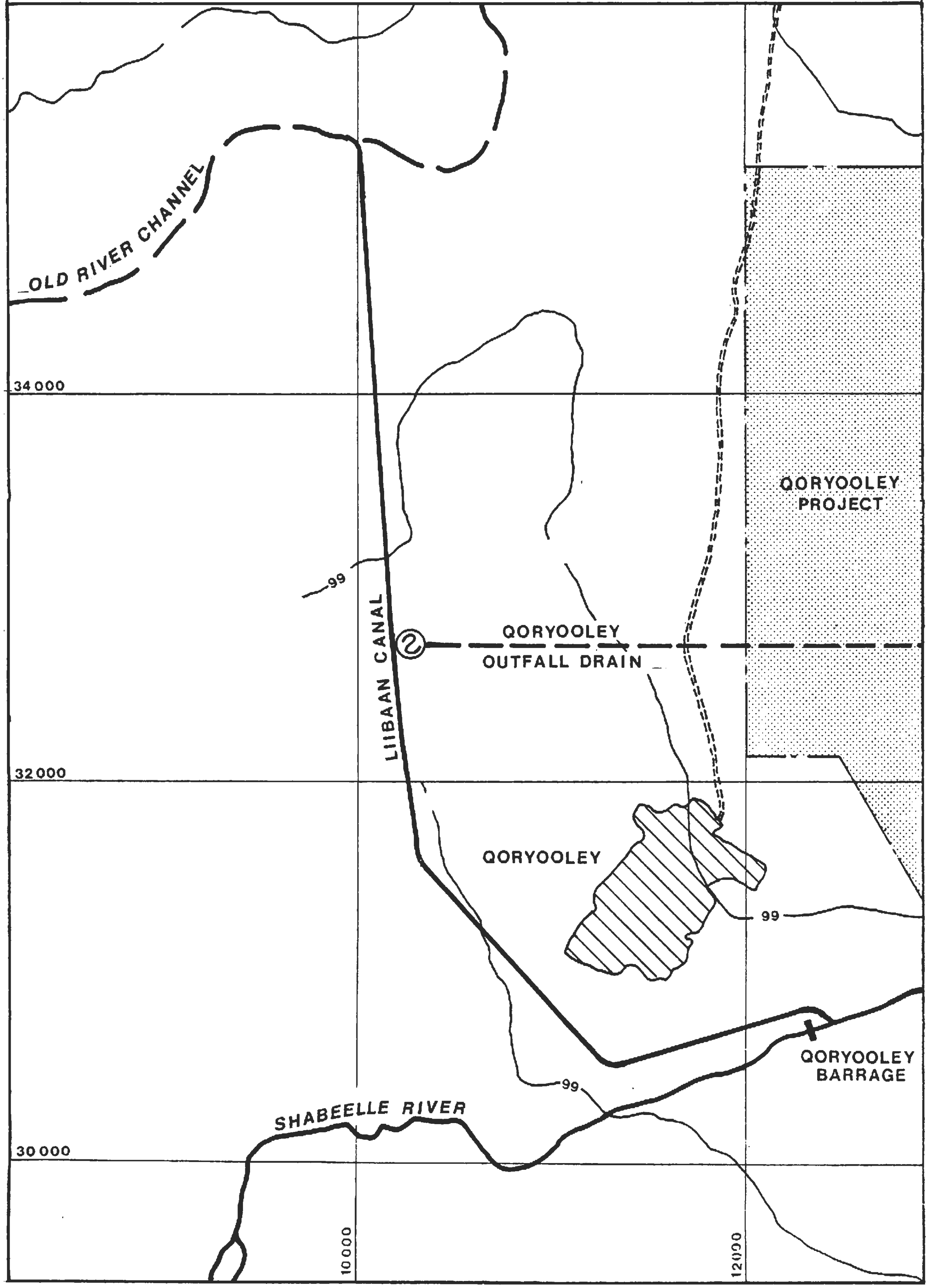
## 8.10 Maintenance

Only the drainage pump station requires any sophisticated maintenance and replacement programme; provision must be made to ensure that all three pumps are in full working order as the success of any surface drainage system lies in its ability to cope with sudden large discharges.

All the drains will require frequent weed clearance and the occasional removal of any gathered silt deposits. The former is to be undertaken by the individual farm units and sufficient free tractor time is available to cut the weeds on a regular basis with the mower bars. Silt clearance and general drain earthwork maintenance can be done by the hydraulic excavators used for branch and distributary canal clearance; most silt should be excluded from the drainage system and the time required for removal should be almost negligible.

FIGURE 8.2

DRAINAGE WATER DISPOSAL ROUTE



# CHAPTER 9

# FLOOD CONTROL

## 9.1 Flooding Risk

It is quite normal for the river to be flowing at bank full stage for long periods in the Study Area. During the der season of 1977, the river flowed continuously for almost two months at less than 50 mm from the natural river bank level. The operation of the barrages artificially extends this high water level upstream of the barrages and in these areas levels were uniformly high from July 10th 1977 to January 8th 1978.

To prevent serious flooding, flood bunds have been constructed, sometimes to a height greater than 1.5 m, along both banks of the river and along most of its length within the Study Area. These are usually set back from the normal river edge by about 10 to 20 m. However, in many places the banks have been broken down by the passage of people fetching water or crossing the river in cable operated boats. In addition, hippopotami often damage the banks and this is the prime cause of the minor floods that plague the Study Area. The last major flood occurred in 1960 when a breach upstream of the Study Area allowed water to flow down the outside of the river banks and flood extensive areas around Janaale.

During the present studies the highest water levels occurred during the peak of the 1978 gu flood. Daily records are only available for this period from the gauge just downstream of the new Gayweerow barrage site (see Annex II). The peak level attained was 68.84 m (based on A39 benchmark). This was the highest level that the local farmers could ever remember and during the five days of very high flow it was necessary to build small earth bunds to prevent minor flooding. At no time during the studies did any significant flooding occur within the Project Area, a sharp contrast to the areas further upstream near Jowhar where very serious breaching occurred.

## 9.2 Flood Bund Levels

The existing flood protection banks are inadequate because they require constant attention to prevent the high river waters from breaking through the low spots. It is fortunate that the clay soils are very stable and even very small bunds can hold back the flood water in an emergency. A lot of needless time is spent patching up these small banks and the provision of substantial bunds with adequate freeboard can avoid this. Table 9.1 provides the required bund top levels (based on A39 benchmark) for all chainages along the Project Area section of the river, taking the inlet channel as 0 m (see Annex II, Chapter 3). This gives a freeboard of 0.91 m above the highest recorded level just downstream of the new barrage site.

## TABLE 9.1

### Flood Bund Levels (based on A39 benchmark)

| Chainage (m) | Position | Bund top level (m) |
|---|---|---|
| 0 | Inlet channel | 70.02 |
| 550 | Upstream side of barrage | 69.95 |
| 550 | Downstream side of barrage | 69.75 |
| 3 200 | | 69.36 |
| 3 500 | | 69.32 |
| 4 100 | | 69.23 |
| 4 600 | ) | 69.16 |
| 5 700 | ) Gayweerow | 69.00 |
| 6 300 | | 68.92 |
| 6 700 | | 68.86 |
| 7 400 | | 68.76 |
| 7 700 | ) | 68.72 |
| 8 800 | ) Jasiira | 68.56 |
| 9 200 | ) | 68.50 |
| 9 500 | | 68.46 |
| 9 800 | | 68.42 |
| 10 200 | Liibaan canal | 68.36 |

The flood bund will be linked, at the upstream end, to the inlet channel, the Asayle canal and the Tawakal branch canal banks, and at the downstream end to the Liibaan canal banks. This means that the complete Project Area is protected not only from direct flooding by the river but also from flood waters sweeping down the right bank of the river from a major breach further upstream.

# CHAPTER 10

## ALTERNATIVE SYSTEM

### 10.1 Gravity Supply

With the water supply system described in Chapter 6 only 573 ha NCA are fed by gravity, all the remaining 3 390.5 ha NCA being supplied by distributary head irrigation pump stations. Recent experience at Libsoma with a large pump station abstracting directly from the River Shabeelle has shown the possible difficulties of pumping water with a very heavy silt load. Due to some doubts expressed by the Client, therefore, an alternative water supply system was sought which would increase the area fed by gravity.

By remodelling the upper 8.9 km of the Asayle canal, the Tawakal branch can be supplied from this and all the Tawakal distributaries, apart from T1 and T2.2, can be fed by gravity. This increases the NCA under gravity from 573 ha to 1 662.5 ha, equivalent to almost 50% of the intensively irrigated areas. This alternative has been considered in full and is presented, with complete financial and economic analyses, as a second method of water supply for the project.

### 10.2 Layout Changes Caused by the Alternative Supply

The gravity supply from the Asayle canal of the Tawakal branch canal and its distributaries does not involve any significant alteration to the majority of the irrigation layouts. Everything associated with the Gayweerow branch canal is unaffected, as are all the infield layouts on the Tawakal side. Apart from T1, T3 and T5, the only changes to the Tawakal distributaries are that the irrigation pump stations are replaced by movable weir head regulators. The basic changes, therefore, lie solely in the Tawakal branch canal itself and the detailed head layouts of T1, T3 and T5. Figure 10.1 shows the full layout of the alternative system in the area where the Asayle canal would meet the Tawakal branch canal. This complete area has been covered by a comprehensive topographic survey with all significant points of detail recorded; this has been plotted on a drawing held in Annex X.

### 10.3 Asayle Canal Remodelling

The Asayle canal relies upon Janaale barrage to provide a constant supply level. The accepted upstream gauge level for satisfactory operation of the main canals at Janaale is 4.20. This level was maintained easily throughout the 1977 der season and has been taken as the design water level for remodelling purposes. This is equivalent to a water level of 71.17 m (based upon A39 benchmark).

The total design discharge for the remodelled canal has been taken as 5.85 $m^3/s$ at the head compared with the existing maximum capacity of about 4.0 $m^3/s$. This is made up of three components:-

(i) 2.10 $m^3/s$ to supply the 1 600 ha NCA of land fed directly from the upper 8.9 km of the canal.

(ii) 3.25 $m^3/s$ to feed all the Tawakal branch canal requirements, together with an allowance for seepage losses.

(iii) 0.50 $m^3/s$ to pass into the lower section of the Asayle canal to supply the requirements of the few farms operating on the left bank.

The increased capacity of the canal means that the existing regulation structures will be inadequate and therefore new structures are necessary. A new head regulator and one cross regulator at chainage 6 400 m are necessary. In addition two road crossings, at chainages 200 and 4 800 m, are required.

The existing bed width of the canal is about 3.5 to 4.0 m, totally inadequate to pass the required discharge; the remodelled section would have a typical bed width of 7.0 m. Because of the required increase in width it would be necessary to remove some of the existing distributary outlets and provide new ones. In the upper section, of 8.9 km length, there are 25 existing outlets on the right bank, but only 14 on the left bank. Therefore, by more extensive remodelling of the left bank than the right, it should be possible to limit the number of new outlets to 14. The required changes in water level are small and therefore the operation of the existing outlets on the right bank should not be adversely affected.

To assess the quantity of earthworks required for remodelling, survey work was carried out along the length of the upper section of canal. A complete long profile of bank levels was made, together with cross-sections at chainages 4 700 and 8 900 m. This information is recorded in Annex X and has been used, together with a normal Lacey regime equation design for the new canal, to estimate the required earthworks quantity for remodelling to be 160 000 $m^3$.

As shown on Figure 10.1 the upper section of the Asayle canal can sweep straight around into the line of the Tawakal branch canal (the first 5 700 m of the branch canal are not necessary with this alternative). The water level required at this changeover point to gravity feed the 1 662.5 ha is 69.21 m, 1.96 m lower than the holding level at Janaale. This is just sufficient, and after allowance has been made for the headlosses through the Asayle canal structures an average water slope of 12.0 cm/km remains.

The lower section of the Asayle canal is not of major importance and the nominal discharge of 0.5 $m^3/s$ is considered ample to satisfy the demands made from it. Figure 10.1 shows the lower section offtaking from the remodelled upper section/ Tawakal branch canal line, with its own head regulator. No remodelling of this lower part will be necessary apart from the short section by the head regulator.

## 10.4 Sedimentation

The successful operation of the project relies upon the settling of a large proportion of the suspended sediment load in the river water before it reaches the branch canals and the night storage reservoirs. If this is not achieved then rapid siltation, especially in the reservoirs, will affect the flow of water in the project.

Consequently some form of sedimentation basin is necessary to replace the inlet channel which settles the sediment before it enters the Tawakal branch canal when water supply is solely from Gayweerow barrage. An existing reservoir has already been constructed adjacent to the Asayle canal near to Tawakal. This is unused and has been adopted as a sedimentation basin for the settling of water supplied by the Asayle canal (see Figure 10.1).

The existing minimum bank level of the reservoir is 70.06 m, providing a freeboard of at least 0.6 m. A simple open reservoir is not sufficient for sedimentation purposes as short circuiting and selective channel formation within the silt deposits would occur. In addition silt clearance with a dragline would be impossible. Therefore, a system of bunds is required to force the water to move in a zig-zag path, hence greatly increasing the effectiveness of the basin (see Figure 10.1). By direct comparison with the cross-section design of the inlet channel (Section 6.3) it has been possible to form an equivalent channel with an effective length of 520 m (compared with 800 m for the inlet channel). The earthworks to form the bund pattern can be found from the basin channels as the design bed level of 66.06 m is 2.2 m below the existing bed level.

Water control into and out of the basin can be achieved by two large gated pipe regulators. In addition a cross regulator on the Asayle canal between the inlet and outlet structures is needed so that the entire canal discharge can be routed through the sedimentation basin. The cross regulators can also provide a vehicular crossing point of the Asayle canal to replace the one lost by the removal of the existing regulator at chainage 8 900 m.

As with all the other basins or reservoirs in the Project Area, pre-construction infiltration testing and augering in the bed of the basin will be necessary to see if it is likely to have excessive seepage losses. This can be caused by sand and gravel deposits associated with the old river channels and, if encountered, may require the site of the reservoir to be changed.

## 10.5 Tawakal Branch Canal Changes

By feeding the branch canal from the Asayle canal, the first 5 700 m of the branch canal can be eliminated. However, because of the efficient cut and fill section adopted for this first stretch, the total saving in earthworks is only 41 000 $m^3$. The prerequisite for eliminating this section is that the direction of flow along T1 is reversed, the irrigation pump station and night storage reservoir being moved to the other end (see Figure 10.1).

The basic change to the branch canal, required to provide increased gravity supply, is that the water level is increased by around 2.2 m. This involves the construction of very large canal embankments. The high level canal has been designed using the computer, producing a final fill requirement for the branch canal of 510 000 $m^3$. This is 5.7 times greater than the cut and fill balanced requirement of 89 500 $m^3$ for the entire length of the low level branch canal design.

## 10.6 Distributary Canal Changes

As mentioned in Section 10.5, the direction of flow in T1 has to be reversed so that the pump station can be placed adjacent to the Asayle/Tawakal branch canal line. This is a slightly less efficient solution and involves an additional 13 500 $m^3$ of fill.

The only other distributary canal changes needed are to realign the head reaches of T3 and T5. These can be seen from a comparison of Figure 10.1 with Map 3A.

## 10.7 Alternative Design Advantages

The following advantages of the alternative method of water supply, with an increased area of land fed by gravity, over the normal system can be identified:-

(i) The number of distributary head irrigation pump stations has been reduced from eight to five, to be replaced by movable weir head regulators. This will produce a saving in annual fuel charges and maintenance costs. However, water control will not be much simpler as monitoring of pump engine speeds will be replaced by a constant attention to the upstream head over the movable weir.

(ii) The branch canal is formed in fill rather than balanced cut and fill. The advantage of this is that the small risk of encountering minor sandy deposits is eliminated and the possibility of any clay lining being necessary is reduced.

(iii) Because the first 5 700 m of the Tawakal branch canal can be eliminated, less disruption of the minor offtakes from the right bank of the lower Asayle canal will be caused during implementation. The time between the cutting of supplies and the first planting on the Murale farm can be reduced. Also existing supplies can be maintained more easily to the Nimcooley farm area throughout the Tawakal phase of implementation.

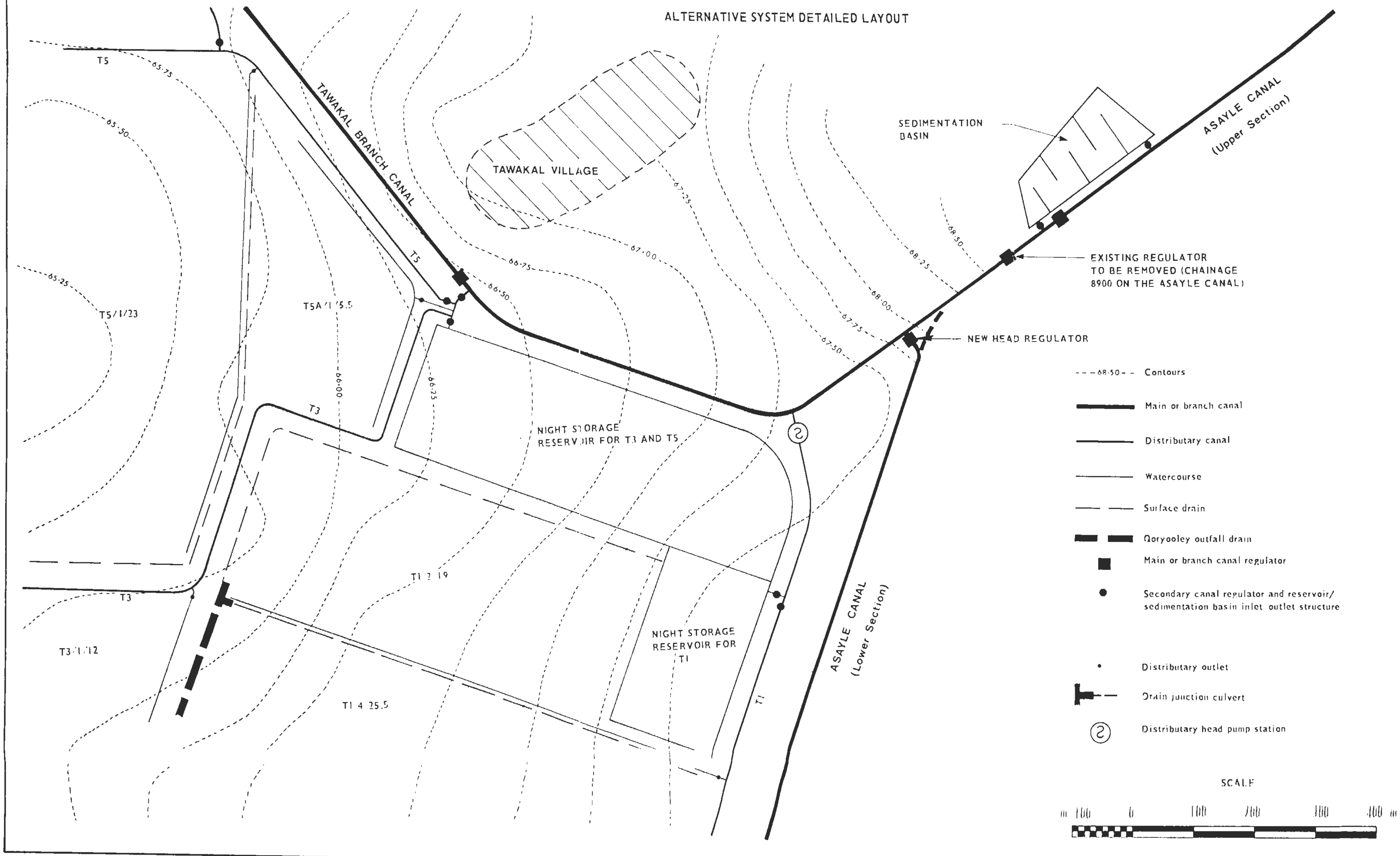
## 10.8 Alternative Design Disadvantages

To be balanced against the advantages of the alternative system are several considerable disadvantages:-

(i) Gravity supply to the Asayle canal relies upon the successful operation of Janaale barrage to maintain the required holding level. This was achieved during the present studies only by the use of temporary measures to fill holes in the vertical sluice gates and inserting stop logs on the overflow weirs. The condition of all the water control equipment on the barrage is rapidly deteriorating and a major refit is required to guarantee the holding of adequate levels in the future.

FIGURE 10.1

ALTERNATIVE SYSTEM DETAILED LAYOUT



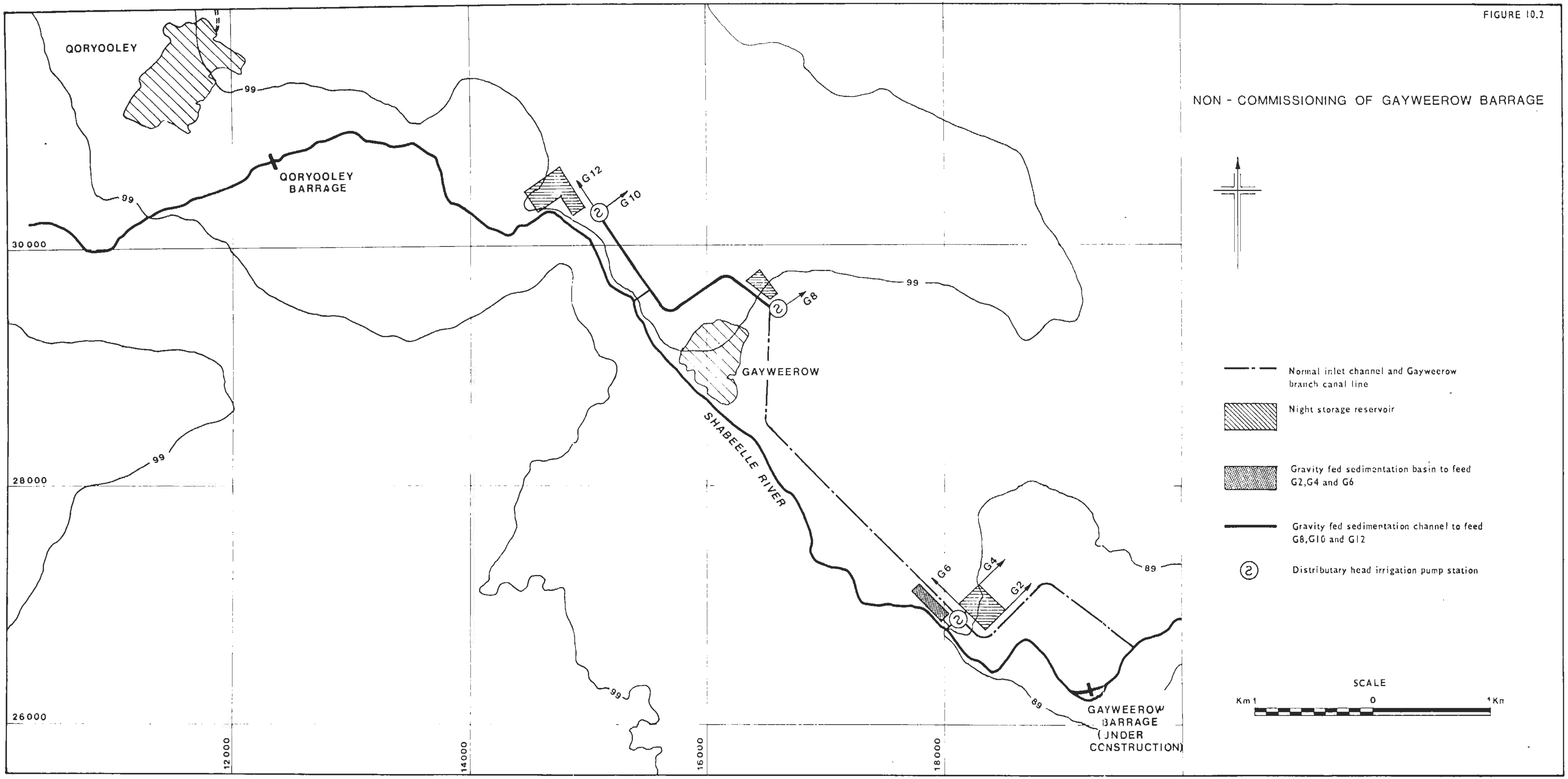


FIGURE 10.2
NON - COMMISSIONING OF GAYWEEROW BARRAGE
QORYOOLEY
QORYOOLEY BARRAGE
GAYWEEROW
SHABEELLE RIVER
GAYWEEROW BARRAGE (UNDER CONSTRUCTION)
G12
G10
G8
G6
G4
G2
99
89
30000
28000
26000
12000
14000
16000
18000
Normal inlet channel and Gayweerow branch canal line
Night storage reservoir
Gravity fed sedimentation basin to feed G2,G4 and G6
Gravity fed sedimentation channel to feed G8,G10 and G12
Distributary head irrigation pump station
SCALE
Km 1
0
1 Km

(ii) The Asayle canal will require extensive remodelling to accommodate the increased discharge. This will be costly and, if not carefully planned, will cause major disruption to the existing water users in the upper section of the canal.

(iii) No control on the abstraction of water from the Asayle canal is made and during periods of shortage the more influential farmers withdraw more than their fair share of water. This means that a guaranteed, steady supply to the Tawakal branch canal would be difficult to achieve unless an efficient controlling authority could be established.

(iv) The available headloss along the canal is only sufficient to provide an average water slope of 12 cm/km. This is equivalent to a Lacey silt factor of about 0.6.

As the canal will be carrying the full sediment load of the river waters, rapid siltation will occur. If the rate of deposition remains similar to present levels (this is probable as the existing canal is steeper but discharges are less) then a minimum of two clearances per year will be necessary.

(v) The division of the water supply into two sources means that two sedimentation basins have to be provided. Inter-travel between the two of one dragline is thought to be impractical and therefore a second dragline will be necessary.

(vi) The amount of earthworks involved in formation of the Tawakal branch canal is vastly increased. 510 000 $m^3$ of fill would have to be borrowed and, as the excavation of deep borrow pits is not recommended, this would cause major problems.

(vii) A little downstream of Tawakal village, the branch canal has to cross a low saddle of land and with the high level of the gravity supply alternative excessive commands of up to 3.0 m occur. This would make the control of seepage losses from the canal difficult.

(viii) Because the branch canal has to be formed in a large depth of fill, the regulator structures have to be constructed high up above ground level. This means that the foundations must be much larger than is usually necessary and increases the risk of differential settlement.

(ix) By dividing the water supply of the project into two sources, the close water control and identity of the project as a single unit is lost.

(x) The alternative supply system is more expensive. A greater initial capital cost of So.Shs. 8.1 million is involved and, although the normal system involves slightly higher annual operation, maintenance and equipment replacement costs, the final internal rate of return for the alternative system is 0.4% lower than the normal supply system.

## 10.9 Non-commissioning of Gayweerow Barrage

The new Gayweerow barrage currently is being built under the requirements of the Ministry of Agriculture grapefruit production scheme. Funding is available and the barrage is regarded as committed. However, to date, only the establishment of a depot and earthwork excavation for the barrage have been completed. Consequently, brief consideration has been given to the implications of the irrigation project being implemented before the commissioning of the Gayweerow barrage.

The first obvious point is that, without the barrage to provide a holding level of 69.11 m, the gravity supply of the inlet channel and branch canals with pumping at the head of distributaries is impossible. The first alternative is to move the distributary head pump stations to form a single large pump station at the river intake point. However, this can quickly be seen to be an inefficient solution with high level canals producing high total earthworks quantities. A second alternative, making use of the gravity supply from the Asayle canal to feed the Tawakal distributaries, is much more economic in this case.

All the Tawakal distributaries can be fed from the remodelled Asayle canal, either by gravity or, in the cases of T1 and T2.2, by distributary head pumping. This is the same as the alternative system outlined in Sections 10.1 to 10.6.

The Gayweerow distributaries can no longer be fed by pumping from a cut and fill branch canal that carries a gravity supply upstream of the new barrage site. The solution is to make the best possible use of the holding level available from Qoryooley barrage. The level can be taken to be 67.20 m.

Provision of two gravity offtakes to supply sedimentation basins would be made. The pump station and night storage reservoir that feed G2, G4 and G6 can be supplied through a sedimentation basin between the river and the pump station. G8, G10 and G12 can be supplied from a long sedimentation channel with its river offtake point midway between the pump station that serves G8 and the one that feeds both G10 and G12. Figure 10.2 shows the schematic layout of the sedimentation basin and channel, together with their offtake points from the river in relation to Qoryooley barrage. There is no opportunity to gravity feed any of the Gayweerow distributaries with this system, even without night storage facilities, and therefore no changes to the distributary head pump stations are made.

A simple comparison of the cost elements for this system with the normal system of inlet channel and gravity supplied branch canal, shows that the capital costs of both are similar. However, with the double offtake, difficult silt removal operations would be necessary. For this reason this system is not proposed for the normal project, nor the alternative system with gravity supply from the Asayle canal feeding the Tawakal distributaries, providing that the Gayweerow barrage is fully commissioned.

# CHAPTER 11

## BUILDINGS AND NON-AGRICULTURAL SERVICES

### 11.1 Pilot Farm

Provision has been made for the construction and operation of a pilot farm within the Project Area. This is to provide training, production testing and research facilities.

The training programmes are designed to cover all project workers from the field supervisors through to farm and project managerial staff. The pilot farm will be run as a full production unit, in other words, a small version of the eight project farm units. This will give the opportunity for testing and modification of the proposed agricultural and irrigation practices in relation to the soils and communal farming of field units. In conjuction with this, a livestock upgrading unit will be provided. A small field research unit is to be installed to cover detailed crop variety selection and agronomic recommendations for the project.

In addition a completely self-contained poultry unit is to be installed for the improvement of hen breeds and egg production.

#### 11.1.1 Location

A site was sought that would include both the old alluvial deposits, especially the Saruda series, and the semi-recent alluvial deposits (se Map 1A, Annex I). A major restraint was that it was necessary to keep the entire farm on the river side of the proposed line of the Qoryooley outfall drain, thus avoiding the necessity for a canal crossing point.

An area very close to Qoryooley was selected which fits into the final five fields on G12 distributary, giving an NCA of 137 ha. There are three soil series encountered in this area; old alluvial deposits, Saruda and Golweyn, and semi-recent alluvial deposits of the Qoryoley series. A major advantage of this area is that it is close to Qoryooley barrage so that a temporary supply canal can easily be built before G12 is constructed.

A similar site approximately 2 km north-east of Jasiira was considered, but was rejected because it covered the area of lowest land in the Project Area and, therefore, before the full surface drainage system is constructed, would be susceptible to flooding by storm water.

#### 11.1.2 Water Supply

Ultimately the entire pilot farm will be watered as an integral part of distributary G12. However, this is not due for construction until year 6 of the implementation programme and, therefore, an alternative temporary source is necessary. This can be supplied by a 430 m long canal feeding directly from the river, 265 m upstream of Qoryooley barrage (see Figure 11.1).

A holding level of 67.20 m at the barrage means that, as long as no night storage facilities are provided, the complete pilot farm can be supplied by gravity. The lack of night storage facilities will not be a problem as the length of the supply canal, including the length coincident with the line of G12, is only 1 100 m. Consequently at night-time the complete canal can be closed at the inlet culvert adjacent to the river.

Because the supply canal will be carrying the full suspended sediment load of the river, it is recommended that it is designed with a minimum Lacey silt factor of 1.0. This will ensure that as much as possible of the silt load is passed straight through and onto the fields.

### 11.1.3 Irrigation Layout

The field layout and infield irrigation supply has been designed such that it fits in with the overall project layout. The farm will cause no interference to the later implementation of the Jasiira farm unit. Figure 11.1 shows the pilot farm irrigation layout and how the supply canal fits into the construction of distributary G12. Note that 3 ha of field unit G12/20/35.5 are reserved for the project headquarters and that, for additional flexibility, field unit G12/22/31 has been provided with two watercourses.

### 11.1.4 Drainage

Initially the Qoryooley outfall drain will not be built and the surface drains will not tie into any main disposal system. It may be necessary, therefore, to link the six drains together along the future line of the outfall drain so that any surface run-off can be guided into the depression (below 65.00 m) at the bottom of field G12/20/35.5

After year 2, the outfall drain will be operational and the drainage of the pilot farm can operate normally.

### 11.1.5 Access

Figure 11.2 shows the roads associated with the pilot farm and also gives an example of the level of access available throughout the project. The roads can be broken down into three categories.

The 10 m major surfaced road provides an all weather road link from the project headquarters to Qoryooley and the surfaced road that links to Janaale, Shalambood and Mogadishu.

The 6 m major earth roads form the main network of communications within the project. Of the 6 m roads shown on Figure 11.2, the road adjacent to the outfall drain can only be built as the drain is excavated and the surface drain control culverts constructed. The road that runs down the east side of field unit G12/18B/21.5 forms an important link, initially to re-establish the road line between Qoryooley and Tugarey which will be disrupted by the pilot farm, and later to join the project headquarters to the main access route alongside the outfall drain.

FIGURE 11.1

PILOT FARM IRRIGATION LAYOUT

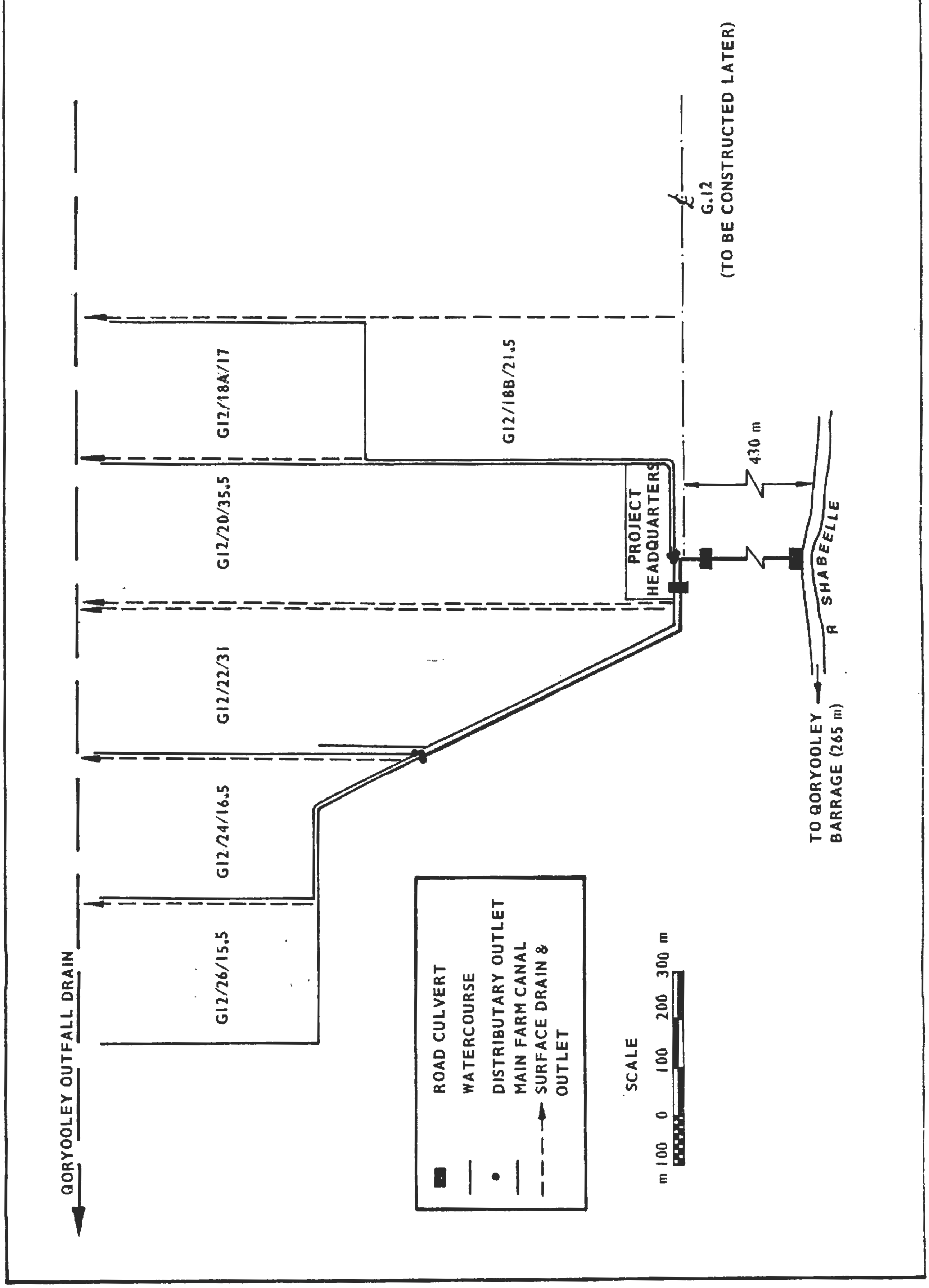
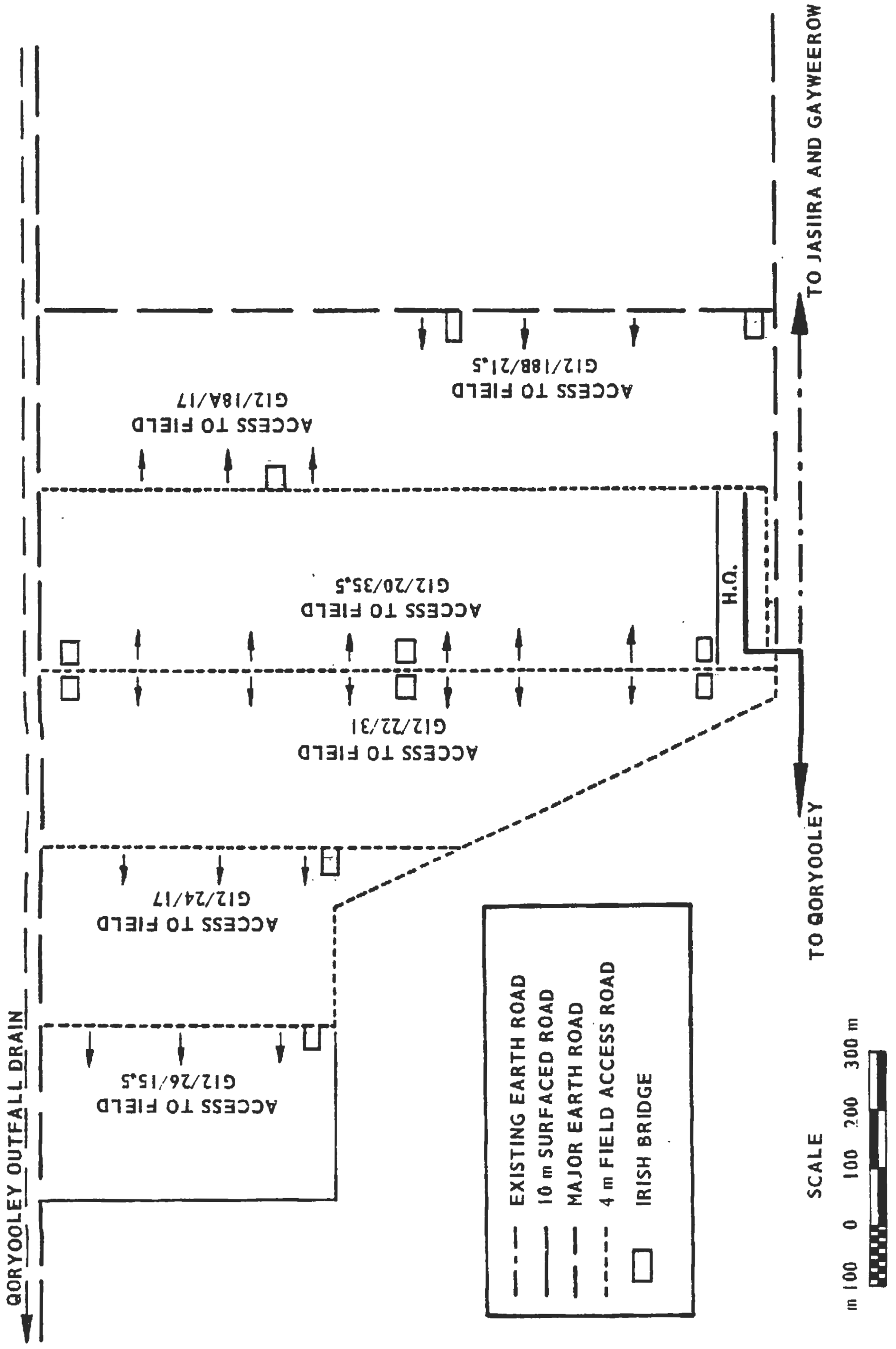


FIGURE 11.2

## PILOT FARM ACCESS



The third group of roads are the 4 m field access roads which run adjacent to and outside of each field's surface drain, providing direct access to the field across the drain. The positions of the Irish bridges to enable crossings in wet conditions are also shown. To form an interlinking system the 4 m roads often run alongside watercourses (see Figure 11.1) but this does not allow any additional field access as the watercourses form a barrier.

## 11.2 Project Headquarters

The project headquarters covers a total of 3 ha at the head of field G12/20/35.5. Within this compound buildings and machinery are held to provide facilities for the following:-

(a) Housing for the project managerial staff.

(b) Dormitory and messing facilities for trainees.

(c) Office space for all the project management staff, research staff, secretaries, clerks and operation staff.

(d) A small laboratory to provide facilities for soil moisture testing, water and soil acidity testing, water salinity determination and weighing of research trial yields.

(e) A mechanical, joinery and electrical workshop. Provision is to be made for welding and metal bending machinery for water control equipment and vehicle repairs, drilling, sawing and planing operations, puncture repairs and general vehicle maintenance operations. More sophisticated operations, such as engine cylinder head removal, are not to be attempted here.

(f) A diesel fuel tank of minimum capacity 13 500 litres.

(g) A chemical and fertiliser transit store through which the products going to all the farms must pass.

(h) A grain store to accommodate the production from the pilot farm, together with a drying slab.

(j) Research and pilot farm buildings, including stores, machinery sheds, a covered area for training and a drying slab.

(k) Livestock facilities including concrete feeding yards, loafing yards, a silage pit, maize stover silo, stores, mixing shed and a crush.

(l) The pilot farm poultry unit. This comprises a breeding house, rearing house, incubator room, wash room and egg store, together with the necessary equipment.

(m) A coral hard standing for parking project vehicles.

Table 11.1 gives a summary of the building requirements, together with the required gross areas of each type. The electricity supply for all of these is to be provided by a 130 kVA generator set installed in the workshop area.

Investigations were made into the possibility of using the electricity supply of Qoryooley. However this only operates during the evening and is of insufficient capacity to meet the project headquarters' requirements.

## TABLE 11.1

### Project Headquarters Building Requirements

| Building | Gross area ($m^2$) |
|---|---|
| Project manager's house | 200 |
| Other project management staff houses (7 No.) | 1 050 |
| Assistant management staff houses (3 No.) | 300 |
| Dormitory and mess | 400 |
| Workshop | 240 |
| Fuel tank | - |
| Grain store | 60 |
| Drying slabs | 75 |
| Chemical and fertiliser transit store | 200 |
| Machinery sheds | 60 |
| Pilot farm stores | 40 |
| Research and training facilities | 90 |
| Livestock stores | 150 |
| Mixing shed | 20 |
| Concrete feeding yards | 150 |
| Silage pit | - |
| Maize stover silo | - |
| Crush | - |
| Poultry unit | 156 |
| Coral hard standing | 300 |

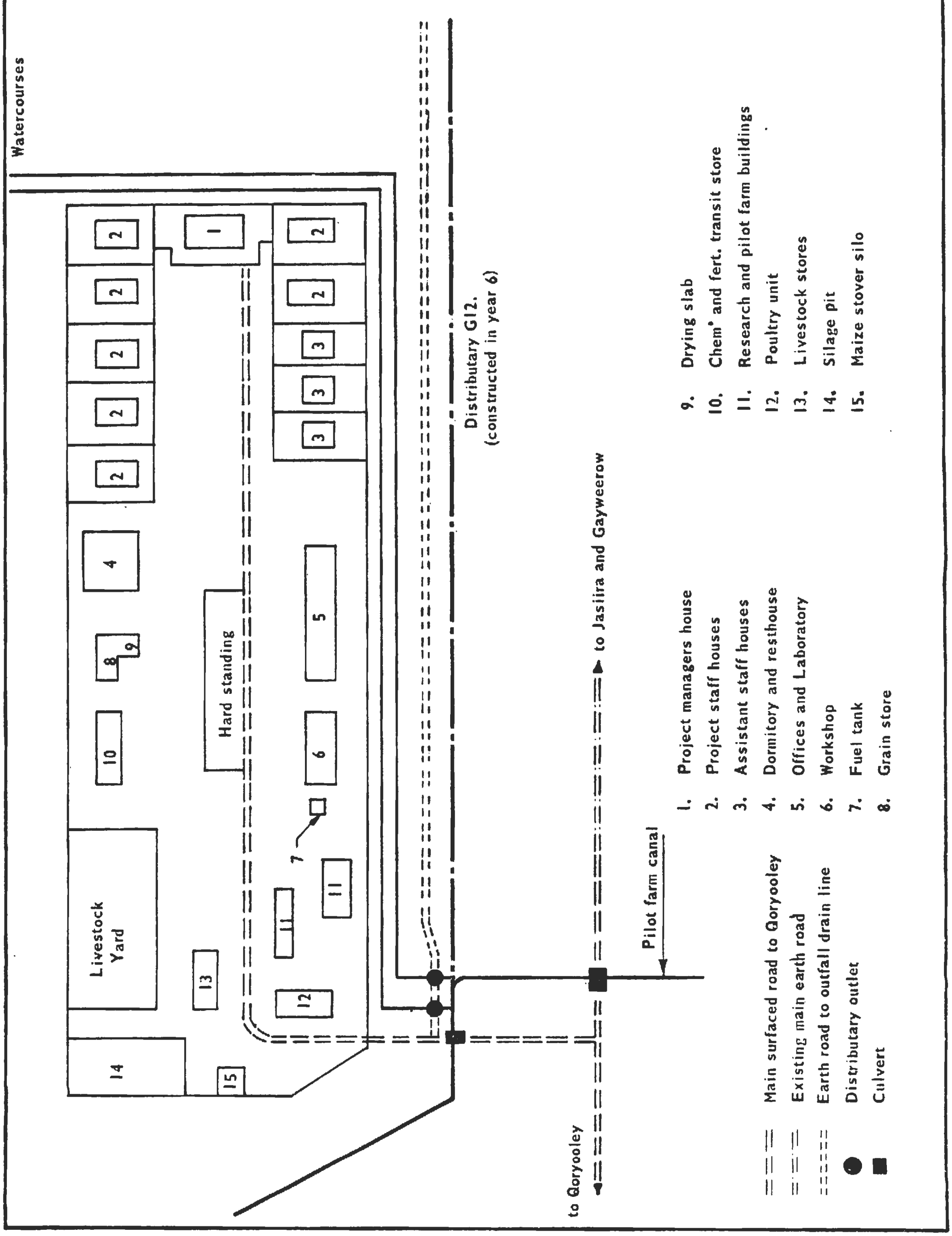
Each building must be fitted especially to collect the available rain water as this is to be used as the potable water supply for the headquarters. This procedure is common practice, both on the banana farms in the Study Area and at Jowhar sugar estate, and should guarantee an adequate water supply in all but the severest of drought periods.

For general domestic water supply the headquarters is to be provided with a 5 000 litre water tanker to collect water from the tubewell close to Janaale which is pumped continuously as a source of reasonable quality water. Throughout the Project Area groundwater is too saline for drinking purposes and the cost of treatment plants for groundwater or canal water would be excessive.

Figure 11.3 shows a layout for the project headquarters' buildings. The prime consideration in determing a layout is that the houses are placed towards the east so that they will be upwind of the farm buildings, during both the north-east and south-east monsoon winds.

FIGURE 11.3

# PROJECT HEADQUARTERS LAYOUT



## 11.3 Meteorological Station

As part of the responsibilities of the pilot farm it is recommended that a meteorological station is installed and operated at a representative site within the bounds of the pilot farm. The only existing station in the Study Area is at the farm training centre close to Janaale and is described in detail in Annex II, Section 1.5. This can be used as a standard pattern for the farm station.

During the present investigations a large variation in weather conditions was observed over the Study Area but, as the records of only one meteorological station were available, no indication could be obtained as to whether these variations were persistent or random.

A second station could rectify this and provide valuable information concerning climatic variation within the Study Area. The pilot farm is an obvious site as skilled staff will be available to help operate it and provide training facilities for operating staff from all over Somalia. As the benefit would be not only to the project but also to the Study Area and Somalia in general, the cost of the meteorological station has not been included in the project.

## 11.4 Farm Unit Headquarters

Each farm unit requires a small central farm building area with housing for the manager together with the essential farm building requirements. The exact requirements depend upon the size of the farm. As with the project headquarters the houses should be placed upwind of the farm buildings.

Each farm unit headquarters will require a 15 kVA generator set to provide electricity and each building must be adapted to collect rain water as the supply of potable water. As with the project headquarters the domestic water supply will be provided by a tanker. A workshop will be required to do only simple work such as oil changes and puncture repairs.

## 11.5 Surfaced Road

Included in the costs of the project is the provision of a surfaced road to link the headquarters to the main road at Qoryooley, a total distance of 1.4 km. This road can be extended at any time to include the villages of Jasiira and Gayweerow and ultimately to link back to the existing surfaced road by passing over Gayweerow bridge. This would entail a total of 12.3 km of new road and the complete length has been shown on the layouts (Map 3A).

## 11.6 Access within the Project

Figure 11.2 has been given and discussed in Section 11.1, to provide a typical example of field access within the project. Every field is provided with a 4 m access road adjacent to, but outside of, the surface drain and examination of the pilot farm example can show how these fit into the main access network.

There are no surfaced roads within the project and all traffic must pass along the graded earth roads provided on both sides of every branch canal, distributary canal, main collector drain and the outfall drain. Crossing points are available at every pump station, head regulator, movable weir cross regulator, pipe cross regulator and drain junction structure. This provides an adequate network of crossing points in all cases except for the Qoryooley outfall drain which forms a major east-west barrier. To alleviate this a total of five road culverts have been provided along its length.

## 11.7 Village Water Supplies

The existing villages in and around the Project Area cannot obtain their water supplies from wells because of the salinity of the groundwater. Consequently all their water has to come from the river, either directly or from the main and minor canals that cover a large proportion of the area. During the dry season these canals are often dry and villagers have to walk as far as 7 km to obtain water from the remaining pools of river water.

Likewise, with the new project canals, there will be no water in them during the dry season. However, the opportunity arises to make use of the night storage reservoirs to provide a village water supply that can last through the dry season. This involves over-deepening of the bed to give sufficient storage depth below the minimum operational level to supply the village needs, together with the losses due to evaporation and seepage, over the entire dry season (see Section 6.7).

However, careful control must be made to ensure that, due to human and livestock pollution, the reservoir does not become a major health hazard. It is for this reason that the entire perimeter of all the reservoirs must be fenced off and special provisions made for the abstraction of water.

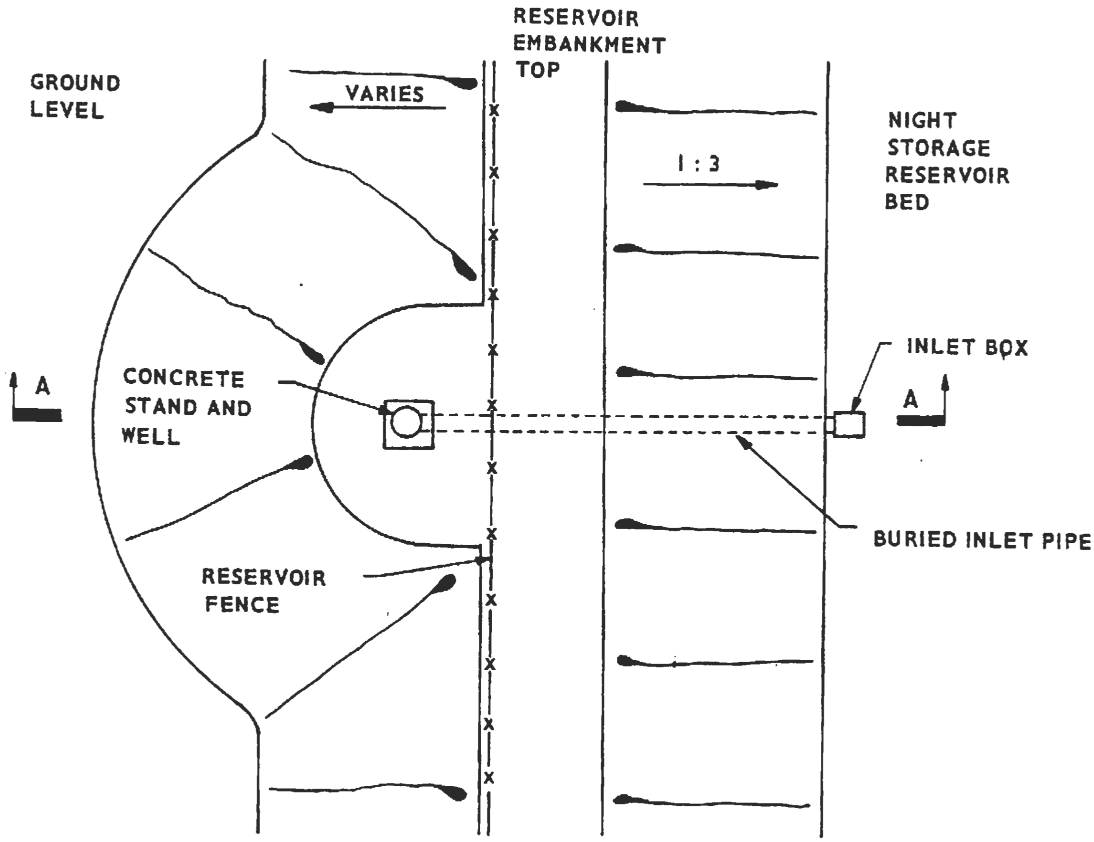
Figure 11.4 shows a simple well system that could be used to provide the village water supply to outside of the reservoir fence. The inlet box should be positioned away from the reservoir inlet/outlet culvert to avoid excessive amounts of silt settling into the inlet pipe, and its lip level set just above the reservoir bed level to stop sediment falling in. The inlet pipe should be large enough to allow hand clearance and the silt trap deep enough to allow a long period of operation before the fine sediment deposited there has to be cleared out. Water can then be extracted from the well in the traditional manner.

## 11.8 Livestock Water Supply

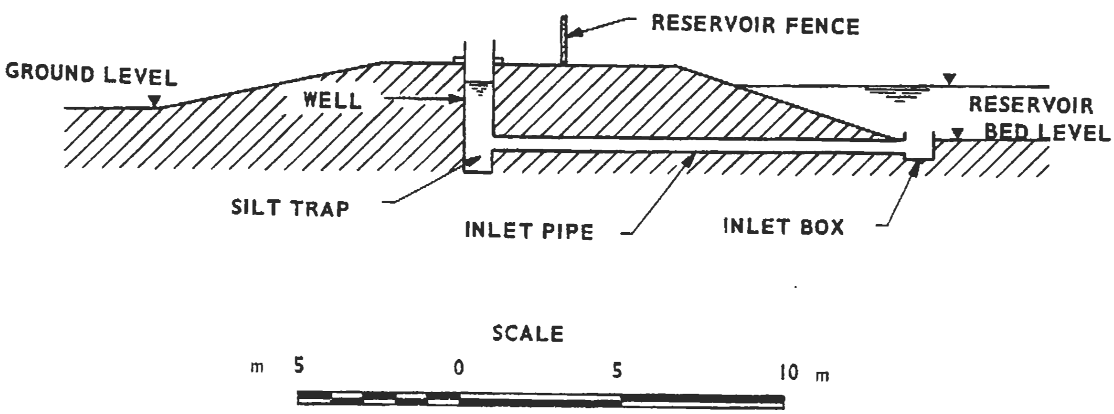
During the dry season the existing livestock population faces the same problems as the villagers; with no canal water supplies they have to be herded long distances to seek out the available water. This wasteful process can be eliminated by the use of the night storage reservoirs during the dry season. However, at all costs, cattle must be barred from drinking directly out of the reservoir. It is proposed that, where possible, water is siphoned out of the reservoirs through plastic tubing into the watering troughs. In some instances the ground level will be too high to permit this and the use of hand pumps will be needed.

FIGURE 11.4

VILLAGE WATER SUPPLY WELL




SECTION A - A



## 11.9 Vehicles

Table 11.2 lists the project vehicles, apart from all agricultural equipment that are needed. All the vehicles are to have diesel engines so that only diesel fuel tanks need to be provided at the project headquarters and the farm unit headquarters. The only exceptions are the motorcycles, but these will cause no problems because their low fuel consumption means that they can fill up with petrol at Qoryooley.

The dragline is required for desilting of the inlet channel, and the hydraulic excavators for branch and distributary canal clearance operations. The 5 tonne lorry is needed to distribute diesel drums to the pump stations; the hydraulic arm will be helpful in pump replacement and water control equipment repair operations.

The water tanker provides domestic water to the farm unit headquarters and the project headquarters by bringing it from a tubewell near Janaale. The Land Rovers are for project management staff, and the motorcycles are for the canal pump operator, livestock extension workers, assistant pilot farm managers and the farm unit managers.

**TABLE 11.2**

**Non-agricultural Project Vehicles**

| Vehicle | No. required |
|---|---|
| Rope operated dragline | 1* |
| Hydraulic excavators with desilting buckets | 2 |
| 5 tonne lorry with hydraulic lifting arm | 1 |
| 5 000 litre water tanker | 1 |
| Land Rover station wagons | 2 |
| Land Rover long wheel base pick-ups | 4 |
| Motorcycles | 15 |

Note: * 2 are required if the alternative system is adopted

# CHAPTER 12

# COSTS

## 12.1 Unit Rates

Unit rates for engineering construction are based on the local rates for the 1977 variation order for the Jowhar offstream storage reservoir. These are given on the basis of the duty free import of all construction plant and materials. Table 12.1 summarises all the main items.

As a check, other estimates were obtained from local contractors and a large variation was observed in many of the individual rates. These rates, however, were not consistently higher or lower than the Jowhar rates. As percentages in relation to the corresponding Jowhar rates, they ranged from 29% to 171%.

Where suitable local rates were not available, international rates for the Juba sugar project and for similar works in Nigeria and Iraq, have been modified for the project.

## 12.2 Compound Rates

The unit rates given in Table 12.1 have been used, together with detailed quantities for individual items, to produce the costs for all major engineering operations and structures required in the project. The complete list of compound rates for engineering and services costs is given in Table 12.2.

Certain items included in Table 12.2, such as vehicles and siphon tubing, have been costed on the basis of CIF Mogadishu costs plus an allowance of 10% for customs handling and clearance (25% for lorries). Land preparation costs have been assessed from the operation times and the local hire of the appropriate machinery.

## 12.3 Pumping Equipment Costs

The costs of pumps, pump engines and ancillary equipment, fully fitted, have been determined from a detailed analysis of the recent quotations for the Juba sugar project, where similar pumps are to be installed.

The additional civil cost has been taken as 50% of the total cost for all the pumps and fittings.

## TABLE 12.1

### Unit Rates for Engineering Costs (1977 time base)

| Item | Unit | Cost (So. Shs.) |
|---|---|---|
| Excavation to form associated embankments | $m^3$ | 7.60 |
| Supply and lay dry stone pitching 0.30 m thick with backing | $m^2$ | 290 |
| Ditto but 0.20 m thick | $m^2$ | 217 |
| 0.1 m thick concrete blinding layer | $m^2$ | 70 |
| Mass concrete in structures | $m^3$ | 737 |
| Reinforced concrete grade A | $m^3$ | 817 |
| Extra over cost for sulphate resisting cement | $m^3$ | 40 |
| Plane shuttering | $m^2$ | 135 |
| Soffit shuttering | $m^2$ | 120 |
| Mild steel reinforcement | tonne | 10 000 |
| Mesh reinforcement (A142) | $m^2$ | 155 |
| 1.20 m diameter spigot and socket concrete pipe including bedding | m | 1 650 |
| Ditto, 1.05 m diameter | m | 1 400 |
| Ditto, 0.90 m diameter | m | 1 100 |
| Ditto, 0.75 m diameter | m | 760 |
| Ditto, 0.60 m diameter | m | 510 |
| Ditto, 0.45 m diameter | m | 370 |
| Ditto, 0.375 m diameter | m | 310 |
| Ditto, 0.30 m diameter | m | 255 |
| Ditto, 0.225 m diameter | m | 208 |
| Protective bituminous Flintkote paint | $m^2$ | 20 |
| Land forming | $m^3$ | 8.0 |
| Land forming survey | ha | 90 |

TABLE 12.2

Compound Rates for Engineering and Service Costs

| Item | Unit | Cost (So. Shs.) |
|---|---|---|
| 1. Land preparation | | |
| Removal of large trees | No. | 750 |
| Light to medium dense bush clearance | ha | 500 |
| Dense thicket clearance | ha | 1 150 |
| Removal of existing structures | No. | 5 000 |
| 2. Earthworks | | |
| Formation of watercourse channel | km | 6 000 |
| 3. Canal structures | | |
| 3.10 $m^2$ vertical lift sluice gate head regulator and road bridge deck | No. | 369 000 |
| Ditto 2.20 $m^2$ | No. | 317 000 |
| Ditto 1.54 $m^2$ | No. | 317 000 |
| 2 x 2.5 m movable weir regulator and road bridge deck | No. | 463 000 |
| Ditto, 1 x 4 m | No. | 425 000 |
| Ditto, 1 x 3 m | No. | 318 000 |
| Ditto, 1 x 2.5 m | No. | 276 000 |
| Ditto, 1 x 2.0 m | No. | 231 000 |
| Ditto, 1 x 1.6 m | No. | 215 000 |
| 4 x 1.20 m diameter pipe regulator | No. | 553 000 |
| Ditto, 2 x 1.20 m | No. | 304 000 |
| Ditto, 2 x 1.05 m | No. | 245 000 |
| Ditto, 1 x 1.20 m | No. | 180 000 |
| Ditto, 1 x 1.05 m | No. | 145 000 |

cont/

TABLE 12.2 (cont.)

| Item | Unit | Cost (So. Shs) |
|---|---|---|
| 1 x 0.90 m diameter pipe regulator | No. | 115 000 |
| Ditto, 1 x 0.75 m | No. | 90 000 |
| Ditto, 1 x 0.60 m | No. | 71 000 |
| Ditto, 1 x 0.45 m | No. | 56 000 |
| 100 l/s distributary outlet | No. | 13 500 |
| Ditto, 60 l/s | No. | 11 200 |
| Tail water escape | No. | 32 000 |
| Road culvert (0.90 m diameter pipe) | No. | 61 000 |
| 6 m road bridge | No. | 316 000 |
| Inverted siphon, 0.5 $m^3/s$ | No. | 125 000 |
| 4. Drain structures | | |
| 0.45 m diameter main collector drain junction | No. | 24 000 |
| Ditto, 0.60 m diameter | No. | 30 000 |
| Ditto, 0.75 m diameter | No. | 38 000 |
| Surface drain junction culvert | No. | 5 300 |
| 1 x 0.45 m diameter pipe road culvert | No. | 26 000 |
| Ditto, 2 x 1.05 m diameter | No. | 122 000 |
| Ditto, 2 x 1.20 m diameter | No. | 140 000 |
| Ditto, 3 x 1.20 m diameter | No. | 250 000 |
| Surface water escape | No. | 35 000 |
| 5. Infield structures and equipment | | |
| 100 l/s watercourse fall or splitter | No. | 6 000 |
| 60 l/s watercourse fall or splitter | No. | 3 600 |

cont/

TABLE 12.2 (cont.)

| | Item | Unit | Cost (So. Shs.) |
|---|---|---|---|
| | Portable watercourse check | No. | 600 |
| | 40 mm plastic siphon tubing | km | 18 000 |
| | Irish bridge | No. | 2 000 |
| 6. | Surfaced roads | km | 1 325 000 |
| 7. | Buildings | | |
| | Houses and offices | $m^2$ | 2 300 |
| | Workshops and storehouses | $m^2$ | 1 800 |
| | Storesheds | $m^2$ | 1 500 |
| 8. | Vehicles | | |
| | Land Rover | No. | 55 000 |
| | Water tanker | No. | 188 000 |
| | Lorry with hydraulic arm | No. | 250 000 |
| | Motorcyles | No. | 6 000 |
| | Rope operated dragline | No. | 575 000 |
| | Hydraulic excavator | No. | 338 000 |
| 9. | Miscellaneous | | |
| | Village water supply well | No. | 14 000 |
| | Fencing | m | 54 |
| | 130 kVA generator and fittings | No. | 126 000 |
| | 15 kVA generator and fittings | No. | 26 500 |

## 12.4 Project Implementation and Final Capital Costs

Figure 12.1 is the proposed implementation programme for the project, broken down into the three phases of development: pilot farm, Tawakal development and Gayweerow development. The reasons for this phasing and the exact details of the implementation are discussed in Annex IX.

The total construction period of five and three quarters years is long and for this reason the engineering work has been divided into three parts, corresponding directly to three phases of implementation. All the quantities have been summed individually for these three parts and the capital costs assessed totally separately. Only at the very end are the three components brought together to produce a total project capital cost.

Appendix C gives the detailed capital cost estimates for the project based on the system with normal water supply totally from Gayweerow barrage. The costs are broken down into ten separate bills and the summary of these, producing the final project capital cost of So. Shs. 90 630 000 is given in Appendix E.

The detailed capital cost estimates for the alternative project, making use of the Asayle canal for part of the water supply, are given in Appendix D. Only the bills that vary from the normal supply bills are given in Appendix D. The final summary of the capital costs is given in Appendix E, alongside the summary for the normal supply project. The latter is So. Shs. 8.1 million less than the alternative supply system capital cost.

## 12.5 Annual Operation and Maintenance Costs

Appendix F gives the detailed estimates of annual operation and maintenance costs for the normal supply condition. The situation has been assessed not only for the project in full production (year 7 onwards) but also for each year prior to this as the project builds up to full implementation.

Maintenance costs have been assessed, in a good number of cases, on a percentage basis of the original capital cost (e.g. buildings 2%, vehicles 10%). Operation costs are based on the expected operation hours and fuel consumption of the machine together with a cost of So. Shs. 1.58/l of diesel fuel and So. Shs. 2.27/l of petrol An additional 10% has been allowed for oil and lubricants. The costs of operator salaries are not given in the estimates but are included on the staff roll (Annex VIII).

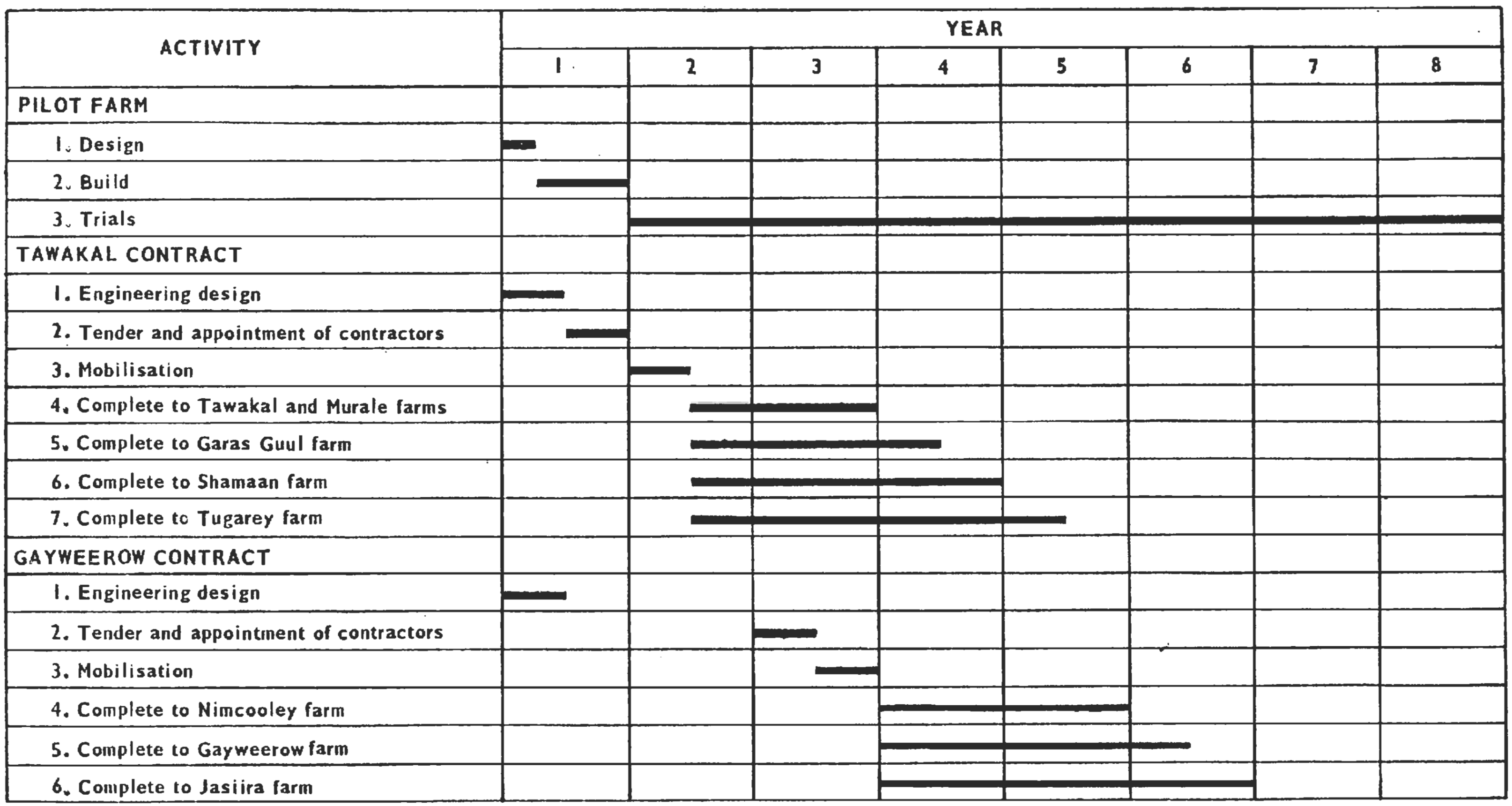
Appendix G gives the elements of annual operation and maintenance charges that are different for the case of the alternative water supply from the Asayle canal from the normal water supply arrangement. Appendix H summarises the total annual operation and maintenance costs for both water supply alternatives, together with an estimate of the foreign exchange element in annual costs.

## 12.6 Equipment Replacement Costs

Table 12.3 gives the expected lifespan of the items of equipment that will need replacement within the project horizon of 30 years. All other structures and equipment, apart from those replaced on an annual basis, such as workshop tools, have been taken as having a lifespan greater than the project horizon. The lifespans have been used to generate the replacement schedule costs given in Appendix I.

FIGURE 12.1

## CONSTRUCTION PROGRAMME



## TABLE 12.3

### Lifespan of Equipment

| Item | Lifespan (years) |
|---|---|
| Pumps | 10 |
| Pump engines | 5 |
| Generators | 10 |
| Dragline | 10 |
| Hydraulic excavators | 5 |
| Motorcycles | 5 |
| All other vehicles | 8 |

## 12.7 Foreign Exchange

All costs have been broken down to show the expected foreign exchange element. Table 12.4 lists the average foreign exchange content for the various types of operation undertaken during construction and running of the project.

## TABLE 12.4

### Foreign Exchange Elements

| Item | Foreign exchange element (%) |
|---|---|
| Preparatory and survey work | 60 |
| Earthworks | 45 |
| Canal and drainage structures | 55 |
| Pump stations | 75 |
| Workshops | 80 |
| Stores, offices, houses | 65 |
| Electricity supply | 80 |
| Surfaced roads | 50 |
| Earth roads | 40 |
| Bush clearance and land levelling | 50 |
| Engineering and supervision | 65 |
| Communications systems | 95 |
| Lorries | 80 |
| All other vehicles | 90 |

# PART III

# MASTER PLAN FOR THE STUDY AREA

# CHAPTER 13

# MASTER PLAN INTRODUCTION

## 13.1 Study Area

The complete Study Area, shown on Figure 13.1, was originally outlined by the Project for the Water Control and Management of the Shebelli River, Somalia (HTS Ltd, 1969) as a total of 70 000 ha of which 50 000 ha were on the left bank of the river and 20 000 ha on the right bank. These areas were calculated from uncontrolled photo mosaics using an approximate linear scale. The true-to-scale maps now available have shown that the area of the study is in fact 67 410 ha.

Under the terms of the agreement, a master plan for the Study Area is to be provided and Part III of this annex covers the general planning criteria and services required for the engineering development of the projects identified within the Master Plan.

## 13.2 Qoryooley Project Feasibility Study

The selection of 5 000 ha of land for a detailed feasibility study was undertaken at the beginning of the Master Plan work. The area selected, the Qoryooley project, has been studied in depth in Part II of this annex and many of the resultant compound costs (for example for the construction of distributary canals) have been used directly in the outline cost estimations for the Master Plan projects.

The feasibility study area is shown on Figure 13.1, the Study Area proposed development map, as project area A1.

## 13.3 Development Projects

In addition to the feasibility study project, a further eight major projects have been identified for the Master Plan. These range from simple upgrading or rehabilitation of existing irrigation areas (e.g. der flood project) to projects of similar sophistication to the Qoryooley project feasibility study. The full list of the nine projects, together with their references on Figure 13.1, is:-

| | |
|---|---|
| A1 | Qoryooley project (feasibility study) |
| A2 | Faraxaane project |
| A3 | Asayle project |
| A4 | Der flood project |
| A5 | Mukoy Dumis project |
| A6 | Banana drainage project |
| A7 | Ministry of Agriculture grapefruit production scheme |
| A8 | Shalambood project |
| A9 | Golweyn project |

The projects have been given outline costings so that they can be compared, where appropriate, on an economic basis and a priority ranking undertaken. The only exception to this is the Ministry of Agriculture grapefruit production scheme. In this case the project has already received funding and construction of the water supply system has begun; consequently the scheme is regarded as committed. However, certain fundamental technical doubts have arisen during the present studies concerning the scheme (Section 14.6).

## 13.4 Technical Constraints and Priorities

### 13.4.1 Barrages

The successful operation of any irrigation system in the Study Area depends upon the holding of water levels by the river barrages. During the present studies, with adequate water available most of the time, levels were maintained quite well, although only with the help of stop-gap measures. The condition of the barrages, especially Janaale, is deteriorating rapidly and it is essential that steps are taken to remedy this situation. The detailed descriptions of the barrages, together with the recommended repair works, are given in Chapter 3.

### 13.4.2 Gravity and Pumped Supplies

In the 1969 Shebelli report (HTS Ltd., 1969) it was believed that gravity irrigation of both left and right bank areas should be possible from canals offtaking at the existing Janaale barrage and the new Gayweerow barrage (under construction).

This has generally been shown to be true for the left bank with the land sloping away in the south-east and south of the Study Area to only 62 to 64 m. However, the long canal distances involved mean that in the tail areas command will only be minimal. This can be seen from the Bokore canal where command at the tail pool, 16.0 km from the headworks and with only poor water slopes for the second half of this, is practically zero. It is inevitable, therefore, that, for the provision of adequate water supplies to these areas, some element of pumping will be necessary. This is confirmed by the presence of 31 pumps on the banks of the main canals already existing in the Study Area.

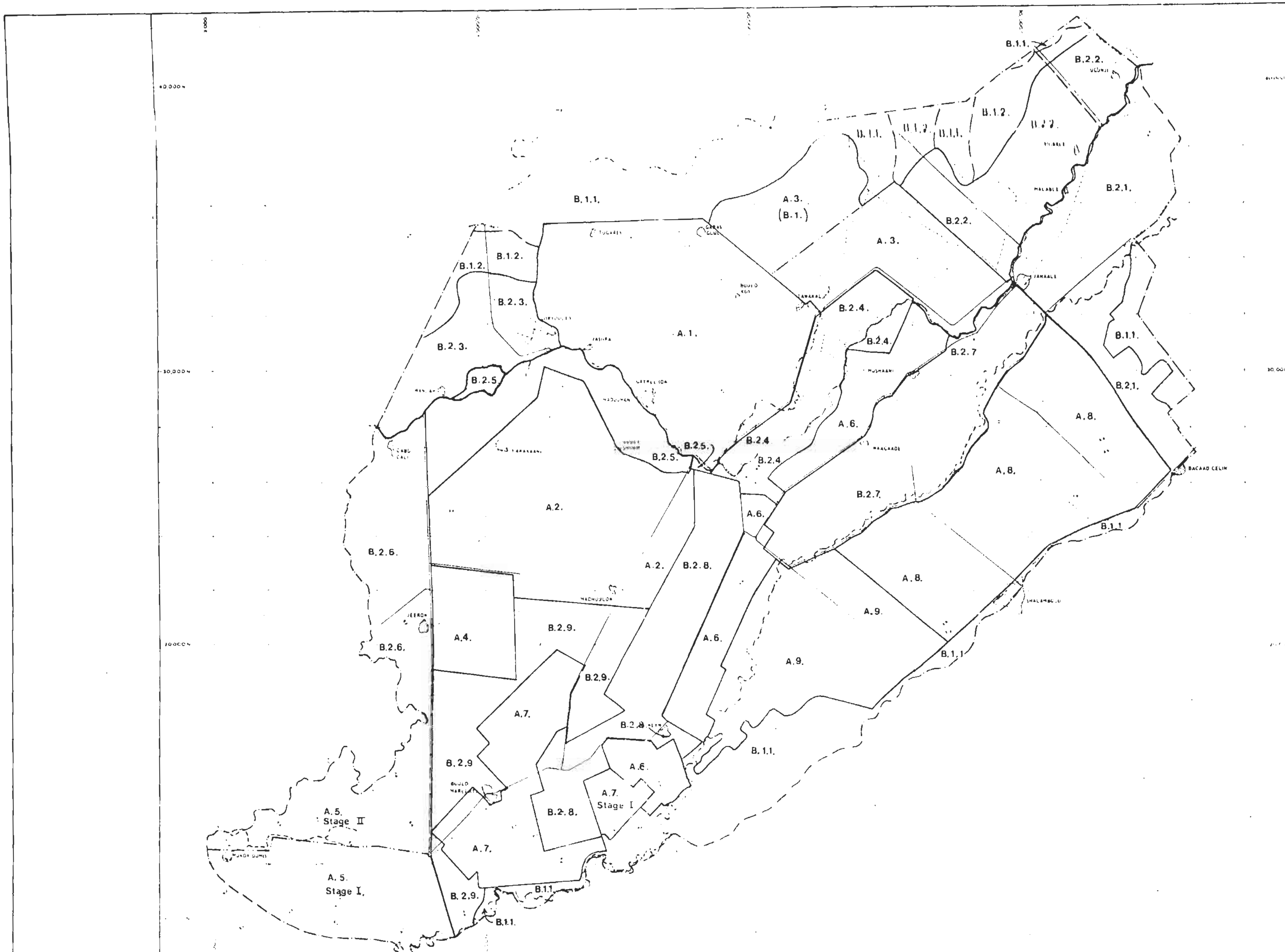
On the right bank of the river the situation is rather different. The 1969 Shebelli report stated, 'it appears that the land slopes away from the river towards the proposed boundary of the area'. This is now known not to be the case and the land only falls away from the river for about 2.5 to 3.0 km, rising again after this towards the old river channel that forms the northern boundary of the Study Area. The right bank land is therefore a rather awkward area and it is only because of the variability of the central depression between the river and the old channel that the Qoryooley and Asayle projects can be successfully irrigated. However, unless very large commands are to be accepted upstream, it does mean practically all of the Qoryooley project has to be fed by pumping.

### 13.4.3 Water Availability

The criteria for the development of the Study Area with respect to water availability are discussed in detail in Annex II.

FIGURE 13.1

## STUDY AREA – PROPOSED DEVELOPMENT




**A. PROJECT AREAS**

- A.1 Qoryooley project
- A.2 Faraxaane project
- A.3 Asayle project
- A.4 Der flood project
- A.5 Mukoy Dumis project
- A.6 Banana drainage project
- A.7 EDF grapefruit production scheme
- A.8 Shalambood project
- A.9 Golweyn project

**B. LAND DEVELOPMENT CLASSIFICATION AND ZONES**

- B.1 Non-development zones
  1. Acacia woodland
  2. Marginal agriculture
- B.2 Existing systems with upgraded technical services
  1. Janaale zone
  2. Degwariiri zone
  3. Bandar zone
  4. Majabto zone
  5. Haduuman zone
  6. Jeerow zone
  7. Waagade zone
  8. Primo Secondario Banana zone
  9. Tahliil zone

TOPOGRAPHICAL LEGEND

SCALE

The fundamental constraint that arises from this is that in the gu season water consumption, and consequently the net cultivated area, is to remain the same as with the existing irrigated agriculture within a gross project area. This limits many of the projects to a cropping intensity of less than 50% in the gu season. The der season cropping, however, is not under this constraint as adequate water is available.

### 13.4.4 Soil Salinity and Land Classification

Over the entire Study Area of 67 410 ha a reconnaissance soil survey, with a sampling density of one site per square kilometre, was undertaken. This provided information about each soil horizon concerning particle size distributions, exchangeable bases, cation exchange capacity, pH, and the electrical conductivity of the soil saturation extract. This latter parameter enabled a soil salinity map to be produced (Map 1C), showing that throughout the Study Area, apart from an area of bush on the northern edge, and some land close to the Dhamme Yaasiin canal, salinity represents, at worst, only a moderate hazard. Over more than 90% of the area the salinity hazard is low or negligible.

The basic information has been combined with other soil characteristics, especially drainability and the presence of old river channels to produce a land suitability map (1B) of the Study Area. This divides the land into four classes for surface irrigation of annual crops excluding paddy rice, namely: highly suitable, suitable, moderately suitable and unsuitable. Ninety-one per cent of the Study Area falls into the suitable and moderately suitable classes.

The remaining 9% has been classified as unsuitable. These areas are mostly thin strips of land associated with the old river channels, with only four significant blocks occurring:-

(i) An area either side of the Asayle canal. Below Tawakal this forms only a thin strip and only has a minor influence upon the Qoryooley project. However, the north-eastern end, upstream of Tawakal, is much more substantial and reduces the land available for the Asayle project significantly.

(ii) Much of the land on either side of the Dhamme Yaasiin between kilometres 1.5 and 10.3. This affects both the Shalambood project and the Waagade development zone.

(iii) An area in the middle of the Shalambood project area and part of the Janaale zone.

(iv) A large stretch of bush land on the northern edge of the Study Area. This area is not considered suitable for any form of agricultural development.

### 13.4.5 Topographical Surveying

The first priority in the detailed planning of any irrigation development is the provision of detailed topographical information. Only then can canal and drain lines, together with the detailed layout of fields, be correctly positioned. Ideally the entire Study Area should be surveyed on a 250 m grid or finer and the results used to plot contours at 250 mm intervals.

### 13.4.6 Night Storage

As irrigation during the night is generally not practised in the Study Area, a large amount of water is wasted at night by seepage into old river channels and unused reservoirs. This is an unacceptable situation, especially as development of other projects further upstream will soon be placing severe limits upon the availability of water. Therefore priority must be given, where possible, to the provision of night storage facilities so that full utilisation of the available water can be made.

### 13.4.7 Sedimentation

Night storage can only be successful if sedimentation facilities are provided to remove much of the heavy silt load from the river water. If this is not done the night storage reservoirs or canals will silt up rapidly and become inoperative. Sedimentation also provides the opportunity for having main canals operating for much longer periods before routine clearance of the silt is needed, hence reducing maintenance costs for the system. The reduction of channel capacities due to silt and weed growth is the greatest single factor causing problems of inadequate water distribution along the existing main canals in the Study Area.

## 13.5 Development Zones

In addition to the land covered by the nine identified development projects the complete Study Area remaining has been divided into land development zones. The first category (B.1) includes all the land that is considered to be unsuitable for any form of development, covering existing bush areas that cannot profitably be cleared for irrigation development and marginal agricultural areas that do not warrant upgrading.

The second category covers the development zones identified for general improvement of their existing irrigation systems. The zones are:-

| | |
|---|---|
| B.2.1 | Janaale zone |
| B.2.2 | Degwariiri zone |
| B.2.3 | Bandar zone |
| B.2.4 | Majabto zone |
| B.2.5 | Haduuman zone |
| B.2.6 | Jeerow zone |
| B.2.7 | Waagade zone |
| B.2.8 | Primo Secondario banana zone |
| B.2.9 | Tahliil zone |

The proposals for upgrading of the first eight of these, together with any features peculiar to a zone, are described in detail in Chapter 15. The ninth zone (Tahliil) is diverse in nature and it is recommended that the possibility of absorbing it into one or more of the Faraxaane, der flood or grapefruit projects should be considered. Only when further survey information is available can exact proposals for development be made for this zone.

CHAPTER 14

# DEVELOPMENT PROJECTS (ENGINEERING)

This chapter describes and costs the engineering services required for the eight development projects other than the Qoryooley project feasibility study. The latter project provides many of the basic cost elements directly but does not represent a model system upon which all of the other projects should be based. The requirements and technical difficulties will only in part be similar and therefore each project must be treated individually.

## 14.1 Faraxaanne Project

### 14.1.1 Rationale

This project is the most similar to the feasibility study and has developed from the alternative area for the feasibility study outlined in the Inception Report (Annex XI). The land is already well cultivated and the improvement of this agriculture is the main priority. However, similar to the feasibility study, no significant and functional main canalisation system exists at present and the upgrading process will require the installation of a completely new main canal system.

The Shebelli report (HTS Ltd., 1969) identified this project in descriptive terms, stating that with a holding level of about 69.0 m at Gayweerow barrage a new canal, the Faraxaane, could serve much of the area between the Wadajir and the Bokore canals.

### 14.1.2 Location

The gross project area identified on Figure 13.1 is 5 800 ha, from which it is expected that about 5 000 ha gross of irrigated land will be found after elimination of unsuitable areas due to topography or soils. Taking a land use efficiency of 80% (cf Qoryooley project 77%) this will represent a net cultivated area (NCA) of 4 000 ha.

The northern boundary of the project has been fixed by the most likely line of the new canal. The levee formation close to the river forces the canal line slightly away from the river and therefore, to avoid a thin strip of land between the canal and the main surfaced road being included in the project area, the southern edge of the main road has been taken as the northern boundary. The western edge is formed by the Bokore canal and the eastern boundary by the limit of the banana plantations irrigated from the Primo Secondario canal. The north-west limit is determined from the best line for the canal. The remaining boundary, the southern, has been fixed partly by the SISAB secondary of the Bokore canal, but mostly by the limitations of practical irrigation supply.

### 14.1.3 Existing Irrigation

Over the complete project the most effective irrigation at present is supplied by small canals directly from the river. The tortuous lines of these canals make them difficult to assimilate into a major project and reorganisation of the minor canals and infield works will be necessary.

Some of the water supply is from the Wadajir canal. However, the headworks of this are 8.2 km upstream of Qoryooley barrage and whenever water levels fall slightly the water supply is excluded from the canal. It may be possible to make use of part of this canal in a new system, even if only as a source of earth for an adjacent new canal. However, this can only be seen from the results of a detailed topographic survey.

The lack of any distributary outlets means that the area inside the project that is watered from the Bokore canal is negligible.

The total NCA of all the existing irrigated agriculture has been estimated to be 2 324 ha.

### 14.1.4 Main Canals

With the new barrage at Gayweerow bridge under construction, a holding level of just over 69 m will shortly be available from here and can be used to gravity-supply most of the project. The main canal will need to follow the line of the main surfaced road for about 9 km before turning away from the river and heading towards the village of Faraxaane (see Figure 14.1). By raising the right hand bank of this canal full flood protection for the project, but not the main road or the village of Buulo Shiikh, can be obtained.

A second, much shorter main canal will also be necessary to supply the areas towards Madhuulow. This has tentatively been shown along the line of the existing Wadajir canal.

### 14.1.5 Sedimentation

The new Gayeerow barrage is being constructed at an offstream site in a bend of the river. The river will ultimately be realigned through this and the old bend of the river, now isolated, will be used as a sedimentation basin for canals taking off from the left bank of the river. Although no technical information was available, it has been assumed that this basin will provide adequate sedimentation facilities for the Faraxaane project.

### 14.1.6 Irrigation Methods

The organisation of irrigation proposed is similar to that for the feasibility study. Watering is based on a 12 hour irrigation day and therefore night storage facilities will be necessary. From the experience encountered in the feasibility study area it is expected that distributary head offstream night storage reservoirs will be more suitable than onstream storage and the project has been costed on this basis.

Infield irrigation is to be similar, with field units fed from the distributary canals through distributary outlets and permanent watercourses. The fields will be irrigated either by long furrows or small basin irrigation.

The complete land forming of all fields is regarded as essential.

### 14.1.7 Pumping

With only a 2 m contour survey of the area, it is difficult to predict the exact limits of gravity irrigation. However, it is expected that some of the project land will require the assistance of low pumping to ensure adequate water distribution and an area of 1 000 ha NCA has been taken as supplied by pumping, the remaining 3 000 ha receiving water solely by gravity.

The use of simple, diesel driven pumps situated at the head of distributaries where necessary is recommended.

### 14.1.8 Drainage

The inclusion of a simple surface drainage system is vital for the removal of excess irrigation water and storm run-off. The groundwater under this area is of very poor quality and, despite the lack of intensive irrigation, is encountered at a relatively shallow depth. At most boreholes the water table is between 6 and 10 m below ground level. This may lead, therefore, to the relatively rapid build up in water levels to damaging levels, making the inclusion of deep field drainage a necessity at an earlier date than on the Qoryooley project.

The disposal of drainage water may cause problems, although if only non-saline surface run-off is being removed, it should be possible to pump into the Bokore canal. Figure 14.1 shows this arrangement with the pump station adjacent to the SISAB secondary canal. However, this situation must be reviewed carefully and if extensive use of the Bokore canal is planned it may be necessary to provide for alternative disposal of the water.

### 14.1.9 Costing

Table 14.1 summarises the initial capital investment costs for the engineering construction of the project and the annual operation and maintenance costs. The costs of project staffing are not included in these figures.

Capital costs have, where appropriate, been assessed from the estimates for the feasibility study. However, for some particular items preliminary designs have been undertaken and the quantities calculated. Likewise the annual charges have been found from a direct comparison, although a significant reduction has been made because of the smaller proportion of pumped water supplies. The cost of equipment and machinery replacement has been included in the annual operation and maintenance costs.

TABLE 14.1

Faraxaane Project Cost Estimate

| Item | Cost (So. Shs. x 1 000) |
|---|---|
| Land forming | 16 360 |
| Bush and thicket clearance | 110 |
| Large tree removal | 375 |
| Formation of main canals (including flood protection works) | 4 560 |
| Night storage reservoir embankments | 2 250 |
| Distributary canal earthworks | 4 043 |
| Formation of watercourses | 888 |
| Excavation of surface drains | 1 125 |
| Excavation of main drains | 1 763 |
| Main canals structures | 3 568 |
| Reservoir inlet/outlet structures | 1 184 |
| Distributary canal structures | 6 810 |
| Distributary outlets | 2 108 |
| Surface drain culverts | 1 016 |
| Main drain structures | 1 672 |
| In-field equipment (fords included) | 1 676 |
| Irrigation pump stations | 1 124 |
| Drainage pump station | 992 |
| Buildings | 17 240 |
| Services (vehicles, electricity etc.) | 4 280 |
| Contingencies (20%) | 14 629 |
| Engineering design and supervision | 6 886 |
| TOTAL capital cost | 94 659 |
| Feasibility study costs | 2 076 |
| Annual operation and maintenance cost | 1 716 |

## 14.2 Asayle Project

### 14.2.1 Rationale and Location

The Asayle canal offtakes from the river at Janaale barrage and runs down the right bank of the river for 15.8 km. The land commanded by the canal can be divided into three areas.

(i) The land between the canal and the river. This area forms a depression from which it is very difficult to remove any surface water and consequently suffers greatly from waterlogging problems. Present irrigation in this area is poor and relies more heavily upon the river than the Asayle canal for its water supply. The improvement of this land is covered by the Majabto development zone in Section 15.8.

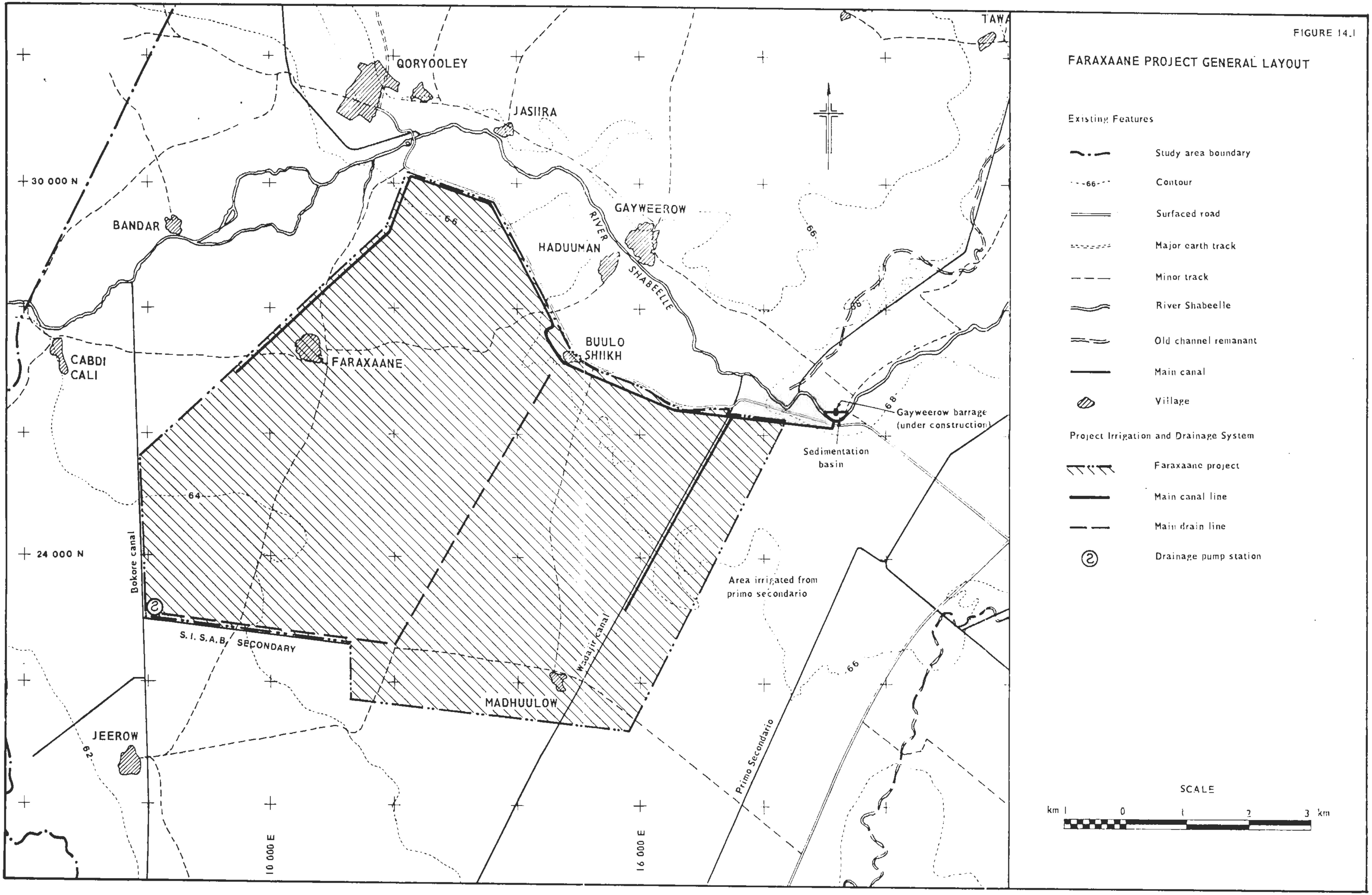


FIGURE 14.1
FARAXAANE PROJECT GENERAL LAYOUT
Existing Features
Study area boundary
66
Contour
Surfaced road
Major earth track
Minor track
River Shabeelle
Old channel remanant
Main canal
Village
Project Irrigation and Drainage System
Faraxaane project
Main canal line
Main drain line
Drainage pump station
SCALE
km 1 0 1 2 3 km
QORYOOLEY
JASIIRA
TAWA
30 000 N
24 000 N
10 000 E
16 000 E
BANDAR
GAYWEEROW
HADUUMAN
RIVER SHABEELLE
BUULO SHIIKH
FARAXAANE
CABDI CALI
Gayweerow barrage (under construction)
Sedimentation basin
Area irrigated from primo secondario
Bokore canal
S.I.S.A.B. SECONDARY
Wadajir canal
MADHUULOW
JEEROW
Primo Secondario
66
64
62
68

(ii) The land on the right bank of the canal from Janaale to the existing (but non-operational) cross regulator at Tawakal. This is large area of more than 4 000 ha gross, holding an existing net cultivated area of only 1 464 ha. This low intensity of cropping is a major drawback, with a large system of poorly maintained minor canals serving the farms spread out over the area. Even during periods of adequate riverflow major difficulties in water distribution arise, with the outlying farms and smallholders perhaps receiving only one irrigation during a season.

The Asayle project covers the rationalisation of this farm land by drawing the complete cultivated area into a much smaller unit, remodelling the Asayle canal and providing the new area with distributary canals and infield irrigation works (Figure 14.2).

(iii) The land on the right bank of the canal between Tawakal and the river outfall. This area will be taken over by the canal system designed for the Qoryooley project. Therefore, assuming that the lower cost alternative for the feasibility study with all the water supply from the Gayweerow barrage is adopted rather than the more expensive option of remodelling the Asayle canal to feed part of the project, this area will no longer be part of the land commanded by the Asayle canal.

### 14.2.2 Land Suitability

A large area of approximately 600 ha of land classified as unsuitable for development occurs within the boundaries of the project and special provision will have to be made to avoid this. The required gross cultivated area is about 2 000 ha (to contain the 1 464 ha NCA) and the boundary shown on Figure 14.2 encloses sufficient land to provide this, on top of the area lost to unsuitable land. This is thought to correspond approximately to the limit of successful gravity irrigation.

### 14.2.3 Janaale Barrage

The success of gravity feed down the Asayle canal relies upon the correct operation and maintenance of the barrage at Janaale. At the present time the water control equipment is rapidly deteriorating and will soon be inadequate to provide the close control required. Therefore priority must be given to the repair of the barrage before any remodelling of the Asayle canal is considered.

### 14.2.4 Asayle Canal Remodelling

The existing cross-section of the Asayle canal is rather small, with a bed width of about 3 m and a typical water surface width of 7 m. However, adequate water slopes of nearly 25 cm/km over the upper 8.9 km of the canal mean that the canal capacity, shortly after silt clearance, is more than 4 $m^3/s$. With the loss of the Qoryooley project land the required capacity of the canal, at inlet, should be less than 3 $m^3/s$.

However, to provide adequate command in the region of Tawakal it will be necessary to raise the tail of the upper section of canal slightly. This will entail significant remodelling.

Because of the change in water levels the existing cross regulatros will have to be replaced. However, only one of these is in an operational condition and new structures are needed. The provision of three regulation structures and two road culverts (see Figure 14.2) is expected to be sufficient to provide complete control for full distributary canal coverage of the cultivated area of 1 464 ha.

### 14.2.5 Gravity Supply and Night Storage

It is hoped that, as with the existing irrigation in the Asayle project area, the supply of water can be entirely by gravity. However, the provision of night storage facilities requires, apart from pre-sedimentation of the water, a considerable amount of head. For distributary head reservoir night storage a minimum of 0.6 m of command is required for the reservoir and its structures. Consequently, it is not possible to provide night storage together with a gravity supply. However, the lack of this facility is thought to be less critical in this particular case for two reasons:-

(i) The larger farmers will be able to include some element of night-time watering for basin irrigation.

(ii) Excess night-time flows not used by the project can be passed down the Asayle canal and returned to the river for use further downstream.

### 14.2.6 Siltation

The Asayle canal becomes choked with silt rather more quickly than, for example, the Primo Secondario canal. This is due not to the lack of adequate water slopes, but to the insufficient demand for water from the minor canals. Because of this the canal discharges are too small to maintain the heavy sediment load in suspension. The new project must ensure that not only the Asayle canal but also the secondary canals are designed such that as much as possible of the sediment can be passed right along the canals and onto the fields.

### 14.2.7 Infield Irrigation

The standard system of field units, with distributary outlets and permanent watercourses is recommended for the project, with complete land forming of all the fields. However, with the majority of the heavy sediment load being passes onto the field it may be inappropriate to use long furrow irrigation as these will silt up rapidly. The more extensive use of traditional basin irrigation is therefore envisaged.

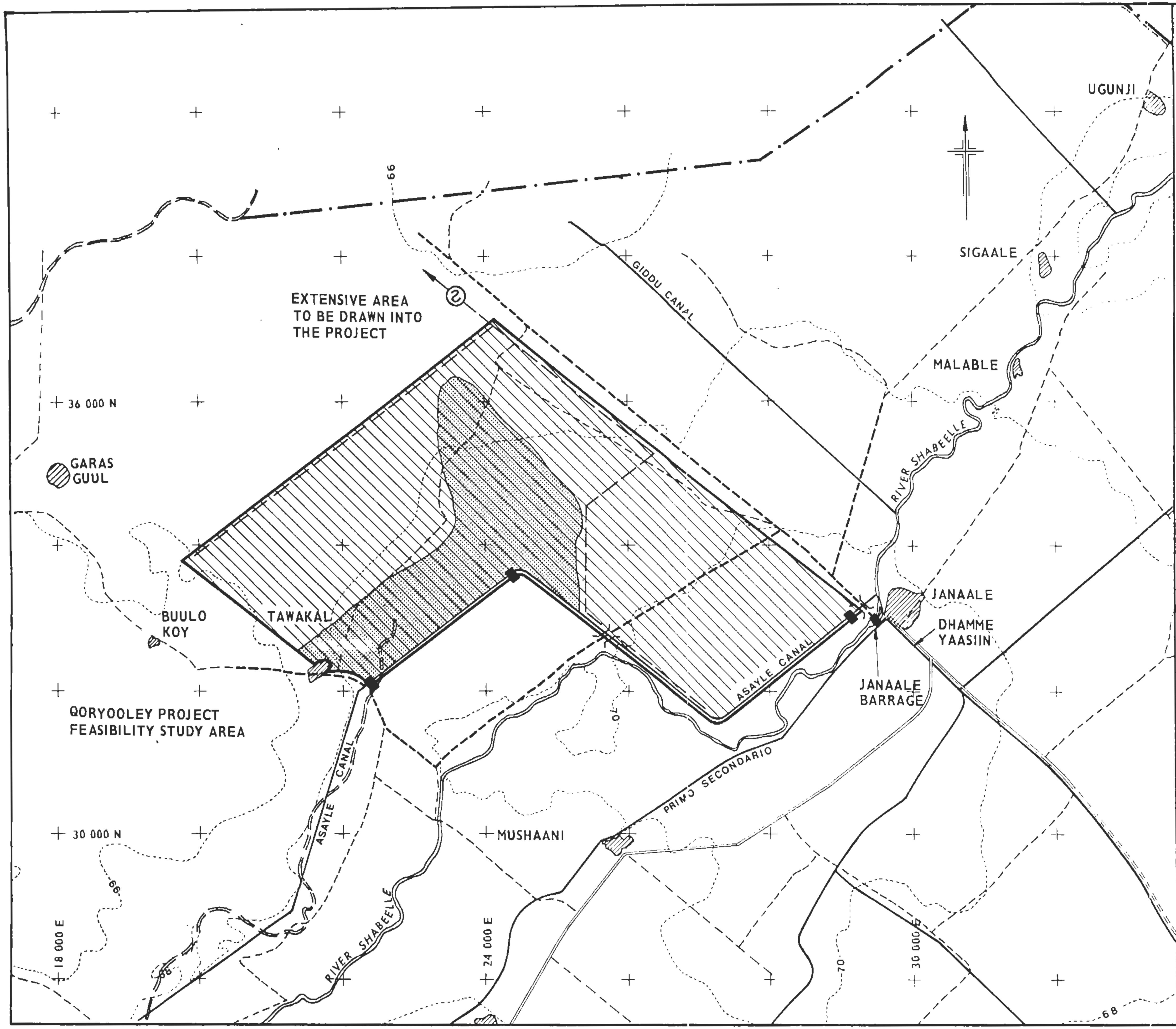
### 14.2.8 Drainage

The project costings include for the provision of a surface drainage system but no deep drainage. This should be perfectly adequate as the water table drops away rapidly from the river in this region.

The disposal of drainage water may pose problems as no suitable old river channel can be seen on the photographs close to the disposal point. One possible solution may be to reverse the direction of the drain shown in Figure 14.2 on

FIGURE 14.2

ASAYLE PROJECT GENERAL LAYOUT




Existing Features

| | |
|---|---|
| | Study area boundary |
| --66-- | Contour |
| | Surfaced road |
| | Major earth track |
| | Track |
| | River Shabeelle |
| | Old channel remnant |
| | Main canal |
| | Village |
| | Class 6 land unsuitable for irrigation development |

Proposed Irrigation and Drainage System

| | |
|---|---|
| | Important tracks |
| | Asayle project |
| | Main drain lines |
| | Road bridge |
| | Regulator on the Asayle canal |
| 2 | Drainage pump station |

SCALE

m 1 0 1 2 3 m

the north-west boundary of the project, pass the water under the Tawakal branch canal of the Qoryooley project and thence into the drainage system for this project.

### 14.2.9 Costings

Table 14.2 summarises the initial capital investment costs for the engineering construction of the project and the annual operation and maintenance costs. The volume of earthworks required for remodelling has been calculated from the survey information of the canal in Annex X.

Because no desilting works are provided the cost element due to silt clearance has been increased over that required for the feasibilty study.

**TABLE 14.2**

**Asayle Project Cost Estimates**

| Item | Cost (So. Shs. x 1 000) |
|---|---|
| Land forming | 5 988 |
| Bush and thicket clearance | 315 |
| Remodelling of Asayle canal | 912 |
| Distributary canal earthworks | 1 480 |
| Formation of watercourses | 325 |
| Excavation of surface drains | 412 |
| Excavation of main drains | 645 |
| Asayle canal structures | 2 669 |
| Distributary canal structures | 1 200 |
| Distributary outlets | 772 |
| Surface drain culverts | 372 |
| Main drain structures | 612 |
| In-field equipment (including fords) | 613 |
| Drainage pump station | 545 |
| Buildings | 4 870 |
| Services (vehicles, electricity, etc.) | 1 209 |
| Contingencies (20%) | 4 588 |
| Engineering design and supervision | 2 173 |
| TOTAL capital cost | 29 700 |
| Feasibility study costs | 760 |
| Annual operation and maintenance cost | 517 |

## 14.3 Der Flood Project

### 14.3.1 Rationale

The object of this scheme is to make use of the spare river water in October to flood irrigate large rectangular basins to a depth of about 0.3 m. The water is then allowed to infiltrate slowly and when the soil surface is dry enough to walk on, sesame is planted. This will grow using the residual moisture held within the rooting depth of the mature plant (see Section 2.4) combined with a small contribution from the der season rainfall which usually falls in November and early December.

This method of irrigation is already widespread in the Study Area and is used for the growth of sesame. No other crops are grown in this way, and it is thought that the limited rooting depth of many crops on heavy clay soils precludes their irrigated production in this manner.

### 14.3.2 Location

With the production of only one main crop per year the project must make as much use as possible of the existing main canal system. In addition, an area where the current land use is low must be found, otherwise the increased net agricultural benefit will not justify the required capital investment.

The only area to satisfy both of these requirements is the almost totally unused land on the left bank of the Bokore canal just downstream of the SISAB secondary canal. Use can be made of these two canals to gravity feed a gross area of 1 200 ha (960 ha NCA).

The Bokore canal forms the western boundary, and the SISAB secondary, the northern. The other two boundaries can only be fixed exactly after detailed survey work has been completed, but for planning purposes the simple parallelogram shape shown in Figure 14.3 has been adopted.

### 14.3.3 Bokore Canal

The top 7.6 km of the Bokore canal have an average water slope of 15 cm/km and are generally slow to silt up. In fact, close to the headworks, a sandy bed deposit rather than clay was observed. The maximum recorded discharge at the head was 2.7 $m^3/s$; this occurred despite the breakdown of the hydraulic excavator and therefore the canal was not cleared of silt during the present studies. The capacity after clearance is expected to be greater than 3.0 $m^3/s$. Unlike the other main canals in the Study Area, the complete set of nine head regulator gates were fully operational.

The complete der flood project will have a maximum water requirement of only 0.5 $m^3/s$ and therefore the Bokore canal is capable of providing this without any remodelling. The only work necessary is to provide a new cross regulator on the canal to control the water level at the point of the SISAB canal head regulator.

### 14.3.4 Distributaries

The SISAB canal will require remodelling and new head and cross regulators. From it three new distributary canals can be fed to provide gravity supplies to the entire project.

### 14.3.5 Field Unit

The basic field unit will be rather different from the standard field recommended for the Qoryooley project. From preliminary investigation a suggested layout is to have five rectangular basins of 4 ha each with permanent perimeter bunds, fed from one single permanent watercourse. Water could be fed into the basin through permanent concrete pipes bedded into the side of the watercourse. A simple bung would keep them closed when not in use.

The whole project would have a total of 48 field units, each requiring a total water application of 0.35 m during the month of October. If these are watered eight at a time for five days the required watercourse discharge will be 160 l/s and therefore the individual basin discharge 32 l/s. Note that night-time irrigation will pose no problems as the basins can be left to fill by themselves without any difficulties.

Within the large basins land levelling (as opposed to land forming for furrow or small basin irrigation) will be necessary and an estimated 450 $m^3$/ha of earthmoving will be needed. The final levels do not have to be as accurate as with land forming since efficient use of water is not as critical, and therefore a considerably cheaper rate of working can be adopted.

### 14.3.6 Food Crop Production

Sesame production, especially at harvest time, is a labour intensive process and therefore a considerable number of people will require settling in and around the project area. To provide their own food, the cropping part of the project will have to be reserved for maize. This can be done by breaking some of the large basins up into smaller units and irrigating them in the traditional manner for maize.

A total NCA of 229 ha already exists in the project area and therefore, within the constraint of not increasing the cropped areas in the gu season, a total of 57 large basins can be subdivided for maize production in the gu season. The area of maize required in the der season will depend upon the specific requirements for food and the success of the previous season's crop.

### 14.3.7 Drainage

Despite the simplicity of this project, the provision of a surface drainage system to draw off excess irrigation water that will not infiltrate, and to remove storm run-off, is still essential. Each large basin should be provided with an outlet pipe (similar to the inlet) to permit the drainage of water into a field unit shallow drain. These drains can then pass the water into the disposal drains shown on Figure 14.3. No junction structures will be necessary as the drainage system will not be operated as frequently as under an intensive irrigation project. Likewise, no special provision needs to be made for the disposal of the infrequent drainage waters outside the project area.

### 14.3.8 Costing

Because the der flood project is rather different from the Qoryooley project most of the items had to be assessed individually. Table 14.3 summarises the project capital investment costs and the annual operation and maintenance costs.

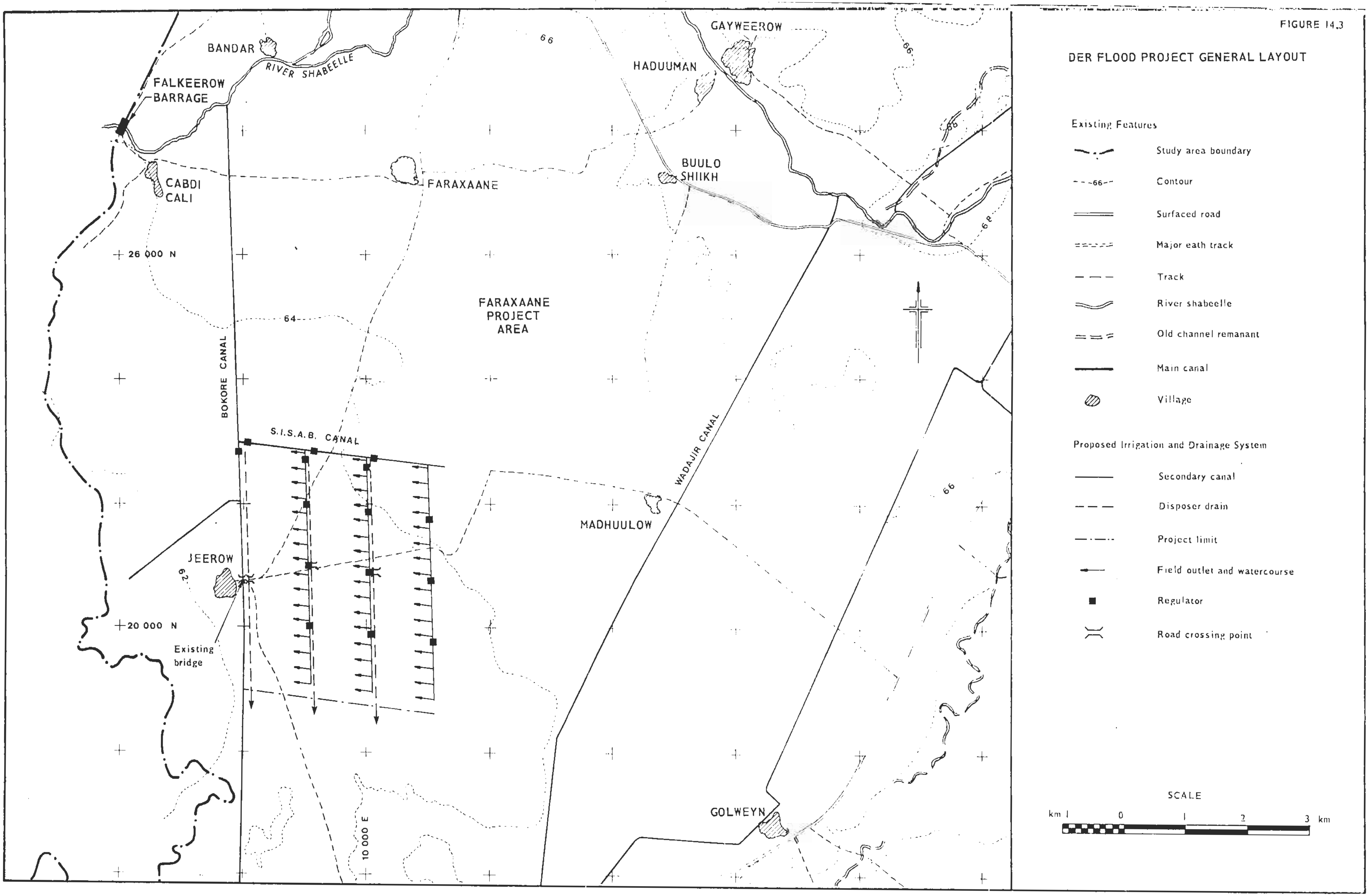
TABLE 14.3

Der Flood Project Cost Estimates

| Land | Cost (So.Shs.x 1 000) |
|---|---|
| Land levelling | 1 814 |
| Bush and thicket clearance | 48 |
| SISAB canal remodelling | 319 |
| Distributary canal earthworks | 838 |
| Formation of watercourses | 213 |
| Formation of basin bunds | 1 167 |
| Excavation of surface drains | 270 |
| Excavation of disposal drains | 135 |
| Canal structures | 2 270 |
| Drain culverts | 183 |
| Distributary outlets | 763 |
| Infield equipment | 165 |
| Buildings | 215 |
| Services | 53 |
| Contingencies (20%) | 1 691 |
| Engineering design and supervision | 840 |
| TOTAL capital cost | 10 984 |
| Feasibility study cost | 498 |
| Annual operation and maintenance cost | 88 |

FIGURE 14.3

DER FLOOD PROJECT GENERAL LAYOUT



## 14.4 Mukoy Dumis Project

### 14.4.1 Rationale

The aim is to provide a fully equipped irrigation and drainage system to a project in an area of previously undeveloped land. This means that no reduction in the net agricultural benefits has to be made because of the agricultural production that would occur in the 'without project' case. However, this has to be balanced against the additional development costs (especially bush clearance) and the limited cropping intensity possible. The latter arises because no increase in the gu season cropped areas, over and above the existing level, can be allowed due to the critical water supply situation in June and July. Clearly the cropping in a new development area will therefore have to be limited to the der season only.

### 14.4.2 Location

The only significant area of undeveloped land in the Study Area occurs in the extreme south-west corner between the study boundary and the lower reaches of the Bokore canal. This encompasses a total area of 4 270 ha. No other areas of uncultivated land exist other than the narrow marginal strips of bush close to the south-eastern and northern boundaries of the Study Area. These are all sufficiently broken and isolated from the main water supplies, with often very awkward topography for surface irrigation methods, not to warrant further investigation.

### 14.4.3 Development Areas

Two major old river channels run through the area, one forming the southern boundary of the project and the other splitting the area approximately into two by running from the tail pool of the Bokore and Primo Secondario canals to Mukoy Dumis. This latter channel forms a major break in the area and for this reason the project has been broken into two development phases. Only the first phase, formed from the slightly larger area of 2 210 ha in the southern half, has been considered for costing purposes at this stage.

Within this area the land is rather broken and uneven, with many isolated depressions and ridges shown by the 2 m contours. Because of this, and because the line of the new Golweyn to Jelib road cuts through this area, the phase one land has been sub-divided into four zones. These have a total NCA of 1 650 ha.

### 14.4.4 Bush Clearance and Land Forming

Apart from some small areas close to the Bokore canal, that were cleared in a futile attempt to gravity feed them from the canal, the entire project is covered in light to medium bush which will have to be cleared. In addition significant strips of dense thicket occur, especially adjacent to the old river channels, which will involve lengthy machine hours to remove. Few large trees exist.

The land forming quantities are expected to be greater than for the feasibility study. This arises from the broken nature of the micro-topography in the bush compared to the semi-remodelled existing cultivated areas. However, no additional earthmoving will be necessary to remove existing canals and for this reason the final figure of 500 $m^3$/ha has been adopted for this project.

### 14.4.5 Bokore Canal

The existing Bokore canal can be used to convey the project water requirement of about 2 $m^3$/s the 16 km from the river to the project. However, the water slopes in the second half of the canal are only 10 cm/km and require steepening before the canal can carry sufficient water. This involves deepening the bed, together with increasing the canal width. Over the complete 16 km this represents a large volume of earthworks for remodelling.

### 14.4.6 Pumping

The deepening of the bed of the Bokore canal to increase water slopes means that there is no possibility of gravity feeding the project and pumping is necessary. At first the possibility of a single pump station at the tail pool was investigated. However, outline designs quickly showed that, with the rather broken topography of the area, excessive command and consequently very large earthworks quantities would occur. The second alternative of distributary head irrigation pump stations was therefore adopted and costed.

### 14.4.7 Sedimentation and Night Storage

Night storage facilities at the head of distributaries will be provided and therefore some sedimentation facility must be provided. It is difficult to determine the optimum position for this, but remodelling and enlarging of the tail pool of the Bokore and Primo Secondario canals may prove to be an attractive proposition.

### 14.4.8 Infield Works

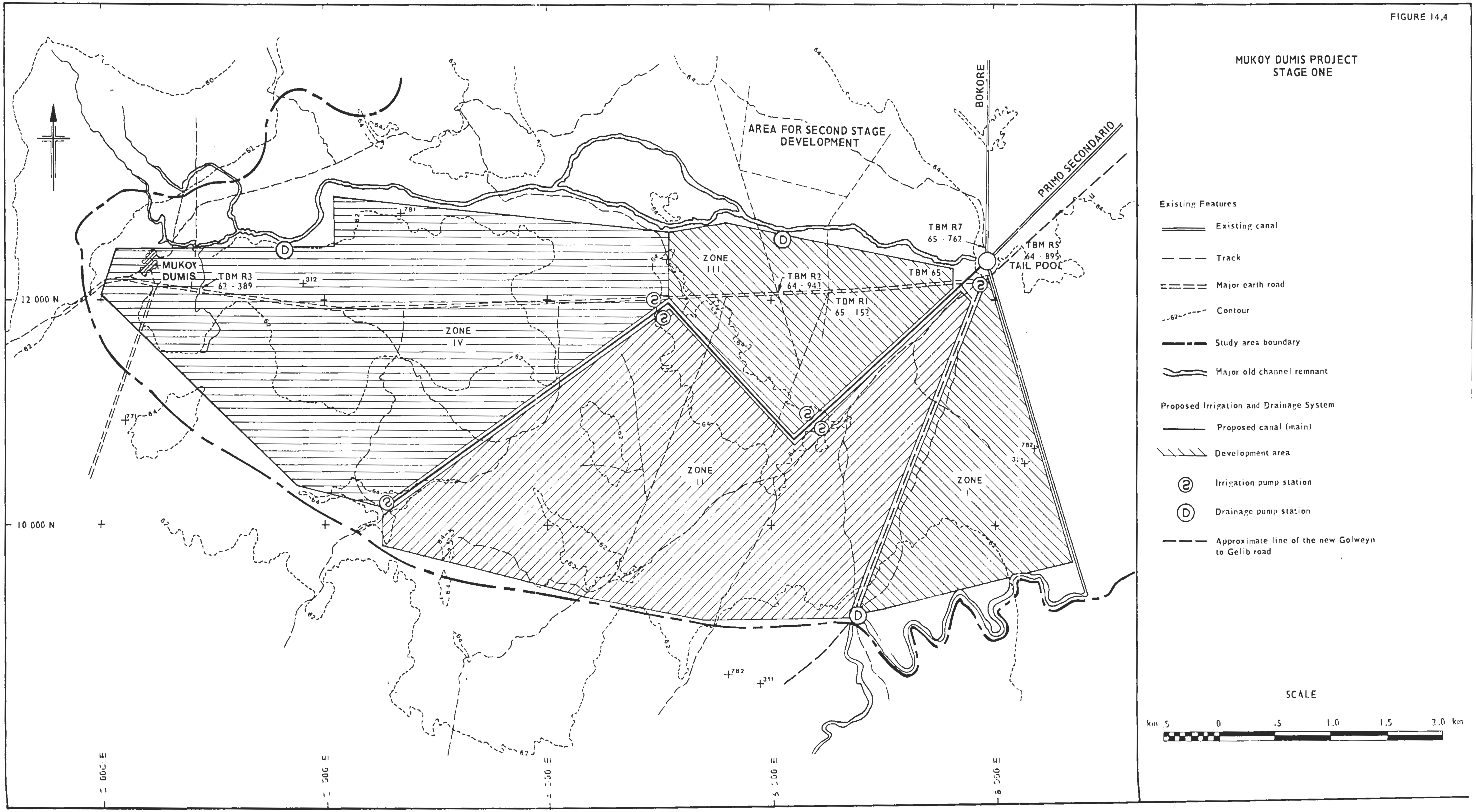
The provision of field units similar to the feasibility study with distributary outlets and permanent watercourses is proposed.

### 14.4.9 Drainage

The rather broken nature of the area makes it impractical to collect all the water from the surface drainage system to a single point for disposal. Instead Figure 14.4 shows three drainage pump stations which can lift the water directly from the main collector drains into the old river channels.

The groundwater table is very deep in this area and the soils marginally lighter than the majority of the Study Area. Consequently no problems of waterlogging or salinity are envisaged and no provision for deep drainage has been made.

FIGURE 14.4


MUKOY DUMIS PROJECT
STAGE ONE
AREA FOR SECOND STAGE DEVELOPMENT
BOKORE
PRIMO SECONDARIO
MUKOY DUMIS
TBM R3 62 · 389
TBM R7 65 · 762
TBM R5 64 · 895
TAIL POOL
TBM 65
TBM R2 64 · 942
TBM R1 65 · 152
ZONE III
ZONE IV
ZONE II
ZONE I
12 000 N
10 000 N
Existing Features
Existing canal
Track
Major earth road
Contour
Study area boundary
Major old channel remnant
Proposed Irrigation and Drainage System
Proposed canal (main)
Development area
Irrigation pump station
Drainage pump station
Approximate line of the new Golweyn to Gelib road
SCALE
km .5 0 .5 1.0 1.5 2.0 km

### 14.4.10 Costing

Table 14.4 summarises the capital investment costs and the annual operation and maintenance costs for the project. The building cost includes for a similar level of management as the feasibility study together with the settling of a large number of families to farm the land. This is necessary as Mukoy Dumis is the only significant village in the entire project area.

**TABLE 14.4**

**Mukoy Dumis Project Cost Estimates**

| Land | Cost (So.Shs.x 1 000) |
|---|---|
| Land forming | 6 749 |
| Bush and thicket clearance | 1 115 |
| Remodelling of Bokore canal and tail pool | 1 155 |
| Formation of main canals | 608 |
| Night storage reservoir embankments | 928 |
| Distributary canal earthworks | 1 668 |
| Formation of watercourses | 366 |
| Excavation of surface drains | 464 |
| Excavation of main drains | 552 |
| Bokore canal new structures | 958 |
| Main canal structures | 1 051 |
| Distributary canal structures | 2 338 |
| Distributary outlets | 870 |
| Surface drain culverts | 419 |
| Main drain structures | 690 |
| Infield equipment (including fords) | 691 |
| Irrigation pump stations | 1 855 |
| Drainage pump stations | 840 |
| Buildings | 7 112 |
| Services (vehicles, electricity etc.) | 1 766 |
| Contingencies (20%) | 6 439 |
| Engineering design and supervision | 3 043 |
| TOTAL capital cost | 41 677 |
| Feasibility study cost | 1 199 |
| Annual operation and maintenance cost | 834 |

## 14.5 Banana Drainage Project

### 14.5.1 The Need for Drainage

Over the forty years of intensive irrigation within the areas now producing bananas, the groundwater table has steadily been rising, at rates of up to 0.84 m/year but generally at much slower rates. Over much of the area that has been irrigated for the longest period (a narrow strip running between Janaale and the Qoryooley road) the groundwater table is now less than 2 m below ground level.

This is having several serious consequences upon the banana production:-

(i) Waterlogging of the root zone is causing yield reduction and generally feeble vegetative growth.

(ii) The proximity of the groundwater table restricts the rooting depth of the bananas, making them more prone to collapse during the strong jilal winds.

(iii) The poor condition of the plants has left them more susceptible to nematode infection. This problem reduces the life span of the banana plants and replanting is now necessary about every four years.

(iv) The amount of infiltration and downward movement of water through the soil has become limited. Irrigation of a banana field begins after a fallow period and at the start infiltration is usually adequate. However, this slowly deteriorates until after three or four years the farmers have difficulty in applying sufficient water to the soil before the risk of damage due to surface waterlogging forces them to drain off the uninfiltrated water into the depression around the field.

Despite the reduced downward movement of water, no significant build up of salinity to hazardous levels has been observed.

The proposal for the banana drainage project is to install a main disposal drain line through the banana production areas so that, where necessary, fields can be installed with buried field drains which are linked to the main drain. This would provide the opportunity to halt the rise in water tables, increase the downward movement of water through the root zone and thereby increase the possible yields from the banana plants. This is to be done in conjunction with a programme for nematode control to help increase the life span of the plants.

The drainage system can also be used for the disposal of surface water from the area. This will allow the drainage of excess irrigation water and storm run-off together with the dewatering of the stagnant pools of water that occur in the areas.

### 14.5.2 Main Drain Line

The majority of the banana farms are irrigated from the Primo Secondario canal and, by running a parallel main drain line, effective drainage of a large proportion of the bananas can be achieved with the single main disposal line.

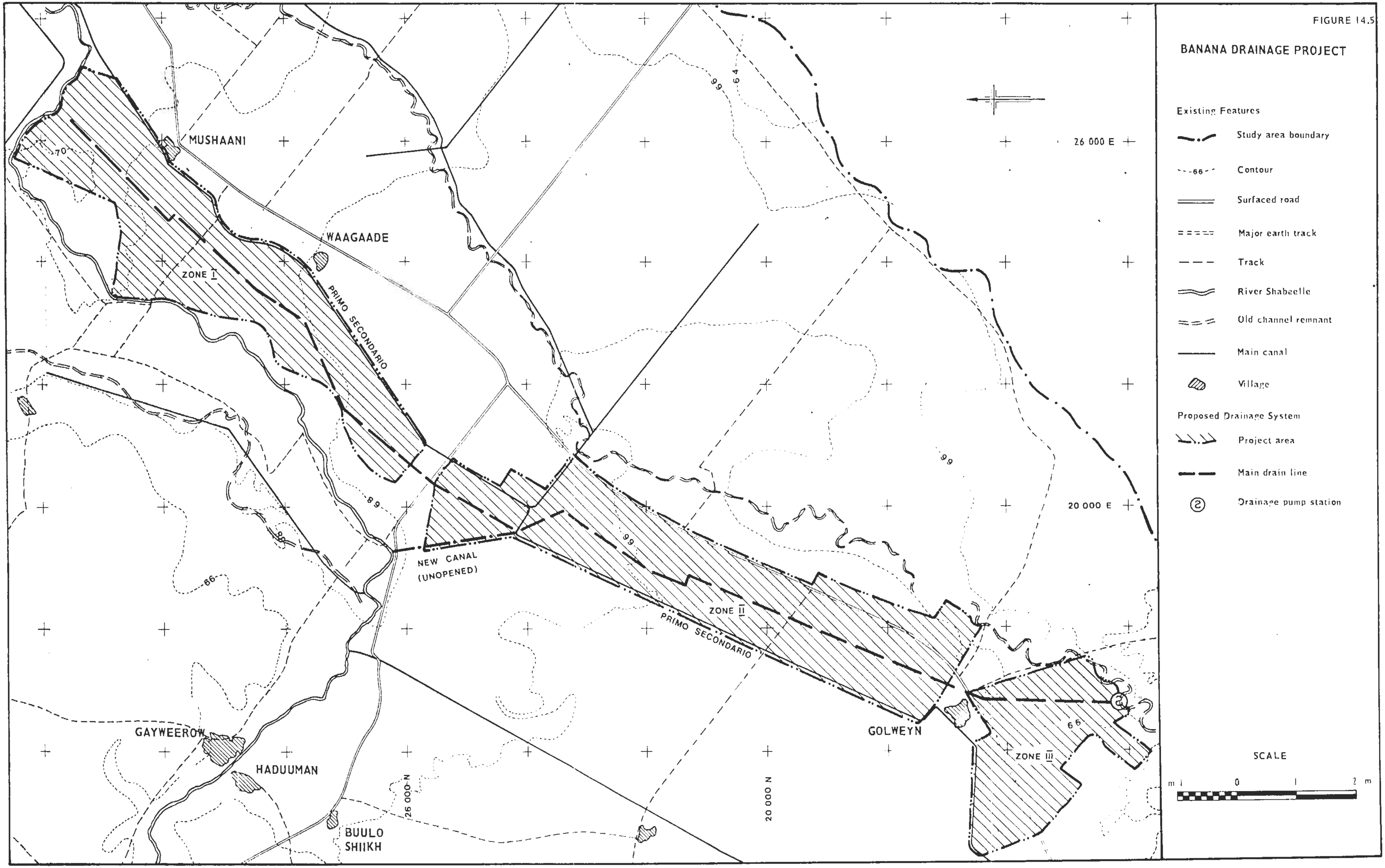


FIGURE 14.5
BANANA DRAINAGE PROJECT
Existing Features
Study area boundary
66
Contour
Surfaced road
Major earth track
Track
River Shabeelle
Old channel remnant
Main canal
Village
Proposed Drainage System
Project area
Main drain line
2
Drainage pump station
SCALE
m 1
0
1
2 m
MUSHAANI
WAAGAADE
ZONE I
PRIMO SECONDARIO
NEW CANAL
(UNOPENED)
ZONE II
PRIMO SECONDARIO
ZONE III
GOLWEYN
GAYWEEROW
HADUUMAN
BUULO
SHIIKH
26 000 E
20 000 E
26 000 N
20 000 N
70
99
89
66

Figure 14.5 shows the drain line in relation to the Primo Secondario and the banana farms it would be able to drain. The banana farms have been broken into three zones and Table 14.5 gives the details of these. The total NCA of 1 560 ha is equivalent to 46% of the entire productive area of bananas in the complete Study Area.

TABLE 14.5

**Banana Zones included in the Drainage Catchment**

| Zone | Gross cultivated area (ha) | Net cultivated area (ha) |
|---|---|---|
| I | 1 150 | 634 |
| II | 1 140 | 628 |
| III | 540 | 298 |
| TOTAL | 2 830 | 1 560 |

Zone I, covering the bananas locked between the Primo Secondario canal and the river, corresponds to the area most urgently requiring drainage and the provision of buried field drains would start in this zone. Zones II and III are more recent and the groundwater table is generally lower than Zone I. However, it is in these areas that the fastest rise in water tables is occurring and therefore it is only a matter of time before the demand for drainage becomes as urgent as that of Zone I.

Figure 14.5 shows a preliminary drain line 21.5 km long. By keeping the drain as shallow as possible and by making full use of the natural slope of the land, which falls about 6 m along the length of the drain, it is possible to keep the earthworks to the absolute minimum. The only structures needed will be the culverts for essential road access and minor canal crossing points plus one inverted siphon to pass the drain underneath the Primo Secondario canal in Zone II.

### 14.5.3 Disposal Pump Station

Water drained from the base of the root zone will be highly saline and therefore must be disposed of safely without being pumped back into any canals or the river to contaminate the irrigation supplies. This can be guaranteed by pumping the drainage waters into the old river channel at the tail of the drain. The required static lift to do this is estimated to be only 3 m.

Using a total drainable surplus of 40 l/s/km$^2$, made up of 23 l/s/km$^2$ deep percolation and 17 l/s/km$^2$ surface run-off, the total discharge for the pump station becomes 0.624 m$^3$/s. This does not include an allowance for storm run-off because the main drain discharge is limited by the pumping capacity from collector drains (see below).

### 14.5.4 Infield Drainage Layout

The banana fields in the narrow strip adjacent to the main drain are to be linked to the main drain by means of a deep collector. Water will have to be pumped from this into the main shallow drain. Both surface water and subsoil water will be drained by gravity into the collector.

Figure 14.6 shows an idealised layout of two 5 ha fields fitted out with the infield drainage system. This size of field is typical of the banana plantations and the arrangement shown has been used to assess the cost of installing deep drains and the associated works. There is normally sufficient space for a deep drain without reducing the cultivated area.

The buried field drains are assumed to be at a depth of 2 m and a spacing of 60 m. This corresponds to the 'most likely' sub-surface drainage conditions assumed in Section 4.6 of the drainage chapter. A control culvert is provided for each 5 ha field to throttle the release of storm run-off into the collector drain. In addition an underpass is provided to take the deep collector drain under one of the minor canals.

The complete field drainable surplus is only 4 l/s and therefore the pump needs only to be very small. Indeed a simple 75 mm centrifugal pump can be expected to pass 20 l/s. Consequently by buying a portable pump of this type with flexible inlet and outlet hosing, a total of five collector drains could be dewatered in rotation by one single pump. For practical reasons this has been limited to three collectors served by one pump.

The success of buried field drainage on the clay soils of the Study Area is, as yet, unproven although initial results from the trials at Jowhar sugar estate appear promising. It is imperative therefore, that preliminary field trials are undertaken before the expense of the main drain and disposal pump station can be justified, to confirm the suitability of the soils for buried field drainage. For this purpose a trial close to the river could be set up and the drainable surplus, because of its insignificant quantity, pumped into the river without harmful effects.

### 14.5.5 Improved Irrigation Supplies

Much work can be done to improve the irrigation networks supplying the banana farms. Minor canals can be realigned and simple control structures built to replace the time consuming process of breaking and reforming earth bunds. However, this work has not been included as part of the banana drainage project and the general recommendations for improvement are restricted to Chapter 15.

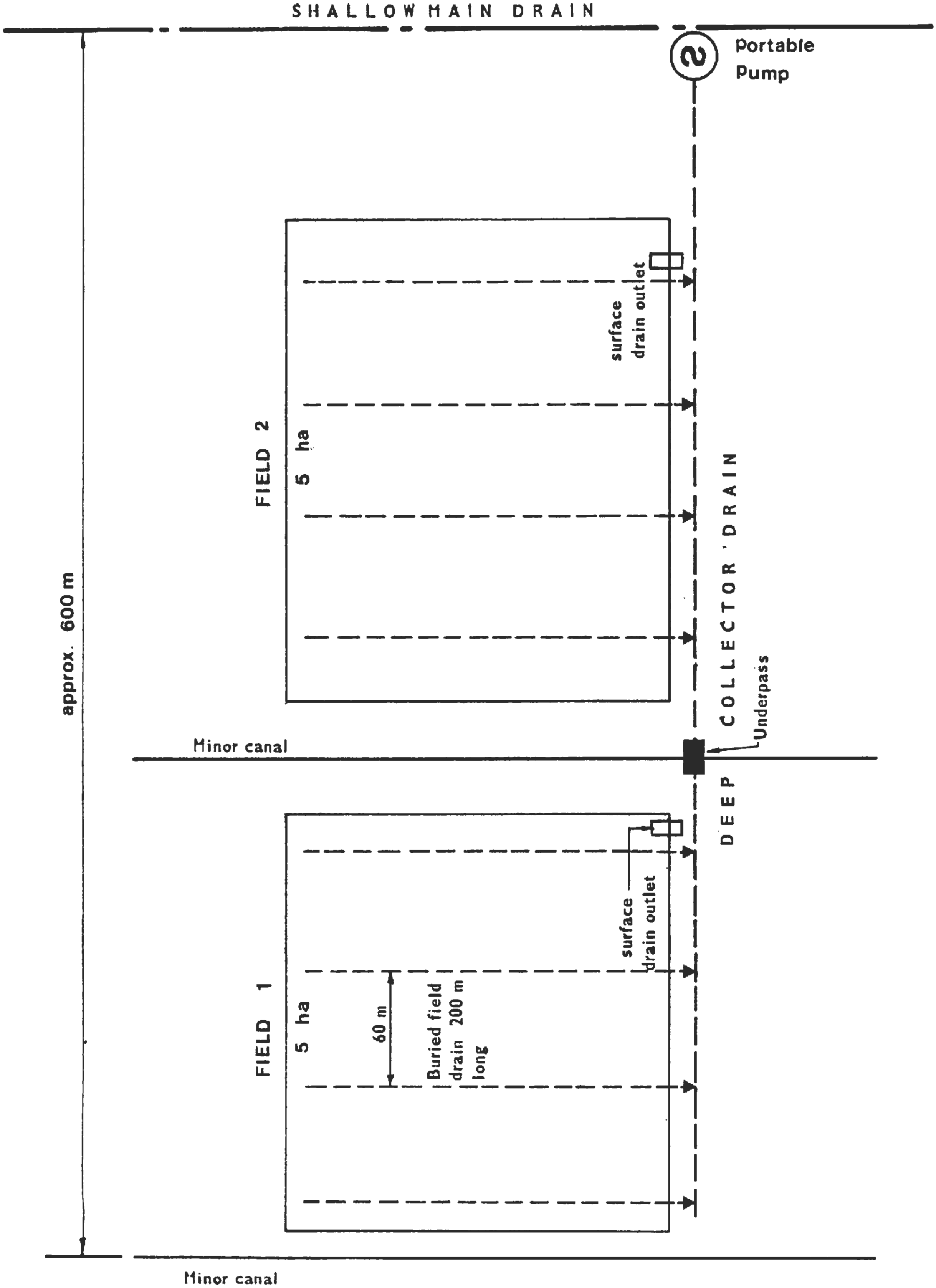
### 14.5.6 Costing

The costs for this project are broken down into two discrete sections:-

(i) The capital investment costs and annual operation and maintenance costs of the main drain, its structures and the disposal pump station.

FIGURE 14.6

IDEALISED FIELD LAYOUT FOR BURIED DRAINS



(ii) The capital investment costs and annual operation and maintenance costs of the infield works, including buried field drains, deep collector drain, surface drain culverts, underpass and pumping costs. This is based on the 10 ha unit shown in Figure 14.6, and finally expressed as a cost per hectare.

Table 14.6 summarises the costs for both of these sections.

**TABLE 14.6**

**Banana Drainage Project Cost Estimates**

| Item | Cost (So.Shs.x 1 000) |
|---|---|
| 1. Main drain and disposal pump station | |
| Land clearance | 50 |
| Excavation of main drain | 532 |
| Road and canal culverts | 1 767 |
| Inverted siphon under Primo Secondario | 125 |
| Disposal pump station | 486 |
| Engineering design and supervision | 296 |
| Contingencies (20%) | 592 |
| TOTAL capital cost | 3 848 |
| Feasibility study cost | 765 |
| Annual operation and maintenance cost | 90 |
| 2. Infield drainage system (for a 10 ha unit) | |
| Laying buried field drains | 23.6 |
| Excavation of deep collector drain | 46.7 |
| 2 surface drain control culverts | 10.6 |
| Underpass | 20.0 |
| Part cost of a portable pump | 3.0 |
| Realignment of the minor canalisation | 2.8 |
| Contingencies (20%) | 21.3 |
| Engineering design and supervision | 10.7 |
| Sub-total | 138.7 |
| Add on 25% for non-ideal field layout | 34.7 |
| TOTAL capital cost for 10 ha | 173.4 |
| TOTAL capital cost per hectare | 17.3 |
| Annual operation and maintenance cost per hectare | 0.24 |

## 14.6 Ministry of Agriculture Grapefruit Production Scheme

### 14.6.1 The Scheme

The final designs for a grapefruit plantation in the region of Buulo Mareerta were completed in October 1973. This was for the intensive irrigation of 1 386 ha of grapefruit together with a further 966 ha of annual food crop production. The original proposal was for the grapefruit to be trickle irrigated but this has since been changed to traditional surface irrigation techniques.

Funding for the project is already available and construction work has started on the main water supply system. It is for this reason that the scheme is regarded as committed and therefore already an integral part of the development programme for the Study Area. To date (May 1978) the earthworks excavation for the new Gayweerow barrage, and the new canal to link the barrage to the existing Primo Secondario canal (see Map 1F) have been completed. However, structural work has not started on the barrage and the only reinforced concrete works so far undertaken are the culvert to pass the canal water under the Qoryooley road and several inverted siphons under the new link to maintain existing water supplies. The exact areas to be planted have yet to be finally fixed but the figures listed in Table 14.7 are the original land areas presented in the final design reports. The six zones correspond to the area A7 shown on Figure 13.1.

TABLE 14.7

Ministry of Agriculture Grapefruit Production Scheme Areas

| Zone | Crop | Gross cultivated area (ha) | Net cultivated area (ha) |
|---|---|---|---|
| I | Grapefruit | 325 | 230 |
| II | Grapefruit | 645 | 465 |
| III | Grapefruit | 875 | 691 |
| Sub-total | | 1 845 | 1 386 |
| IV, V, VI | Annual crops | 1 190 | 966 |
| TOTAL | | 3 035 | 2 352 |

The complete engineering proposals for the scheme were never available for the present study and therefore the exact details could not be ascertained. However, the following details are known:-

(i) River water supply was to be from the new barrage site, using the new link to feed the Primo Secondario.

(ii) Because the grapefruit are perennial, the development of groundwater resources will be necessary to provide irrigation water during the dry season and any other times of water shortage.

### 14.6.2 Primo Secondario

With the implementation of the scheme, considerable disruption and remodelling of the Primo Secondario canal will be necessary, with the new link canal joining it almost 14 km from the headworks.

The existing canal is by far the best in the Study Area with water slopes and discharges large enough to keep the majority of the river suspended sediment in motion. Because of this, the required maintenance time is vastly reduced and any modifications to the canal must ensure that this situation is not affected. The prime consideration must be, therefore, to maintain the existing discharges along the upper section of the canal and only provide the additional water requirements of the grapefruit scheme from the link canal. The only drawback to this is that only part of the water supplied to the grapefruit scheme will have benefitted from the sedimentation facilities to be provided at the new barrage site.

The alternative is to divide the Primo Secondario into two, with the upper section isolated from the link canal and a lower section. In this case the link canal would provide water not only to the grapefruit scheme but also to all the existing users of the lower section of the Primo Secondario. The discharges in the upper section would be approximately halved and, with the existing canal size, flow velocities would reduce dramatically. This would lead to the rapid siltation of the canal which could only be remedied by a complete remodelling to reduce the size of the canal.

### 14.6.3 Possible Difficulties

During the present studies two very basic doubts about the rationale of grapefruit production for the world market by a scheme in the Study Area have arisen. These are not engineering problems, but because they are so fundamental a brief outline is given:-

(i) Grapefruit is a very sensitive crop to soil salinity. With an electrical conductivity of the soil saturation paste extract ($EC_e$) of only 2.4 mmhos/cm a 10% yield reduction can be expected. When mixed irrigation with river water (80%) and groundwater (20%) is considered the expected yield reduction due to ultimate salinity levels is predicted to be nearly 50%. A full description of this problem is given in Annex II, Section 9.7.

(ii) The Study Area, especially in the more open areas away from the river typified by Buulo Mareerta, suffers for several months of the year from strong winds. This means that all the small areas of existing grapefruit cultivation produce a fruit that is heavily scarred. This may be unacceptable for the world market and some method of avoiding wind damage must be found.

## 14.7 Shalambood Project

### 14.7.1 Project Identification

An area of approximately 8 000 ha gross was identified by the Shebelli report (HTS Ltd., 1969) for remodelling. This corresponded to the areas commanded by the Dhamme Yaasiin canal and the proposal was for the refitting of this canal and the construction of a completely new distributary canal system. Provision of new field outlets and watercourses was only considered necessary for the redeveloped land, with the 3 400 ha of banana farms then in existence using, where possible, their own minor canal systems.

Provision of new water control structures and a complete refitting of the Dhamme Yaasiin head regulator were costed, together with new inspection roads and a complete surface drainage system. Sub-surface drainage was not considered necessary.

In 1977 a pre-feasibility report (State Planning Commission) reconsidered this same area for remodelling and essentially the same engineering proposals were made. In addition the possible hazards of rapid siltation were mentioned and the inclusion of irrigation tubewells to provide supplementary water supplies made. With this report the gross project area was 7 200 ha and the maximum NCA planted at one time 4 500 ha. This was mostly to be cropped with upland rice.

Combining the information available from these two previous reports with the data supplied by the present studies, this area has again been proposed for remodelling. With the use of true-to-scale maps, unavailable for the other reports, and by the exclusion of some land at the tail of the system to be fed from the new Gayweerow barrage, the final gross project area has been reduced to 6 255 ha. Figures 13.1 and 14.7 outline the proposed area of the Shalambood project.

### 14.7.2 Existing Land Use and Irrigation

The area planted under perennial crops, estimated to be 3 400 ha in 1969, has been rapidly declining and the current figure is only 358 ha. All the areas that used to rely on a pumped supply have gone out of production. In their places a significant area of annual crop production, both on co-operatives and government-run farms, together with a number of smallholdings, has been developed. The total NCA of annual crops is estimated to be 1 743 ha.

There are, however, significant areas of abandoned land and in some of these significant bush growth has re-established itself. The failure of this land to be developed is basically because of the inability of the existing main and secondary canal systems to provide water supplies.

The majority of the existing system is in a poor condition and the following faults have been identified:-

(i) The system is under-used because of the abandoned areas.

(ii) Regulation gates are in a poor condition, making water control difficult.

(iii) The spacing between secondary canals is too great to allow proper water distribution.

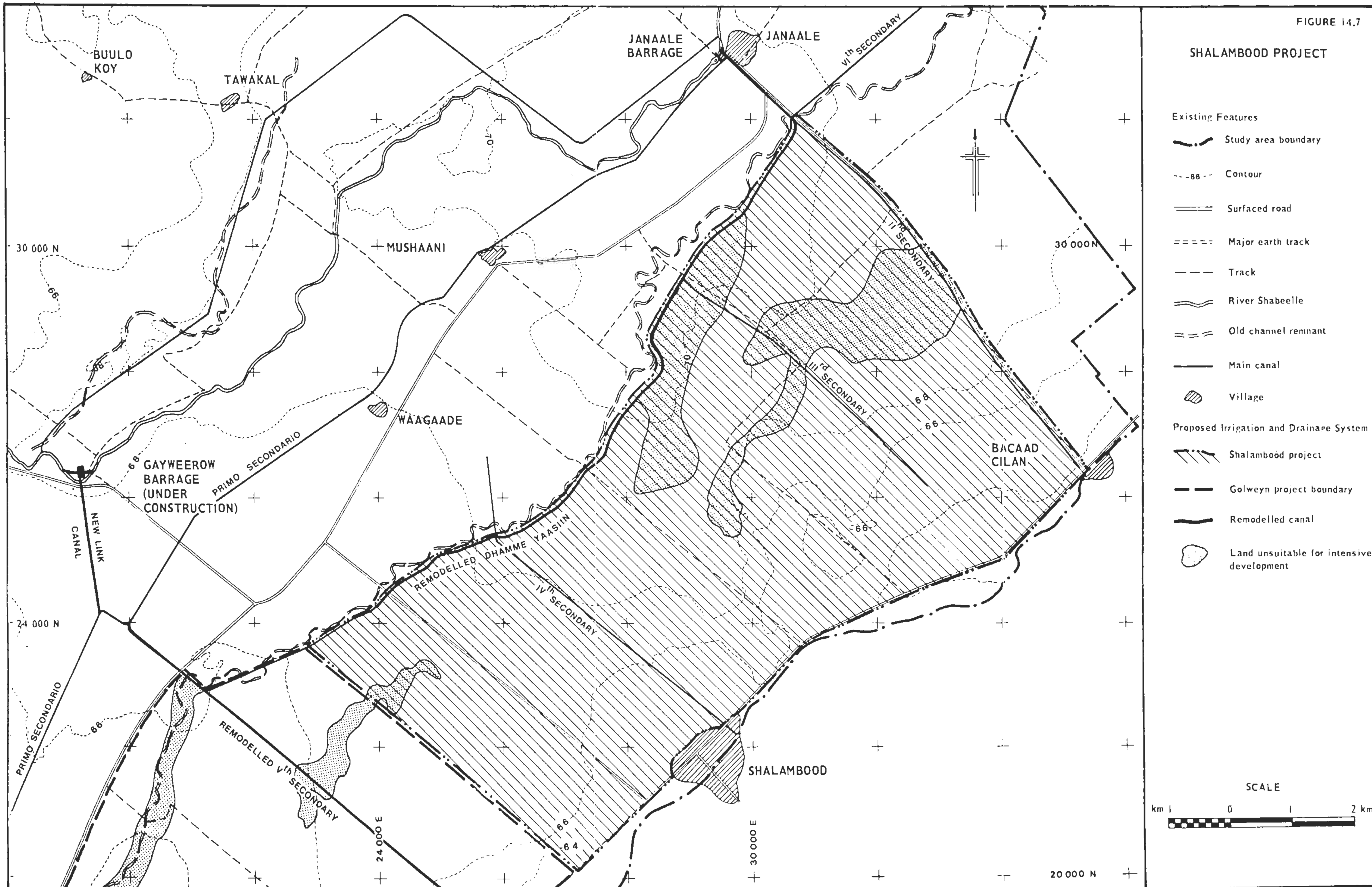



FIGURE 14.7

SHALAMBOOD PROJECT

(iv) In some reaches canal commands are excessive and seepage through the banks occurs.

(v) At the tails of the secondary canals command is often insufficient to provide gravity supplies.

(vi) The main canal silts up rapidly and only minimal discharges can reach the tail.

(vii) No surface drainage is available to draw off excess irrigation water, storm run-off or the stagnant water held in the many borrow pits and trenches in the area.

(viii) The land is generally uneven, making efficient water distribution within a field impossible.

Remodelling and refitting of the system should be able to rectify all of these problems.

### 14.7.3 Land Suitability

The area of the project lies entirely on the Saruda soil series and this is mostly classified as moderately suitable land. However, a total of about 850 ha lies on land that has been downgraded to unsuitable because of salinity and drainability reasons (see Figure 14.7).

### 14.7.4 Cultivated Areas

The unsuitable land has been excluded from the project leaving a gross cultivated area of 5 405 ha or, at 80% efficiency, a total NCA of 4 324 ha. The future planting of the existing perennial area (358 ha) has been allowed for, leaving an area of 3 966 ha for annual cropping. If a 40% gu season cropping intensity is adopted the annual crop NCA in this season will be 1 586 ha. Adding the 358 ha of perennial crops gives a total of 1 944 ha, well within the existing figure of 2 107 ha. This means that the project complies with the constraints imposed due to water shortage.

### 14.7.5 Dhamme Yaasiin Canal

The current capacity of this canal is around 6.5 $m^3/s$ at the head, approximately equal to the new project design requirement. Consequently, the required earthworks for expanding the water section should be relatively small. Despite this, significant remodelling will be necessary to improve the canal banks, both to give extra freeboard in some places and, most importantly, to widen them so that seepage losses can be minimised.

In most cases new cross regulation structures will be necessary as the condition of the existing ones is so bad. However, only refitting of the gates and provision of downstream protection works should be necessary for the head regulator.

### 14.7.6 Sedimentation

Neither of the previous reports coverd any provision for sedimentation facilities, despite (in the earlier case) including night storage facilities. This will be necessary and a suitable place for an on/or offstream sedimentation basin will have to be found. One possible site is on the right hand side of the canal, opposite where the sixth and second secondaries offtake.

### 14.7.7 Night Storage and Distributary Canals

The original 1969 proposal considered day-time watering only with the provision of night storage facilities. The selection of onstream distributary canal nioht storage was made (see Section 2.8) and because of the length of distributary canal involved (up to 7 km) intermediate night storage cross regulators had to be included. These allow successive reaches of the canal to fill up at night to storage level whilst the distributary head regulator continues to admit the normal continuous supply. The idea of onstream night storage has been retained for this report as no other detailed survey work has been done to disclaim this. It does have the advantage over offstream storage that the head required is less. However, sedimentation will be vital otherwise the distributary canals will silt up, making effective water distribution difficult.

A total of nine distributary canals was proposed to replace the four existing ones and, as the water levels would be generally lower, no use of the present distributaries was envisaged. The new canals would reduce the spacing between distributaries to about 1.5 km. It is believed that the closer spacing between canals should make it possible for almost the entire area to be fed by gravity.

### 14.7.8 Groundwater

A few tubewells already exist within the project area to supply supplementary irrigation water to the perennial crops when river levels cannot be maintained. However, the groundwater is all of poor quality (more than 3.0 mmhos/cm) and its use cannot be justified as adequate water supplies, within the cropping constraints described above, can be supplied solely from river water at a minimum of 75% reliability.

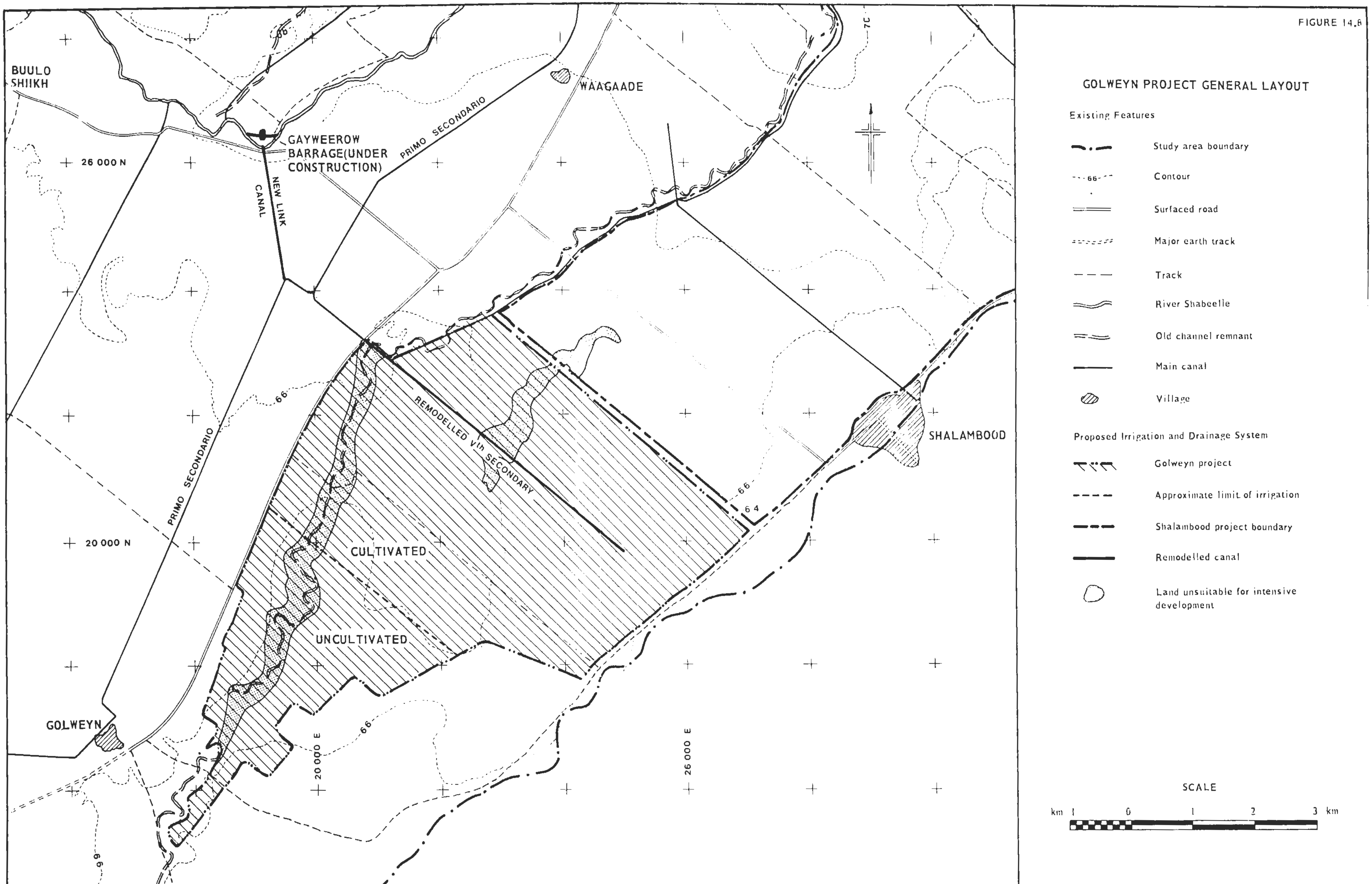
### 14.7.9 Infield Works

Since the 1969 report the condition of the remaining infield canals has deteriorated to such an extent that it is now considered that complete refitting with distributary outlets and watercourses will be necesary. For the purposes of the cost estimates the standard feasibility study field unit has been adopted. Full land forming has been included.

### 14.7.10 Drainage

A complete surface drainage system will be necessary to drain away any excess water. No problems are envisaged with this and disposal into an old river channel can be achieved in the south-west corner of the project. A simple low head pumping station will be necessary to assist with this.

## GOLWEYN PROJECT GENERAL LAYOUT



### 14.7.11 Costing

Table 14.8 summarises the capital investment costs and the annual operation and maintenance cost for the project. The quantity of eathworks for remodelling has been taken from the original work of 1968.

TABLE 14.8

Shalambood Project Cost Estimates

| Item | Cost (So. Shs. x 1 000) |
|---|---|
| Bush and thicket clearance | 1 115 |
| Land forming | 17 685 |
| Remodelling of Dhamme Yaasiin canal | 1 330 |
| Formation of sedimentation basin | 342 |
| Distributary night storage canals | 5 920 |
| Formation of watercourses | 960 |
| Excavation of surface drains | 1 216 |
| Excavation of main drains | 1 906 |
| Dhamme Yaasiin and sedimentation basin structures | 1 624 |
| Distributary canal structures | 9 453 |
| Distributary outlets | 2 279 |
| Surface drain culverts | 1 098 |
| Main drain structures | 1 807 |
| Infield equipment (including fords) | 1 812 |
| Drainage pump station | 788 |
| Buildings | 17 705 |
| Services (vehicles, electricity etc.) | 4 395 |
| Contingencies (20%) | 14 287 |
| Engineering design and supervision | 6 704 |
| TOTAL capital cost | 92 426 |
| Feasibility study cost | 2 244 |
| Annual operation and maintenance cost | 1 609 |

## 14.8 Golweyn Project

### 14.8.1 Location

The Golweyn project is determined by the land that can be commanded from the remodelled and possibly realigned Vth secondary of the Dhamme Yaasiin canal when its water supply is changed to the Primo Secondario canal (see Figure 14.7). This comprises some of the land originally covered by the forerunners of the Shalambood project and a sprawling, poorly irrigated area to the south-west of the Vth secondary which attempts to receive water from this canal.

The total area within the project boundary shown on Figure 14.8 is just over 3 500 ha.

### 14.8.2 Existing Agriculture

A total of 814 ha of poorly watered crops are grown within the project, 186 ha to the north-east of the Vth secondary and the remainder to the south-west. This includes a total of 108 ha of perennial crops. The former areas derive their water solely from the Dhamme Yaasiin. Because of the poor condition of the canal and the long distance to the headworks these areas have great difficulty in obtaining anything other than a minimal irrigation supply. The other areas in the south-west are fed from the Vth secondary, which in turn is supplied not only from the tail of the Dhamme Yaasiin but also by a small link canal from the Primo Secondario. This link is nearly always choked by weeds and silt and only minor flows can be passed. Consequently, the total amount of water the Vth secondary receives is small and the crop yields are poor.

### 14.8.3 Project Cropping

The standard cropping used for the feasibility study has been applied to this project. The gu season annual cropping intensity of 40% must correspond to the present cultivated area of 814 ha less the perennial crop area of 108 ha as this will be retained if the constraints imposed due to water shortages are to be met.

Therefore the total project NCA is 1 873 ha (108 ha perennial, 1 765 ha annual) and assuming a land use efficiency of 80%, the gross cultivated area of the project is 2 341 ha. This is over 1 000 ha less than the total project area, meaning that considerable drawing in and rationalisation of the agriculture can be undertaken. The exact areas to be cultivated, and those to be left, will only be determined after detailed survey work has been completed, but a preliminary boundary between the two has been shown in Figure 14.8, purely for reference. Two old river channels occur within the area and, as these are unsuitable for development, they will not be included in the irrigated land.

### 14.8.4 Vth Secondary Remodelling

The main purpose of the project is to remodel and, if necessary, realign the Vth secondary so that it can make full use of its potential supply from the Primo Secondario canal. A water level of 68.63 m was recorded at the proposed offtake point and, as this represented a quite typical level of operation for the Primo Secondario, no problems should be encountered in gravity feeding the entire cultivated area of the project.

The water requirement for the project will be about 2.5 $m^3/s$ and this will have to be drawn off from the existing discharge in the Primo Secondario. This will not cause any problems as the deficit can be made up by passing an extra 2.5 $m^3/s$ down the new link canal from Gayweerow barrage to satisfy the water requirements of the lower section of the Primo Secondario.

The existing Vth secondary will be of little use to the project and essentially a new canal will have to be built to serve the area. This, however, will only have to be about 5 km long. Along this length a new head regulator, inverted siphon to pass under the Golweyn road, and cross regulators to control feed to the distributary canals will be needed. In addition a sedimentation basin must be provided as the Primo Secondario waters carry the full suspended sediment load of the river water. Otherwise the night storage facilities recommended for the project will not be able to operate.

### 14.8.5 Infield Works

None of the existing minor canalisation of infield works will be suitable for the project and new systems will be needed. These have been based for costing purposes on the standard layout adopted for the feasibility study. The cultivated area will require land forming.

### 14.8.6 Drainage

This area is similar to the Shalambood project area and will require the provision of a complete surface drainage system. Also the groundwater table is well below ground level (more than 20 m) so that no sub-surface drainage will be necessary. Disposal is easily achieved by pumping into the old river channel in the south-west of the project.

### 14.8.7 Costing

Table 14.9 summarises the capital investment costs and the annual operation and maintenence costs for the Golweyn project.

**TABLE 14.9**

**Golweyn Project Cost Estimates**

| Item | Cost (So.Shs. x 1 000) |
|---|---|
| Bush and thicket clearance | 483 |
| Land forming | 7 660 |
| Reconstruction of Vth secondary | 1 900 |
| Formation of sedimentation basin | 148 |
| Formation of night storage reservoir embankments | 1 053 |
| Formation of distributary canals | 1 893 |
| Formation of watercourses | 416 |
| Excavation of surface drains | 527 |
| Excavation of main drains | 826 |
| Vth secondary and sedimentation basin structures | 1 952 |
| Distributary canal and night storage reservoir structures | 3 414 |
| Distributary outlets | 987 |
| Surface drain culverts | 476 |
| Main drain structures | 783 |
| Infield equipment (including fords) | 785 |
| Drainage pump station | 545 |
| Buildings | 8 073 |
| Services (vehicles, electricity etc.) | 2 004 |
| Contingencies (20%) | 5 406 |
| Engineering design and supervision | 2 503 |
| TOTAL capital cost | 34 940 |
| Feasibility study cost | 972 |
| Annual operation and maintenance cost | 767 |

# CHAPTER 15

# UPGRADING OF DEVELOPMENT ZONES (ENGINEERING)

## 15.1 Existing Irrigation

Section 13.5 outlined the eight developement zones in the Study Area that have been selected for general upgrading without inclusion in a major development project. These are shown on Figure 13.1.

These zones are, without exception, presently cultivated to a greater or lesser extent and this chapter deals with the upgrading and rationalisation of the irrigation and drainage services in these areas. These must be read in conjunction with the recommended improvements to the infrastructure and agronomic practices in Annex VI.

The zones are all served by minor canals winding across the land, in most cases feeding directly from the river, although in some cases, for example the Waagade zone, the offtakes are from the existing main canals. The basic difficulties of poor water distribution and availability are caused because of the many problems associated with these minor canals. These have been discussed in Chapter 1 but for convenience are restated here:-

(i) In all the area no operational water control equipment or field outlet gates exist. All water control is achieved either by breaking and remaking the earth banks and bunds or, just occasionally, by opening and closing (with earth) small concrete pipes. This makes the fair distribution of water impossible and the overall efficiency of water use very low.

(ii) The minor canals, either offtaking from canals or the river, wind across the land, in many cases making poor use of the ground contours, and often provide only minimal command over the field level. Because of this much duplication of channels occurs.

(iii) The minor channel slopes are inadequate (for the small discharges they carry) to keep the sediment load in motion. This results, in many cases, not from the land slopes being insufficient but from the attempts to extend the channels as far as possible beyond their natural limits.

(iv) The lack of adequate slope on the minors means that they require silt and weed clearance before the start of every season and, as almost everywhere the banks are too small to accommodate machinery, this work has to be done by hand.

In addition to the canalisation problems, the lack of any surface drainage system and uneven land surfaces make the effective irrigation of these areas difficult. Consequently crop response to irrigation is below what could be achieved.

## 15.2 Upgrading

The constraints imposed by the minor canalisation system mean that the irrigated zones will never have the opportunity to operate under ideal intensive irrigation practices. Despite this, basic engineering improvements can be made vastly to increase the availability, control and distribution of water. This permits a more efficient utilisation of water and general improvement in crop yields. The outline requirements for upgrading are described in the following sections.

Implementation of these proposals requires a rather different approach from a development project. There is no opportunity for either a discrete design package or the construction of the canlisation and drainage systems under a single contract. Instead a continuous survey, design and remodelling process of small areas taken in turn is required. Consequently only the regional offices of the Ministry of Agriculture at Janaalc are in a position to undertake this work.

## 15.3 Survey Information

The correct choice of modified canal lines, field layouts, and drainage lines can only be selected when detailed topographical mapping is available. A system of benchmarks already exists throughout the Study Area and setting out to a sufficient accuracy for upgrading purposes can be based on the 1 : 25 000 maps of the Study Area (Map Series 5). There are no problems in surveying areas on a 250 m grid or finer basis to produce topographical maps with contours at intervals of 250 mm.

In addition to basic contouring all pertinent features should be surveyed, including canals, roads, structures and large trees. The upgrading process cannot afford complete remodelling and must be designed to be in sympathy with or avoid altogether these major objects.

## 15.4 Land Forming

Successful small basin irrigation relies upon a reasonably even land surface with only minor variations in level within an individual basin. If this is not the case the basins have to be sub-divided into very small and inefficient units. Therefore some form of basic land forming is envisaged in the upgrading process. This need not be very sophisticated, and simple bulldozing in conjunction with a 25 m survey grid should produce a significant improvement. With a skilled tractor operator the final result can be only marginally less effective than the results of a full land forming operation.

## 15.5 Minor Canalisation

The canals are small and the area each one commands limited. Consequently it is inappropriate to consider the inclusion of any sedimentation and night storage facilities into the system. This means that the full suspended sediment load of the river waters must be carried by the canals and that either night-time irrigation or complete closure of the canal at night must be considered if large amounts of water are not to be wasted.

Bearing these factors in mind the following recommendations can be made about the upgrading of the minor canals:-

(i) Much duplication of canals occurs. Rationalisation and selection of the best lines can be made on the basis of the survey information. Where necessary, new sections can be constructed and the redundant parts obliterated.

(ii) Where possible, grouping of the supply to minor canals should be undertaken. This enables larger canals to be built, permitting mechanical silt clearance. The larger sections should also provide more efficient water distribution.

(iii) Many of the minor canals, especially on the right bank of the river, extend beyond their maximum effective length. Indeed, in some cases, slopes are minimised so that the water can be passed across the central depression midway between the river and the old river channel on the northern boundary of the project. Therefore upgrading should involve the correct termination of canals so that they can operate in sympathy with the levels of the land they serve.

(iv) Very long minor canals should be avoided. By limiting them to 2 or 3 km it should be possible to close down the entire canal at night, thereby eliminating any need for night storage.

(v) Because of the heavy silt load, good water slopes must be provided to pass the majority of the silt onto the fields. If this is not done then the canals will continue to silt up as rapidly as at present and little improvement in water distribution will be possible With a typical canal discharge of 0.2 $m^3/s$ the minimum water slope should be around 30 cm/km.

(vi) Canal banks should be cleared of bushes and widened. This will limit the amount of seepage losses where commands are high and provide vital access routes along the canal lines.

(vii) Each canal should be provided with a gated concrete head regulator. These are vital so that the canal can be opened and closed at will, providing fine control over the exact discharge. A good example of one already in existence is the head regulator to the Giddu canal; this is a simple concrete structure with a 1.0 m wide vertical lift sluice gate and nut and key operation.

(viii) Each outlet point should be provided with a simple structure (a concrete pipe and plug may be sufficient). This will provide the opportunity for more equitable water distribution and allow access along the banks.

## 15.6 Pumping

The minor canals exist in areas where gravity feed is, if not entirely adequate, at least possible. However, the supply to the canals depends upon high river levels (for those offtaking directly from the river). This can only be maintained, in areas away from the barrages, when river discharges are very high and any fall in discharges can cut off the supply. Consequently the provision of simple pumping facilities may be advisable in cases like this.

A total of 16 pumping units are already in existence along the river banks within the Study Area. These are all the twin discharge pipe centrifugal pumps, either 200 mm or 250 mm diameter, that were made in Shalambood. These are robust, simple, and easily capable of handling the heavy silt load in the river water. Power is provided by belt drive from a tractor.

If the production of these pumps could be restarted they would provide ample capacity to serve the minor canals. If, however, this is not possible, then portable pumps of discharges up to 150 l/s are available that could serve the purpose.

## 15.7 Drainage

Simple surface drainage facilities should be included in the upgrading process to dispose of excess irrigation water and storm run-off. These will not form large linked systems and therefore discharges should be small. This will allow the use of wild junctions in all cases without the provision of a structure. Structures will only be necessary where access routes are crossed.

Disposal of the drainage water may pose problems as quantities will be too small in most cases to justify an expensive disposal drain and pump station. Local depressions will probably form the best answer, with the drainage water being temporarily ponded in these areas and allowed to infiltrate slowly and evaporate.

In certain development zones, such as the Waagade zone, this will probably not be adequate and provision for drainage water disposal will have to be made.

## 15.8 Development Zone Details

Although the general recommendations for upgrading apply to all the eight development zones, certain features can be identified about each one as an individual area (see Figure 13.1).

### 15.8.1 Degwariiri Zone (B.3.2)

This area covers nearly 2 500 ha of land irrigated directly from the river. For the purposes of development the Siigaale and Giddu canals have been taken as minor canals. The total NCA is estimated at 1 157 ha, 33% of which is perennially cropped.

Currently irrigation is extended too far away from the river and upgrading should involve limiting the length of minor canals to about 2 km. This corresponds to the middle of the depression between the river and the old channel on the northern boundary of the Study Area. Drainage water can be allowed to gather in the lowest spots of this depression.

Some of the canal offtake points are as much as 10 km upstream of Janaale barrage and therefore the inclusion of pumping facilities at the heads of minor canals should be considered. Two pump units are already serving this zone.

The Siigaale canal head regulator is oversize for its use and it may be possible to group several canals together to offtake from this point. This would enable better use of the existing structure to be made.

### 15.8.2 Bandar Zone (B.3.3)

This is similar to the Degwariiri zone in that the central depression between the river and old river channel on the northern boundary limits the effective length of the minor canals to about 2 km. This zone, however, is smaller, with a gross area of only 1 815 ha and an estimated NCA of 691 ha.

A main feature of the area is the existing Liibaan canal. This is ineffective in supplying the area, with almost zero flows and zero water slopes along it. The way to improve this would be to remodel the first 3.5 km of the canal and effectively turn it back into a minor canal reaching only as far as the central depression. In this way adequate water slopes could be achieved and a significant area irrigated from it. This, incidentally, would not interfere with the drainage disposal route proposed for the Qoryooley project as the outfall drain would meet the new canal at its tail. Therefore, the tail section of the old Liibaan canal could remain unchanged and act solely as the disposal drain for the Qoryooley project.

### 15.8.3 Janaale Zone (B.3.1)

This zone, gross area 3 165 ha and net area 1 310 ha, is in part similar to the Degwariiri and Bandar zones, because of its supplies from the river. However, an extensive proportion of the area is served from the sixth and second secondaries of the Dhamme Yaasiin. The remodelling of this canal for the Shalambood project must ensure that supplies are maintained to these secondaries.

The disposal of drainage water is likely to prove rather awkward from this area as no suitable channels or depressions are known. It may just be possible to find a suitable area outside the Study Area to the east of the zone.

A further constraint on the development of parts of the zone, fed by the secondary canals, arises because of the presence of lands classed as unsuitable for irrigated agriculture shown on Map 1B. These areas should be avoided and improvements concentrated on the better lands closer to the river.

### 15.8.4 Majabto Zone (B.3.4)

Apart from the small areas on the left bank of the river this zone is formed from the land locked in between the river and the Asayle canal. The gross area is 1 760 ha, but the NCA only 579 ha.

This area is interesting because the irrigated lands receive water both from the Asayle canal and the river. The locked-in land forms a complete basin from which no surface water can escape, consequently standing pools of water are common and some fields are almost permanently covered with up to 300 mm of water.

Upgrading can therefore be considered for this area if provision for a complete surface drainage system is made. This could be achieved by linking the field drains to a single drain between the river and Asayle canal.

Final disposal would require pumping back into the river near the site of the new Gayweerow barrage.

However, much of the land at the north-eastern end of the zone is classified as unsuitable for development (see Map 1B) and therefore the true effectiveness of a surface drain will be limited to the better soils in the zone.

### 15.8.5 Haduuman Zone (B.3.5)

This area is formed by the land trapped between the river and the northern boundary of the Faraxaane project which coincides with the main road to Qoryooley for much of its length. This fairly narrow strip is one of the most intensively cultivated areas of annual crops in the Study Area, with a total NCA of 823 ha fitted into a gross area of 1 940 ha. The irrigation supply is almost totally from minor canals offtaking directly from the river.

This zone represents perhaps the best area for upgrading and significant benefits should be obtainable from the provision of better minor canals and a more even land surface. Drainage, however, is a rather awkward problem, especially in the east; this area tends to be locked in by the main road and areas of standing water do occur. Provision must be made to allow for the drainage of this and, assuming the implementation of the Faraxaane project, the way to do this will be to pass it under the road and new project canal. Disposal would then be taken care of by the Faraxaane project surface drainage system.

### 15.8.6 Jeerow Zone (B.3.6)

This zone forms part of the larger unit of land enclosed between the Bokore canal and the left branch of the river that runs down from Falkeerow barrage in a south-westerly direction. Irrigation in this area is from both the river and the Bokore canal, although the area served by the latter is restricted because of the limited outlet facilities. The Jeerow secondary makes the most significant contribution of any single canal. The gross area within the Study Area is 2 325 ha, supporting a total NCA of 1 028 ha.

There is no doubt that much more extensive use of the Bokore canal can be made in this area together with some of the land outside the Study Area. The provision of several new large minor canals, on a similar standing to distributary canals, should be sufficient to gravity feed almost all of this expanded area.

### 15.8.7 Waagade Zone (B.3.7)

This zone of 3 790 ha covers the land in between the Primo Secondario and the Dhamme Yaasiin canals. None of this land is included in the banana drainage project. The area is covered in mixed annual and perennial cropping, broken up by many small areas of bush and standing water. The total NCA is only 903 ha.

This is, despite the low intensity of land use, an old and well established area of irrigated agriculture. Consequently upgrading of minor canal systems is likely to prove difficult and of only marginal benefit. The greatest single improvement would come from the provision of a surface drainage system to drain off the standing water and allow better water control in the fields. A complete system of surface drains could be linked to the banana drainage project main disposal drain just after it passes under the Primo Secondario canal.

### 15.8.8 Primo Secondario Banana Zone (B.3.8)

This zone, of a total area of 2 835 ha, is by far the most important area of banana production outside the land covered by the banana drainage project. The total NCA of 1 372 ha is dominated by an estimated 1 018 ha of perennial crops. It the possible benefits from the banana drainage project accrue then it will be logical to take th is zone as the next stage in the provision of buried field drainage to control groundwater levels.

# CHAPTER 16

## SHABEELLE RIVER AUTHORITY

### 16.1 General

Several previous reports have mentioned the need for an authority to control the abstraction of water from the River Shabeelle and the United Nations commissioned a survey in late 1963. The report, by Signor Dante A. Caponera, was published in 1964 and included a draft water law. The Somali Government adopted this draft and the Organisation of Water Law No. 13 was passed and became effective on 1st August 1966, but this law has never been implemented.

The allocation of water has now passed to the Director of Land and Water in the Ministry of Agriculture who is responsible for the whole of Somalia.

This department maintains canals and barrages in the Study Area and is also responsible for flood control on the River Shabeelle and the River Juba; the department should form the kernel of the new authority.

### 16.2 Need for an Authority

At present most of the work of a river authority is carried out by the Land and Water Department of the Ministry of Agriculture which has pursued a 'laissez faire' policy until recently. This was quite realistic in view of the shortage of management in Somalia and any policy of control on the river inevitably means restrictions on water supply to one group or another and such restrictions, however minor, often have the effect of ossifying the existing structure of irrigation. In practice the Ministry of Agriculture has actively encouraged the installation of pumps to serve village co-operatives in an attempt to increase overall agricultural production.

In the past, farmers have regarded water shortage as an act of God and indeed the Government could not increase the volume of water available although it could enforce scheduling within a canal system.

The inauguration of Jowhar offstream storage reservoir brings about a radical change in the situation. The reservoir will take water from the river at times of surplus and return it in times of need, but to obtain the most effective use of the reservoir will require careful assessment and control of the releases. In the der season there should be little problem in filling the reservoir in most years, but the decision to release water will be a very delicate one involving a choice, amongst other things, between completing the irrigation of annual crops in December and January or saving the water for perennial crops in February and March. In the gu season the choice becomes more difficult because, in most years, there is insufficient water to fill the reservoir and yet releases are desperately needed in June and July. Therefore the decision to divert into the reservoir (and deprive optimistic farmers downstream), is almost political in character and cannot be taken without full knowledge of the situation and comprehensive planning.

## 16.3 Responsibility of the Shabeelle River Authority

The role of the Shabeelle River Authority may be summarised as follows:-

- (a) flood control
- (b) river gauging
- (c) monitoring groundwater levels and quality
- (d) registering present users of river water and groundwater
- (e) development planning for the river basin
- (f) licensing of abstraction of river water and groundwater both for present users and future users
- (g) operation of Jowhar offstream storage reservoir
- (h) enforcement of controls on abstraction
- (i) maintenance of barrages and canals
- (j) control of water-borne diseases
- (k) development of fisheries.

These functions are elaborated below.

### 16.3.1 Flood Control

The channel of the River Shabeelle has a capacity of about 400 $m^3/s$ at Beled Weyn, 140 $m^3/s$ at Mahaddaay Weyn and 60 $m^3/s$ at Falkeerow. At times of flood most of the surplus water is spilled between Jalalaqsi and Jowhar but flooding has been known to occur over much of the length of the river between Jowhar and Falkeerow. The Jowhar offstream storage reservoir should be commissioned in 1979 and designs are in hand for the Duduble flood relief channel and head regulator. Once these schemes are in operation, the risk of flooding downstream of Sabuun will be very much reduced although not entirely eliminated; the Shebelli river report (HTS Ltd, 1969) recommended two flood relief channels in the vicinity of Duduble, and so far there has been little progress on these works.

The control of floods will require skilled operation of the flood relief head regulator gates and the Jowhar offstream storage reservoir and it must be borne in mind that the latter is intended primarily for storage and not for flood prevention.

The operation will be based on simulated studies of past riverflows and a knowledge of the current discharge at Beled Weyn.

The work of the flood control section will also include the repair and maintenance of existing river banks and formation of new banks or raising and strengthening of existing banks. For this reason the flood control section should also be responsible for river gauging.

In certain areas, particularly just north of Janaale, flooding is caused by hippopotami breaking river banks. There will have to be a programme for the control of these animals in certain areas.

### 16.3.2 River Gauging

There are supposed to be river staff gauges at:-

| | |
|---|---|
| Beled Weyn | at road bridge |
| Buulo Berde | at road bridge |
| Duduble | - |
| Dinlawe | - |
| Sabuun | at uncontrolled river section |
| Jowhar | upstream bridge |
| Buulo Jameeco | - |
| Balcad | 13 km upstream of road bridge |
| Afgooye | at road bridge |
| Awdheegle | at road bridge |
| Janaale | at barrage |
| Qoryooley | at barrage |
| Falkeerow | at barrage |

However, not all these are in operation. In addition there will be a measuring point at the new Gayweerow barrage and we recommend the establishment of staff gauges at Sablaale and Hawaay. All of these gauges should have the stage-discharge curve checked by current metering and this is a major task involving repeated measurements particularly at times of low or high flows. It will then be found necessary to remove certain gauges because sites are not satisfactory, for example, the Afgooye road bridge is frequently partially blocked by floating or submerged trees and this obviously affects the readings.

There also needs to be frequent communication with most sites and daily communication with the most important areas such as Beled Weyn and Janaale.

The records of levels and discharges at each site must be maintained at head office and at Jowhar and should be published as part of the annual report.

### 16.3.3 Monitoring Groundwater Levels and Quality

Irrigation is essential for the expansion of agricultural production in Somalia in the next few years and yet surface water supplies are limited. It is therefore important to assess the groundwater potential throughout the river valley since this is one of the few possibilities available for expansion of water supplies. However, the role of groundwater in the Study Area is described in this report and there seems to be little scope outside the Study Area due to very low rates of recharge from the river and increasing salinity away from the river.

Apart from irrigation, water is also required for industry and domestic purposes, particularly in Mogadishu, and this water is likely to come from the groundwater of the Shabeelle Basin. At the moment the potential for abstraction outside the Study Area is not known, yet it may be possible to develop local schemes on a conjunctive use basis (using groundwater when river supplies are short). On the other hand there is a danger that aquifers will be overdeveloped and salt water intrusion is likely. At the moment there is not the expertise within Somalia to forecast such events and such work is better left to specialist consultants. Nonetheless it is a simple matter to record water levels and measure the water quality at selected wells or boreholes and these data, when collected over several years, can be of utmost importance to the specialist.

To help in future studies it is essential that borehole logs, the results of any tests etc., be recorded and lodged with the Shabeelle River Authority. A further point is that the simulation studies for the Jowhar offstream storage reservoir make a generous allowance for seepage into the reservoir bed and any such seepage will recharge the aquifer.

Therefore it is recommended that a programme of monitoring commence forthwith.

### 16.3.4 Registration of Water Users

At present no records are maintained of river water users and it is a simple matter for any person to break the river banks and take water by means of an open channel through the bank. As development proceeds and water becomes more scarce, controls and rationing will become necessary yet such controls will be impossible without a central registry of water users.

The register must also include tubewells although there is no need to worry about hand dug wells since these are almost invariably too shallow to have a major influence on the overall use of water and the rates of abstraction are very low.

### 16.3.5 Development Planning for the River Basin

There is already a shortage of water in the gu season and this shortage will get worse as development proceeds; clearly future developments must be controlled so that the shortages do not become disastrous. The control of such development is a matter fundamental to the economy of Somalia and must be seen as such. Accordingly such planning involves the widest and deepest consideration and decisions should be thoroughly publicised so that all concerned, particularly authority staff, fully understand the necessity for following the plan even though some people may disagree. That is not to say that the plan must be rigidly adhered to once formalised; on the contrary, the plan should be reviewed and modified periodically to allow for changing economic circumstances. Other reasons for change will arise in the agricultural sector, where improved varieties may enable major changes to be made, or the need for industrialisation of processing may itself impose additional demands on the planting and harvesting programme or the selection of varieties of crops.

It is for these reasons that the members of the governing body must be drawn from the widest sources.

### 16.3.6 Licensing of Abstraction

Once the register of existing users has been compiled, and the development plan agreed, new developments may be permitted. If the development plan is widely known and clearly understood the licensing of new abstraction, either from the river or from the tubewells, should not be too difficult. It is important to realise that decisions on such matters must be made quickly - there is a tendency in Somalia for approvals to be given slowly but authorities rarely reject applications. This is insufficient for such a vital commodity as water - the policy must be so clear that the authority may reject rapidly unsuitable applications for permits to abstract water.

The licensing of abstractions brings in another point. At the moment, refined mathematical analysis of the operation of Jowhar offstream storage reservoir shows that the reservoir will not be able to meet the gu season water requirements in one year out of every four years in the future. Clearly it would be unwise to permit further abstractions from the river in the gu season. Likewise permission to irrigate new perennial crops with river water will rarely be given. However, there seems to be considerable scope for the expansion of irrigation in the months August to November although such schemes will have to compete economically with projects where cropping intensities of 150% are possible. Therefore it is likely that many of the new licences will have restrictions on them. This situation must be compared with the present where the restrictions are imposed by the nature of the river. For example, nobody can abstract in a dry period, and moreover many canals can only take water when the river is in flood.

### 16.3.7 Operation of Jowhar Offstream Storage Reservoir and Barrages

There are two major functions of the Shabeelle River Authority - flood control and allocation of water supplies. For each of these the role of the Jowhar offstream storage reservoir is vital and the operation of the reservoir must be planned carefully so that the maximum possible benefit is obtained from the river water. The discharge of the river above Jowhar is quite unpredictable but, despite this, the Authority must be able to guarantee supplies downstream. That is not to say that the Authority must guarantee a set amount of water for every month of the year. On the contrary, before the start of each season the Authority must consider the volume of water available (mainly the amount stored in the reservoir), and then balance this with the water requirements downstream. In most years there will not be too much difficulty in the der season but the gu season will nearly always cause problems since there will rarely be sufficient water in the reservoir in March to meet all requirements for June and July. At these times the Authority must err on the side of caution since there is no sense in the Authority promising to supply water and then failing; indeed if it is not able to fulfil its promises it will have great difficulty in regaining confidence and re-establishing its authority.

To determine the policy of operation, the Authority must review the situation before the start of each season and before the start of the dry season. It must then determine the method of operation for that season, publicise the policy, and ensure that it is implemented. The policy will generally involve the operation of the reservoir and restrictions on the use of canals offtaking from the river.

In Annex II of this report it has been shown that the River Shabeelle is just about capable of supporting the existing level of irrigation development although water supplies are very critical in June and July. To provide the irrigation water required will require skilled operation of the reservoir and for the simulation studies the following assumptions were made:-

(a) crop water requirements were based on theoretical calculations

(b) field application efficiencies were assumed to be 60% and water distribution efficiencies 75% giving an overall efficiency of 45%

(c) planting dates for all crops were modified to give the most extensive area under irrigation.

(d) areas of existing and proposed irrigation were mainly compiled from a desk study after discussion with the Ministry of Agriculture

(e) discharges were measured from existing records based on existing stage-discharge curves; these records ceased in 1973.

In the case of assumptions (a), (d) and (e) there was no alternative and there is every reason to believe that these assumptions are justified. The forecasting of field application efficiency is a subjective matter but the choice of 60% is believed to be realistic even though efficiencies are as low as 20% at present. Assumption (c) was made since this is the only way that the irrigation areas may be optimised. At present rainfall and riverflows are important factors in the cropping programme but the river regime downstream of Jowhar will change with the commissioning of the reservoir and farmers must take full advantage of this.

Farmers have demonstrated their adaptability by modifying their planting dates to take advantage of high riverflows or heavy rainfall but in the future the situation will be different - the river discharge will be predictable over a period of two months or so and individual farmers must plan their operations to fit in with the programme for the whole river. They will only do this if they have confidence in the River Authority.

It will be imperative for the Authority to ensure that all farmers are aware of the need to change and the consequent advantages to themselves.

Closely allied to operation of the reservoir will be the control of the barrages at Janaale, Gayweerow, Qoryooley and Falkeerow and these barrages should come under the same section together with all canals which need to be run by the Authority.

The operation of Jowhar offstream storage reservoir and the control of abstraction downstream will not be an easy task but it is the prime purpose of the Authority and it must be made clear to all that the intention of the Authority is to expand production not to restrict it and any controls which are imposed are to assist this expansion.

### 16.3.8 Enforcement of Controls on Abstractions

The Authority will not only have to do the basic planning for water abstraction, it will also have to ensure that the plans are enforced. No matter how skilfully the plans are prepared, nor how well they are presented, there will always be someone who tries to drill a tubewell without a licence or to abstract water from the river illegally.

At this stage it is not considered necessary to delegate power of prosecution to the Authority as the regional governors and district commissioners are quite capable of maintaining the law, but it will be the duty of the Authority to look out for infractions and bring them to the notice of the police. In all such cases the greatest deterrent is the certainty that the offender is going to be caught; if the chances of prosecution are high there is much less temptation to break regulations.

It is therefore important for the Authority to be vigilant in safeguarding its resources. To this end it will be necessary to institute river patrols which are the most efficient method of surveillance and allow mobility when movement by vehicles is restricted by impassable tracks and roads.

### 16.3.9 Maintenance of Barrages and Canals

The maintenance of canals and barrages in Somalia may be divided into three elements, maintenance of earth canals, maintenance of concrete structures, and overhaul and replacement of water control equipment such as steel sluice gates.

The maintenance of earth canals and drains is a comparatively easy job comprising desilting, weed cutting, maintenance of inspection roads and repairs of breaches caused mainly by burrowing animals. The bulk of this work can be done by draglines, hydraulic excavators with reed cutting attachments, and wheeled loading shovels; canal maintenance can be carried out with a minimum of professional supervision even though it is expensive (about 5% of the capital costs per year).

Concrete structures, on the other hand, give very little trouble throughout their lifetime provided that they are properly designed and constructed. If there is trouble, however, as downstream of Falkeerow barrage, the repairs require the greatest expertise both in the designer and the executing agency. Such repairs are usually costly since they require extensive temporary works or, alternatively, long shut-downs of the canals.

In the case of water control equipment, repairs are often very complicated and parts, particularly gearing, must be manufactured outside Somalia. From the state of the existing gates it appears that the water of the Shabeelle is particularly corrosive and the choice of suitable materials and methods to reduce corrosion will be most important.

### 16.3.10 Control of Diseases

The Authority must make itself responsible for the control of water-borne diseases, particularly bilharzia and malaria. In the case of bilharzia, or schistosomiasis as it is also called, the Authority will have to maintain a programme of chemical dosing of canals to control the snail population.

Schistosomiasis is prevalent in the Study Area, as it is in many irrigated areas throughout the world. The latent effects of this disease are often not obvious since most of the individuals affected gear their activities to their physical capacities but many of these cases break down under the stress of extra work such as contract labouring.

Therefore it is important for the development of the area that the spread of the disease is controlled as rapidly as possible and then the incidence reduced.

The schistosome is a parasite which spends half of its life cycle in man and the other half in aquatic snails. Schistosomiasis haematobium is thought to occur in the area but there is a chance that Schistosomiasis mansoni also occurs.

The eradication of the disease requires a simultaneous attack on the parasite both in the human stage of the cycle and in the snail stage. It is possible to control the snail population but if snails return they will be infected by the humans and the infestation will continue.

It is an obligation of an irrigation authority to control the snail population but the treatment of human cases is a general health problem and should be the responsibility of the appropriate authorities. Nevertheless, in an infested area individuals can safeguard themselves by elementary hygiene and avoiding contact with waters which may be infested.

For control of the snails, copper sulphate is now being replaced by a chemical, N-tritylmorpholine, which is sold by Shell Chemicals under the trade name Frescon. This has been shown to be non-toxic to animal and fish life at the dilutions used in canals and to have negligible effects on the yields of irrigated crops, but the emulsifiable concentrate which is most commonly used requires careful handling. An alternative to Frescon is Bayliscide manufactured by Bayer of the Federal Republic of Germany.

For the Study Area the best way to apply the Frescon would be to dose the canals at a concentration of 0.075 ppm for five days and this should give effective control of snails in the irrigation system if repeated six times per year. It would also be necessary to spray the standing water in drains and in canals during the dry season. The equipment required is very simple, just drums with constant flow outlets and knapsack sprayers and it is considered that two men would be able to cover the whole Study Area.

Another possible method for the control of schistosomiasis is the planting of the 'soap tree' by the main canal. This tree, also known as the 'Endog berry' bears berries which contain a natural detergent. When the berries fall into the water the detergent is released and acts as a molluscicide. It appears that the detergent has no toxic effects on fish or man and the tree is known to grow in the region of Kismaayo. The use of this method should also be studied but it will take time for the tree to be established and chemical methods should be used in the meantime. These methods will control the spread of schistoomiasis but the treatment of people who are already infected should, of course, be dealt with by the Ministry of Health.

In the case of malaria a great deal can be done by adequate surface drainage to reduce areas of standing water or by spraying ponds. In the case of canals it may be possible to encourage fish, as is the present practice in the Dhamme Yaasiin near Janaale, but this is unlikely to be possible on a large scale because most canals will be left without water in the dry season. As in the case of schistosomiasis the treatment of infected persons should be left to the Ministry of Health.

### 16.3.11 Development of Fisheries

At present fresh water fish does not form an important part of the diet of the average Somali, even by the river, although fishing of some sort appears to be almost continuous at the barrages. It may be that the numbers and size of fish are limited by the frequent drying up of the river but with the inauguration of Jowhar offstream storage reservoir the regime of the river will change and influence the development of fish in the river.

The potential for fishing along the river must be studied and there is a possibility, although rather remote, that the method of operation of Jowhar reservoir may be determined by the requirements of fish.

## 16.4 Structure of the Authority

The Authority should be established an an autonomous institution possessing juridicial personality and financial autonomy. Many such para-statal bodies exist already in Somalia and in theory each is responsible to a particular minister although in practice many appear to act almost as independent entities reporting to the President's office. The prime purpose of the Authority is to enable the waters of the Shabeelle to be used for the development of agriculture and it is essential that the authority comes under the Secretary of State for Agriculture. However, many other bodies have interests in agriculture and the River Shabeelle and it is suggested that there should be a steering committee to advise the general manager of the Shabeelle River Authority. The following organisations should be among those represented on the steering committee:-

Ministry of Agriculture
Ministry of Industry
Ministry of Finance
State Planning Commission
Settlement Development Agency
Water Development Agency
National Banana Board
Middle Shabeelle Regional Governor's office
Lower Shabeelle Regional Governor's office.

## 16.5 Inauguration of the Authority

Jowhar offstream storage reservoir should become operational during 1979 and the reservoir will be filled during the der season. In the early stages the Consultant will be responsible for operating the reservoir but it is important that permanent staff are appointed immediately so that they may be trained by the Consultant during the early stages of operation. It must be emphasised that responsible staff are required and the present resources of the Land and Water Department of the Ministry of Agriculture are inadequate to operate the reservoir successfully. The commissioning should be seen as the opportunity to inaugurate the Shabeelle River Authority.

During the commissioning, and after the Consultant's staff have departed, important decisions on the operation of the reservoir will be required constantly. Therefore the Shabeelle River Authority should be formed immediately.

# APPENDICES

# APPENDIX A

# MAIN CANAL SURVEY

# APPENDIX A

## MAIN CANAL SURVEY

During July and August 1977 a ground survey of the eight main canals in the Study Area was carried out and the detailed results are given in this appendix. The following details are recorded:-

(i) the chainage of each structure, given as a distance in kilometres from the offtake point.

(ii) the type of structure; the following coding has been used:-

H - head regulator

X - cross regulator

OG - field outlet feeding by gravity. If no gates are associated with this (see (iv)) it will be an earth breach in the canal bank, either culverted or not.

OP - field outlet fed by pumping, if gates are associated with this the offtake will have the ability to act as a gravity offtake as well.

RB - road bridge or culvert.

FB - foot bridge.

(iii) the bank the structure is situated on, either left (L) or right (R).

(iv) the number of gates associated with the structure.

(v) the width of each gate in metres.

(vi) the number of gates fully operational (i.e. those gates that can be opened manually without extra assistance from lifting gear, sledge hammers etc.)

(vii) the canal command in metres. This is a visual estimate of the height of the water level in the canal at the time of the survey above the surrounding ground level.

(viii) the freeboard in metres of the canal banks above the water level at the time of the survey.

(ix) the canal width in metres measured as an inter-bank width at water level but including silt banks, unless dry, when the bed width is given.

(x) the condition of weeds in the canal, coded as follows:-

G - good, with little or no weed growth and no interference to the flow of water.

F - fair, with minimal weed growth only affecting the areas close to the banks.

P - poor, with weed growth choking the canal down to as much as 50% of its clear cross-section.

B - bad, with over 50% of the section lost to dense weed growth.

(xi) crossing points over the canal, either vehicular (V) or pedestrian (P).

# MAIN CANAL SURVEY

CANAL: SIGAALE

SHEET NO: 1/2

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 4 | 2.0 | 1 | | | 3.0 | P | P | Inlet heavily silted |
| 0.4 | RB | | | | | | | 2.5 | | V | |
| 0.6 | OG | R | 1 | 0.5 | 0 | 0.5 | 1.3 | | P | | |
| 1.4 | OG | R | 1 | 0.7 | 0 | 1.0 | 1.0 | | P | | |
| 1.5 | OG | R | | | | 1.5 | 1.1 | | P | | |
| 2.0 | OG | R | | | | 1.5 | 1.1 | | P | | |
| 2.7 | OG | R | 1 | 1.0 | 0 | 1.0 | 1.1 | | P | | |
| 3.5 | OG | R | | | | 0.5 | 0.7 | | P | | End of Study Area of at Km 3.6 |
| 3.9 | OG | L | | | | <0.5 | 0.9 | | B | | |
| 4.0 | OG | R | | | | <0.5 | 0.8 | | B | | |
| 4.4 | FB | | | | | | 1.0 | 2.0 | B | P | |
| 4.5 | OG | R | | | | <0.5 | 1.2 | | B | | |
| 4.5 | OG | R | | | | <0.5 | | | | | |
| 4.5 | OG | R | 1 | 0.8 | 0 | <0.5 | | 1.5 | B | | |
| 4.5 | X | | | | | | | | | | Mud and wattle only |
| 4.8 | OG | L | | | | 0.5 | 0.9 | 1.5 | P | | |
| 4.9 | X | | 1 | 1.0 | 0 | | 0.7 | | B | | Bypassed |
| 5.2 | OG | R | | | | 1.0 | 0.5 | | B | | |
| 5.2 | OG | L | | | | 1.0 | | | B | | |
| 5.5 | OG | L | | | | 1.0 | 0.6 | | B | | |
| 6.0 | OG | R | | | | <0.5 | 0.6 | | P | | |
| 6.1 | OG | L | | | | <0.5 | 0.5 | | P | | |
| 6.3 | OG | R | | | | <0.5 | 0.6 | | B | | |
| 6.4 | OG | L | | | | <0.5 | 0.4 | | F | | |
| 6.4 | FB | | | | | | | 2.5 | P | P | |
| 6.5 | OG | L | | | | <0.5 | 0.5 | | | | |
| 6.5 | OG | R | | | | <0.5 | | | | | |
| 6.6 | OG | L | | | | <0.5 | 0.5 | | P | | |
| 6.6 | OG | L | | | | <0.5 | 0.6 | | P | | |
| 6.7 | OG | L | | | | 0.5 | 0.4 | | P | | |
| 6.7 | OG | R | | | | 0.5 | | | P | | |
| 6.8 | OG | L | | | | 0.5 | 0.5 | | P | | |
| 7.2 | OG | R | | | | 0.5 | 0.4 | | P | | |
| 7.3 | OG | L | | | | | | | | | |
| 7.4 | OG | R | | | | 0.5 | 0.3 | | P | | |
| 7.5 | OG | L | | | | 0.5 | 0.5 | 2.5 | P | | |
| 7.5 | OG | L | | | | 0.5 | 0.4 | | P | | |
| 7.6 | OG | R | | | | 0.5 | 0.5 | | P | | |
| 7.7 | OG | L | | | | 0.5 | 0.4 | | P | | |

CANAL: SIGAALE

SHEET NO: 2/2

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7.7 | OG | R | | | | 0.5 | 0.4 | | P | | |
| 7.9 | OG | R | | | | | | | | | |
| 7.9 | OG | R | | | | 0.5 | 0.3 | | B | | |
| 8.1 | OG | L | | | | <0.5 | | | P | | |
| 8.1 | OG | R | | | | | | | | | |
| 8.3 | OG | L | | | | | | | | | |
| 8.4 | OG | R | | | | <0.5 | 0.6 | 3.0 | P | | |
| 8.4 | OG | L | | | | | | | | | |
| 8.5 | OG | L | | | | | | | | | |
| 8.5 | FB | | | | | | | 2.5 | | P | |
| 8.5 | OG | L | | | | <0.5 | 0.7 | | | B | |
| 8.6 | OG | L | | | | | | | | | |
| 8.6 | OG | R | | | | | | | | | |
| 8.7 | OG | L | | | | <0.5 | 0.6 | | | P | |
| 8.7 | OG | R | | | | | | | | P | |
| 8.8 | OG | L | | | | | | | | B | |
| 8.8 | OG | L | | | | <0.5 | 0.5 | | | B | |
| 8.8 | OG | R | | | | | | | | B | |
| 8.9 | OG | R | | | | <0.5 | 0.6 | | | B | |
| 9.0 | OG | R | | | | <0.5 | | | | P | |
| 9.0 | OG | R | | | | <0.5 | | | | P | |
| 9.1 | OG | R | | | | 0 | 0.5 | 3.0 | | P | |
| 9.5 | | | | | | | | 3.0 | | B | End of canal at reservoir No inflow to it. |

# MAIN CANAL SURVEY

CANAL: GIDDU

SHEET NO: 1/1

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 1 | 1.0 | 1 | | | | B | P | Head regulator closed at time of survey. |
| 0.6 | RB | | | | | 2.5 | 0.8 | | B | V | |
| 1.5 | OG | L | | | | 2.0 | 0.8 | | P | | |
| 1.7 | OG | R | | | | 1.5 | 0.8 | | F | | |
| 2.0 | OG | R | | | | 1.0 | 0.8 | | P | | |
| 2.0 | OG | L | | | | 1.0 | 0.8 | | P | | |
| 2.1 | OG | L | | | | 1.0 | 1.1 | | P | | |
| 2.2 | OG | R | | | | 1.0 | 1.0 | | P | | |
| 2.3 | OG | R | | | | 1.0 | 1.0 | | P | | |
| 2.3 | OG | L | | | | 1.0 | 0.9 | | P | | |
| 2.6 | FB | | | | | 1.0 | | | B | P | |
| 2.8 | OG | L | | | | 0.5 | 0.8 | 3.0 | B | | |
| 2.8 | OG | R | | | | 1.0 | 0.8 | | B | | |
| 2.9 | OG | R | | | | <0.5 | 0.7 | | B | | |
| 3.0 | OG | L | | | | 0.5 | 1.0 | | B | | |
| 3.1 | OG | R | | | | 0.5 | 1.1 | | B | | |
| 3.3 | OG | L | | | | <0.5 | 1.1 | | B | | |
| 3.3 | OG | R | | | | <0.5 | | | B | | |
| 3.4 | FB | | | | | | 1.0 | 2.5 | B | | |
| 3.5 | OG | L | | | | <0.5 | | | B | | |
| 3.5 | OG | R | | | | <0.5 | | | B | | |
| 3.8 | OG | R | | | | <0.5 | | | B | | |
| 4.9 | OG | L | | | | 0 | 1.1 | | B | | |
| 4.9 | OG | R | | | | 0 | | | B | | |
| 5.0 | OG | R | | | | 0 | 1.2 | | B | | |
| 5.2 | OG | R | | | | 0 | 1.3 | 2.5 | B | | |
| 5.5 | OG | L | | | | 0 | 1.4 | | B | | |
| 5.5 | OG | R | | | | 0 | | | B | | |
| 5.5 | OG | R | | | | 0 | | | B | | |
| 5.6 | OG | L | 1 | 1.1 | 0 | 0 | | | B | | Gate bypassed |
| 5.6 | OG | L | | | | 0 | | | | | |
| 5.6 | OG | R | | | | 0 | | | | | |
| 5.7 | OG | L | | | | 0 | | 2.5 | B | | |
| 5.7 | FB | | | | | | | | | P | |
| 5.7 | OG | R | | | | 0 | | | B | | |
| 5.7 | OG | R | | | | 0 | | | B | | |
| 6.1 | OG | R | | | | 0 | 1.3 | | B | | |
| 6.1 | OG | L | | | | 0 | | | B | | |
| 6.1 | FB | | | | | 0 | 1.3 | | B | P | Wattle bund at end of canal. |

# MAIN CANAL SURVEY

CANAL: ASAYLE

SHEET NO: 1/3

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 2 | 1.3 | 0 | | | | F | P | |
| 0.2 | RB | | | | | | | 5.0 | P | V | |
| 0.3 | OG | R | 1 | 0.45 | 1 | 1.5 | 0.1 | | P | | |
| 0.5 | OG | R | 1 | 0.45 | 1 | 1.5 | 0.1 | | P | | |
| 0.5 | OG | R | 1 | 0.45 | 1 | 2.0 | 0.1 | | P | | |
| 0.8 | OG | R | | 0.45 | 1 | 1.5 | 0.1 | | B | | |
| 0.8 | OG | L | | | | 1.5 | | | B | | |
| 0.8 | OG | R | | | | 1.5 | | | B | | |
| 1.3 | OG | R | 1 | 0.45 | 0 | 1.5 | 0.1 | | B | | |
| 1.4 | OG | R | 1 | 0.45 | 0 | 1.5 | 0.1 | | B | | |
| 1.4 | OG | L | | | | 1.5 | 0.3 | | B | | |
| 1.7 | OG | R | 1 | 1.25 | 0 | 1.5 | 0.2 | | B | | |
| 2.0 | OG | R | 1 | 0.45 | 0 | 1.0 | 0.4 | | P | | |
| 2.3 | FB | | | | | | 0.1 | | | P | |
| 3.0 | OG | L | | | | 1.0 | 0.2 | | B | | |
| 3.5 | OG | L | | | | 1.5 | 0.3 | | B | | |
| 4.3 | OG | R | 1 | 0.45 | 1 | 1.0 | 0.3 | | P | | |
| 4.3 | OG | L | | | | 1.0 | 0.3 | | P | | |
| 4.5 | FB | | | | | | | | | P | |
| 4.8 | X | | 2 | 1.3 | 0 | | | 5.0 | P | V | |
| 4.8 | OG | R | 1 | 0.45 | 0 | 1.5 | | | P | | |
| 6.0 | OG | R | 1 | 0.45 | 0 | 0.5 | | | B | | |
| 6.2 | OG | R | 1 | 0.45 | 0 | 0.5 | | | B | | |
| 6.3 | OG | R | 1 | 0.8 | 1 | 0.5 | | | P | | |
| 6.3 | OG | R | 1 | 0.8 | 1 | 0.5 | | | P | | |
| 6.3 | OG | R | 1 | 0.8 | 1 | 0.5 | | | P | | |
| 6.4 | OG | L | 1 | 0.45 | 0 | 0.5 | | | P | | |
| 6.4 | X | | 3 | 1.0 | 3 | | 0.3 | 5.0 | P | P | |
| 6.6 | RB | | | | | | | 5.0 | P | V | Bridge collapsed into water |
| 6.6 | OG | R | 1 | 0.45 | 0 | 0.5 | 0.0 | | P | | |
| 6.9 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | P | | |
| 6.9 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | P | | |
| 7.2 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.4 | | B | | |
| 7.2 | OG | R | | | | 0.5 | | | B | | |
| 7.5 | OG | R | | | | <0.5 | | | B | | |
| 7.5 | FB | | | | | | | | | P | |
| 7.6 | OG | L | 1 | 0.45 | 0 | <0.5 | 0.3 | | B | | |

CANAL: ASAYLE



| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7.7 | OG | R | 1 | 0.45 | 0 | <0.5 | 0.3 | | B | | |
| 8.3 | OG | L | 1 | 0.45 | 0 | <0.5 | 0.5 | | B | | |
| 8.3 | OG | L | 1 | 0.45 | 0 | <0.5 | | | | | |
| 8.3 | OG | R | 1 | 0.45 | 0 | <0.5 | | | B | | |
| 8.4 | OG | R | | | | 0.5 | 0.3 | | B | | |
| 8.9 | OP | L | 1 | 0.45 | 0 | <0.5 | | | P | | |
| 8.9 | OG | R | 1 | 0.45 | 0 | <0.5 | | | P | | |
| 8.9 | OG | R | 1 | 0.45 | 0 | <0.5 | | | P | | |
| 8.9 | OG | L | 1 | 0.45 | 0 | <0.5 | | | P | | |
| 8.9 | X | | 2 | 1.3 | 0 | | 0.3 | 5.0 | P | V | Tawakal |
| 9.5 | OG | L | 1 | 0.45 | 0 | <0.5 | | | P | | |
| 9.5 | OG | R | 1 | 0.45 | | 1.0 | | | P | | |
| 10.1 | OG | L | 1 | 0.45 | 0 | 1.0 | | | P | | |
| 10.1 | OG | R | 1 | 0.45 | 0 | 1.0 | 0.4 | | P | | |
| 10.4 | FB | | | | | | | | | P | |
| 10.4 | OG | L | 1 | 0.45 | 0 | 0.5 | | | P | | |
| 10.4 | OG | R | 1 | 0.45 | 0 | 0.5 | 0.3 | | P | | |
| 10.5 | OG | L | 1 | 0.45 | 0 | <0.5 | | | F | | |
| 10.5 | OG | R | 1 | 0.45 | 0 | 0.5 | 0.2 | | F | | |
| 10.9 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.4 | | F | | |
| 11.2 | OG | R | 1 | 0.45 | 0 | 0.5 | 0.4 | | F | | |
| 11.2 | OG | L | | | | | | | F | | |
| 11.7 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |
| 11.9 | OG | L | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 11.9 | OG | R | 1 | 0.45 | 0 | <0.5 | 0.5 | | F | | |
| 11.9 | FB | | | | | | | | | P | |
| 12.3 | OG | L | 1 | 0.45 | 0 | <0.5 | | 5.0 | G | | Canal clearance reached this point |
| 12.3 | OG | R | 1 | 0.45 | 0 | <0.5 | | | G | | |
| 12.3 | OG | L | 1 | 0.45 | 0 | <0.5 | | | G | | |
| 12.3 | OG | R | 1 | 0.45 | 0 | <0.5 | | | G | | |
| 12.3 | X | | 2 | 1.3 | 0 | <0.5 | | | F | V | |
| 12.6 | OG | R | 1 | 0.45 | 0 | <0.5 | 0.3 | | F | | |
| 12.6 | OG | R | | | | <0.5 | | | F | | |
| 12.8 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | 4.0 | F | | |
| 13.0 | OG | L | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 13.0 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 13.3 | FB | | | | | | | | | P | |
| 13.6 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |
| 13.6 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 13.7 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |

CANAL: ASAYLE

SHEET NO: 3/3

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 13.9 | OG | R | 1 | 0.6 | 0 | 0.5 | 0.3 | | F | | |
| 13.9 | OG | L | 1 | 0.6 | 0 | 0.6 | | | F | | |
| 14.2 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |
| 14.2 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 14.8 | FB | | | | | | | | | P | |
| 14.8 | OG | L | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |
| 14.8 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 15.1 | OG | R | 1 | 0.45 | 0 | 0.5 | 0.3 | | F | | |
| 15.1 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 15.1 | OG | L | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 15.1 | OG | R | 1 | 0.45 | 0 | 0.5 | | | F | | |
| 15.4 | OG | R | 1 | 0.45 | 0 | | | | F | | |
| 15.4 | X | | 3 | 1.3 | 0 | | | | P | V | |
| 15.6 | OG | L | | | | <0.5 | | | B | | |
| 15.6 | OG | R | | | | <0.5 | | | B | | |
| 15.8 | | | | | | | | | | | Junction with river |

## MAIN CANAL SURVEY

CANAL: DHAMME YAASIIN

SHEET NO: 1/1

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 5 | 1.75 | 2 | | 0.3 | 9.0 | G | V | |
| 0.3 | PB | | | | | 3.0 | | | | P | |
| 1.1 | RB | | | | | 4.0 | 0.3 | 9.0 | B | V | |
| 1.4 | OG | L | 1 | 1.0 | 1 | 4.0 | | | P | | VI secondary |
| 1.5 | OG | L | 1 | 1.0 | 1 | 4.0 | | | | | |
| 1.5 | OG | L | 3 | 1.25 | 1 | 4.0 | 0.0 | | | | II secondary |
| 1.5 | OG | L | 1 | 1.0 | 0 | 4.0 | | | | | |
| 1.5 | X | | 4 | 2.0 | 0 | | 0.0 | 8.0 | B | P | |
| 3.5 | OG | L | 1 | 1.0 | 1 | 2.5 | 0.2 | 6.0 | B | | |
| 3.9 | OG | R | 1 | 0.75 | 1 | 2.0 | | | | | |
| 4.5 | OG | L | 1 | ? | 0 | | | | | | III secondary |
| 4.5 | X | | 5 | 1.4 | 0 | 2.0 | 0.4 | 7.0 | B | V | |
| 4.9 | OG | R | 1 | 0.75 | 0 | 2.0 | | | | | |
| 7.4 | OG | R | 1 | 1.0 | 0 | 1.5 | | | P | | |
| 7.7 | OG | R | 1 | 1.0 | 0 | 1.5 | 0.4 | 5.0 | P | | |
| 8.1 | OG | L | 1 | 1.0 | 0 | 1.5 | | | P | | |
| 8.3 | OG | R | 1 | 1.0 | 1 | 1.5 | | | P | | |
| 8.4 | RB | | | | | | | | P | V | |
| 8.7 | OG | L | 2 | 1.0 | 1 | 1.5 | | | P | | |
| 10.3 | OG | R | 2 | 1.0 | 0 | 1.0 | | | | | |
| 10.3 | OG | L | 2 | 1.25 | 1 | 1.0 | | | P | | IV secondary |
| 10.3 | X | | 3 | 1.15 | 1 | 1.0 | 0.4 | 5.0 | P | P | |
| 11.7 | OG | L | 1 | 1.0 | 0 | 1.5 | | | P | | |
| 11.9 | OG | L | 1 | 1.0 | 1 | 1.0 | | | P | | |
| 12.0 | OG | R | 1 | 1.0 | 1 | 1.0 | | | P | | |
| 12.2 | OG | R | 1 | 1.0 | 1 | | 0.8 | | P | | |
| 12.3 | X | | 4 | 0.93 | 0 | | | | B | V | Shalambood road |
| 12.6 | OG | L | 1 | 1.0 | 1 | 1.0 | 0.8 | 4.0 | P | | |
| 13.5 | OG | R | 1 | 1.0 | 0 | 1.0 | | | P | | |
| 13.9 | OG | L | 1 | 1.0 | 1 | 1.0 | | | G | | |
| 13.9 | RB | | | | | | | | | V | |
| 14.0 | OG | R | 1 | 1.0 | 0 | 1.0 | 1.0 | 4.0 | P | | |
| 14.9 | OG | R | 1 | 0.5 | 0 | 1.0 | 1.0 | 3.0 | B | | |
| 15.2 | OG | L | 1 | 0.85 | 1 | 1.0 | | | P | | |
| 15.7 | OG | L | 1 | 1.0 | 0 | 0.5 | | | F | | |
| 15.7 | OG | L | 2 | 1.2 | 1 | 0.5 | | | F | | V secondary |
| 15.7 | OG | L | 1 | 1.0 | 0 | 0.5 | | | F | | |
| 15.7 | OG | L | 1 | 1.0 | 0 | 0.5 | | | F | | |
| 15.7 | OG | R | 2 | 1.1 | 0 | 0.5 | 1.0 | 2.0 | F | | Link canal to Primo Secondario |

## MAIN CANAL SURVEY

CANAL: PRIMO SECONDARIO

SHEET NO: 1/4

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 3 | 1.1 | 0 | 1.0 | 0.4 | | G | P | |
| 0.0 | OG | L | 1 | 0.5 | 0 | 1.0 | | | G | | |
| 0.1 | RB | | | | | | 0.6 | 9.0 | G | V | |
| 0.5 | OG | L | 1 | 1.2 | 0 | 1.0 | 0.5 | | G | | |
| 0.9 | OG | L | 1 | 0.9 | 0 | 1.0 | 0.5 | | F | | |
| 1.2 | OG | L | 1 | 0.9 | 1 | 1.5 | 0.5 | | F | | |
| 1.5 | OG | L | 1 | 1.2 | 0 | 1.0 | 0.1 | | G | | |
| 1.5 | X | | 4 | 1.2 | 0 | 1.0 | 0.2 | | G | P | |
| 1.8 | OG | L | 1 | 0.9 | 0 | 1.0 | | | F | | |
| 2.6 | OG | L | | | | 1.0 | | 7.0 | F | | |
| 2.7 | OG | R | 1 | 0.7 | 1 | 0.5 | 0.3 | | F | | |
| 3.0 | OG | L | 1 | 0.7 | 1 | 0.5 | 0.4 | | F | | |
| 3.6 | OP | L | 1 | 0.9 | 0 | 0.5 | | | F | | |
| 4.1 | OG | L | 1 | 0.7 | 0 | 0.5 | | 6.0 | G | | |
| 4.1 | OP | R | | | | | | | G | | |
| 4.1 | OG | R | 1 | 1.0 | 1 | 0.5 | 0.4 | | G | | |
| 4.1 | FB | | | | | | | 8.0 | G | P | |
| 4.1 | OG | L | | | | 0.5 | | | G | | |
| 4.5 | RB | | | | | 0.5 | 0.5 | 7.0 | G | V | |
| 4.8 | OG | L | | | | 1.0 | 0.3 | | G | | |
| 5.0 | OG | R | 1 | 0.7 | 1 | 1.0 | | | G | | |
| 5.0 | FB | | | | | 1.0 | | 10.0 | G | | Mushaani village |
| 5.3 | OG | L | 1 | 0.9 | 1 | 1.0 | 0.3 | | G | | |
| 5.4 | OP | R | 1 | 0.9 | 0 | | | | G | | |
| 5.9 | OG | L | 1 | 0.75 | 1 | 0.5 | 0.5 | | G | | |
| 5.9 | RB | | | | | | | 7.0 | G | V | |
| 6.3 | OG | L | 1 | 0.9 | 0 | 0.5 | 0.7 | | G | | |
| 6.3 | OG | R | | | | 1.0 | | | G | | |
| 6.3 | OG | L | 1 | 0.5 | 1 | 1.0 | 0.7 | | G | | |
| 6.5 | OG | R | 1 | 0.8 | 1 | 1.0 | 0.7 | | G | | |
| 6.5 | RB | | | | | | | 7.0 | G | V | Majabto road |
| 7.1 | OG | R | 1 | 0.7 | 1 | 0.5 | 0.5 | | G | | |
| 7.1 | OG | R | 1 | 0.85 | 1 | 0.5 | | | G | | |
| 7.5 | OG | R | 1 | 1.0 | 1 | 0.5 | 0.4 | | G | | |
| 7.6 | RB | | | | | | | 7.0 | G | V | |
| 7.8 | OG | L | 1 | 0.6 | 1 | 1.0 | 0.4 | | G | | |
| 7.9 | OG | L | 1 | 1.0 | 0 | 1.0 | | | G | | |
| 7.9 | OG | R | 1 | 1.0 | 1 | | | | G | | |
| 7.9 | FB | | | | | | | | | P | |

CANAL: PRIMO SECONDARIO

SHEET NO: 2/4

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 8.0 | OP | R | 1 | 1.0 | 1 | | | | G | | |
| 8.4 | RB | | | | | | 0.3 | 7.0 | G | V | |
| 8.8 | OG | L | 1 | 0.8 | 1 | 1.0 | 0.3 | | F | | |
| 8.8 | OP | R | 1 | 1.0 | 1 | | | | F | | |
| 9.3 | OP | R | 1 | 1.0 | 1 | | | | F | | |
| 9.5 | OG | L | 1 | 1.0 | 0 | 1.0 | 0.05 | | F | | Zero freeboard from |
| 10.0 | OG | L | 1 | 1.0 | 1 | | 0.00 | | F | | Km 9.5 to Km 10.8 |
| 10.0 | OP | R | 1 | 1.0 | 1 | | 0.00 | | F | | |
| 10.0 | FB | | | | | | 0.00 | | F | P | |
| 11.2 | OG | R | 1 | 1.0 | 1 | 1.5 | | | F | | |
| 12.1 | RB | | | | | | | 6.0 | P | V | Qoryooley road |
| 12.3 | OG | L | 1 | 1.0 | 0 | 1.0 | | | P | | |
| 12.3 | OP | R | 1 | 1.0 | 1 | | 0.6 | | P | | |
| 13.0 | OP | R | 1 | 1.0 | 0 | | 0.4 | | P | | |
| 13.2 | OG | L | 1 | 0.8 | 1 | 1.0 | | | P | | |
| 13.2 | OG | L | 1 | 1.4 | 1 | 1.0 | | | P | | Link to Dhamme Yaasiin |
| 13.2 | OP | L | 1 | 0.85 | 0 | 1.0 | | | P | | |
| 13.2 | X | | 3 | 1.25 | 1 | | | 4.0 | B | P | |
| 13.9 | OP | R | 1.0 | 1 | 1.0 | | | | P | | |
| 14.3 | RB | | | | | | 1.0 | 6.0 | F | V | |
| 14.3 | OP | L | 1 | 0.8 | 0 | | | | F | | |
| 14.7 | OG | R | 1 | 1.05 | 1 | 0.5 | | | F | | |
| 15.0 | OP | L | 1 | 0.8 | 1 | | | | F | | |
| 15.5 | OG | R | 1 | 0.9 | 1 | 1.0 | 0.6 | | F | | |
| 15.8 | OP | L | 1 | 0.5 | 0 | 1.0 | | | F | | |
| 16.3 | OP | R | 1 | 1.0 | 1 | | | | F | | |
| 16.3 | RB | | | | | | 1.2 | 7.0 | F | V | |
| 16.7 | OG | L | 1 | 0.8 | 1 | 0.5 | | | F | | |
| 16.8 | OP | R | 1 | 1.0 | 1 | | | | F | | |
| 17.1 | OP | L | 1 | 1.0 | 1 | | 1.2 | | F | | |
| 17.5 | OG | L | | | | 1.0 | | | F | | |
| 17.7 | OG | R | 1 | 0.7 | 0 | 0.5 | | | F | | |
| 18.0 | OP | R | 1 | 0.7 | 1 | 0.5 | | | F | | |
| 18.0 | OP | L | 1 | 1.1 | 1 | | | | F | | |
| 18.0 | X | | 3 | 1.2 | 0 | | 1.2 | 7.0 | F | P | |
| 18.7 | RB | | | | | | 1.0 | 8.0 | F | V | Madhuulow road |
| 19.0 | OP | L | 1 | 0.9 | 0 | 0.5 | | | F | | |
| 19.0 | OG | R | 1 | 0.9 | 1 | 1.0 | | | F | | |
| 19.3 | OP | L | 1 | 1.0 | 0 | | | | F | | |
| 19.3 | OG | R | 1 | 1.0 | 1 | 1.0 | | | F | | |
| 19.5 | OG | R | 1 | 0.85 | 1 | | 1.2 | | F | | |
| 19.5 | OP | L | 1 | 0.85 | 0 | | | | F | | |

CANAL: PRIMO SECONDARIO

SHEET NO: 3/4

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20.0 | OG | R | 1 | 0.6 | 0 | 1.5 | | | F | | |
| 20.1 | OP | R | 1 | 1.0 | 1 | 1.0 | 0.8 | | F | | |
| 20.2 | OG | L | 1 | 0.8 | 1 | 1.0 | | | F | | |
| 21.4 | OG | L | 1 | 0.8 | 1 | 1.0 | | | P | | |
| 21.4 | OG | R | 1 | 0.5 | 0 | 1.0 | | | P | | |
| 21.7 | OP | R | 1 | 1.05 | 1 | | 1.0 | | P | | |
| 21.7 | OG | L | 1 | 1.0 | 0 | 1.0 | | | P | | |
| 21.7 | OP | L | 1 | 0.9 | 1 | 1.0 | | | P | | |
| 21.7 | OG | L | 1 | 1.0 | 1 | 1.0 | | | P | | |
| 21.7 | X | | 2 | 0.9 | 0 | | 1.5 | 7.0 | | P | |
| 21.9 | RB | | | | | | 1.0 | 7.0 | G | V | Golweyn |
| 22.0 | OG | R | 1 | 1.0 | 1 | 1.0 | | | G | | |
| 22.0 | OG | R | | | | 0.5 | | | G | | |
| 22.3 | OG | R | | | | 0.5 | | | G | | |
| 22.4 | FB | | | | | | 1.1 | 8.0 | G | P | |
| 22.4 | OG | R | 1 | 0.5 | 0 | 0.5 | | | G | | |
| 22.6 | OG | L | 1 | 0.5 | 0 | 0.5 | 1.3 | | G | | |
| 22.8 | OP | L | 1 | 1.0 | 1 | | | | G | | |
| 22.9 | OG | R | 1 | 0.95 | 1 | 0.5 | 1.3 | | G | | |
| 23.4 | OP | L | 1 | 0.5 | 1 | | 1.2 | | F | | |
| 23.5 | OP | L | 1 | 1.0 | 1 | 0.5 | 1.0 | | P | | |
| 24.3 | OG | R | | | | 0.5 | 1.0 | | F | | |
| 24.3 | OG | R | 1 | 0.9 | 0 | 0.5 | | | F | | |
| 24.3 | OP | L | 1 | 0.9 | 0 | 0.5 | | | P | | |
| 24.9 | RB | | | | | | 1.0 | 7.0 | F | V | |
| 25.3 | OG | L | 1 | 0.5 | 0 | 0.5 | | | F | | |
| 25.3 | OG | R | 1 | 0.9 | 1 | 0.5 | 0.8 | | F | | |
| 25.4 | OG | L | 1 | 0.9 | 1 | 0.5 | | | F | | |
| 25.5 | OG | L | 1 | 0.5 | 0 | 0.5 | 0.8 | | F | | |
| 26.0 | OG | R | 1 | 0.9 | 1 | 0.5 | 0.4 | | G | | |
| 26.0 | RB | | | | | | | 6.0 | G | | Antonyo |
| 26.0 | OP | L | 1 | 2.0 | 1 | | 1.0 | | G | | |
| 26.2 | X | | 3 | 0.9 | 3 | | | | G | P | |
| 26.3 | - | R | | | | | | | | | Junction with Wadajir |
| 26.8 | OG | R | 1 | 0.9 | 0 | 0.5 | 1.0 | | F | | |
| 26.8 | OG | R | 1 | 0.9 | 0 | 0.5 | 1.0 | | F | | |
| 26.9 | OG | L | 1 | 1.0 | 0 | 0.5 | 1.2 | | F | | |
| 27.8 | OG | R | 1 | 0.9 | 1 | 0.5 | 1.0 | | F | | |
| 28.6 | OG | L | 1 | 1.0 | 1 | 1.0 | 0.9 | | G | | |
| 28.7 | OG | L | 1 | 0.9 | 0 | 1.0 | | | G | | |
| 29.4 | OG | R | 1 | 0.9 | 1 | 1.0 | 0.8 | | G | | |

CANAL: PRIMO SECONDARIO



| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 29.4 | OG | R | 1 | 0.9 | 0 | 1.0 | | | G | | |
| 29.6 | OG | R | 1 | 0.7 | 0 | 0.5 | 0.4 | | G | | |
| 29.8 | FB | | | | | | | 7.0 | G | P | Buulo Mareerta |
| 30.0 | X | | 5 | 0.9 | 0 | | 0.8 | 6.0 | G | V | |
| 30.1 | OG | R | 1 | 0.9 | 0 | < 0.5 | 1.0 | | G | | |
| 30.4 | OG | L | 1 | 0.9 | 0 | < 0.5 | 0.8 | | G | | |
| 30.7 | OG | R | 1 | 0.9 | 1 | < 0.5 | 1.0 | | F | | |
| 30.8 | OG | R | 1 | 0.5 | 0 | < 0.5 | 1.2 | | F | | |
| 30.8 | OG | L | 1 | 0.9 | 1 | < 0.5 | | | F | | |
| 31.3 | OG | L | 1 | 0.9 | 1 | < 0.5 | 1.0 | | F | | |
| 31.7 | OG | R | | | | < 0.5 | | | P | | |
| 32.0 | OG | R | 1 | 0.5 | 0 | < 0.5 | | | P | | |
| 32.2 | OG | L | 1 | 0.85 | 1 | 0.5 | | | P | | |
| 32.5 | OG | R | | | | 0.5 | | | P | | |
| 32.5 | OG | L | 1 | 0.65 | 1 | 0.5 | 1.0 | | P | | |
| 32.7 | X | | 5 | 0.9 | 0 | | | | F | V | Tail pool on Bokore |

# MAIN CANAL SURVEY

CANAL: WADAJIR

SHEET NO: 1/1

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 4 | 1.35 | 0 | 0.0 | 1.0 | 4.5 | F | P | |
| 0.6 | RB | | | | | 0.0 | | 5.0 | P | V | |
| 1.6 | FB | | | | | 0.0 | 0.3 | | P | P | |
| 1.6 | OG | R | 1 | 0.55 | 1 | 0.5 | 0.6 | | P | | |
| 2.4 | OG | L | 1 | 0.55 | 0 | 1.0 | 0.6 | | F | | |
| 2.4 | OG | R | | | | 1.0 | 0.6 | | F | | |
| 2.6 | OG | R | | | | 1.0 | 0.5 | | F | | |
| 2.7 | OG | L | 1 | 0.55 | 0 | 1.0 | 0.5 | | F | | |
| 2.7 | OG | R | 1 | 0.75 | 0 | 1.0 | 0.5 | | F | | |
| 2.7 | X | | 5 | 0.85 | 1 | 1.0 | 0.8 | 4.5 | F | V | |
| 4.0 | OG | R | | | | 2.5 | | | P | | |
| 5.6 | OG | L | 1 | 0.55 | 0 | 3.5 | | | P | | |
| 5.6 | OG | R | 1 | 0.55 | 0 | 3.5 | | | P | | |
| 5.6 | RB | | | | | 3.0 | 0.8 | 7.0 | F | V | Madhuulow. |
| 6.2 | OG | L | 1 | 0.85 | 0 | 2.5 | 0.1 | | P | | |
| 6.6 | OG | R | 1 | 0.55 | 0 | 2.5 | 0.2 | | P | | |
| 7.2 | OG | L | | | | 2.0 | 0.3 | | P | | |
| 7.7 | OG | R | | | | 1.5 | 0.2 | | P | | |
| 7.7 | OG | R | 1 | 0.55 | 0 | 1.5 | 0.2 | 5.0 | B | | |
| 8.4 | OG | L | 1 | 0.55 | 0 | 1.5 | 0.2 | | B | | |
| 8.4 | OG | R | 1 | 0.8 | 1 | 1.5 | 0.2 | | B | | |
| 8.9 | OG | L | 1 | 0.55 | 1 | 1.5 | 0.3 | | B | | |
| 8.9 | OG | R | 1 | 0.55 | 1 | 1.5 | 0.3 | | B | | |
| 9.0 | OG | R | 1 | 0.55 | 0 | 1.5 | 0.3 | | B | | |
| 9.4 | OG | R | 1 | 0.55 | 0 | 1.5 | 0.3 | | B | | |
| 9.9 | OG | L | | | | 1.5 | 0.5 | | B | | |
| 9.9 | OG | R | 1 | 0.55 | 1 | 1.5 | 0.5 | | B | | |
| 9.9 | FB | | | | | | | 6.0 | B | P | |
| 10.3 | OG | | | | | 1.5 | 0.3 | | P | | |
| 10.7 | OG | R | 1 | 0.55 | 0 | 1.0 | 0.5 | | P | | |
| 11.1 | OG | R | 1 | 0.55 | 0 | 1.0 | 0.5 | | F | | |
| 11.5 | OG | R | 1 | 0.55 | 0 | 0.5 | 0.6 | | F | | |
| 12.6 | X | | 5 | 0.85 | 0 | 0.5 | 1.2 | | | V | Tail at the junction with Primo Secondario |

## MAIN CANAL SURVEY

CANAL: LIIBAAN

SHEET NO: 1/1

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | | | | 1.5 | 11.0 | P | | River |
| 0.1 | H | | 7 | 0.85 | 0 | 1.0 | | 8.0 | B | V | |
| 0.2 | OG | R | 1 | 0.85 | 0 | 1.0 | 0.8 | | B | | |
| 0.3 | OG | R | | | | 1.0 | 0.8 | | B | | |
| 0.6 | FB | | | | | | 0.7 | 10.0 | B | P | |
| 0.8 | OG | R | 1 | 0.85 | 0 | 1.0 | 0.8 | | B | | |
| 1.5 | OG | L | 1 | 0.85 | 0 | 0.5 | 0.8 | | F | | |
| 1.5 | OG | R | 1 | 0.85 | 1 | 0.5 | 0.8 | 10.0 | F | | |
| 1.8 | FB | | | | | | 0.8 | 10.0 | P | P | |
| 2.4 | OG | L | 1 | 0.85 | 0 | 1.0 | 0.5 | | P | | |
| 2.4 | X | | 7 | 0.85 | 0 | 1.0 | 0.8 | | B | V | Falkeerow road |
| 4.1 | OG | L | 1 | 0.85 | 0 | 1.0 | 1.0 | | P | | |
| 4.1 | OG | R | 1 | 0.85 | 1 | 1.0 | | | P | | |
| 4.4 | OG | L | | | | 1.0 | | | P | | |
| 4.6 | OG | L | 1 | 1.0 | 1 | 1.0 | 1.0 | | P | | |
| 4.6 | OG | R | 1 | 0.85 | 1 | 1.0 | | | P | | |
| 4.6 | X | | 7 | 0.85 | 4 | | 0.8 | 9.0 | B | P | |
| 5.7 | OG | R | | | | 0.5 | | | B | | |
| 6.0 | OG | R | | | | 0.5 | | | B | | |
| 6.1 | OG | L | | | | 0.5 | | | B | | Edge of Study Area at Km 6.3 : old river channel crosses |
| 6.4 | OG | L | | | | <0.5 | 1.0 | 4.0 | G | | |
| 6.5 | OG | R | | | | <0.5 | | | G | | |
| 6.6 | OG | R | | | | <0.5 | 0.0 | | G | | |
| 6.9 | OG | L | | | | <0.5 | | | P | | |
| 6.9 | OG | R | | | | <0.5 | 0.0 | 2.0 | P | | |
| 7.0 | OG | L | | | | <0.5 | 0.2 | | P | | |
| 7.0 | FB | | | | | | | 2.0 | | P | |
| 7.1 | OG | L | | | | <0.5 | | | P | | |
| 7.2 | OG | L | | | | <0.5 | 0.3 | 1.0 | P | | |
| 7.3 | OG | R | | | | <0.5 | | | P | | |
| 7.3 | OG | L | | | | <0.5 | | | P | | |
| 7.4 | OG | L | | | | <0.5 | | | P | | |
| 7.7 | OG | L | | | | 0.0 | | | B | | |
| 8.1 | RB | | | | | | 1.0 | | | V | |
| 8.3 | OG | R | | | | 0.0 | 1.0 | | B | | |
| 8.3 | | | | | | | | 0.8 | | | Tail at reservoir |

# MAIN CANAL SURVEY

CANAL: BOKORE

SHEET NO: 1/2

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | H | | 9 | 0.9 | 9 | 2.0 | 0.2 | 5.0 | F | P | |
| 1.1 | RB | | | | | 2.0 | 0.5 | | F | V | Cabdi Cali road |
| 2.1 | OG | L | 3 | 0.55 | 0 | 2.0 | 0.2 | 5.0 | F | | |
| 2.2 | OG | R | | | | 2.0 | 0.2 | 5.0 | F | | |
| 3.4 | OG | L | 1 | 0.7 | 1 | 2.0 | 0.4 | | F | | |
| 3.6 | OG | R | 1 | 0.5 | 0 | 2.0 | | | F | | |
| 5.0 | OG | L | 1 | 0.7 | 0 | 2.5 | 0.4 | | P | | |
| 5.4 | OG | R | 1 | 0.9 | 0 | 2.0 | 0.2 | | P | | |
| 5.4 | OG | L | 3 | 0.55 | 3 | 2.0 | 0.3 | | P | | SISAB secondary |
| 6.2 | OG | R | 3 | 0.55 | 2 | 2.0 | 0.3 | | F | | Jeerow secondary |
| 6.6 | OG | R | 1 | 0.55 | 0 | 2.0 | 0.3 | | F | | |
| 6.6 | OG | L | 1 | 0.55 | 0 | 2.0 | 0.3 | | F | | |
| 7.5 | OG | L | 1 | 1.0 | 0 | 2.0 | 0.3 | | F | | |
| 7.7 | X | | 9 | 0.9 | 0 | | 0.2 | 10.0 | F | V | Jeerow |
| 7.8 | OG | L | 1 | 0.9 | 1 | 1.5 | 0.3 | | F | | |
| 8.0 | OG | L | 1 | 0.9 | 0 | 1.5 | 0.3 | | F | | |
| 8.5 | OG | R | 1 | 1.0 | 1 | 1.5 | 0.4 | | P | | |
| 8.7 | OG | L | 1 | 0.65 | 1 | 1.5 | 0.3 | | P | | |
| 9.4 | OG | L | 1 | 0.75 | 1 | 1.5 | 0.3 | | P | | |
| 9.4 | OG | L | 1 | 0.60 | 1 | 1.5 | 0.5 | | P | | |
| 9.5 | OG | R | 3 | 0.5 | 0 | 1.5 | 0.5 | | P | | Mukoy Dumis secondary |
| 10.7 | OG | L | 1 | 0.5 | 1 | 1.5 | 0.4 | | P | | |
| 11.4 | OG | L | 1 | 0.6 | 1 | 1.5 | 0.7 | | P | | |
| 12.1 | OG | L | 1 | 0.55 | 1 | 1.0 | 0.5 | | P | | |
| 12.5 | OP | R | | | | | 0.4 | | P | | |
| 12.7 | OG | L | 1 | 0.9 | 0 | 1.0 | | | P | | |
| 12.7 | OG | R | 1 | 0.5 | 1 | 0.5 | | | P | | |
| 12.8 | OG | L | 1 | 0.65 | 0 | 1.0 | 0.5 | | P | | |
| 13.3 | X | | 5 | 1.8 | 0 | 1.0 | | 10.0 | P | V | |
| 14.1 | OG | R | | | | 1.0 | | | P | | |
| 14.3 | OG | L | 1 | 0.5 | 0 | 1.0 | | | P | | |
| 14.9 | OG | R | 1 | 1.0 | 0 | 0.5 | 0.4 | | P | | |
| 15.5 | OG | R | | | | 0.5 | 0.2 | | P | | |
| 15.8 | OP | R | 1 | 1.0 | 0 | | 0.3 | | P | | |
| 16.0 | X | | 9 | 0.85 | 1 | | 0.5 | 10.0 | B | V | Entrance to tail pool |
| 16.1 | OG | R | 1 | 1.0 | 0 | 0.5 | 1.0 | | B | | |

| Chainage (km) | Structure | Bank | Number of gates | Width of each gate (m) | No. of gates fully operational | Command (m) | Freeboard (m) | Canal width (m) | Weed condition | Crossing points | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 16.1 | - | L | | | | | | | B | | Tail of Primo Secondario |
| 16.1 | X | | 7 | 0.9 | 3 | | 0.6 | | F | V | Exit from tail pool |
| 16.3 | OG | L | | | | 0.5 | 0.4 | | F | | Buulo Mareerta road |
| 16.5 | OG | L | | | | 0.5 | 0.4 | | F | | |
| 16.8 | OG | L | | | | 0.5 | 0.3 | | F | | |
| 16.9 | OG | R | | | | 0.5 | 0.2 | | F | | |
| 17.0 | OG | R | | | | 0.5 | | | P | | |
| 17.1 | OG | L | 1 | 0.5 | 0 | 0.5 | 0.4 | | P | | |
| 17.3 | OG | L | | | | 0.5 | 0.4 | | P | | |
| 17.5 | OG | L | | | | < 0.5 | 0.4 | | P | | |
| 17.5 | OG | R | | | | | | | P | | |
| 17.8 | OG | L | | | | < 0.5 | | | B | | |
| 17.9 | OG | R | | | | < 0.5 | | | B | | |
| 18.2 | OG | L | | | | < 0.5 | 0.2 | | P | | |
| 18.3 | OG | L | 1 | 0.6 | 0 | < 0.5 | 0.3 | | P | | |
| 18.3 | OG | L | | | | < 0.5 | | | P | | |
| 18.4 | OG | L | | | | < 0.5 | | | P | | |
| 18.4 | OG | L | | | | < 0.5 | | | P | | |
| 18.5 | OG | L | | | | < 0.5 | | | P | | Tail at old river channel |

# APPENDIX B

# LAND FORMING TRIALS

## APPENDIX B

## LAND FORMING TRIALS

Three trial areas within the Qoryooley project were selected and surveyed on a 25 m grid. Each trial covered a total area of 6.25 ha, all the trials being orientated with the dip and strike parallel to the edges of the areas. Table B.1 provides the details of each trial, and Tables B.2, B.3 and B.4 give the detailed results of each trial together with the fill calculations. In each 25 m square (at the centre of which is the co-ordinate point) the central figure is the measured ground level (to an arbitrary datum), the lower figure is the required level after land forming is complete, and the upper figure is the difference between the two (fill requirements negative).

The plane of best fit is found by fitting graphically a best line to the average elevations in both directions. From the two resultant slopes, and knowing the plane of best fit passes through the average elevation of the complete plot in the middle of the area, the required elevations can be found for each co-ordinate point.

The total fill required is then simply the sum of the negative differences, multiplied by the area of one 25 m square. As a check, the amount of cut is also calculated and should agree with the fill required (small rounding errors will occur).

### TABLE B.1

### Detail of Land Forming Trials

| Trial | Approximate co-ordinates of the NW corner | Maximum variation in land level (m) | Fill required ($m^3$/ha) | Present land use |
|---|---|---|---|---|
| I | 14 500/32 250 | 1.03 | 300 | cultivated |
| II | 16 800/34 150 | 0.61 | 295 | uncultivated but cleared |
| III | 17 000/32 600 | 0.34 | 195 | cultivated |

# TABLE B.2

## Land Forming Trial I

| Grid | A | B | C | D | E | F | G | H | I | J | Average elevation (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.12 | 0.17 | 0.02 | 0.06 | 0.05 | 0.08 | - 0.03 | - 0.17 | - 0.06 | - 0.07 | |
| 1 | 7.86 | 7.81 | 7.95 | 7.90 | 7.90 | 7.87 | 7.97 | 8.10 | 7.98 | 7.99 | 7.93 |
| | 7.98 | 7.98 | 7.97 | 7.96 | 7.95 | 7.95 | 7.94 | 7.93 | 7.92 | 7.92 | |
| | 0.05 | 0.07 | 0.04 | - 0.07 | 0.04 | 0.02 | 0.04 | 0.07 | 0 | 0 | |
| 2 | 8.01 | 7.99 | 8.01 | 8.11 | 7.98 | 8.01 | 8.06 | 0.08 | 8.00 | 8.00 | 8.03 |
| | 8.06 | 8.06 | 8.05 | 8.04 | 8.03 | 8.03 | 8.02 | 8.01 | 8.00 | 8.00 | |
| | 0.05 | - 0.05 | 0.07 | 0.04 | 0.05 | 0.06 | - 0.03 | - 0.11 | 0.03 | 0.02 | |
| 3 | 8.09 | 8.19 | 8.06 | 8.08 | 8.06 | 8.05 | 8.13 | 8.20 | 8.05 | 8.06 | 8.10 |
| | 8.14 | 8.14 | 8.13 | 8.12 | 8.11 | 8.11 | 8.10 | 8.09 | 8.08 | 8.08 | |
| | -0.05 | - 0.06 | 0.02 | 0.01 | - 0.03 | 0.02 | - 0.04 | - 0.03 | - 0.01 | - 0.04 | |
| 4 | 8.28 | 8.29 | 8.20 | 8.20 | 8.23 | 8.18 | 8.23 | 8.21 | 8.18 | 8.21 | 8.22 |
| | 8.23 | 8.23 | 8.22 | 8.21 | 8.20 | 8.20 | 8.19 | 8.18 | 8.17 | 8.17 | |
| | 0.03 | 0.01 | - 0.05 | - 0.12 | - 0.07 | 0.03 | 0.05 | 0.03 | 0 | 0.03 | |
| 5 | 8.28 | 8.30 | 8.35 | 8.41 | 8.35 | 8.25 | 8.22 | 8.23 | 8.25 | 8.22 | 8.29 |
| | 8.31 | 8.31 | 8.30 | 8.29 | 8.28 | 8.28 | 8.27 | 8.26 | 8.25 | 8.25 | |
| | -0.05 | - 0.01 | - 0.03 | - 0.06 | - 0.13 | - 0.05 | 0.01 | 0.06 | 0.07 | 0.11 | |
| 6 | 8.44 | 8.40 | 8.41 | 8.43 | 8.49 | 8.41 | 8.34 | 8.28 | 8.26 | 8.22 | 8.37 |
| | 8.39 | 8.39 | 8.38 | 8.37 | 8.36 | 8.36 | 8.35 | 8.34 | 8.33 | 8.33 | |
| | -0.02 | 0.01 | - 0.03 | - 0.06 | - 0.02 | - 0.08 | - 0.07 | 0.07 | 0.04 | 0.25 | |
| 7 | 8.49 | 8.46 | 8.49 | 8.51 | 8.46 | 8.52 | 8.50 | 8.35 | 8.37 | 8.15 | 8.43 |
| | 8.47 | 8.47 | 8.46 | 8.45 | 8.44 | 8.44 | 8.43 | 8.42 | 8.41 | 8.41 | |
| | 0.08 | - 0.01 | 0.04 | - 0.02 | - 0.05 | - 0.06 | - 0.07 | 0.04 | 0.02 | 0.22 | |
| 8 | 8.48 | 8.57 | 8.51 | 8.56 | 8.58 | 8.59 | 8.59 | 8.47 | 8.48 | 8.28 | 8.51 |
| | 8.56 | 8.56 | 8.55 | 8.54 | 8.53 | 8.53 | 8.52 | 8.51 | 8.50 | 8.50 | |
| | 0.05 | 0.07 | - 0.04 | - 0.02 | - 0.04 | - 0.16 | - 0.12 | - 0.09 | 0.15 | 0.07 | |
| 9 | 8.59 | 8.57 | 8.67 | 8.64 | 8.65 | 8.77 | 8.72 | 8.68 | 8.43 | 8.51 | 8.62 |
| | 8.64 | 8.64 | 8.63 | 8.62 | 8.61 | 8.61 | 8.60 | 8.59 | 8.58 | 8.58 | |
| | 0.05 | 0.17 | 0.01 | - 0.11 | - 0.14 | - 0.15 | - 0.08 | 0.03 | 0.10 | - 0.01 | |
| 10 | 8.67 | 8.55 | 8.70 | 8.81 | 8.83 | 8.84 | 8.76 | 8.64 | 8.56 | 8.67 | 8.70 |
| | 8.72 | 8.72 | 8.71 | 8.70 | 8.69 | 8.69 | 8.68 | 8.67 | 8.66 | 8.66 | |
| Average elevation (m) | 8.32 | 8.31 | 8.34 | 8.37 | 8.35 | 8.35 | 8.35 | 8.32 | 8.26 | 8.23 | 8.32 |
| Fill | 0.43 | 0.50 | 0.20 | 0.11 | 0.14 | 0.21 | 0.06 | 0.23 | 0.41 | 0.71 | 3.00 |
| Cut | 0.12 | 0.13 | 0.15 | 0.46 | 0.48 | 0.50 | 0.48 | 0.47 | 0.07 | 0.12 | 2.98 |

Best fit slope 0.03% →

Best fit slope 0.33% ↑

Total fill = 3.00 x 25 x 25 = 1 875 $m^3$

Fill per ha = $\frac{1\ 875}{6.25}$ = 300 $m^3$

## TABLE B.3

### Land Forming Trial II

| Grid | A | B | C | D | E | F | G | H | I | J | Average elevation (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | -0.11 | -0.04 | 0 | 0.02 | 0.05 | 0.01 | 0 | 0.05 | 0 | -0.06 | |
| 1 | 8.83 | 8.78 | 8.76 | 8.75 | 8.74 | 8.80 | 8.83 | 8.79 | 8.86 | 8.94 | 8.81 |
| | 8.72 | 8.74 | 8.76 | 8.77 | 8.79 | 8.81 | 8.83 | 8.84 | 8.86 | 8.88 | |
| | -0.07 | -0.03 | 0.02 | 0.06 | 0.09 | 0.11 | 0.07 | 0.06 | 0 | -0.11 | |
| 2 | 8.75 | 8.73 | 8.70 | 8.67 | 8.66 | 8.66 | 8.72 | 8.74 | 8.82 | 8.95 | 8.74 |
| | 8.68 | 8.70 | 8.72 | 8.73 | 8.75 | 8.77 | 8.79 | 8.80 | 8.82 | 8.84 | |
| | -0.07 | -0.04 | 0.07 | 0.08 | 0.10 | 0.10 | 0.11 | 0.02 | -0.09 | -0.17 | |
| 3 | 8.71 | 8.70 | 8.61 | 8.61 | 8.61 | 8.63 | 8.64 | 8.74 | 8.87 | 8.97 | 8.71 |
| | 8.64 | 8.66 | 8.68 | 8.69 | 8.71 | 8.73 | 8.75 | 8.76 | 8.78 | 8.80 | |
| | -0.01 | 0.03 | 0.08 | 0.06 | 0.06 | 0.03 | 0.03 | 0.02 | 0.01 | -0.17 | |
| 4 | 8.62 | 8.60 | 8.57 | 8.60 | 8.62 | 8.67 | 8.69 | 8.71 | 8.74 | 8.94 | 8.86 |
| | 8.61 | 8.63 | 8.65 | 8.66 | 8.68 | 8.70 | 8.72 | 8.73 | 8.75 | 8.77 | |
| | 0.01 | 0.04 | 0.14 | 0.10 | 0.03 | -0.01 | -0.07 | -0.07 | -0.02 | -0.13 | |
| 5 | 8.56 | 8.55 | 8.55 | 8.52 | 8.61 | 8.67 | 8.75 | 8.76 | 8.73 | 8.86 | 8.66 |
| | 8.57 | 8.59 | 8.61 | 8.62 | 8.64 | 8.66 | 8.68 | 8.69 | 8.71 | 8.73 | |
| | 0 | -0.02 | 0 | 0 | 0.01 | -0.16 | -0.10 | -0.11 | -0.05 | -0.06 | |
| 6 | 8.53 | 8.57 | 8.57 | 8.58 | 8.59 | 8.78 | 8.74 | 8.76 | 8.72 | 8.75 | 8.66 |
| | 8.53 | 8.55 | 8.57 | 8.58 | 8.60 | 8.62 | 8.64 | 8.65 | 8.67 | 8.69 | |
| | -0.10 | -0.02 | 0 | -0.05 | -0.04 | -0.13 | -0.09 | -0.02 | -0.02 | 0.01 | |
| 7 | 8.59 | 8.54 | 8.53 | 8.59 | 8.60 | 8.71 | 8.69 | 8.63 | 8.61 | 8.64 | 8.61 |
| | 8.49 | 8.51 | 8.53 | 8.54 | 8.56 | 8.58 | 8.60 | 8.61 | 8.63 | 8.65 | |
| | -0.22 | -0.08 | -0.03 | -0.01 | -0.01 | -0.01 | -0.07 | 0.02 | 0.08 | 0.04 | |
| 8 | 8.68 | 8.56 | 8.53 | 8.52 | 8.54 | 8.56 | 8.64 | 8.56 | 8.52 | 8.58 | 8.57 |
| | 8.46 | 8.48 | 8.50 | 8.51 | 8.53 | 8.55 | 8.57 | 8.58 | 8.60 | 8.62 | |
| | 0.03 | -0.02 | 0 | 0.01 | -0.02 | 0.07 | 0.12 | 0.15 | 0.15 | 0.04 | |
| 9 | 8.39 | 8.46 | 8.46 | 8.46 | 8.51 | 8.44 | 8.41 | 8.39 | 8.41 | 8.54 | 8.45 |
| | 8.42 | 8.44 | 8.46 | 8.47 | 8.49 | 8.51 | 8.53 | 8.54 | 8.56 | 8.58 | |
| | -0.05 | -0.04 | -0.06 | 0.03 | 0.07 | 0.09 | 0.11 | 0.14 | 0.10 | 0 | |
| 10 | 8.43 | 8.44 | 8.48 | 8.40 | 8.38 | 8.38 | 8.38 | 8.36 | 8.42 | 8.54 | 8.42 |
| | 8.38 | 8.40 | 8.42 | 8.43 | 8.43 | 8.47 | 8.49 | 8.50 | 8.52 | 8.54 | |
| Average elevation (m) | 8.61 | 8.59 | 8.58 | 8.57 | 8.59 | 8.63 | 8.65 | 8.64 | 8.67 | 8.77 | 8.63 |
| Fill | 0.04 | 0.07 | 0.31 | 0.36 | 0.41 | 0.41 | 0.44 | 0.46 | 0.36 | 0.09 | 2.95 |
| Cut | 0.63 | 0.30 | 0.09 | 0.06 | 0.07 | 0.31 | 0.33 | 0.20 | 0.16 | 0.70 | 2.85 |

Best fit slope 0.15% ↓

Best fit slope 0.07% ←

Total fill = 2.95 x 625 = 1 844 $m^3$

Fill per ha = $\frac{1\,844}{6.25}$ = $\underline{\underline{295\ m^3}}$

## TABLE B.4

### Land Forming Trial III

| Grid | A | B | C | D | E | F | G | H | I | J | Average elevation (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | -0.04 | -0.03 | 0.01 | 0.01 | -0.05 | -0.03 | -0.07 | -0.02 | -0.03 | -0.11 | |
| 1 | 8.43 | 8.44 | 8.42 | 8.45 | 8.53 | 8.53 | 8.59 | 8.57 | 8.60 | 8.70 | 8.53 |
| | 8.39 | 8.41 | 8.43 | 8.46 | 8.48 | 8.50 | 8.52 | 8.55 | 8.57 | 8.59 | |
| | -0.05 | -0.02 | 0.09 | 0.05 | 0.02 | 0.06 | -0.02 | 0.03 | -0.08 | -0.04 | |
| 2 | 8.46 | 8.45 | 8.36 | 8.43 | 8.48 | 8.46 | 8.56 | 8.54 | 8.67 | 8.65 | 8.51 |
| | 8.41 | 8.43 | 8.45 | 8.48 | 8.50 | 8.52 | 8.54 | 8.57 | 8.59 | 8.61 | |
| | -0.09 | 0.03 | -0.04 | 0.05 | 0.03 | -0.06 | 0.05 | 0.12 | -0.03 | -0.02 | |
| 3 | 8.51 | 8.41 | 8.50 | 8.44 | 8.48 | 8.59 | 8.50 | 8.46 | 8.63 | 8.64 | 8.52 |
| | 8.42 | 8.44 | 8.46 | 8.49 | 8.51 | 8.53 | 8.55 | 8.58 | 8.60 | 8.62 | |
| | 0.07 | 0 | -0.02 | -0.02 | 0 | 0.10 | 0.01 | -0.08 | -0.04 | -0.01 | |
| 4 | 8.36 | 8.45 | 8.49 | 8.52 | 8.52 | 8.44 | 8.55 | 8.67 | 8.65 | 8.64 | 8.53 |
| | 8.43 | 8.45 | 8.47 | 8.50 | 8.52 | 8.54 | 8.56 | 8.59 | 8.61 | 8.63 | |
| | 0.03 | 0.05 | -0.02 | 0.03 | -0.04 | -0.01 | 0.10 | 0.02 | 0 | 0 | |
| 5 | 8.41 | 8.41 | 8.50 | 8.48 | 8.57 | 8.56 | 8.47 | 8.58 | 8.62 | 8.64 | 8.52 |
| | 8.44 | 8.46 | 8.48 | 8.51 | 8.53 | 8.55 | 8.57 | 8.60 | 8.62 | 8.64 | |
| | 0.09 | 0.02 | -0.08 | -0.01 | 0.07 | -0.02 | -0.04 | 0.05 | 0 | 0 | |
| 6 | 8.37 | 8.46 | 8.58 | 8.54 | 8.48 | 8.59 | 8.63 | 8.57 | 8.64 | 8.66 | 8.55 |
| | 8.46 | 8.48 | 8.50 | 8.53 | 8.55 | 8.57 | 8.59 | 8.62 | 8.64 | 8.66 | |
| | 0.04 | 0.09 | 0.01 | 0.08 | 0.01 | 0.02 | 0.03 | 0 | 0.07 | 0.04 | |
| 7 | 8.43 | 8.40 | 8.50 | 8.46 | 8.55 | 8.56 | 8.57 | 8.63 | 8.58 | 8.63 | 8.53 |
| | 8.47 | 8.40 | 8.51 | 8.54 | 8.56 | 8.58 | 8.60 | 8.62 | 8.65 | 8.67 | |
| | -0.06 | -0.04 | -0.01 | -0.01 | 0 | -0.02 | 0.01 | 0 | -0.04 | -0.01 | |
| 8 | 8.54 | 8.54 | 8.53 | 8.56 | 8.57 | 8.61 | 8.60 | 8.64 | 8.70 | 8.69 | 8.60 |
| | 8.48 | 8.50 | 8.52 | 8.55 | 8.57 | 8.59 | 8.61 | 8.64 | 8.66 | 8.68 | |
| | -0.03 | 0 | -0.15 | -0.07 | 0 | -0.04 | 0.02 | -0.03 | 0.08 | 0.03 | |
| 9 | 8.52 | 8.51 | 8.68 | 8.63 | 8.58 | 8.64 | 8.60 | 8.68 | 8.59 | 8.66 | 8.61 |
| | 8.49 | 8.51 | 8.53 | 8.56 | 8.58 | 8.60 | 8.62 | 8.65 | 8.67 | 8.69 | |
| | -0.03 | -0.04 | 0.05 | -0.02 | -0.07 | -0.01 | -0.01 | 0.04 | 0.07 | 0.07 | |
| 10 | 8.54 | 8.57 | 8.50 | 8.60 | 8.67 | 8.63 | 8.65 | 8.63 | 8.62 | 8.64 | 8.61 |
| | 8.51 | 8.53 | 8.55 | 8.58 | 8.60 | 8.62 | 8.64 | 8.67 | 8.69 | 8.71 | |
| Average elevation | 8.46 | 8.46 | 8.51 | 8.51 | 8.54 | 8.56 | 8.57 | 8.60 | 8.63 | 8.66 | 8.55 |
| Fill | 0.23 | 0.19 | 0.16 | 0.22 | 0.13 | 0.18 | 0.22 | 0.26 | 0.22 | 0.14 | 1.95 |
| Cut | 0.30 | 0.13 | 0.32 | 0.13 | 0.16 | 0.19 | 0.14 | 0.13 | 0.22 | 0.19 | 1.91 |

Best fit slope 0.05% ↑

Best fit slope 0.09% ←

Total fill = 1.95 x 25 x 25 = 1 219 $m^3$

Fill per ha = $\frac{1\ 219}{6.25}$ = $\underline{\underline{195}}$ $m^3$

# APPENDIX C

## FEASIBILITY STUDY DETAILED CAPITAL COST ESTIMATES

# APPENDIX C

## FEASIBILITY STUDY

## DETAILED CAPITAL COST ESTIMATES

This appendix contains the detailed capital cost estimates for the Qoryooley project under the normal situation of all the water supply being from Gayweerow barrage. Each item is split up into the pilot farm, Tawakal contract and Gayweerow contract. The items are grouped into 10 separate bills, namely:-

| | |
|---|---|
| Bill 1 | Land preparation |
| Bill 2 | Earthworks |
| Bill 3 | Canal and reservoir structures |
| Bill 4 | Drain structures |
| Bill 5 | Pump stations |
| Bill 6 | Infield structures and equipment |
| Bill 7 | Surface road |
| Bill 8 | Buildings |
| Bill 9 | Services and equipment |
| Bill 10 | Engineering design and supervision |

For each bill the expected foreign exchange element is given.

## BILL 1 LAND PREPARATION

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 1.1 | Bush clearance (light to medium) | ha | 500 | - | - | 120 | 60 | 30 | 15 |
| 1.2 | Bush clearance (dense thicket) | ha | 1 150 | - | - | 20 | 23 | 10 | 12 |
| 1.3 | Tree removal | No. | 750 | - | - | 200 | 150 | 300 | 225 |
| 1.4 | Land levelling | $m^3$ | 8 | 69 000 | 552 | 1 165 000 | 9 320 | 748 000 | 5 984 |
| 1.5 | Land levelling survey | ha | 90 | 137 | 12 | 2 329.5 | 210 | 1 497 | 135 |
| 1.6 | Removal of existing structures (including Asayle canal) | No. | 5 000 | - | - | 4 | 20 | 6 | 30 |
| | Sub-total | | | | 564 | | 9 783 | | 6 407 |
| | Add 10% for contingencies | | | | 620 | | 10 761 | | 7 041 |
| | Foreign exchange (50%) | | | | 310 | | 5 381 | | 3 521 |

## BILL 2 EARTHWORKS

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 2.1 | Remodelling of Asayle canal | $m^3$ | 7.60 | - | - | - | - | - | - |
| 2.2 | Reforming of sedimentation basin on Asayle canal | $m^3$ | 7.60 | - | - | - | - | - | - |
| 2.3 | Excavation and formation of inlet channel embankments | $m^3$ | 7.60 | - | - | 43 000 | 327 | - | - |
| 2.4 | Excavation from outfall and main drain, reservoirs and borrow areas to form distributary canal embankments and associated (to drains and canals) earth roads | $m^3$ | 7.60 | 5 000 | 38 | 438 000 | 3 329 | 288 000 | 2 189 |
| 2.5 | Excavation in cut to form branch canal embankments | $m^3$ | 7.60 | - | - | 90 000 | 684 | 31 000 | 234 |
| 2.6 | Excavation from reservoirs to form associated reservoir embankments | $m^3$ | 7.60 | - | - | 148 000 | 1 125 | 160 000 | 1 216 |
| 2.7 | Excavation of surface drains to form watercourse embankment strips | $m^3$ | 7.60 | 7 100 | 54 | 86 000 | 654 | 55 000 | 418 |
| 2.8 | Formation of watercourse channels | km | 6 000 | 7.1 | 43 | 86 | 516 | 55 | 330 |
| 2.9 | Excavation for outfall drain beyond project boundary | $m^3$ | 7.60 | - | - | 60 000 | 456 | - | - |
| 2.10 | Formation of river flood bund | $m^3$ | 7.60 | - | - | - | - | 47 000 | 357 |
| | Sub-total | | | | 135 | | 7 091 | | 4 744 |
| | Add 10% for contingencies | | | | 149 | | 7 800 | | 5 218 |
| | Foreign exchange (45%) | | | | 67 | | 3 510 | | 2 348 |

## BILL 3 CANAL AND RESERVOIR STRUCTURES INCLUDING W.C. EQUIPMENT

| | Item | Unit | Rate (So.Shs. '000) | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| 3.1 | 3.10 $m^2$ vertical lift gate head regulator and road bridge deck | No. | 369 | - | - | - | - | - | - |
| 3.2 | 2.20 $m^2$ vertical lift gate head regulator and road bridge deck | No. | 317 | - | - | 1 | 317 | - | - |
| 3.3 | 1.54 $m^2$ vertical lift gate head regulator and road bridge deck | No. | 317 | - | - | - | - | 1 | 317 |
| 3.4 | 2 x 2.5 m movable weir and road bridge deck | No. | 463 | - | - | 1 | 463 | - | - |
| 3.5 | 4 m movable weir and road bridge deck | No. | 425 | - | - | 1 | 425 | - | - |
| 3.6 | 3 m movable weir and road bridge deck | No. | 318 | - | - | 1 | 318 | 2 | 636 |
| 3.7 | 2.5 m movable weir and road bridge deck | No. | 276 | - | - | 1 | 276 | - | - |
| 3.8 | 2.0 m movable weir and road bridge deck | No. | 231 | - | - | 1 | 231 | 1 | 231 |
| 3.9 | 1.6 m movable weir and road bridge deck | No. | 215 | - | - | - | - | 1 | 215 |
| Total carried forward | | | | | | | 2 030 | | 1 399 |

BILL 3 (cont.)

| Item | Unit | Rate (So.Shs. '000) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|
| Total brought forward | | | | | | 2 030 | | 1 399 |
| 3.10 4 x 1.20 m pipe regulator | No. | 553 | - | - | - | - | - | - |
| 3.11 2 x 1.20 m pipe regulator | No. | 304 | - | - | 1 | 304 | - | - |
| 3.12 2 x 1.05 m pipe regulator | No. | 245 | - | - | 3 | 735 | 4 | 980 |
| 3.13 1 x 1.20 m pipe regulator | No. | 180 | - | - | 2 | 360 | 2 | 360 |
| 3.14 1 x 1.05 m pipe regulator | No. | 145 | - | - | 2 | 290 | 4 | 580 |
| 3.15 1 x 0.90 m pipe regulator | No. | 115 | 1 | 115 | 10 | 1 150 | 4 | 460 |
| 3.16 1 x 0.75 m pipe regulator | No. | 90 | - | - | 6 | 540 | 1 | 90 |
| 3.17 1 x 0.60 m pipe regulator | No. | 71 | - | - | 2 | 142 | 3 | 213 |
| 3.18 1 x 0.45 m pipe regulator | No. | 56 | - | - | - | - | 1 | 56 |
| 3.19 100 l/s distributary outlets | No. | 13.5 | 2 | 27 | 26 | 351 | 12 | 162 |
| 3.20 60 l/s distributary outlets | No. | 11.2 | 4 | 45 | 78 | 874 | 66 | 739 |
| Total carried forward | | | | 187 | | 6 776 | | 5 039 |

BILL 3 (cont.)

| Item | Unit | Rate (So.Shs. '000) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|
| Total brought forward | | | | 187 | | 6 776 | | 5 039 |
| 3.21 Tail water escapes | No. | 32 | 1 | 32 | 13 | 416 | 8 | 256 |
| 3.22 0.90 m pipe culvert | No. | 61 | 2 | 122 | - | - | - | - |
| 3.23 6 m road bridge | No. | 316 | - | - | - | - | - | - |
| 3.24 Inverted siphon | No. | 125 | - | - | 1 | 125 | - | - |
| Sub-total | | | | 341 | | 7 317 | | 5 395 |
| Add 10% contingencies | | | | 375 | | 8 049 | | 5 825 |
| Foreign exchange (55%) | | | | 206 | | 4 427 | | 3 203 |

## BILL 4 DRAIN STRUCTURES

| | Item | Unit | Rate (So.Shs.) | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| 4.1 | 0.45 m main collector drain junction | No. | 24 | - | - | 2 | 48 | 1 | 24 |
| 4.2 | 0.60 m main collector drain junction | No. | 30 | - | - | 7 | 210 | - | - |
| 4.3 | 0.75 m main collector drain junction | No. | 38 | - | - | - | - | 3 | 114 |
| 4.4 | Surface drain junction | No. | 5.3 | - | - | 113 | 599 | 77 | 408 |
| 4.5 | 1 x 0.45 m road culvert | No. | 26 | - | - | 1 | 26 | - | - |
| 4.6 | 2 x 1.05 m road culvert | No. | 122 | - | - | 1 | 122 | - | - |
| 4.7 | 2 x 1.20 m road culvert | No. | 140 | - | - | 1 | 140 | - | - |
| 4.8 | 3 x 1.20 m road culvert | No. | 250 | - | - | 2 | 500 | - | - |
| 4.9 | Surface water escapes | No. | 35 | - | - | 7 | 245 | 9 | 315 |
| Sub-total | | | | | - | | 1 890 | | 861 |
| Add 10% for contingencies | | | | | - | | 2 079 | | 947 |
| Foreign exchange (55%) | | | | | - | | 1 143 | | 521 |

## BILL 5 PUMP STATIONS

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 5.1 | Pumps, engines and associated civil works for pump station at T1 | sum | - | - | - | - | 333 | - | - |
| 5.2 | Ditto at T7 and T9 | sum | - | - | - | - | 423 | - | - |
| 5.3 | Ditto at T11 and T13 | sum | - | - | - | - | 392 | - | - |
| 5.4 | Ditto at T2 and T4 | sum | - | - | - | - | 540 | - | - |
| 5.5 | Ditto at T2.2 | sum | - | - | - | - | 527 | - | - |
| 5.6 | Ditto at G2, G4 and G6 | sum | - | - | - | - | - | - | 545 |
| 5.7 | Ditto at G8 | sum | - | - | - | - | - | - | 392 |
| 5.8 | Ditto at G10 and G12 | sum | - | - | - | - | - | - | 545 |
| 5.9 | Drainage pump station | sum | - | - | - | - | 992 | - | - |
| | Sub-total | | | | - | | 3 207 | | 1 482 |
| | Add 10% for contingencies | | | | - | | 3 528 | | 1 630 |
| | Foreign exchange (55%) | | | | - | | 1 940 | | 897 |

## BILL 6 INFIELD STRUCTURES AND EQUIPMENT

| | Item | Unit | Rate (So.Shs.) | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| 6.1 | Watercourse fall structures for 100 l/s | No. | 6 000 | - | - | 5 | 30 | 1 | 6 |
| 6.2 | Watercourse fall structures for 60 l/s | No. | 3 600 | - | - | 16 | 58 | 16 | 58 |
| 6.3 | Watercourse splitter for 100 l/s | No. | 6 000 | - | - | 1 | 6 | - | - |
| 6.4 | Watercourse splitter for 60 l/s | No. | 3 600 | 1 | 4 | 3 | 11 | 2 | 7 |
| 6.5 | Portable watercourse checks | No. | 600 | 46 | 28 | 286 | 172 | 181 | 109 |
| 6.6 | Siphon tubing | km | 18 000 | 1.7 | 31 | 18.5 | 333 | 11.9 | 214 |
| 6.7 | Fords | No. | 2 000 | 12 | 24 | 201 | 402 | 127 | 254 |
| | Sub-total | | | | 87 | | 1 012 | | 648 |
| | Add 10% for contingencies | | | | 96 | | 1 113 | | 713 |
| | Foreign exchange (55%) | | | | 53 | | 612 | | 392 |

## BILL 7 SURFACED ROAD

| Item | Unit | Rate (So.Shs. '000) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|
| 7.1 Construction of surfaced road from Qoryooley to the Project HQ | km | 1 325 | 1.4 | 1 855 | - | - | - | - |
| Sub-total | | | | 1 855 | | - | | - |
| Add 10% for contingencies | | | | 2 041 | | - | | - |
| Foreign exchange (10%) | | | | 1 123 | | - | | - |

## BILL 8 BUILDINGS

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 8.1 | Grade one housing (200 $m^2$ each) | $m^2$ | 2 300 | 200 | 460 | - | - | - | |
| 8.2 | Grade two housing (150 $m^2$ each) | $m^2$ | 2 300 | 1 050 | 2 415 | 750 | 1 725 | 450 | 1 035 |
| 8.3 | Grade three housing (100 $m^2$ each) | $m^2$ | 2 300 | 300 | 690 | - | - | - | - |
| 8.4 | Dormitory and mess | $m^2$ | 2 300 | 400 | 920 | - | - | - | - |
| 8.5 | Project HQ offices and laboratory | $m^2$ | 2 300 | 450 | 1 035 | - | - | - | - |
| 8.6 | Storehouses, workshops and farm offices | $m^2$ | 1 800 | 270 | 486 | 550 | 990 | 330 | 594 |
| 8.7 | Storesheds and shelters | $m^2$ | 1 500 | 590 | 885 | 2 140 | 3 210 | 1 460 | 2 190 |
| 8.8 | Drying slabs and feeding yards | $m^2$ | 200 | 225 | 45 | 500 | 100 | 300 | 60 |
| 8.9 | 13 500 l fuel tanks | No. | 31 000 | 1 | 31 | 2 | 62 | - | - |
| 8.10 | 15 500 l fuel tanks | No. | 33 000 | - | - | 3 | 99 | 1 | 33 |
| 8.11 | 17 300 l fuel tanks | No. | 35 000 | - | - | - | - | 12 | 70 |
| 8.12 | Compensation for re-establishing village houses | No. | 500 | 12 | 6 | 573 | 287 | 169 | 85 |
| 8.13 | Irrigation operators' houses | $m^2$ | 1 800 | - | - | 80 | 144 | 20 | 36 |
| 8.14 | Hard standing | $m^2$ | 60 | 300 | 18 | 1 000 | 60 | 600 | 36 |
| 8.15 | Silage pits | No. | 50 000 | 1 | 50 | - | - | - | - |
| 8.16 | Maize stover silo | No. | 5 000 | 1 | 5 | - | - | - | - |
| 8.17 | Livestock crush | No. | 3 000 | 1 | 3 | 5 | 15 | 3 | 9 |
| 8.18 | Poultry housing and stores | $m^2$ | 1 700 | 156 | 281 | - | - | - | - |
| | Sub-total | | | | 7 330 | | 6 692 | | 4 148 |
| | Add 10% for contingencies | | | | 8 063 | | 7 361 | | 4 563 |
| | Foreign exchange (65%) | | | | 5 241 | | 4 785 | | 2 966 |

## BILL 9 SERVICES AND EQUIPMENT

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 9.1 | Village water supply wells | No. | 14 000 | - | - | 5 | 70 | 3 | 42 |
| 9.2 | Fencing for reservoirs, research plots and loafing yard | m | 54 | 1 900 | 103 | 6 030 | 326 | 4 410 | 238 |
| 9.3 | 130 kVA generator set and ancillary equipment | No. | 126 000 | 1 | 126 | - | - | - | - |
| 9.4 | 15 kVA generator set and ancillary equipment | No. | 26 500 | - | - | 5 | 133 | 3 | 80 |
| 9.5 | Furniture for houses, offices, dormitory | sum | - | - | 274 | - | 115 | - | 69 |
| 9.6 | Rent of housing during pilot farm construction | House months | 2 000 | 60 | 120 | - | - | - | - |
| 9.7 | VHF radios and aerials | No. | 15 000 | - | - | 13 | 195 | - | - |
| 9.8 | Office equipment and stationery | sum | - | - | 20 | - | 5 | - | 3 |
| 9.9 | Laboratory equipment | sum | - | - | 35 | - | - | - | - |
| 9.10 | Workshop tools | sum | - | - | 170 | - | 105 | - | 63 |
| 9.11 | Poultry equipment | sum | - | - | 19 | - | - | - | - |
| 9.12 | Land Rovers | No. | 55 000 | 6 | 330 | - | - | - | - |
| 9.13 | Water tanker (5 000 litre) | No. | 188 000 | 1 | 188 | | - | | - |
| 9.14 | 5 tonne lorry with hydraulic lifting arm | No. | 250 000 | - | - | 1 | 250 | - | - |
| 9.15 | Motorcycles | No. | 6 000 | 4 | 24 | 6 | 36 | 5 | 30 |
| 9.16 | Dragline (rope operated) | No. | 575 000 | - | - | 1 | 575 | - | - |
| 9.17 | Hydraulic excavator | No. | 338 000 | 1 | 338 | - | - | 1 | 338 |
| | Sub-total | | | | 1 747 | | 1 810 | | 863 |
| | Add 10% for contingencies | | | | 1 922 | | 1 991 | | 949 |
| | Foreign exchange | | | | 1 548 | | 1 487 | | 685 |

## BILL 10 ENGINEERING DESIGN AND SUPERVISION

| Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|
| 10.1 Allow 10% of the sum total of bills 1, 2, 3, 4, 5, 6, 7, 8 for the engineering design and supervision | sum | - | - | 1 134 | - | 4 070 | - | 2 594 |
| Total | | | | 1 134 | | 4 070 | | 2 594 |
| Foreign exchange (65%) | | | | 737 | | 2 646 | | 1 686 |

# APPENDIX D

## FEASIBILITY STUDY DETAILED CAPITAL COST ESTIMATES FOR THE ALTERNATIVE SUPPLY SYSTEM

# APPENDIX D

## FEASIBILITY STUDY

## DETAILED CAPITAL COST ESTIMATES

## FOR THE ALTERNATIVE SUPPLY SYSTEM

This appendix gives the detailed capital cost estimates for the Qoryooley project for the alternative case where water supply to the Tawakal canal is by means of the Asayle canal. Only those bills that vary from the total bill costs given in Appendix C are given here. These are:-

| | | |
|---|---|---|
| Bill | 1A | Land preparation (alternative) |
| Bill | 2A | Earthworks (alternative) |
| Bill | 3A | Canal and reservoir structures (alternative) |
| Bill | 5A | Pump stations (alternative) |
| Bill | 9A | Services and equipment (alternative) |
| Bill | 10A | Engineering design and supervision (alternative) |

Details of the expected foreign exchange element are given with each bill. For Bills 4, 6, 7 and 8 see Appendix C.

## BILL 1A LAND PREPARATION (ALTERNATIVE)

| Item | | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 1.1 | Bush clearance (light to medium) | ha | 500 | - | - | 120 | 60 | 30 | 15 |
| 1.2 | Bush clearance (dense thicket) | ha | 1 150 | - | - | 20 | 23 | 10 | 12 |
| 1.3 | Tree removal | No. | 750 | - | - | 200 | 150 | 300 | 225 |
| 1.4 | Land levelling | $m^3$ | 8 | 69 000 | 552 | 1 165 000 | 9 320 | 748 000 | 5 984 |
| 1.5 | Land levelling survey | ha | 90 | 137 | 12 | 2 329.5 | 210 | 1 497 | 135 |
| 1.6 | Removal of existing structures (including Asayle canal) | No. | 5 000 | - | - | 23 | 115 | 6 | 30 |
| Sub-total | | | | | 564 | | 9 878 | | 6 401 |
| Add 10% for contingencies | | | | | 620 | | 10 866 | | 7 041 |
| Foreign exchange (50%) | | | | | 310 | | 5 433 | | 3 521 |

## BILL 2A EARTHWORKS (ALTERNATIVE)

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 2.1 | Remodelling of Asayle canal | $m^3$ | 7.60 | - | - | 160 000 | 1 216 | - | - |
| 2.2 | Reforming of sedimentation basin on Asayle canal | $m^3$ | 7.60 | - | - | 19 000 | 144 | - | - |
| 2.3 | Excavation and formation of inlet channel embankments | $m^3$ | 7.60 | - | - | - | - | 43 000 | 327 |
| 2.4 | Excavation from outfall and main drain, reservoirs and borrow areas to form distributary canal embankments and associated (to drains and canals) earth roads | $m^3$ | 7.60 | 5 000 | 38 | 452 000 | 3 435 | 288 000 | 2 189 |
| 2.5 | Excavation from outfall and main drain, reservoir and borrow areas to form branch canal embankments | $m^3$ | 7.60 | - | - | 510 000 | 3 876 | 31 000 | 234 |
| 2.6 | Excavation from reservoirs to form associated reservoir embankments | $m^3$ | 7.60 | - | - | 148 000 | 1 125 | 160 000 | 1 216 |
| 2.7 | Excavation of surface drains to form watercourse embankment strips | $m^3$ | 7.60 | 7 100 | 54 | 86 000 | 654 | 55 000 | 418 |
| 2.8 | Formation of watercourse channels | km | 6 000 | 7.1 | 43 | 86 | 516 | 55 | 330 |
| 2.9 | Excavation for outfall drain beyond project boundary | $m^3$ | 7.60 | - | - | 60 000 | 456 | - | - |
| 2.10 | Formation of river flood bund | $m^3$ | 7.60 | - | - | - | - | 47 000 | 357 |
| | Sub-total | | | | 135 | | 11 422 | | 5 071 |
| | Add 10% for contingencies | | | | 149 | | 12 564 | | 5 578 |
| | Foreign exchange (45%) | | | | 67 | | 5 654 | | 2 510 |

## BILL 3A CANAL AND RESERVOIR STRUCTURES INCLUDING W.C. EQUIPMENT (ALTERNATIVE)

| | Item | Unit | Rate (So.Shs. '000) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 3.1 | 3.10 $m^2$ vertical lift gate head regulator road bridge deck | No. | 369 | - | - | 1 | 369 | - | - |
| 3.2 | 2.20 $m^2$ vertical lift gate head regulator and road bridge deck | No. | 317 | - | - | - | - | - | - |
| 3.3 | 1.54 $m^2$ vertical lift gate head regulator and road bridge deck | No. | 317 | - | - | - | - | 1 | 317 |
| 3.4 | 2 x 2.5 m movable weir and road bridge deck | No. | 463 | - | - | 1 | 463 | - | - |
| 3.5 | 4 m movable weir and road bridge deck | No. | 425 | - | - | 1 | 425 | - | - |
| 3.6 | 3 m movable weir and road bridge deck | No. | 318 | - | - | 2 | 636 | 2 | 636 |
| 3.7 | 2.5 m movable weir and road bridge deck | No. | 276 | - | - | 1 | 276 | - | - |
| 3.8 | 2.0 m movable weir and road bridge deck | No. | 231 | - | - | 2 | 462 | 1 | 231 |
| 3.9 | 1.6 m movable weir and road bridge deck | No. | 215 | - | - | 1 | 215 | 1 | 215 |
| Total carried forward | | | | | | | 2 846 | | 1 399 |

BILL 3A (cont.)

| Item | Unit | Rate (So.Shs. '000) | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|
| | | | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| Total brought forward | | | | | | 2 846 | | 1 399 |
| 3.10 4 x 1.20 m pipe regulator | No. | 553 | - | - | 2 | 1 106 | - | - |
| 3.11 2 x 1.20 m pipe regulator | No. | 304 | - | - | 1 | 304 | - | - |
| 3.12 2 x 1.05 m pipe regulator | No. | 245 | - | - | 3 | 735 | 4 | 980 |
| 3.13 1 x 1.20 m pipe regulator | No. | 180 | - | - | 2 | 360 | 2 | 360 |
| 3.14 1 x 1.05 m pipe regulator | No. | 145 | - | - | 2 | 435 | 4 | 580 |
| 3.15 1 x 0.90 m pipe regulator | No. | 115 | 1 | 115 | 10 | 1 150 | 4 | 460 |
| 3.16 1 x 0.75 m pipe regulator | No. | 90 | - | - | 6 | 540 | 1 | 90 |
| 3.17 1 x 0.60 m pipe regulator | No. | 71 | - | - | 2 | 142 | 3 | 213 |
| 3.18 1 x 0.45 m pipe regulator | No. | 56 | - | - | - | - | 1 | 56 |
| 3.19 100 l/s distributary outlets | No. | 13.5 | 2 | 27 | 26 | 351 | 12 | 162 |
| 3.20 60 l/s distributary outlets | No. | 11.2 | 4 | 45 | 92 | 1 030 | 66 | 739 |
| Total carried forward | | | | 187 | | 8 999 | | 5 038 |

BILL 3A (cont.)

| Item | Unit | Rate (So.Shs. '000) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|
| Total brought forward | | | | 187 | | 8 999 | | 5 039 |
| 3.21 Tail water escapes | No. | 32 | 1 | 32 | 10 | 320 | 8 | 256 |
| 3.22 0.90 m pipe culvert | No. | 61 | 2 | 122 | - | - | - | - |
| 3.23 6 m road bridge | No. | 316 | - | - | 2 | 632 | - | - |
| 3.24 Inverted siphon | No. | 125 | - | - | - | - | 1 | 125 |
| Sub-total | | | | 341 | | 9 951 | | 5 420 |
| Add 10% contingencies | | | | 375 | | 10 946 | | 5 962 |
| Foreign exchange (55%) | | | | 206 | | 6 020 | | 3 279 |

## BILL 5A PUMP STATIONS (ALTERNATIVE)

| | Item | Unit | Rate (So.Shs.) | Pilot farm Quantity | Pilot farm Amount ('000) | Tawakal contract Quantity | Tawakal contract Amount ('000) | Gayweerow contract Quantity | Gayweerow contract Amount ('000) |
|---|---|---|---|---|---|---|---|---|---|
| 5.1 | Pumps, engines and associated civil works for pump station at T1 | sum | - | - | - | - | 324 | - | - |
| | (Items 5.2, 5.3 and 5.4 not used) | | | | | | | | |
| 5.5 | Pumps, engines and associated civil works for pump station at T2.2 | sum | - | - | - | - | 527 | - | - |
| 5.6 | Pumps, engines and associated civil works for pump station at G2, G4, G6 | sum | - | - | - | - | - | - | 545 |
| 5.7 | Pumps, engines and associated civil works for pump station at G8 | sum | - | - | - | - | - | - | 392 |
| 5.8 | Pumps, engines and associated civil works for pump station at G10, G12 | sum | - | - | - | - | - | - | 545 |
| 5.9 | Drainage pump station | sum | - | - | - | - | 992 | - | - |
| Sub-total | | | | | - | | 1 843 | | 1 482 |
| Add 10% for contingencies | | | | | - | | 2 027 | | 1 630 |
| Foreign exchange (55%) | | | | | - | | 1 115 | | 897 |

BILL 9A SERVICES AND EQUIPMENT (ALTERNATIVE)

| Item | Unit | Rate (So.Shs.) | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|
| | | | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| Sum of all items except 9.16 brought forward from Bill 9, Appendix C | - | - | - | 1 747 | - | 1 235 | - | 863 |
| 9A.16 Dragline | No. | 575 000 | - | - | 1 | 575 | 1 | 575 |
| Sub-total | | | | 1 747 | | 1 810 | | 1 438 |
| Add 10% contingencies | | | | 1 922 | | 1 991 | | 1 582 |
| Foreign exchange | | | | 1 548 | | 1 487 | | 1 145 |

## BILL 10A ENGINEERING DESIGN AND SUPERVISION (ALTERNATIVE)

| Item | | Unit | Rate | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|---|---|
| | | | (So.Shs.) | Quantity | Amount ('000) | Quantity | Amount ('000) | Quantity | Amount ('000) |
| 10A.1 | Allow 10% of the sum total of Bills 1A, 2A, 3A, 4, 5A, 6, 7, 8, for the engineering design, documentation and supervision | sum | - | - | 1 134 | - | 4 696 | - | 2 643 |
| Total | | | | | 1 134 | | 4 696 | | 2 643 |
| Foreign exchange (65%) | | | | | 737 | | 3 052 | | 1 718 |

# APPENDIX E

## FEASIBILITY STUDY CAPITAL COST SUMMARIES

This appendix summarises the detailed capital cost estimates given in Appendices C and D. The two methods of water supply, alternative 1 with all the water coming from Gayweerow barrage, and alternative 2 which makes use of the Asayle canal to supply all the Tawakal farms, are given in parallel so that direct comparison between the two can easily be made.

| | | Pilot farm | | Tawakal contract | | Gayweerow contract | |
|---|---|---|---|---|---|---|---|
| | | Total ('000) | Foreign ('000) | Total ('000) | Foreign ('000) | Total ('000) | Foreign ('000) |
| Alternative 1 : Normal Supply | | | | | | | |
| Bill 1 | Land preparation | 620 | 310 | 10 761 | 5 381 | 7 041 | 3 521 |
| Bill 2 | Earthworks | 149 | 67 | 7 800 | 3 510 | 5 218 | 2 348 |
| Bill 3 | Canal structures | 375 | 206 | 8 049 | 4 427 | 5 825 | 3 203 |
| Bill 4 | Drain structures | - | - | 2 079 | 1 143 | 947 | 521 |
| Bill 5 | Pump stations | - | - | 3 528 | 1 940 | 1 630 | 897 |
| Bill 6 | Infield structures | 96 | 53 | 1 113 | 612 | 713 | 392 |
| Bill 7 | Surfaced road | 2 041 | 1 123 | - | - | - | - |
| Bill 8 | Buildings | 8 063 | 5 241 | 7 361 | 4 785 | 4 562 | 2 966 |
| Bill 9 | Services | 1 922 | 1 548 | 1 991 | 1 487 | 949 | 685 |
| Bill 10 | Engineering | 1 134 | 737 | 4 070 | 2 646 | 2 594 | 1 686 |
| Total | | 14 400 | 9 285 | 46 751 | 25 931 | 29 479 | 16 219 |
| TOTAL for all 3 contracts | | So.Shs. 90 630 000 | | | | | |
| Foreign exchange | | So.Shs. 51 435 000 | | | | | |
| Alternative 2 : Asayle Supply | | | | | | | |
| Bill 1A | Land preparation | 620 | 310 | 10 866 | 5 433 | 7 041 | 3 521 |
| Bill 2A | Earthworks | 149 | 67 | 12 564 | 5 654 | 5 578 | 2 510 |
| Bill 3A | Canal structures | 375 | 206 | 10 946 | 6 020 | 5 962 | 3 279 |
| Bill 4 | Drain structures | - | - | 2 079 | 1 143 | 947 | 521 |
| Bill 5A | Pump stations | - | - | 2 027 | 1 115 | 1 630 | 897 |
| Bill 6 | Infield structures | 96 | 53 | 1 113 | 612 | 713 | 392 |
| Bill 7 | Surfaced road | 2 041 | 1 123 | - | - | - | - |
| Bill 8 | Buildings | 8 063 | 5 241 | 7 361 | 4 785 | 4 562 | 2 966 |
| Bill 9A | Services | 1 922 | 1 548 | 1 991 | 1 487 | 1 582 | 1 145 |
| Bill 10A | Engineering | 1 134 | 737 | 4 696 | 3 052 | 2 643 | 1 718 |
| Total | | 14 400 | 9 285 | 53 642 | 29 301 | 30 658 | 16 949 |
| TOTAL for all 3 contracts | | So.Shs. 98 700 000 | | | | | |
| Foreign exchange | | So.Shs. 55 535 000 | | | | | |

# APPENDIX F

## FEASIBILITY STUDY ANNUAL OPERATION AND MAINTENANCE COSTS, DETAILED ESTIMATES

The detailed estimates of annual operation and maintenance charges for the Qoryooley project are given in this appendix for each of the years building up to full production. The individual items are grouped into seven categories, namely:-

| | |
|---|---|
| M1 | Earthworks |
| M2 | Structures |
| M3 | Pump stations |
| M4 | Infield structures and equipment |
| M5 | Surfaced road |
| M6 | Buildings |
| M7 | Services and equipment |

## Annual Maintenance Cost (So.Shs. x '000)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| M1 | Earthworks | | | | | | | |
| M1.1 | Hydraulic excavators | | | | | | | |
| | (a) Service and maintenance | - | 27 | 27 | 27 | 27 | 54 | 54 |
| | (b) Fuel, oil and lubricants | - | 0.8 | 0.8 | 23.9 | 31.7 | 44.9 | 58.8 |
| M1.2 | Dragline | | | | | | | |
| | (a) Service and maintenance | - | - | - | 46 | 46 | 46 | 46 |
| | (b) Fuel, oil and lubricants | - | - | - | 9.4 | 15.2 | 22.3 | 27.3 |
| M1.3 | Earth graders hire and fuel | - | 0.2 | 0.2 | 11.6 | 15.2 | 21.5 | 23.8 |
| N.B. | Clearance of watercourses, surface drains and all weed cutting charged to agricultural costs. | | | | | | | |
| | Sub-total | - | 28.0 | 28.0 | 117.9 | 135.1 | 188.7 | 209.9 |
| | Foreign exchange | - | 17.0 | 17.0 | 71.0 | 81.0 | 113.0 | 127.0 |
| M2 | Canal and Drain Structures | | | | | | | |
| M2.1 | Ramps at regulators (So.Shs. 200 per structure) | - | 0.4 | 0.4 | 5.2 | 6.8 | 10.0 | 11.8 |
| M2.2 | Structure dry inspection and repair | - | - | - | - | 3.0 | 3.0 | 5.0 |
| | Total carried forward | - | 0.4 | 0.4 | 5.2 | 9.8 | 13.0 | 16.8 |

## Annual Maintenance Costs (So.Shs x '000) (cont.)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| | Total brought forward | - | 0.4 | 0.4 | 5.2 | 9.8 | 13.0 | 16.8 |
| M2.3 | Repair of gauges | - | - | - | - | 0.3 | 0.3 | 0.5 |
| M2.4 | Daily paid labour at So.Shs. 10 per man day | - | - | - | - | 0.2 | 0.2 | 0.3 |
| M2.5 | Repair of distributary outlets | - | - | - | - | - | - | 13.5 |
| M2.6 | Sundries | - | - | - | - | 1.2 | 1.2 | 2.0 |
| | Sub-total | - | 0.4 | 0.4 | 5.2 | 11.5 | 14.7 | 33.1 |
| | Foreign exchange | - | 0.2 | 0.2 | 3.0 | 6.6 | 8.5 | 19.1 |
| M3 | Pump Stations | | | | | | | |
| M3.1 | Civil maintenance (2%) | - | - | - | 13.9 | 21.0 | 27.3 | 30.9 |
| M3.2 | Pump and engine maintenance (1%) | - | - | - | 13.9 | 21.0 | 27.3 | 30.9 |
| M3.3 | Running cost | - | - | - | 141.6 | 176.7 | 235.1 | 279.0 |
| | Sub-total | - | - | - | 169.4 | 218.7 | 289.7 | 340.8 |
| | Foreign exchange | - | - | - | 111.8 | 144.3 | 191.2 | 224.0 |

## Annual Maintenance Cost (So.Shs. x '000) (cont.)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| M4 | Infield Structures and Equipment | | | | | | | |
| M4.1 | Watercourse structure repair | - | - | - | - | - | - | 6.0 |
| M4.2 | Replacement of portable checks | - | 2.8 | 2.8 | 13.3 | 19.9 | 26.2 | 30.8 |
| M4.3 | Replacement of siphon tubing | - | 3.1 | 3.1 | 23.6 | 36.4 | 51.8 | 57.8 |
| M4.4 | Repair of fords | - | - | - | - | - | - | 10.0 |
| | Sub-total | - | 5.9 | 5.9 | 36.9 | 56.3 | 78.0 | 104.6 |
| | Foreign exchange | - | 3.2 | 3.2 | 20.3 | 31.0 | 42.9 | 57.5 |
| M5 | Surfaced road | - | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 |
| | Sub-total | - | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 |
| | Foreign exchange | - | 20.4 | 20.4 | 20.4 | 20.4 | 20.4 | 20.4 |
| M6 | Buildings (2%) | - | 161.3 | 161.3 | 249.6 | 308.5 | 369.3 | 399.7 |
| | Sub-total | - | 161.3 | 161.3 | 249.6 | 308.5 | 369.3 | 399.7 |
| | Foreign exchange | - | 104.8 | 104.8 | 162.2 | 200.5 | 240.0 | 259.8 |

## Annual Maintenance Cost (So.Shs. x '000) (cont.)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| M7 | Services and Equipment | | | | | | | |
| M7.1 | Village wells | - | - | - | 1.1 | 1.4 | 2.0 | 2.2 |
| M7.2 | Fencing | - | 2.1 | 2.1 | 7.3 | 8.6 | 11.4 | 13.3 |
| M7.3 | Electricity supply | | | | | | | |
| | (a) Plant maintenance | - | 2.5 | 2.5 | 4.1 | 5.2 | 6.2 | 6.8 |
| | (b) Fuel and oil | - | 46.2 | 46.2 | 62.1 | 72.7 | 83.3 | 88.6 |
| M7.4 | Furniture replacement | - | 13.7 | 13.7 | 17.2 | 19.5 | 21.8 | 22.9 |
| M7.5 | Office equipment and stationery | - | 9.0 | 9.0 | 10.8 | 12.0 | 13.2 | 13.8 |
| M7.6 | VHF radios and aerials | - | - | - | - | 10.0 | 10.0 | 10.0 |
| M7.7 | Laboratory equipment | - | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 | 3.5 |
| M7.8 | Workshop tools | - | 42.5 | 42.5 | 58.4 | 69.0 | 79.6 | 84.9 |
| M7.9 | Hand tools | - | - | - | - | 1.6 | 1.6 | 2.5 |
| M7.10 | Land Rovers | | | | | | | |
| | (a) Service and repairs | 22.0 | 22.0 | 27.5 | 33.0 | 33.0 | 33.0 | 33.0 |
| | (b) Fuel and oil | 17.4 | 17.4 | 21.8 | 26.1 | 26.1 | 26.1 | 26.1 |
| Total carried forward | | 39.4 | 158.9 | 168.8 | 223.6 | 262.6 | 291.7 | 307.6 |

## Annual Maintenance Cost (So.Shs. x '000) (cont.)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| | Total brought forward | 39.4 | 158.9 | 168.8 | 223.6 | 262.6 | 291.7 | 307.6 |
| M7.11 | Water tanker | | | | | | | |
| | (a) Service and repairs | - | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 |
| | (b) Fuel and oil | - | 10.4 | 10.4 | 20.9 | 20.9 | 31.3 | 31.3 |
| M7.12 | Lorry | | | | | | | |
| | (a) Service and repairs | - | - | - | 25.0 | 25.0 | 25.0 | 25.0 |
| | (b) Fuel and oil | - | - | - | 4.4 | 4.4 | 8.7 | 8.7 |
| M7.13 | Motorcycles | | | | | | | |
| | (a) Service and repairs | 0.6 | 1.2 | 1.8 | 4.2 | 6.0 | 7.8 | 9.0 |
| | (b) Fuel and oil | 1.5 | 3.0 | 4.5 | 10.5 | 15.0 | 19.5 | 22.5 |
| | Sub-total | 41.5 | 188.5 | 200.5 | 303.6 | 348.9 | 399.0 | 419.9 |
| | Foreign exchange | 27.6 | 125.5 | 133.5 | 202.1 | 232.2 | 265.6 | 279.5 |

# APPENDIX G

## FEASIBILITY STUDY, ALTERNATIVE WATER SUPPLY SYSTEM

## ANNUAL OPERATION AND MAINTENANCE COSTS

## DETAILED ESTIMATES

This appendix covers the detailed estimates of the annual operation and maintenance costs for the Qoryooley project for the alternative water supply system with feed from the Asayle canal.

Only those groups that are different from the normal supply system are listed in this appendix.

## Alternative System: Annual Maintenance Cost (So. Shs. x '000)

| | Item | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|---|
| M1A | Earthworks | | | | | | | |
| M1A.1 | Hydraulic excavators | | | | | | | |
| | (a) Service and maintenance | - | 27 | 27 | 27 | 27 | 54 | 54 |
| | (b) Fuel, oil and lubricants | - | 0.8 | 0.8 | 30.8 | 38.6 | 51.8 | 65.7 |
| M1A.2 | Dragline | | | | | | | |
| | (a) Service and maintenance | - | - | - | 46 | 46 | 92 | 92 |
| | (b) Fuel, oil and lubricants | - | - | - | 9.4 | 15.2 | 33.5 | 41.0 |
| M1A.3 | Earth grader hire and fuel | - | 0.2 | 0.2 | 11.6 | 15.2 | 21.5 | 23.8 |
| N.B. | Clearance of watercourses, surface drains and all weed cutting charged to agricultural costs. | | | | | | | |
| | Sub-total | - | 28.0 | 28.0 | 124.8 | 142.0 | 252.8 | 276.5 |
| | Foreign exchange | - | 16.8 | 16.8 | 74.9 | 85.2 | 151.7 | 165.9 |
| M3A | Pump Station | | | | | | | |
| M3A.1 | Civil maintenance (2%) | - | - | - | 8.4 | 11.9 | 18.1 | 21.8 |
| M3A.2 | Pump and engine maintenance (1%) | - | - | - | 8.4 | 11.9 | 18.1 | 21.8 |
| M3A.3 | Running cost | - | - | - | 96.2 | 109.5 | 167.9 | 211.8 |
| | Sub-total | - | - | - | 113.0 | 133.3 | 204.1 | 255.4 |
| | Foreign exchange | - | - | - | 73.5 | 86.6 | 132.7 | 166.0 |

# APPENDIX H

# FEASIBILITY STUDY ANNUAL OPERATION AND MAINTENANCE COSTS SUMMARY

## Feasibility Study Annual Operation and Maintenance Costs Summary ('000 So.Shs.)

| | Year 1 | Year 2 | Year 3 | Year 4 | Year 5 | Year 6 | Year 7 onwards |
|---|---|---|---|---|---|---|---|
| **Normal water supply system solely from Gayweerow barrage** | | | | | | | |
| 1. Earthworks | - | 28.0 | 28.0 | 117.9 | 135.1 | 188.7 | 209.9 |
| 2. Canal and drain structures | - | 0.4 | 0.4 | 5.2 | 11.5 | 14.7 | 33.1 |
| 3. Pump stations | - | - | - | 169.4 | 218.7 | 289.7 | 340.8 |
| 4. In-field structures and equipment | - | 5.9 | 5.9 | 36.9 | 56.3 | 78.0 | 104.6 |
| 5. Surfaced road | - | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 |
| 6. Buildings | - | 161.3 | 161.3 | 249.6 | 308.5 | 369.3 | 399.7 |
| 7. Services | 41.5 | 188.5 | 200.5 | 303.6 | 348.9 | 399.0 | 419.9 |
| TOTAL | 41.5 | 424.9 | 436.9 | 923.4 | 1 119.8 | 1 380.2 | 1 548.8 |
| Foreign exchange component | 27.6 | 271.1 | 279.1 | 590.8 | 716.0 | 881.6 | 987.3 |
| **Alternative water supply system with the Asayle canal feeding the Tawakal branch canal** | | | | | | | |
| 1. Earthworks | - | 28.0 | 28.0 | 124.8 | 142.0 | 252.8 | 276.5 |
| 2. Canal and drain structures | - | 0.4 | 0.4 | 5.2 | 11.5 | 14.7 | 33.1 |
| 3. Pump stations | - | - | - | 113.0 | 133.3 | 204.1 | 255.4 |
| 4. In-field structures and equipment | - | 5.9 | 5.9 | 36.9 | 56.3 | 78.0 | 104.6 |
| 5. Surfaced road | - | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 | 40.8 |
| 6. Buildings | - | 161.3 | 161.3 | 249.6 | 308.5 | 369.3 | 399.7 |
| 7. Services | 41.5 | 188.5 | 200.5 | 303.6 | 348.9 | 399.0 | 419.9 |
| TOTAL | 41.5 | 424.9 | 436.9 | 873.9 | 1 041.3 | 1 358.7 | 1 530.0 |
| Foreign exchange component | 27.6 | 271.1 | 278.9 | 556.4 | 662.5 | 861.8 | 968.2 |

# APPENDIX I

## NON-AGRICULTURAL EQUIPMENT
## REPLACEMENT SCHEDULE COSTS

Non-agricultural Equipment Replacement Schedule Costs ('000 So.Shs.)

| Year | Normal supply | Alternative supply |
|---|---|---|
| 6 | 6 | 6 |
| 7 | 344 | 344 |
| 8 | 6 | 6 |
| 9 | 937 | 663 |
| 10 | 562 | 382 |
| 11 | 729 | 729 |
| 12 | 968.5 | 968.5 |
| 13 | 6 | 6 |
| 14 | 2 065 | 1 516 |
| 15 | 782 | 422 |
| 16 | 1 039 | 1 614 |
| 17 | 966 | 966 |
| 18 | 194 | 194 |
| 19 | 772 | 498 |
| 20 | 678.5 | 498.5 |
| 21 | 674 | 674 |
| 22 | 664 | 664 |
| 23 | 6 | 6 |
| 24 | 2 065 | 1 516 |
| 25 | 1 002 | 642 |
| 26 | 1 227 | 1 802 |
| 27 | 801 | 801 |
| 28 | 311 | 311 |
| 29 | 717 | 443 |
| 30 | 374 | 194 |

# APPENDIX J

## COMPARISON OF COSTS OF SURFACE IRRIGATION AND SPRINKLER IRRIGATION

# APPENDIX J

## COMPARISON OF COSTS OF SURFACE IRRIGATION AND SPRINKLER IRRIGATION

The costs of surface irrigation systems and sprinkler systems are compared in Tables J.1 and J.2. For the surface irrigation system, costs are those used in this report, whereas the sprinkler costs are taken from the draft report 'Citrus State Farm, Lower Khalis Project, Iraq' (MMP May 1977). Annual costs have been modified in line with Somali conditions but capital costs will be broadly comparable with those in Iraq. Conditions in Iraq are similar to the Project Area with alluvial soils on a river flood plain, but the area selected for citrus has soils which are lighter in texture than those of the Study Area and the local topography is slightly more broken.

For the surface irrigation system proposed, low head pumping is required for about 90% of the area. In any sprinkler irrigation system in the Project Area, medium head pumping will be required. For the sake of comparison, the low head pumping has been included as part of the surface irrigation system as the distribution system would be re-designed to eliminate the low lift pumps. For the same reason, the cost of distributary canals has also been included as part of the surface irrigation system. This is a conservative assumption since most of the canal earthworks are taken from drain excavations. In general, only those costs have been considered which vary between the two systems.

The sprinkler system was based on a hose-pull system using permanent main pipes and laterals. One or more sprinklers are located on a small diameter flexible hose supplied from the lateral. The length of hose is restricted to about 50 m because of friction losses in the pipe. After operating in each position, the hose is pulled through to the next setting and so on, until the irrigation cycle is complete. The ideal layout is shown in Figure J.1.

The basic costing was made for a 140 ha unit but costs have been given per hectare for this study. The 140 ha unit fits in well with the system proposed for the Qoryooley project where the average area served by a distributary is about 280 ha. For comparison of costs, annual costs have been discounted at a rate of 8% over thirty years. The comparison shows that the sprinkler system costs So. Shs. 9 000 more per hectare than a surface irrigation system, and, because replacement costs for sprinkler equipment are so much higher, the discounted cost of the sprinkler system is So. Shs. 15 000 more per hectare. The major influence in favour of surface irrigation is the relatively low cost of land levelling (estimated at So. Shs. 4 647/ha), this would have to increase four fold before the economics of the sprinkler system become attractive, such an increase is not likely to occur within the Study Area.

## TABLE J.1

**Cost Estimates for Surface Irrigation System (So. Shs./ha)**

| | | | | | Net present value |
|---|---|---|---|---|---|
| 1. | Capital Costs | | | | |
| | | | | Costs | |
| | (a) | Pumps and diesel engines | | 1 237 | |
| | (b) | Watercourses and farm channels | | 485 | |
| | (c) | Land levelling | | 4 647 | |
| | (d) | Distributary canals and structures | | 3 556 | 9 925 |
| 2. | Replacement Costs | | | | |
| | | | Life (years) | NPV | |
| | (a) | Pumps and diesel engine | 15 | 390 | |
| | (b) | Watercourse maintenance So. Shs. 98/year | | 1 103 | |
| | (c) | Land levelling | 10 | 3 150 | |
| | (d) | Distributary canals maintenance So. Shs. 113.50/year | | 1 278 | 5 921 |
| 3. | Operation and Maintenance Costs | | | | |
| | | | | Annual cost | |
| | (a) | Labour, 140 manhours at So. Shs. 2.00/h | | 280 | |
| | (b) | Diesel fuel | | 62 | |
| | (c) | Pump maintenance and repair | | 279 | |
| | (d) | Farm channels | | 320 | |
| | | | | 941 | 10 594 |
| TOTAL NPV | | | | | 26 440 |

## TABLE J.2

### Cost Estimates for Sprinkler System (So. Shs./ha)

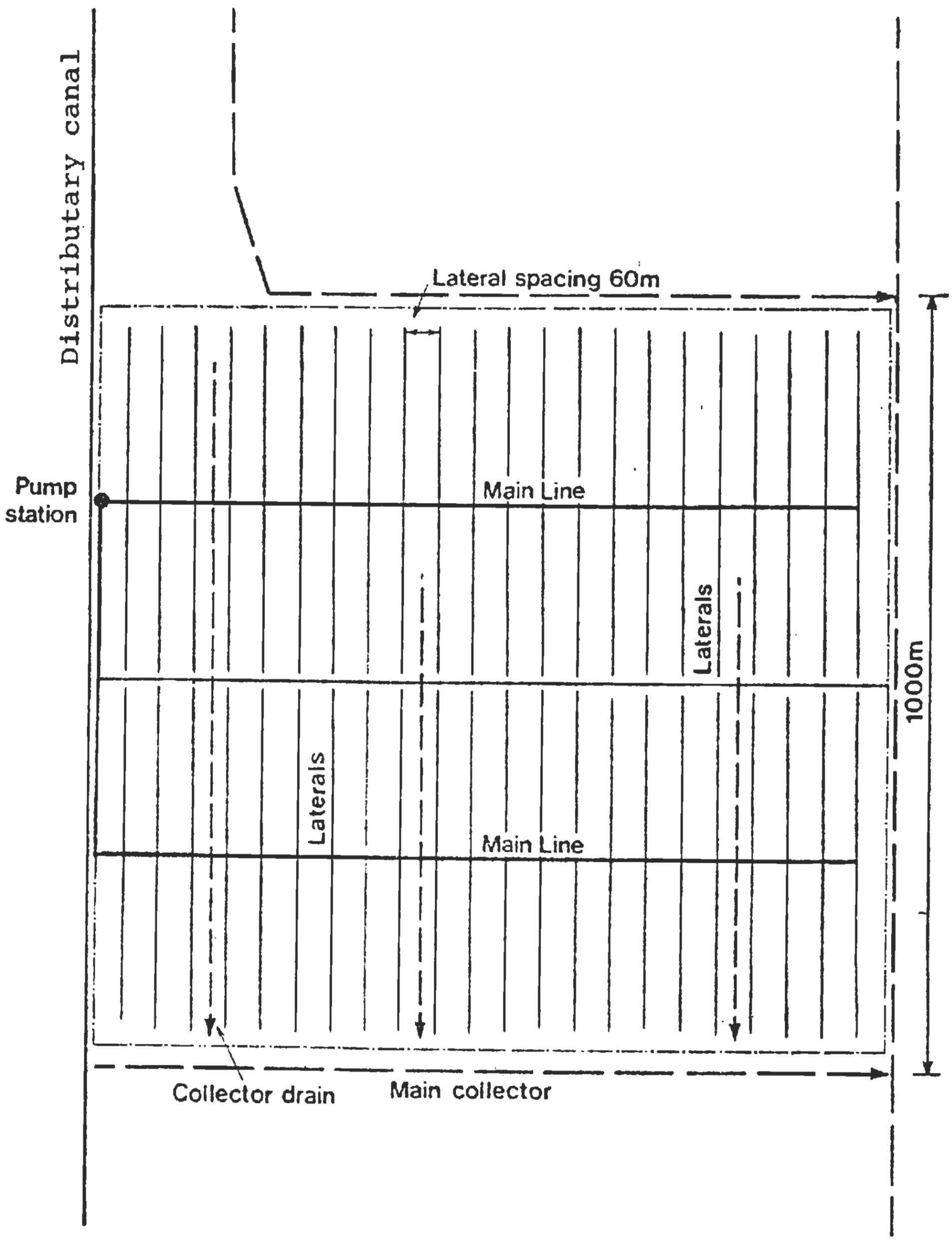
| | | | | | Net present value |
|---|---|---|---|---|---|
| 1. | Capital Costs | | | | |
| | | | | Cost | |
| | (a) | Pumps and diesel engines | | 2 070 | |
| | (b) | Buried asbestos-cement pipe | | 12 570 | |
| | (c) | Polyethylene hoses | | 3 210 | |
| | (d) | Sprinklers | | 1 070 | 18 920 |
| 2. | Replacement Costs | | | | |
| | | | Life (years) | NPV | |
| | (a) | Pumps and diesel engines | 15 | 650 | |
| | (b) | Buried asbestos-cement pipe | 35 | - | |
| | (c) | Polyethylene hoses | 5 | 5 850 | |
| | (d) | Sprinkler heads | 7 | 1 200 | 7 650 |
| 3. | Operation and Maintenance Costs | | | | |
| | | | | Annual cost | |
| | (a) | Labour, 70 manhours at So. Shs. 2.00/h | | 140 | |
| | (b) | Diesel fuel | | 750 | |
| | (c) | Pump maintenance and repair | | 80 | |
| | (d) | Sprinkler maintenance and repair | | 300 | |
| | | | | 1 270 | 14 999 |
| TOTAL cost of farm works of sprinkler system | | | | | 41 619 |

Note: Figures are modified to allow for Somali conditions.

Source: Draft Citrus State Farm Report, Lower Khalis Project, Iraq, Sir M. MacDonald & Partners, May 1977.

Figure J.1

Layout of Sprinkler System used in Cost Comparison



# APPENDIX K

# SEDIMENT CARRYING CAPACITIES OF CANALS

# APPENDIX K

## SEDIMENT CARRYING CAPACITIES OF CANALS

For the design of all new canals, the use of Lacey equations is recommended. Generally speaking, there is sufficient slope within the Study Area to permit the use of Lacey formulae and these criteria have proved extremely successful in the right conditions elsewhere. Whilst the Lacey equations are a reliable guide to the design of new canals, they give no indication of the sediment carrying capacity of canals which are designed to other parameters. The maintenance of existing canals in the Study Area is a constant problem and further work is required to obtain a theoretical understanding of the nature of this problem. This appendix is an attempt to indicate the nature and extent of the problem but the subject is so complex and so few data are available in Somalia, that it is impossible to give more than a general guide. Figure 2.4 in Annex II indicates that the median diameter of suspended sediment in the Dhamme Yaasiin canal is 0.07 mm. Research work in the United States indicates that suspended material of a particle size of less than 0.062 mm will not settle under normal conditions in canals or rivers. Studies have therefore been directed at the carrying capacities of alluvial channels for sediments greater than 0.062 mm. In the case of the Dhamme Yaasiin canal, the median particle size of the material greater than 0.062 mm is approximately 0.08 mm.

For all new work, Table K.1 shows the carrying capacity of canals for sediments of a median size of 0.1 mm. It should be emphasised that the capacity varies considerably with median particle size and is probably much less for smaller particles.

### TABLE K.1

**Typical Carrying Capacity of Alluvial Channels (t/m width/d)**

| Mean velocity (m/s) | 0.3 | 1.0 | 3.0 |
|---|---|---|---|
| Depth of flow | | | |
| 0.3 m | 1 | 100 | 720 |
| 3.0 m | 0 | 200 | 3 600 |

Notes: Figures apply to sands with median particle size of 0.1 mm.
Temperature of water 16°C

Source: Discharge of sands and mean velocity relationships in sand-bed streams. B.R. Colby, US Geological Survey.

For the Primo Secondario canal, the cross-section measured was 11.9 $m^2$, the average depth 1.6 m, and the velocity approximately 0.5 m/s (from Drawing 45 701/13 - Annex X and survey data).

By interpolation from Colby's graphical results, the carrying capacity of the Primo Secondario canal is predicted to be of the order of 15 t/m width/d. This is equivalent to a sediment concentration of about 217 ppm of material above a median particle size of 0.062 mm. To this figure, therefore, a concentration of finer particles must be added. This calculation must be treated with caution since no correction has been made for the higher water temperature in the Study Area and it is thought that the carrying capacity of heavily laden water is greater than the figures shown in Table K.1. These two corrections would tend to cancel each other out.

However, there is a more serious objection to this analysis as simple tests showed small (half a litre) sediment samples settling completely in less than four hours and water at the tail of some canals is relatively clear at times. Obviously these factors must be considered and yet there are no established research findings which are of use in this situation. There is considerable scope for research on this topic in Somalia but the subject is so complex that there is no guarantee that positive results will be obtained.

# BIBLIOGRAPHY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