Somali Democratic Republic
Ministry of Agriculture

Farahaane Irrigation Rehabilitation Project

Design Report

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CHAPTER 1

INTRODUCTION

1.1 General

This report describes the criteria adopted in the preparation of designs and tender documents for the Farahaane Irrigation Rehabilitation Project. A chapter on implementation is also included.

The report should be read in conjunction with the following documents:

a) Tender Documents (comprising 1 volume and an Album of Drawings).
b) Water Management Report.
c) Topographical and Cadastral Survey Report.

1.2 Background

The Farahaane Irrigation Rehabilitation project lies in the Lower Shebelle valley, about 100 km south west of Mogadishu. It is part of a series of rehabilitation projects which the Government is intending to invest in within the Genale - Bulo Marerta area. A feasibility study for the project is contained in the Genale Irrigation Rehabilitation Project feasibility study completed by Tippetts-Abbett-McCarthy-Stratton (TAMS) in October 1986.

The aim of the project is the rehabilitation of the Farahaane irrigation area (4 798 ha net) including the installation of a drainage system, and rehabilitation of Qorioley and Falkeerow barrages.

The objective of this design phase was to produce detailed designs and tender documents for the necessary rehabilitation works, based on the feasibility study undertaken by TAMS. The Agreement for Consultancy Services between the Ministry of Agriculture (MOA) and Sir M MacDonald & Partners (MMP) was signed on 16 September 1987, the main features of the services being:

- carry out additional surveys and investigations as necessary, including the cadastral survey of a 300 ha sample area;

- design all features of the required rehabilitation works to the Farahaane Project Area, and the Qorioley and Falkeerow barrages, and prepare tender documents suitable for international competitive bidding;

- prepare a cost estimate of the works;

- assist in tendering procedure and prepare a tender evaluation report.

Fieldwork in Somalia was carried out between September 1987 and January 1988, with the designs and draft tender documents completed in March 1988.

1.3 Existing Situation

a) Introduction

The Farahaane Project is located within the Genale-Bulo Marerta irrigation system in the lower Shebelle flood plain. The area is characterised by a fairly level topography, fine textured soils and a tropical semi-arid climate.

The project covers a gross area of some 6 250 ha between the Shebelle river to the north, the Wadajir canal to the east, the village of Madhulow and the Sisab canal to the south and the Bokore and Farahaane canals to the west. The area is currently farmed by some 2 000 families, of whom 90% have a holding less than 2 ha.

The area is commanded by the Qorioley and Gayweerow barrages. Qorioley barrage is in need of rehabilitation, but Gayweerow barrage (constructed in 1982) is in good condition. Falkeerow barrage is located downstream of the project and does not command any of the area; however it is in need of rehabilitation and is included as part of the project works.

b) Qorioley and Falkeerow Barrages

The Qorioley and Falkeerow barrages are located some 21 km and 28 km downstream of Genale respectively, and were constructed in 1955. Both structures have concrete bases, piers, abutments and bridge deck, with 9 fabricated steel lifting gates.

The concrete substructures, bases and piers are in reasonably good condition with only minor damage. Bridge decks are badly worn, and their narrow width (only about 3 m) has caused traffic to damage the gate superstructure, handrailing etc. Some spalling of concrete from the underside of the bridge decks has also occurred. The gates are all in poor condition and many are inoperable.

Downstream erosion is also a serious problem. There are no remaining protection works and large scour holes some 100 m long exist at both barrages. The downstream concrete aprons are also damaged and undermining has occurred, particularly at Falkeerow.

Further details of the barrages are given in Chapter 2.

c) Irrigation System

Water for the project area is provided by numerous canals offtaking from the left bank of the Shebelle. These include the Wadajir, Farahaane and Bokore primary canals and several smaller canals. Both the Wadajir and Bokore also serve areas to the south of the project. A network of secondary, tertiary and field channels cover the area but many are in poor condition. There are very few control or regulation structures and those that do exist are often inoperable.

The crops grown are predominantly maize in the Gu season and sesame in the Der. There are also small areas of vegetables, water melons and perennial fruit crops. Cropping intensities and yields are both low. The main irrigation method is small basins about 25 x 25 m in size. Short furrows are also used, and some large basins of 1 - 6 ha are used for sesame cultivation. The canals flow continuously but irrigation is generally only carried out during daylight and so water wastage is common. Irrigation efficiencies are very low - probably less than 20%.

The Ministry of Agriculture is responsible for maintenance of the primary and secondary canal system. Some desilting and weed clearance is carried out annually but it is hampered by poor access along the canals. The tertiary and quaternary systems are the responsibility of individual farmers and as such the degree of maintenance carried out varies from farm to farm. There is no drainage system.

d) Roads

Access into and around the project area is relatively good, although problems are experiences during the rainy seasons. The surfaced road from Genale to Qorioley runs east/west through the northern part or the project, and there are also several earth roads connecting the villages within the area.

1.4 Proposed Works

The proposed works are summarised below and described in further detail in subsequent chapters of this note. A project map is shown in Figure 1.1.

a) Qorioley and Falkeerow Barrages

The existing gates at both barrages would be replaced and the road bridges widened and strengthened. General structural improvements would be carried out together with protective works upstream and downstream.

b) Irrigation System

The irrigation system would be rehabilitated by the remodelling of existing canals, construction of some new canals and the provision of control and regulation structures. The present system of numerous small canals offtaking from the river would be replaced by the remodelled Farahaane primary canal and the new Gayweerow primary canal offtaking from Qorioley and Gayweerow barrages respectively.

The existing canal layout would be maintained as much as possible to avoid disruption to the existing well established system.

c) Drainage System

A surface drainage system would be introduced comprising in-field tertiary and quaternary drains discharging into larger secondary and primary drains. The primary drain would discharge into the existing Bokore canal through a drainage pump station of peak capacity 8.1 m^3/s.

d) Roads

Some 25 km of existing earth roads would be upgraded by surfacing with gravel. Earth access and inspection roads would be provided along all secondary and primary drains and canals.

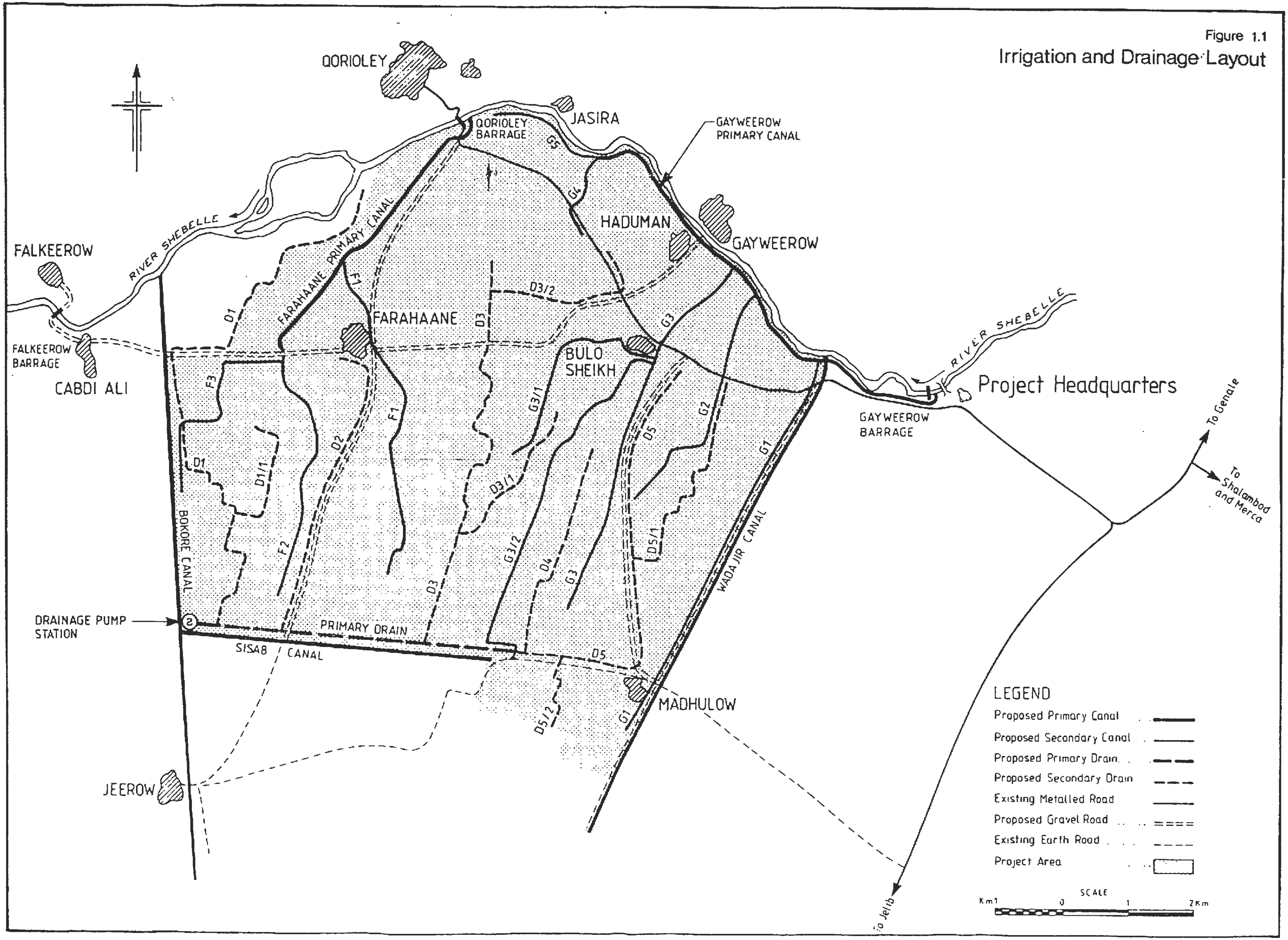
e) Buildings

Buildings and associated services (electricity, water, sewerage) would be provided for project staff at the existing MOA complex near Gayweerow barrage. The buildings to be provided comprise:-

Houses	4 Nr
Administration building	1 Nr
Workshop	1 Nr

In addition operator's quarters would be constructed at seven locations (regulator groups, pump station etc) around the project area.

Figure 1.1
Irrigation and Drainage Layout



CHAPTER 2

BARRAGES

2.1 Existing Situation

The three barrages located within the project area on the River Shebelle are the Gayweerow, Qorioley and Falkeerow barrages. The Gayweerow barrage has been completed in recent times and does not require immediate rehabilitation works. The two other barrages located further downstream have, however, suffered extensive damage to various structural elements and both have developed serious downstream erosion problems which could shortly endanger the structures if not corrected. Rehabilitation works to the Qorioley and Falkeerow barrages are described in the following sections.

The location of the barrages, referred to the upstream Genale barrage, and the corresponding river levels presently ponded upstream (Normal Retention Levels) are as follows:

	Name of Barrage			
	Genale	Gayweerow	Qorioley	Falkeerow
Distance downstream of Genale (km)	0	15.7	26.3	39.7
Normal Retention Level (NRL)	71.17	69.11	67.16	66.11

Note that distances are river kilometres, measured along the channel alignment. The NRL at Gayweerow barrage is that given in previous feasibility studies and is therefore an estimate.

2.1.1 Qorioley Barrage

The Qorioley barrage has a concrete base, piers, abutments and roadway slab. The superstructure is light steel and includes an operator's deck on the level of the gears. The bridge deck of the barrage carries heavy vehicular traffic in spite of its narrowness; the width is only 2.85 m. The length of the structure is 28.50 m. It has a downstream protective apron which is at a higher level near the abutments and deeper in the centre.

The barrage has nine openings, each of these are approximately 2.5 m wide. The gates are fabricated steel slide gates, varying in depth from 2.7 m in the outer bays to 5.5 m in the mid-channel bays. The overall condition of the gates is poor; some are corroded, others have bent spindles or problems with gearing. The steel superstructure supporting the gate guides and operating gear is damaged and bent or broken at several locations; this has been caused by the impact of vehicles using the narrow roadbridge. As a consequence it appears that at any one time no more than three gates are in an operable condition.

The concrete piers and base slab are in reasonably good condition with only minor cracks and spalling. The road curbs and upper surface of the road deck are worn and pitted. Full details of the existing structure are given on Drawing Nr 164701/82.

An erosion pool widens immediately downstream of the abutments where portions of the previous erosion protection works are still in place. The erosion protection consisted of stone masonry and gabion walls. There is also a downstream apron which extends 3 to 7 m from the toe of the piers; this has been broken back by the effects of the erosion and is presently undermined along its entire length.

The scour downstream of the barrage extends for approximately 120 metres, however the width of the scour pool is at a maximum 30 to 40 metres downstream, where the channel has been eroded to a width of 70 metres. At this point the maximum scour depth also occurs; sounding indicates that the channel bed is about 10 metres below existing bank top level. The scour pool has formed symmetrically downstream of the barrage and erosion, particularly on the right bank, has advanced back towards the structure, causing a potential problem to the approach road and water main. There is no erosion evident upstream of the barrage, although a slight shift in the channel alignment towards the right bank has occurred. Details of the scour hole can be seen on Drawing Nr 164701/82.

The subsurface materials are composed mostly of highly plastic clays; a medium dense brown clay layer is followed by a very dense, stiff grey clay which is apparently the foundation layer for the barrage.

2.1.2 Falkeerow Barrage

The Falkeerow barrage is a concrete structure with nine openings which are controlled by fabricated steel slide gates. The gates are approximately 2.5 m wide, excepting the deepest gate which is only 0.85 m wide, presumably designed to provide a scouring function. The depth of the gates varies from 3.1 m in the highest outer bay to 4.5 m in the deeper central section. The deepest gate bays are offset to the right of the structure centre-line giving rise to an assymetrical flow section. The 25 m long barrage is open to vehicular traffic; its road deck is 3.05 m wide.

The barrage has a 0.5 to 2.5 m long downstream protective apron which has an irregular, broken downstream edge. The apron level follows the upstream gate sills, and is thus at a higher elevation near the abutments, but steps down to lower levels in the centre of the river.

It has been reported that the superstructure was originally built entirely of light steel members, but sometime after completion the columns were encased in concrete. An operator's platform, made of light steel angles and channels, is located 3 m above the road deck, on the level of the gate gears.

The condition of the concrete piers, columns and base slabs is reasonably good with only minor damage evident. However, the road deck has deteriorated and extensive spalling of the concrete soffit has occurred.

The mechanical components are corroded or broken and most are inoperable. Full details are given on Drawing Nr 164701/77.

A large scour pool has formed downstream of the barrage; this extends for approximately 120 m. The erosion is more severe at this structure than at the upstream Qorioley barrage, and at its maximum extent has caused the channel to erode to a width of 90 metres and a depth of 12 metres below existing bank top level. The deviation of the river channel to the left immediately downstream of the barrage has probably contributed to the development of the scour pool, and indeed erosion is at its most severe on the right bank as would be expected. The road bridge approach on this bank is on the point of being undermined and unless corrective measures are taken, the structure is in danger of being bypassed. This is not helped by a noticeable shift in the river channel towards the right bank in the upstream reach. Details of the scour hole and river channel can be seen on Drawing Nr 164701/78. Remnants of earlier gabion and stone masonry protection works can be found on both sides of the downstream scour pool. The downstream apron has all but disappeared and the effects of erosion have exposed the thin concrete raft foundation in this region and this has been undermined over most of its length.

Subsurface materials in the area of the barrage are composed of high plasticity clays with occasional sandy layers and lenses. Borings indicate that the top few metres are composed of a medium dense brown clay with a water content above plastic limit. Below this layer is a very dense, stiff, grey clay which appears to be the foundation material for the barrage.

2.2 Approach to Remedial Works

The existing Qorioley and Falkeerow barrages do not show signs of major distress such as excessive settlement, movements, tilting, major cracks or heavily deteriorating concrete. Complete demolition and reconstruction, therefore, does not appear to be necessary. The mechanical components, gates, gears, stems and superstructures would have to be replaced since most are beyond repair or at least beyond their useful life. The existing roadbridge decks have not been designed to carry the present and ever increasing heavy traffic and wider, stronger decks are required. The aim of the proposed works is therefore to retain, repair and upgrade/modify all structural components where possible, and to build such additional works that are necessary to protect the existing structures from further effects of scour or river movement. Structural works that are necessary to improve hydraulic performance or to accommodate new lifting gates and roadbridge decks are also included in the remedial works.

The large scour pools downstream of the barrages should be stabilised to ensure further erosion does not occur, particularly in the regions that could endanger the existing or proposed structural works. There appears to be little benefit in backfilling the scour pools to a section resembling that of the existing river channel. Backfill material would have to be specially selected and compacted, and even with overlying protection works there would be no guarantee that scouring to the extent presently witnessed would not reoccur, unless, of course, extensive monitoring and corrective measures are taken on a regular basis. This approach seems expensive and impractical. Protective works downstream should aim, therefore, to contain scour to the limits of the existing scour pools and provide a high degree of protection immediately downstream of the barrages.

It should be pointed out that the original protective works failed to contain the effects of scour downstream due to a combination of ineffective design, lack of timely maintenance, and in particular through the incorrect operation of the lifting gates. It is apparent that only two or three gates are generally opened to pass river flows (perhaps because the others are inoperable); such operation gives rise to alternate fast moving and stationary areas of water in the channel downstream of the gates. This in turn sets up eddies and gives rise to a large swirling body of water across the entire channel width, with consequent highly erosive effects. Operation following rehabilitation must reduce these effects to a minimum through effective use of all gates across the entire width of the barrages.

There is evidence that floating debris causes damage at the structures. It is considered that the provision of trash racks to each bay is however unnecessary, and the piers and upstream approach works will be modified upstream to minimise this problem, and any trash that does collect will be easily and simply pulled to one side for collection and disposal.

As far as possible the rehabilitation works should use local materials and equipment already available in the country. Also, construction methods with domestic experience should be favoured.

It appears that there is local experience in the construction of concrete walls, slabs, piers, aprons and riprap protection. Coral suitable for riprap and concrete aggregate is available from a source located near the coast at a distance of about 15 to 30 km from the barrages. Because of the availability of these materials and the construction experience with them, concrete and riprap are extensively used in the proposed works. The use of sheet piling has been avoided due to the apparent unavailability of suitable equipment for their insertion.

2.3 Structural Works

The proposed remedial measures at both barrages follow similar approaches, and the works required are thus similar in most respects; small detailing differences occur to account for particular conditions at the two sites. The proposed structural works are presented on Drawing Nrs 164701/78, 79, 82 and 83.

Wider bridgedecks could be provided either by complete demolition of the present structure and the construction of a new deck, or by the widening of the existing deck, suitably repaired and stengthened. The former alternative would give rise to traffic disruption over a long period, and since the existing deck appears to be integral with the piers, demolition would not be straightforward. Widening the existing deck could only be accomplished by extending in the upstream direction; the water main at Qorioley and the structural problems associated with extending piers and adding additional load to the downstream floor preclude the possibility of extending in the downstream direction. The extension will only be possible, therefore, if the new gates are located further upstream in extensions to the existing piers; the existing gate locations being occupied by the widened bridge deck. It is important that the new gates are located a reasonable distance away from the roadbridge and that a substantial kerb and handrailing are provided; this will prevent, or at least limit, the possibility of damage to the gates by vehicular impact.

Following removal of the existing gates and associated superstructure, and the preparation of the existing piers and abutments to provide suitable bearing surfaces, it is proposed to extend the bridge decks by the addition of simply supported reinforced concrete sections. Kerbs and handrailing on the existing decks would be removed on both the upstream and downstream sides; that on the downstream face would be replaced with a more substantial arrangement, similar in all respects to the new kerb and handrailing on the upstream side. A new reinforced concrete wearing surface would be provided to the existing deck and additional support would be provided to the soffit to strengthen the slabs. For this purpose the drawings indicate two steel beams spanning longitudinally below the decks; a suggested procedure for their placement is also given. Strengthening is necessary to ensure that the decks can carry the additional concrete loading and the heavier traffic loading that is certain to occur in future years. The soffits of the existing decks would be repaired; any loose concrete would be hacked off, reinforcement cleaned down, and the face reinstated. Although these repair/remedial works are of a relatively extensive nature they have the advantage of causing the least disruption to traffic flow, which is particularly important at Qorioley.

With the addition of simply-supported extension members upstream, the rehabilitated bridge decks will have a 3.5 metre wide clear roadway and 0.5 metre wide kerbs, including handrailing, on both sides. This deck width will permit the unimpeded crossing of a reasonably large bulldozer; during construction it is possible that machines as large as a Caterpillar D7 will be employed - this has a track width of 2.55 metres and a blade width of 3.81 metres. Such a machine can negotiate the proposed deck; if larger machines, perhaps with larger blade widths are employed, this could be lifted clear of the handrailing, and since the gates are set sufficently away from the deck, the machine could pass unimpeded.

The new gates will be accommodated by reinforced concrete extensions to the existing piers upstream. The pier widths will be increased slightly to 0.7 metres and a one metre wide walkway positioned immediately upstream of the gates will give access to the gate operation gear and additionally facilitate in the removal of any trash that may collect in this region. Note that the pier extensions have been designed to minimise the problem of trash accumulation; the pier noses are well rounded and the pier projection upstream of the gates is minimal.

At Falkeerow barrage there is an existing apron extending for approximately 6 metres upstream of the piers. This appears to be of reasonably substantial construction and it will therefore be retained, however a new reinforced concrete slab will be formed over the existing apron in order to provide a good floor to the gates and spread the loading imposed from the new pier extensions and gates. The new slab will be keyed into the existing concrete foundation slab at a downstream point below the bridgedeck, and a water stop and tie bars will be cast into the slab immediately over the joint between the existing apron and main concrete structure. Any future movement of the foundation will occur at this point, and in view of the fact that water pressures will be high, the water stop will be essential to prevent seepage and thereby limit the possibility of structural damage.

The joint between the existing and new pier extensions also occurs on a similar plane to that in the floor, and although a water stop is not required here, an efficient key should nevertheless be attained to maintain structural integrity and contain the abrasive effects of fast flowing river waters immediately downstream of the gates. Ideally the existing piers should be broken back to expose a reasonable length of reinforcement for bonding the new piers, however it is considered that this operation would be time consuming and difficult to conduct, particularly in view of the fact that the location of existing reinforcement is unknown. It is proposed therefore to achieve adequate bonding by drilling the existing piers and fixing dowel bars down the full height of the pier nose; these will be cast into the new concrete works. The pier faces will, of course, be scabbled and hacked back to provide a good sound overall key. Joints are fully detailed on the design drawings.

At Qorioley barrage there is no apron to the channel bed upstream of the existing piers; a simple reinforced concrete floor slab will therefore be provided for the pier extension works. Joint details will be similar to those described above for both the floor slab and piers.

Structural works required immediately downstream of the existing barrages will aim to provide adequate protection in this region of aggressive scour conditions and thereby prevent the main structure from undermining.

These works comprise the provision of a short concrete stilling basin, with a dentated sill at its downstream limit to launch the fast-moving flows from the gates clear of the channel bed. The end of the basin will be tied into the existing scoured bed by a deep cut-off wall running the full width of the structure. A series of relief drains will be provided through the cut-off to avoid the possibility of underseepage flows building up and giving rise to high uplift pressures beneath the new basin. The floor level of the new basin will vary across the structure in order that the existing structure levels can be accommodated; these details are clearly illustrated on the drawings. Beyond, and surrounding the basin on its left and right hand flanks, in-situ masonry blockwork protection will be provided; further details are given in Section 2.5. Note that fill material below the basin and masonry blockwork areas will have to be carefully compacted; a blend of locally available sand and coral is considered most appropriate for this purpose.

At Qorioley barrage the existing apron is fairly extensive and although broken back and undermined it appears to be generally sound. It is therefore proposed to break back the apron to sound concrete and when placing the new concrete works to insert a water bar and mild steel tie bars to cover for the possibility of settlement and hence cracks developing at this location. The existing apron at Falkeerow is much shorter and comprises stone masonry overlying a concrete raft. It is proposed to remove the masonry and replace with concrete; a water stop and tie bars will be cast into the basin floor at the limit of the underlying concrete raft since this is the point where any future movement will occur.

At both structures the exsting downstream wingwalls have been extended; they have been aligned to follow the scour pool and will provide additional protection to the road approaches in this region of aggresive scour.

2.4 Barrage Gates

The barrage gates are designed to be capable of withstanding and operating against a maximum upstream water level equal to the depth of the gates with no water downstream.

Manually operated, counterbalanced, wheel and axle type lifting gates have been selected in order to achieve maximum operating speed and yet retain simplicity of construction and maintenance. Self-lubricating bearing materials will be used for the wheel bearings and elsewhere wherever possible to reduce maintenance needs to a minimum and yet to ensure continuity of operation as a first consideration.

It is estimated that each gate could be raised against the full upstream head at a speed of 12 cms/minute with one man operating the crank handle. The time required to raise the largest gates on the barrages from a fully closed to fully open position is therefore in the region of 40 minutes. The smaller gates in the outer barrage bays will take a proportionately shorter time to raise. This represents a considerable improvement over the present opening times.

2.5 Channel Protection Works

The river protection works, particularly those in the downstream reach, represent an important component of the barrage rehabilitation proposals. Two forms of protection are provided; in the regions immediately adjacent to the barrage structures masonry blockwork layed on a gravel backing has been provided, while in other areas subject to erosion 0.6 m deep stone rip-rap on a 0.3 m thick gravel backing is considered most appropriate. Details of these works are given on Drawing Nrs 164701/78 and 82.

The function of the blockwork in the downstream areas is to provide a durable and flexible mattress, at relatively low cost, that is capable of resisting the aggressive scour effects in the area immediately surrounding the stilling basin. The blocks will be formed alternately in-situ and held together with mild steel tie bars. The blocks extend almost to the lowest level in the scour pool and terminate at a mass concrete toe wall, designed to give added protection in this region and act as a buffer restraining any tendency of the blockwork to slip down the sides of the scour pool. Beyond the toe wall in the bed a small riprap launching apron has been provided, and riprap extends the full depth of the channel protecting the edge of the blockwork on the banks. Further downstream both banks are protected with riprap until the limit of the scour pool has been reached. This riprap is essentially to protect the banks against wave erosion effects; deep scour is not considered to be a problem in these regions. Some trimming of the banks will be necessary to reduce bank slopes to a minimum 1 in 2 slope.

In the upstream reaches scour is not a problem, and only nominal protection has been provided. At Falkeerow, however, there is a noticeable shift in the channel towards the right bank and this tendency has been corrected through the provision of more extensive training works upstream.

2.6 Stability Analysis

2.6.1 General

Qorioley and Falkeerow barrages operate at present under severe loading conditions and show no signs of failure due to instability, despite downstream scour which has eroded a 1:5 bed slope just downstream of both barrages. During rehabilitation of the barrages loading conditions will become more severe as the stabilising effects of the water in the downstream scour pool are removed (to enable execution of works in the stilling basin area). The stability of the existing structure will be even further compromised if, when dewatered downstream, structural additions are made, resulting in additional upstream loading. The effects on the overall stability of the structure have therefore been investigated to give an indication of the degree of risk involved during the construction phase.

Two critical scenarios have been considered for analysis:

A Barrage structure rehabilitated, downstream protection works not completed, scour hole dewatered and no flow in the river;

B Barrage structure rehabilitated, downstream protection works not completed, scour hole dewatered and water ponded against the barrage upstream to full retention level.

2.6.2 Analysis

Initial calculations show that the most probable mode of failure would be by slip of the upstream slope of the scour hole, due to the force exerted by the weight of the barrage on the top of the slope.

The analysis was carried out using the in-house computer program ISLIP. The program analyses the stability of slopes by means of the slip circle failure criterion, the analysis being based on the theory of Bishop's method of slices.

Soil investigations carried out on site during the pre-design survey show that both barrages are constructed on dense grey clay. The analysis assumed that this clay is homogeneous and that no discontinuities such as sand lenses or layers exist. In practice it is thought probable that some sand may be present but not in a position which will adversely affect the stability of the structure. Further details are given in Appendix A.

In selecting soil stress parameters for analysis such as this, it is important to be conservative. The parameters used are:

- apparent cohesion, 40 kN/m^2
- angle of internal friction, 0^o

The apparent cohesion of 40 kN/m^2 is thought to be low (and hence conservative) as the sample tested was taken from the rather less dense brown clay situated above the barrage formation material. The angle of internal friction of 0^o is also conservative for Scenario A.

Numerous runs of the program were made to optimise the location of the critical slip circle and hence predict the lowest probable factor of safety against slippage.

Assuming the structures themselves do not fail in a manner causing shearing of the foundations then the critical slip circles will commence at the upstream end of the barrage foundations. The factors of safety calculated are tabulated in Table 2.1.

Table 2.1

Factors of Safety against Slip

Scenario	Barrage	
	Qorioley	Falkeerow
A	1.59	1.47
B	1.59	1.41

The Scenario B cases are shown on Figures 2.1 and 2.2.

2.6.3 Conclusion

Under the severe conditions analysed above both barrages have a safety factor against slope failure by slip of greater than 1.4. They can thus be considered to be stable or safe.

Although the assumption of no frictional resistance will be valid for Scenario B (the upstream ponding case) there will be some frictional resistance under Scenario A conditions where pore pressures under the barrage are lower; the safety factors for Scenario A will therefore be conservative. It is important therefore to avoid ponding water upstream and preferable to delay structural rehabilitation works on the barrages until the downstream protection works have been completed.

Figure 2.1
Qorioley Barrage
Slip Circle Analysis

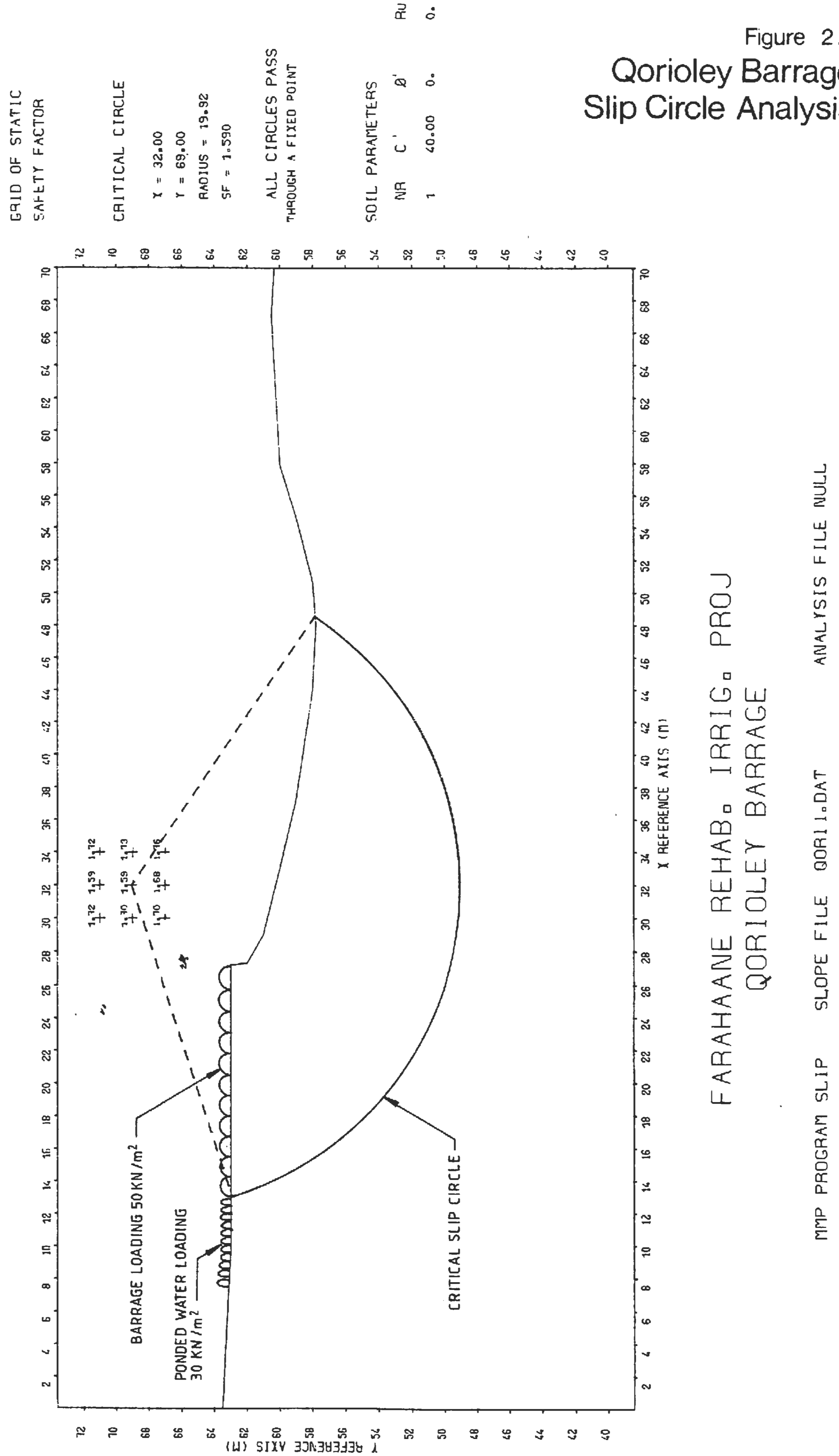
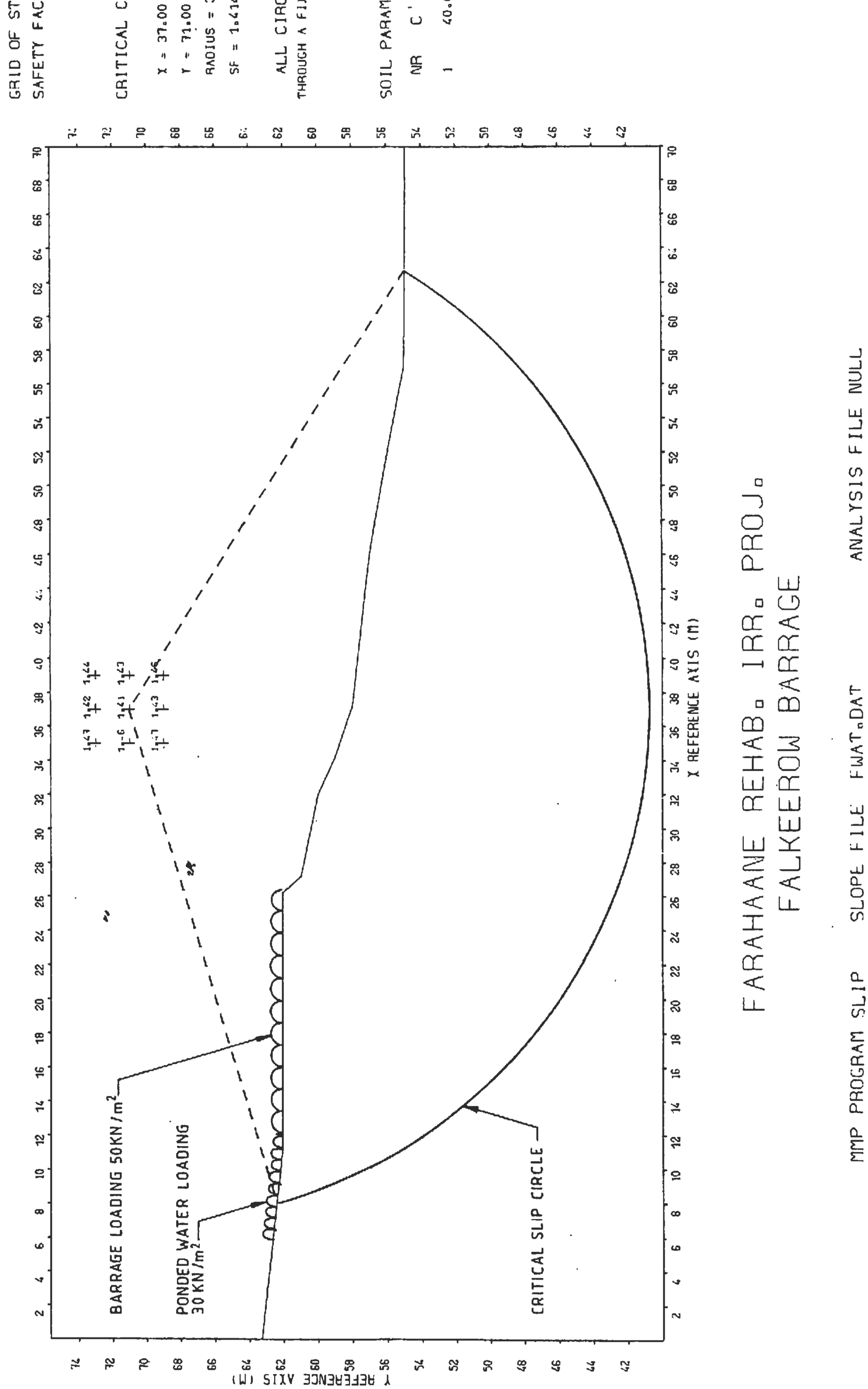


Figure 2.2
Falkeerow Barrage Slip Circle Analysis



2.7 Construction Methods and Programme

Most of the works at the barrages will have to be undertaken in the low flow season between January and March. The Contractor will therefore have to undertake a very intensive programme of works at these times, and he should be assisted by the operation of the upstream Genal and Gayweerow barrages and offtaking structures to limit the river flow downstream of Gayweerow during these periods.

All materials and components programmed for use or erection should be assembled on site before the low flow season commences so that works can proceed and be substantially complete before the end of the dry season. Works not programmed for completion in the first dry season would be carried forward to the next season. Note that a substantial proportion of the works to the bridge deck could probably be carried out in the high flow season.

The stability analysis carried out and presented in the previous section indicates that works downstream of the barrages should be completed before commencing works to the piers, gates and roadbridge deck. This will avoid the possibility of having the combined effects of additional structure loading and high ponded water levels upstream when the downstream scour pool has been dewatered; such a situation could reduce the factor af safety against slip failure. When works are undertaken initially downstream it is also important to guard against the possibility of failure by sliding. This could arise if the soil immediately below the existing concrete structure is allowed to dry out, causing the clay to shrink and lose its cohesive properties. To prevent this failure possibility it is important that, whenever the scour pool is dewatered, a means of maintaining the water table to the underside of the foundation in instigated. A possible solution would be to construct a small bund on the approximate line of the downstream cut-off and pond water upstream; the cut-off wall would then be built up to fulfil this role and coral/sand material would be infilled as work progressed.

Temporary works will be required to bund and divert river flows both upstream and downstream of the structure; the bund downstream will be particularly important to prevent back flow of water pumped from the scour pool.

The Contractor will also have to make provision for an alternative means of crossing the River Shebelle during works on the barrage roadbridge decks. It may be possible for the existing decks to be used throughout this operation, with some form of temporary deck fabricated to span those parts with resurfacing concrete in place, but not sufficiently cured. Alternatively, if the bridge deck is programmed for rehabilitation in the low flow season, a vehicular crossing of the river channel either upstream or downstream of the barrage would be a possibility; bunds used for diversion or containment of pumped waters would form useful roadways. The method adopted to achieve this temporary crossing will be the responsibility of the Contractor, and the method he proposes will be submitted at the time of tendering.

CHAPTER 3

IRRIGATION SYSTEM

3.1 General

The basic criterion adopted in planning the irrigation layout was to follow the existing major canal lines wherever possible so as to minimise earthworks and avoid disruption to the existing irrigation system. Although the existing lines may not be ideal they do tend to run along high ground and there is no reason to radically change their alignment. Some straightening of excessively tortuous sections may however be necessary.

A new primary canal (the Gayweerow primary canal) offtakes from Gayweerow barrage, and following a settling basin (see Section 3.6) runs parallel to the river for some 7.4 km to feed the central and eastern parts of the project area approx 3 000 ha net). Secondary canals offtake from the Gayweerow canal at intervals following the alignment of the existing major channels from north to south. Some of the secondary canals sub-divide to give a typical east-west canal spacing of 1 km.

The Wadajir canal is currently in very poor condition and will not be used to feed the project area. Instead a new parallel secondary canal would be constructed. The Wadajir would however still be required to feed areas outside of the project (about 2 000 ha) and an offtake of 2.2 m^3/s has been provided from the Gayweerow canal. Offtakes have also been provided to serve areas between Gayweerow barrage and the Wadajir canal which are cut off by the Gayweerow canal.

The existing Farahaane primary canal will be remodelled and provided with a settling basin to serve the western part of the project area (approx 1 800 ha net). The canal sub-divides into secondary canals, again generally following existing alignments, running from north to south and spaced at about 1 km intervals.

The existing Bokore and Sisab canals primarily feed land on their right banks outside of the Farahaane area. However, both canals do serve a narrow strip of land totalling some 250 ha in the south-western corner of the project. Due to their poor condition and high cost of remodelling this small area will be fed by other means. The land currently commanded by the Bokore will be supplied by secondary canal F3.

The land currently fed by the Sisab would instead be irrigated from the tails of the secondary canals.

Details of the primary and secondary canals are given in Table 3.1.

Table 3.1

Details of Primary and Secondary Canals

Canal	Number	Total Length (km)
Gayweerow Primary	1	7.4
Farahaane Primary	1	5.0
Secondaries	10	43.2
		55.6

Of the total canal length of 55.6 km only the Gayweerow primary canal (7.4 km) and about 10 km of secondary canals are new. All the other canals following existing alignments and will only require remodelling.

Tertiary canals offtake at intervals along the secondary and in some cases primary canals.

For convenience the project area has been divided into field units of typically 25 - 60 ha net, each field unit being served by one, or in some cases two tertiaries. Each field unit is numbered with respect to the primary and secondary canal it is fed by. The net area of each unit has also been measured (assuming a net to gross ratio of 0.80) and this information is shown on the irrigation layouts in the Album of Drawings. The tertiaries split into quaternaries serving individual farms, and a typical arrangement is shown on the sample area layouts in the Album of Drawings. The project works cover the construction of remodelling of tertiaries and quaternaries such that each farm has direct access to a tertiary or quaternary. Any field channels required to facilitate irrigation of the farm would be the responsibility of individual farmers.

The tertiary and quaternary canals follow existing channels wherever possible, so that the well established field irrigation system is charged as little as possible. It is estimated that there will be some 455 km of tertiary and quaternary canal of which 95 km will be new.

Continuous irrigation is proposed for the peak month. At other times of the year closure at night would be necessary to avoid wastage of water through the canal tail escapes. Further details are included in the accompanying Water Management Report.

3.2 Water Requirements

Individual net water requirements for various crops were estimated by TAMS (1987) using effective rainfall for a 1 in 4 dry year, These have been checked, and are presented in Table 3.2.

Table 3.2

Crop Water Requirements (mm)

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evapotranspiration (mm/d)		5.5	6.0	6.2	5.6	5.1	4.7	4.6	5.0	5.6	5.3	4.7	5.0
Effective rainfall (1)		0	0	0	29	26	35	31	16	0	3	22	2
Maize	(Gu)					15	53	107	88	25			
Legumes	(Gu & Der)	43					40	95	65	3	35	81	113
Sesame	(Der)	73									25	61	130
Watermelon	(Der)	145	26									72	143
Vegetables	(Der)	82	3							18	86	113	149
Perennials		171	167	193	138	133	106	112	140	168	160	119	153

Note (1) 75% reliability (1 in 4 dry year)

Two cropping patterns have been examined - those proposed by TAMS and the World Bank Appraisal Team (1986), as shown in Table 3.3.

Table 3.3

Cropping Patterns

Crop	TAMS Gu %	TAMS Der %	World Bank Gu %	World Bank Der %
Maize	50	-	47	-
Sesame	-	60	-	53
Legumes	15	15	15	15
Vegetables	-	5	-	5
Watermelon	-	5	-	5
Rice	-	-	5	5
	65	85	67	83

Both cropping patterns give an overall cropping intensity of 150%.

The net crop water requirements for these two cropping patterns are shown in Table 3.4.

Table 3.4

Irrigation Requirements (mm net)

Month	Cropping Pattern TAMS	Cropping Pattern World Bank
Jan	61.5	62.7
Feb	1.5	4.0
Mar	-	-
Apr	-	-
May	7.4	16.5
Jun	32.7	40.2
Jul	67.9	69.3
Aug	53.6	53.5
Sep	14.0	23.8
Oct	26.3	38.8
Nov	58.1	63.9
Dec	109.8	112.7
	432.8	485.4

3.3 Canal Discharges

The peak monthly requirement is 112.7 mm net in December (see Table 3.4). However, to allow for future changes to the cropping pattern and increases in cropping intensity the designs have been based on 100% cropping of sesame in December (130 mm net). Adopting a field application efficiency of 60% gives a peak gross field requirement of 217 mm per month which is equivalent to a continuous flow of 0.81 l/s/ha. Assuming distribution losses of 20% between the field and the tertiary canal head regulator gives a discharge at the tertiary canal head regulator of 0.97 l/s/ha, taken as 1.0 l/s/ha for design purposes.

The discharge of the secondary and primary canals has been based on the sum of the offtaking tertiary canals with an allowance for seepage losses as described in Section 3.4 below. These overall seepage losses are about 10% of the tertiary canal discharges, and thus the overall project efficiency is around 45%.

3.4 Canal Design

The primary and secondary canals have been designed using the Lacey Regime equations with a trapezoidal cross section and 1 vertical to 1.5 horizontal side slopes. The width factor has been taken as 0.75 to keep the canals relatively narrow and reduce land acquisition, with the Lacey silt factor in the range of 0.4 to 1.2 to suit ground slopes. A canal design chart is shown in Figure 3.1.

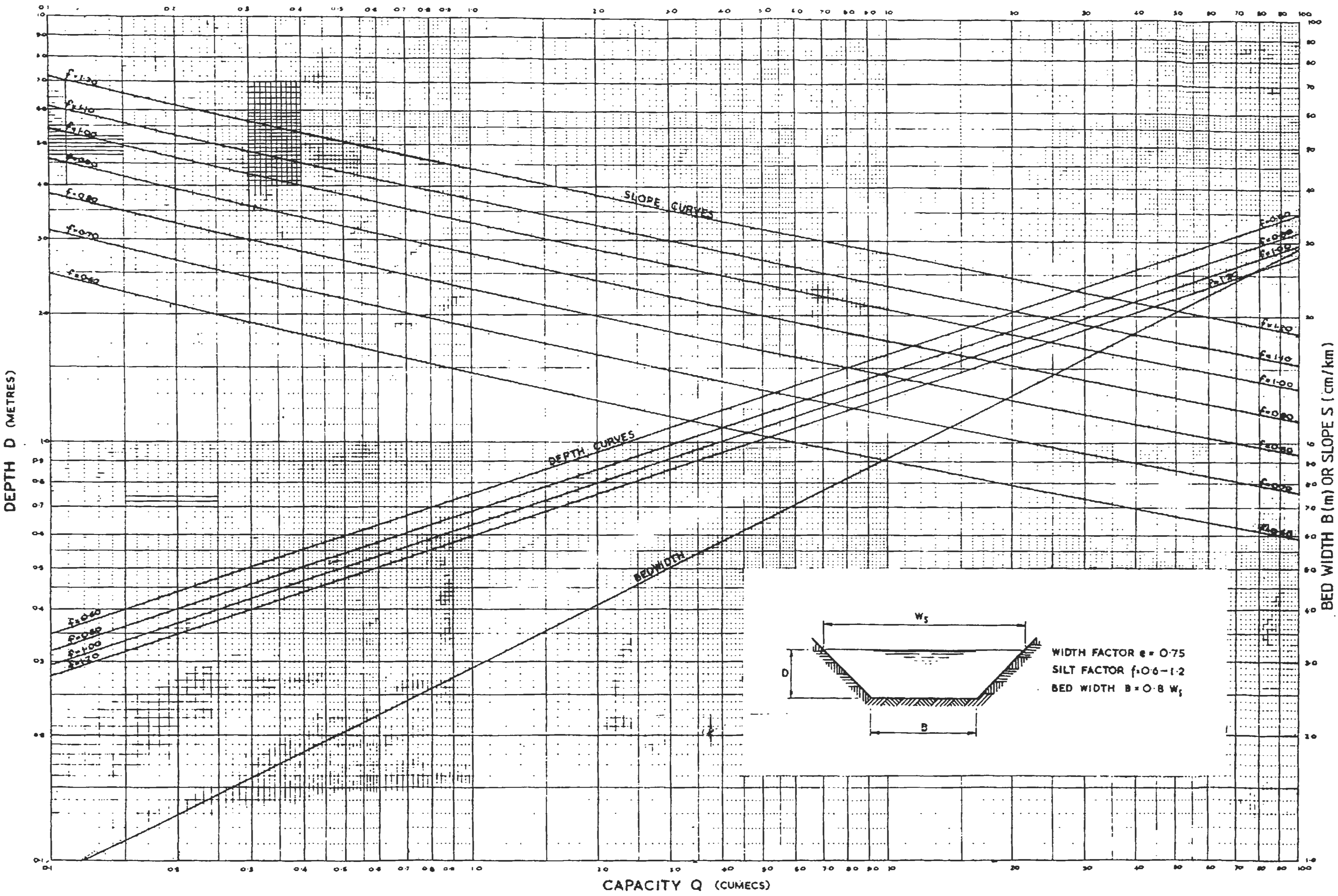
A minimum bed width of 1.0 m and depth of 0.3 m has been taken at the tail of the secondary canals.

Transit losses in the primary and secondary canals have been calculated for each reach at a rate of 1.5 m^3/s per million square metres of wetted perimeter, using the formula:-

$$\text{Losses in reach} = 0.007Q^{1/2}L$$

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

Figure 3.1
Canal Design Chart



Canal cross-section details are given in Table 3.5.

Table 3.5

Canal Cross Section Details

Canal	Bank Top width (m)	Side Slopes Inside (vertical:horizontal)	Outside	Freeboard (m)
Primary	4.0	1:1.5	varies	0.5 - 0.6 [1]
Secondary	4.0 and 1.0	1:1.5	varies	0.4 - 0.5 [1]
Tertiary and Quaternary	0.3	1:1.5	1:1.5	0.15

Note (1) Depending on discharge

The outside bank slopes for primary and secondary canals have been designed to resist a 1 in 5 seepage gradient from design water level to ground level. The bank tops that would be used as inspection roads would be given an outward camber of 1 in 40 to prevent run-off eroding the inside canal slopes.

Tertiary and quaternary canals have been designed using the Manning's equation (roughness 0.030) for a peak nominal discharge of 60 l/s with a range of allowable slopes between 0.10 and 2.0 m/km.

The minimum radius of curvature of the centre lines of canals should be 10 times the bed width.

3.5 Canal Command

The required secondary canal command at each tertiary canal head regulator is made up of several components. These comprise tertiary/quaternary canal command (0.20 m), head loss through the tertiary canal head regulator (0.25 m) and, where applicable, tertiary/quaternary canal slope of 0.10 m/km.

An additional allowance has been made to allow the tertiary head regulator to pass design discharge when the flow in the secondary canal is only half of the design flow. The figure varies with the distance upstream of the cross regulator, as shown in Table 3.7.

Table 3.7

Canal Command

Distance U/S of cross regulator (m)	Loss through tertiary canal head reg (m)	Additional loss at Q/2 (m)	Tertiary/ quaternary command (m)	Total (m)
0 - 250	0.25	0.05	0.20	0.50
250 - 750	0.25	0.10	0.20	0.55
750 - 1250	0.25	0.15	0.20	0.60
>1250	0.25	0.20	0.20	0.65

3.6 Settling Basins

3.6.1 Introduction

During the 1987 field survey, sediment investigations were carried out in order to obtain the sediment concentrations and particle size distribution of the sediment transported in the river at Gayweerow barrage and Qorioley barrage. This data was used to determine the size of the settling basins at the head of the Farahaane and Gayweerow primary canals which have been designed for mechanical clearance by either drag line or long reach hydraulic excavator in accordance with the Terms of Reference. The possibility of clearance by scour sluices has been also investigated and shown to be a feasible alternative, but is not recommended at this stage due to the additional cost, lack of data and difficulty in operating the scour mode. Further details are given in Appendix B.

3.6.2 Sediment Data

Sediment samples were taken on the 25 October 1987 and 11 November 1987. The results of the analysis are summarised in Table 3.8 and further details are given in Appendix C.

Table 3.8

River Sediment Samples

Date	Place	ppm	% of sediment > 0.063 mm
25 October 1987	Qorioley	5 482	44.7%
11 November 1987	Qorioley	2 632	0.9%
25 October 1987	Gayweerow	6 325	26.5%
11 November 1987	Gayweerow	2 469	0.0%

Sediment concentrations as measured on the 25 October 1987 were used in the design which are considered representative of Der flood sediment loads (see

Appendix C). A representative particle size of D = 0.063 mm was used, even though the actual value of the D50 particle size is smaller than this, as the canal velocities will be capable of transporting the wash load to the fields and a very large settling basin would be needed to settle out the wash load. On the 11 November 1987, the quantity of sediment in the river samples with a grain size greater than 0.063 mm was negligible.

3.6.3 Settling Basin Design

The settling basin design is based on the ASCE method (Vetter, 1940). This method gives the design ratio of flow divided by surface water area for a given particle diameter. Figure 3.2 is a plot of the design curves for this method.

The following parameters were used in the design:

T, Water Temperature	=	$20^{o}C$
D, Particle Diameter	=	0.065 mm
Basin Efficiency	=	95%

Using the design curves the design ratio for the above parameters is found to be 0.00115 m/s. The required water surface areas for the Farahaane and Gayweerow settling basins are given in Table 3.9.

Table 3.9

Design Water Surface Areas

Settling Basin	Q Design (m^3/s)	Q/AS (m/s)	Water Surface Area, AS (m^2)
Farahaane	2	0.00115	1 739
Gayweerow	5.9	0.00115	5 130

The chosen length and water surface widths of the basins are given in Table 3.9. These are determined essentially from geometrical considerations to limit the width of the basin and thereby facilitate the clearing of deposited sediment from the basin.

Table 3.9

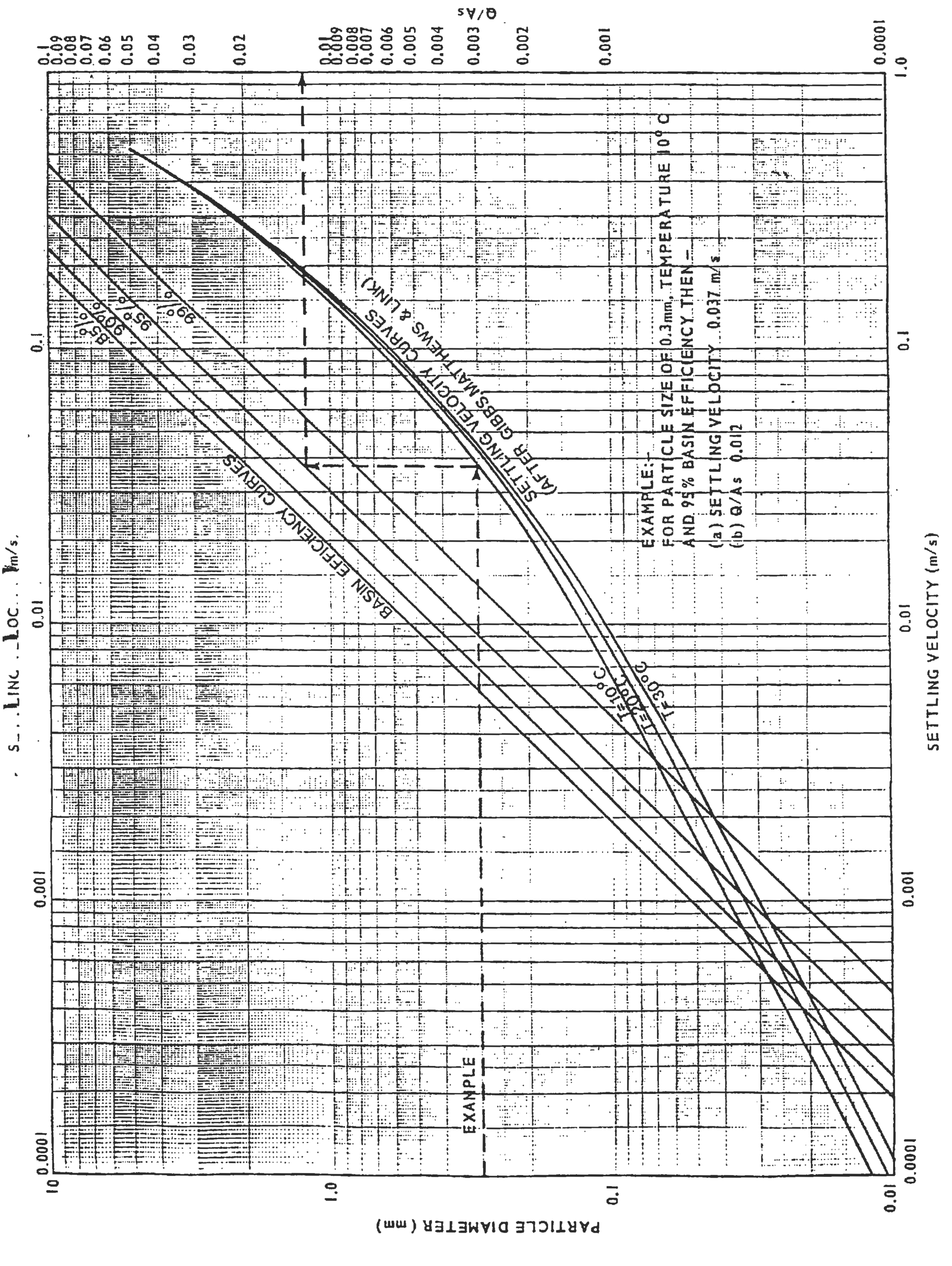
Basin Dimensions

Settling Basin	Length (m)	Water Surface Width (m)	Water Surface Area (m^2)
Farahaane	200	12	2 400
Gayweerow	300	20	6 000

The settling basins have been designed for a maximum through-flow velocity of 0.2 m/s. On the Shield's curve this is equivalent to the threshold of movement for a particle size of 0.063 mm.

SETTLING BASIN DESIGN CURVES
(Specific Gravity of Sediment = 2.65)

Figure 3.2



Using the results for the sediment samples taken on the 25 October 1987, it has been estimated that the amount of sediment settling out in a week during peak sediment concentrations in the Gayweerow and Falkeerow settling basins will be 5 700 tonnes and 2 850 tonnes respectively. Using a sediment density of 19 KN/m^3 the capacity, extra over the flow area, for storage of the sediment over a week (assuming a 24 hour flow) has been determined. This was found to be a depth of 0.97 m and 1.24 m for Farahaane and Gayweerow respectively. The depths of the settling basins have been increased by these amounts thus providing an absolute minimum of 7 days silt storage. At most times of the year however, when the silt load in the river is lower and/or irrigation is not continuous the dead storage provided will significantly exceed 7 days.

The layout of the settling basins as shown on Drawing 164701/30 will allow future extensions of the basins should changes to the system necessitate this.

3.7 Canal Structures

(a) Headworks

Gated head regulators and measuring structures have been provided at the river intakes for the Gayweerow and Farahaane primary canals. In both cases the intakes are located on the left bank of the River Shebelle immediately upstream of the existing barrages, and aligned at 120^o to the river flow to give optimum approach conditions. The head regulators are set back sufficiently from both the river and existing intake structures to obviate the need for special bunding works in the river and avoid interference to existing operational practices during construction. Structure levels are such that estimated maximum flood levels in the river can be adequately contained and prevented from entering the canal system.

At the Farahaane head the estimated peak flood of 55 m^3/s gives a water level in the river of 67.64. A 0.60 metre freeboard to structure top level has been allowed above this figure and the regulator gates have been designed with a 0.20 freeboard above this flood level. Further upstream, at the Gayweerow intake, the estimated peak flood is approximately 65 m^3/s; the new flood embankment levels upstream of the recently completed barrage are assumed to contain such a flow with a 0.60 metre freeboard, giving a flood water level of 69.40. The structure top levels have therefore been set at existing embankment top level and the gates will have a 0.20 m freeboard above the flood level.

To simplify construction, both head regulators are designed to be of mass concrete, and are essentially similar in most respects; the main difference being that at Farahaane is a two bay structure, while that at Gayweerow is three bay to account for higher discharge requirements. Each bay is equipped with a two metre wide twin-spindle geared lifting gate; the gates are identical at each structure and thus interchangable.

The gates are designed for quick and accurate setting to enable regulation of flow into the systems from design ponded levels (referred to as Normal Retention Levels) upstream of the barrages. At Falkeerow Barrage serving the Farahaane system the NRL is 67.16 while that at Gayweerow Barrage has been taken as 69.11 (MMP, 1978), although this latter figure is difficult to confirm since records are not available for the new structure. In both cases small fluctuations in the barrage pond levels will not affect the capacity of the head regulators, and in view of the uncertainty of the 69.11 level at Gayweerow, the headworks have been designed to accept the full design discharge even if the pond level falls to 68.83, (the NRL given by TAMS in their recent feasibility study). Design discharges at the Farahaane and Gayweerow heads are 2.0 m^3/s and 5.9 m^3/s respectively.

Roadbridge decks crossing the regulators have been designed to be of sufficient width to permit the passage of earthmoving equipment used in connection with routine maintenance of the settling basins. Deep cut-off walls have been provided to the upstream face of the structures to prevent damage caused by possible scour in the river channel.

Both structures have been provided with stoplog grooves upstream and downstream to enable one or more bays to be closed off should the need arise.

Separate measuring structures have been provided a short distance downstream of the head regulators. These are round-nosed, broad-crested weirs of mass concrete construction, spanning the full width of the canal. These structures have a good modular range, and the crest level has been chosen to permit accurate measurement of all discharges within the anticipated flow range. Discharge measurement curves for these measuring structures are contained in the Water Management Report (Figure A4). Dewatering pipes have been provided through the crest to simplify dewatering of the upstream reach for inspection purposes. Note that at the Farahaane head the short channel reach separating the head regulator from the measuring weir turns through 30°, and at the Gayweerow head the canal turns through 60° after the measuring weir. These directional changes are necessitated by layout considerations, and since they occur upstream of the settling basins it is important that large radii bends are adopted; Lacey's criteria has therefore been followed.

(b) Primary Canal Cross Regulator

The cross regulator on the Gayweerow primary canal at km 1.900 has been designed to provide water level control at the head of the existing Wadajir canal and new G1 secondary canal. A twin bay mass concrete structure, equipped with two metre wide twin-spindle geared lifting gates (identical to those used at the headworks) has been designed for this purpose. The structure width has been chosen such that the clear width between abutments is equal to the downstream canal bed width, this will ensure optimum hydraulic conditions and thereby minimise the possibility of erosion in the turbulent zone immediately downstream of the stilling basin. The discharge to be passed by the structure under design conditions is 2.50 m^3/s, with a corresponding head loss of 0.40 m. A reinforced concrete road bridge crossing in incorporated in the structure.

Regulation of the upstream water level will be accomplished by manual control of the gates; the water level should be maintained at the design level irrespective of the flow being passed in the canal, and a marker, visible from the gate operation platform, will be provided for this purpose. Both gates should be raised or lowered by equal amounts for regulation, however, should one gate become inoperable it is possible for the design discharge to be passed through one bay; such operational practices should only be maintained for limited periods. Cross regulation at other locations in the canal system is provided 'automatically' through the use of long-crested fixed weirs; these are feasible only where canal discharges are relatively small, and should such an arrangement be adopted at km 1.900 on the Gayweerow primary canal, a crest length in excess of 30 metres would be required to minimise water level variations over the anticipated range of discharges. The alternative gated control arrangement was therefore adopted.

Emergency stop log grooves have been provided to both gate bays at the upstream and downstream structure limits.

(c) Secondary Canal Head Regulators

These structures comprise a gated inlet section, a single or twin pipe through the canal embankment, and an outlet box designed to allow flow measurement.

The pipe inlet has been set back from the offtaking canal to permit changes in bed elevation, dictated by pipe invert setting criteria, to be accomplished without the need for excessive earthworks grades. The headwall at the inlet is from mass concrete; this provides a support for the inlet gate or gates, depending on whether the structure is of high or low capacity and thus provided with one or two pipes. The aim being to keep pipe velocities below 1 m/s and thereby reduce head losses through the structure. The concrete pipe or pipes links the inlet to the outlet box. This box is from lightly reinforced concrete and serves to dissipate the energy of the water issuing from the pipes through the use of baffle walls and then provides a broad-crsted weir measurement facility in the area of less turbulent flow beyond the baffles. The weir has a good modular range and will allow accurate flow measurement over the range of discharges anticipated; a staff gauge is provided upstream of the weir. A small stilling basin has been designed for energy dissipation purposes immediately downstream of the weir.

When each size of regulator is passing its maximum allocated design discharge, head losses of between 0.19 to 0.34 m (depending on its size) must be available for the structure to pass and give accurate measurement of the discharge. This head loss is apportioned to the pipe, downstream baffle wall and round-nosed broad-crested weir (to permit flow in the modular range).

(d) Cross Regulators

Except for the high capacity gated cross regulator at km 1.900 on the Gayweerow primary canal, all other cross regulators on primary and secondary canals will be of fixed-crest duckbill type configuration. Such an arrangement will permit automatic regulation of upstream water levels; the length of the crest being chosen to ensure that when canal flows have reduced by 50% of the design figure, the command level upstream of the structure does not fall by more than 0.05 metres. Positioned centrally in the weir crest a small lifting gate has been provided in all cases; this will enable periodic scouring of the upstream reach to be carried out and allow additional flexibility of operation. Raising the gate during canal filling operations will allow lower reaches of the canal system to be supplied without undue delays.

The design drawings indicate two structure 'Types'; Type 1 is suitable for all discharges up to 0.3 m^3/s while the longer crest provided by the Type 2 arrangement is suitable for all discharges in excess of this figure (subject to a maximum of 0.91 m^3/s). In order to maintain upstream command levels to the specified degree of accuracy, a range of crest setting levels is given on the drawings for each structure type. For both structure types head over the duck-billed weir ranges from approximately 0.07 m to 0.14 m at design discharges. These heads can be maintained, and modular conditions will prevail, when the overall head loss across the structure is at a minimum (less than 0.10 m); higher losses across the structure can, of course, also be adequately accommodated, and the stilling basin downstream of the weir is in fact designed to dissipate flows from the scour gate when it is fully opened.

Road or footbridge alternatives are available in each case. The structures are from mass concrete throughout except for the weir wall which is lightly reinforced. The scour gates are operated directly from the footbridge or from a separate operating platform in the case of the structures incorporating road bridges; this platform is easily accessible from the road bridge. The stilling basin downstream of the gate and weir is designed to provide adequate energy dissipation for a range of tailwater conditions.

Since the available head throughout the system is generally at a premium it is important that precise water level control is available at cross regulators. To achieve this it was considered preferable to provide the fixed weir arrangements shown on the drawings; thus obviating the need for daily manual adjustments and setting (which would be required in the case of gated regulators).

(e) Tertiary Canal Head Regulators

These structures are of similar overall configuration to the secondary canal head regulators, although built to a smaller scale. Inlet and outlet boxes, including the stilling basin section have been designed to be suitable for pre-cast construction (should the Contractor consider this preferable), and are 'one-size' components for use anywhere in the system. Two pipe sizes are necessary to accommodate the discharge range and for each pipe size there is a corresponding broad-crested weir profile section to be set in the outlet box. The smaller 0.225 m diameter pipe is suitable for supplying discharges up to 40 l/s, while the larger 0.3 m pipe is designed for flows up to 80 l/s. The structure can generally accommodate head losses greater than 0.25 m, but should this figure be reduced, modular flow conditions at the measuring weir may be difficult to maintain. A staff gauge, calibrated to give direct discharge measurements has been provided upstream of the measuring weir. Separate pre-cast baffle planks have been designed for insertion in the inlet and outlet boxes; the former to provide a lip to canal bed level in front of the pipe invert, while the latter serves as a baffle wall to arrest the flow before it reaches the measuring weir.

(f) Canal Box Culvert

A canal box culvert has been provided to replace the existing road bridge over the Bokore canal at km 1.1. The structure is a triple barrelled 1.8 m square reinforced concrete box culvert with pitching protection upstream and downstream. The poor condition of the Bokore canal at present limits its capacity to about 3 m^3/s. The culvert however has a maximum capacity of about 10 m^3/s to match the channel dimensions and allow for future remodelling and expansion.

(g) Canal Pipe Culvert

Canal pipe culverts have been located along primary and secondary canals to provide road crossings. The structure comprises a concrete pipe with mass concrete headwalls and pitching protection at the inlet and outlet. Culvert capacity has been based on a maximum flow velocity of 0.7 m/s with a nominal minimum head loss of 0.10 m.

This structure requires the road to be ramped up to achieve the required minimum cover of 0.9 m over the pipe. Hence it is not suitable for metalled roads, in which cases an inverted siphon has been used.

(h) Canal Inverted Siphon

This structure has been provided where canals cross metalled roads where it is undesirable to ramp the road up to any extent. It is similar to the pipe culvert, but instead of mass concrete headwalls incorporates reinforced concrete inlet and outlet boxes thus allowing the pipe to be depressed.

The 3 x 1.2 m diameter siphon at km 0.55 on the Farahaane primary canal has been provided with gates so that the structure serves the additional purpose of a cross regulator.

(i) Canal Footbridge

A footbridge has been provided on the Gayweerow primary canal at Haduman village to give access from Haduman to the ferry to Gayweerow village. The structure comprises a simple reinforced concrete deck with handrailing on one side on reinforced concrete abutments. The deck width has been set at 1.5 m to allow room for cattle etc.

(j) Canal Aqueduct

A canal aqueduct has been provided at the tail of secondary canal G3/2 to cross the primary drain and serve the area south of Madhulow.

The aqueduct pipe is of ductile iron 0.35 m dia with mass concrete inlet and outlet headwalls. A washout chamber discharging into the drain is incorporated in the aqueduct to enable the pipe to be emptied for maintenance or cleared of any silt.

(k) Secondary Canal Tail Escape

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

The structure consists of an inlet box, three sides of which act as an inlet weir, connected to a 0.45 m diameter pipe passing through the canal bank to the drain. An outlet box has been provided to dissipate energy at the pipe outlet. The discharge of the escape will be 0.35 m^3/s with a head on the weir of about 0.20 m. An aeration pipe 0.15 m diameter has been provided in the upstream end of the culvert pipe to prevent rapid fluctuations in discharge should the entrance become drowned.

(l) Tertiary and Quaternary Canal Structures

Tertiary and quaternary canal structures comprise falls, culverts and checks. An estimate of the requirements has been made from the sample area although the final quantities and locations can only be determined during the implementation stage.

Falls

Falls have been designed as simple mass concrete fixed weir structures suitable for precasting. A standard fall height of 0.50 m has been adopted, although reduced drops could be accommodated by a transition in the downstream channel.

Falls are required where the ground slope exceeds the maximum allowable channel slope of 2 m/km.

Culverts

Culverts are required to provide access for field roads over tertiary or quaternary canals. The structure comprises a 0.30 diameter concrete pipe with a mass concrete inlet and outlet box.

Checks

Checks are reqiured at major tertiary and quaternary canal divisions to control and regulate flow. The structure comprises a simple mass concrete wall forming a rectangular weir section in the channel. Flow control can be achieved either by bunding the weir section or placing a simple flashboard or gate in the slots provided.

CHAPTER 4

THE DRAINAGE SYSTEM

4.1 Introduction

There is no drainage system in the area at present and the occurrence of surface water lying in low areas for significant periods of time demonstrates the need for surface drainage.

Water table levels are generally between 5 m and 10 m below ground level, although there are some areas (particularly in Farahaane village) where the water table depth is less than 2 m. Hence there is a possible danger, in the long term, that the water table may rise into the root zone in parts of the project.

In accordance with the Terms of Reference, the drainage system provided is essentially for surface run-off only. However, modifications could be made in the future to accommodate field drainage pipes should this become economically justified.

4.2 Drainage Run-off Rate

The drainage run-off rate has been based on a 1 in 5 year 24 hours storm with a maximum of four days ponding on the fields. An analysis of the Genale rainfall records gave a design storm of 90 mm and a subsequent water balance of rainfall, infiltration and evaporation gave a design run-off rate of 1.3 l/s/ha gross.

The gross project area is about 6 250 ha, giving a total peak design run-off of 8.1 m^3/s.

4.3 Drainage System

The drainage system comprises quaternary, tertiary, secondary and primary drains. The quaternary and tertiary drains comprise the infield drainage system, and the secondary and primary drains form the main disposal system. The approximate total lengths are given below:

Table 4.1

Drain Lengths

Drain	Length (km)
Tertiary and quaternary	450.0
Secondary (10 Nr)	47.8
Primary (1 Nr)	4.9

The quaternary and tertiary drains collect run-off from the individual farms, and generally every farm has direct access to a tertiary or quaternary drain. The tertiary drains discharge into secondary drains which are generally aligned from north to south at intervals of about 1 km. The secondary drains discharge into the primary drain which runs from east to west parallel to the existing Sisab Canal. The primary drain discharges into the Bokore Canal via the drainage pumping station.

As far as possible, all drains have been aligned along farm boundaries or other features such as roads to avoid disruption to the existing irrigation system.

4.4 Drain Depths

The Terms of Reference specify that the infield drainage system (tertiary and quaternary drains) should be 1.5 m deep. This depth is well in excess of surface drainage requirements (typically 0.5 m) and will have only a limited effect on the water table after it has risen to intercept the drains. This was discussed with MOA during the field studies and in the Inception Report (MMP, 1987), but it was agreed that the infield drains would be designed at 1.5 m depth in accordance with the Terms of Reference.

The 1.5 m deep tertiary drains would not be deep enough to receive buried field drains should they become necessary at a later date. It was estimated in the Inception Report that a minimum depth of 2.0 m would be needed, and thus some remodelling of the tertiaries would be required if field drains are installed in the future.

There are two alternatives for constructing the main drainage system (primary and secondary drains). They could be constructed initially to accommodate the 1.5 m tertiary and quaternary drains with deepening at a later date should buried field drains be installed. Alternatively, they could be constructed to the full depth initially.

Table 4.2 compares the present value cost of primary and secondary drains and structures (excluding the drainage pump station) for the two options:

a) primary and secondary drains based on 1.5 m deep tertiaries, deepened after Year 15 to accommodate 2.0 m deep tertiaries;

b) primary and secondary drains based on 2.0 m deep tertiaries.

Table 4.2

Cost Comparison of Primary and Secondary Drain Depths

	Option	Cost ($ m)
a)	1.5 m depth deepened to 2.0 m after Year 15[1]	3.47
b)	2.0 m depth	3.75

Note: (1) Assuming interest rate of 7%

The cost difference is relatively small, and so obtion b) has been adopted. Thus the main drainage system will be constructed to full depth initially and will be able to accommodate 2.0 m tertiaries in the future with no remodelling.

4.5 Drain Hydraulic Design

Primary and secondary drains have been designed using Mannings equation with a roughness coefficient of 0.030. The normal minimum design water surface slope has been taken as 10 cm/km, with the maximum slope based on tractive force theory with a value of critical tractive force of 12.5 N/m^2 corresponding to stiff colloidal clays. In practice, as ground slopes are relatively flat, the maximum slope was never reached. The cross section is trapezoidal with a bed width to depth ration of 3 and a minimum bed width of 1.0 m. The design water level has been set at a minimum of 2.0 m below the lowest ground level in the adjacent field. A 4 m wide access road has been provided on one bank. This should be a minimum of 0.15 m high, increased to a maximum of 1.0 m high where excess material is to be disposed of. The road however should be reduced to the minimum height over culverts etc to reduce structure lengths. A 5 m wide reservation has been specified on the opposite bank for maintenance purposes. The side of the drain on which the access road is located should be determined on site.

The tertiary and quaternary drains will have a nominal depth of 1.5 m throughout and a minimum slope of 10 cm/km. To reduce loss of land, no drain embankments or roads have been provided.

The drain cross section details are summarised in Table 4.3

Table 4.3

Drain Cross Section Details

Drain	Inside Side Slopes	Berm Width (m)	Road Width (m)	Bed Width (m)
Primary and secondary	1:1.5	5.0 and 1.0	4.0 adjacent to 1.0 berm	1.0 min
Tertiary and quaternary	1:1.5	-	-	0.5

The minimum radius of curvature of the centre line of drains should be 10 times the bed width, although this may be reduced to 7 times bed width if space is severely limited.

4.6 Drain Structures

a) Box Culverts

A box culvert has been provided along the primary drain to provide a road crossing where the design discharge is too large for pipe culverts to be used. The structure is a triple barrel reinforced concrete box 1.6 m square with a design head loss of 0.10 m.

b) Pipe Culverts

Four types of pipe culvert have been used in the design of the main drainage system, occurring at drain junctions and road crossings. They comprise pipe culverts (single, double or triple) with different inlet and outlet arrangements to suit the discharge and head loss requirements.

Type 1 culverts are used at the junction between tertiary drains and secondary or primary drains. The structures comprise mass concrete inlet and outlet boxes linked by a concrete pipe, and serve two purposes. Firstly, the orifice type inlet box provides a throttle on the inflow to the drainage system. Secondly, the culvert is designed as an energy dissipator. The outlet box is set so as to induce an hydraulic jump within the pipe or box for the design flow, with the maximum drop between upstream and downstream drain bed level and no flow in the downstream drain.

Two pipe sizes have been specified - 0.30 m dia for tertiary drains serving less than 40 ha (corresponding to a peak flow of about 50 l/s) and 0.375 m dia for tertiaries serving from 40 - 80 ha (100 l/s peak flow). These two types are designated Types 1A and 1B respectively. A third type (Type 1C) is required where the tertiary drain is required to take secondary canal tail escape flow. In this case a 0.45 m dia Type 4 culvert (see below) is specified to accommodate the potential additional flow from the tail escape. Type 1 culverts have been located at suitable points along the primary and secondary drains. There are typically 1, 2 or 3 inlets per field units, depending on the unit size, shape and topography.

Type 2 culverts are simple road culverts comprising a pipe with pitching protection at the inlet and outlet. The pipe diameters have been determined from a maximum velocity in the pipe of 1.0 m/s. The design head loss is 0.10 m.

Culvert Types 3 and 4 are provided where head losses greater than 0.10 m are required and for junctions between primary and secondary drains. They both have mass concrete inlet boxes incorporating weirs which are set so as to avoid appreciable draw-down or backing-up in the drain upstream. The Type 3 has a depressed mass concrete outlet box, whereas the Type 4 has a baffled reinforced concrete outlet box based on the USBR recommendations (ref 'Hydraulic Design of Stilling Basins and Bucket Energy Dissipators'). The difference between the two is that the Type 4 has more dissipation of energy than the Type 3 and is used for higher discharges through and headlosses across the culvert.

A summary of primary and secondary drain culverts is given in Tables 4.4 and 4.5.

Table 4.4

Road Culverts

Discharge (Cumecs)	Head Loss through Culvert (m)		
	0.10	0.11 - 0.20	0.21 - 6.0
≤3.39	TYPE 2	TYPE 3	TYPE 4
>3.39	BOX	BOX	TYPE 4

Table 4.5

Junction Culverts

Discharge (Cumecs)	Head Loss through Culvert (m)	
	0.10 - 0.20	0.21 - 6.00
≤3.39	TYPE 3	TYPE 4
>3.39	BOX	TYPE 4

c) Drain Underpass

A drain underpass has been provided where secondary drain D1 passes underneath secondary canal F3. The structure is essentially an extended Type 2 culvert. Canal lining membrane has been provided upstream and downstream of the underpass to prevent excessive seepage gradients from the canal into the drain.

The underpass would interrupt access along the drain and hence a pipe culvert has been provided in the canal to carry the drain access road.

d) Tertiary and Quaternary Drain Culverts

These are designed to give access across the tertiary and quaternary drains. The structure is a simple pipe culvert of diameter 0.30 m for drains serving areas less than 40 ha, and 0.375 m for drains serving areas of 40 - 80 ha. The culverts will be sited to suit existing tracks and access routes and hence, although an estimate of the overall requirements has been made from the sample area, the exact numbers and locations can only be determined during construction.

4.7 Drainage Pumping Station

The pump station pumps water from the primary drain into the Bokore canal. Three 2.7 cumec vertical axial or mixed flow pumps have been selected to deliver the design discharge of 8.1 cumecs against a maximum static lift of 8 m. A fourth standby pump has been provided.

The pumps are identical to facilitate the procurement of spare parts and maintenance, and can be operated at variable speeds to accommodate a wide range of drain discharge.

Each pump is powered by a 280 kw diesel engine which has a clutch to disengage the drive to the pump, enabling the engines to be run and maintained without pumping during the dry season. To ensure that the batteries need to start the diesel engines are kept fully charged during the dry season when pumping is not required, a solar powered battery charging system has been specified. Also the batteries will be connected together so that one engine can be jump started from another.

The pumps would be started and primed manually but mercury float switches have been provided so that they are turned off automatically when the water level drops below the design minimum.

Four skid fuel tanks and two daily tanks contain fuel for 14 hours pumping for each engine, and a 30 m^3 bulk storage tank located about 40 m from the pump station would enable three engines to be operated for an additional 140 hours. The bulk storage tank has been located above ground with an adjacent ramp to enable gravity filling from diesel tankers. The fuel would be pumped using rotary hand pumps into the day tanks from where it would gravity feed to the skid tanks and the engines.

The discharge pipework from each pump is separate. This avoids having valves which would deteriorate if not regularly used.

A hand operated over-head travelling crane has been specified to enable the diesel engines or the pumps to be removed onto the pumphouse deck slab for maintenance. A corrugated roof has been provided for protection against rain and sun. Steel gantry girders and stanchions support the crane and the roof.

The pump station inlet and sump have been designed to minimise turbulence and prevent vortexes by providing splitters to divide each bay and including fillets behind each intake. Also the average flow velocities in the sump have been kept low at 0.2 m/s.

The sump floor level has been set three metres below the design minimum pumping level to ensure that the minimum submergence requirements of the pumps are satisfied.

Other features of the pump station are the inclined trash rack to prevent debris entering the sump, the two metre wide walkway raking platform and two access hatches to permit entry to the sump.

The reinforced concrete elements of the pump station have been designed to BS 8110 and using the design criteria set out in Chapter 7. The worst combination of earth and hydrostatic pressures, dead and imposed loads have been considered for design. The overall stability of the pump station has been checked; it was assumed that the movement joint between the wingwall and the main structure helped restrain the wingwall against overturning and sliding, and the factor of safety against rotation was calculated. To minimise concrete attack by the salts in the drainage water and the soil, sulphate resisting cement has been specified.

4.8 Disposal of Drainage Water

The drainage water will be pumped into the existing Bokore canal, and therefore it is necessary to ensure that:

- the quality of the drainage water will not be injurious to downstream users;
- the Bokore canal has sufficient capacity to accommodate the drainage water.

a) Quality Considerations

To check on the suitability of the drainage water for irrigation downstream of the pump station, it is necessary to carry out a monthly salinity balance. Three cases have been considered:

(i) Initial - the period immediately after commissioning of the project. Generally the water table will be well below the drain bed levels at this time and the only water entering the drainage system will be surface run-off.

(ii) Intermediate - the period when the water table has risen and is just intercepted by the drains. It is estimated that about 0.3 mm/day would be intercepted by the drains, in addition to the surface run-off.

(iii) Final - the final (steady state) situation where buried field drains are installed and the water table has risen to such an extent that all water lost to deep percolation is intercepted by the drainage system.

For the initial case where virtually all the water entering the drainage system comes from surface run-off, the quality will be similar to that of the river water and will be suitable for re-use. Estimates of the drainage water quality and quantity for the 'intermediate' and 'final' conditions are shown in Table 4.6.

Table 4.6

Drainage Water Salinity

Month	River Salinity (dS/m)[1]	Drainage Water			
		INTERMEDIATE		FINAL	
		Salinity (dS/m)	Flow (m^3/s)	Salinity (dS/m)	Flow[2] (m^3/s)
Jan	0.97	1.66	0.35	2.19	1.13
Feb	1.18	1.18	0.01	2.71	0.03
Mar	0.95	-	-	-	-
Apr	0.92	0.67	0.10	0.71	0.10
May	1.19	2.20	0.23	2.20	0.23
Jun	0.96	1.50	0.33	1.96	0.67
Jul	0.81	1.17	0.42	1.73	1.31
Aug	0.54	0.89	0.35	1.19	1.01
Sep	0.42	0.92	0.21	0.95	0.26
Oct	0.45	0.90	0.25	0.98	0.49
Nov	0.82	1.31	0.37	1.81	1.10
Dec	0.85	1.27	0.49	1.91	2.03

Notes:

(1) Average figures from 'Genale Bulo Marerta Project' (MMP 1978)

(2) Flows based on net irrigable area of 4 798 ha and 1 in 4 dry year.

The calculated figures for drainage water salinity are, for all months except January and February, conservative as the salinity of the irrigation water has been taken as river water salinity. During all months except January and February, part of the irrigation requirements are met by rainfall which will effectively reduce the salinity of the irrigation water and the drainage water.

The following table shows the tolerance to salinity of various crops that are likely to be grown in the area.

Crop	Tolerance
Maize	Moderatively sensitive
Sesame	Moderatively sensitive
Legumes	Sensitive
Vegetables	Moderatively sensitive
Water melon	Moderatively sensitive
Bananas	Sensitive

Maximum allowable water salinity levels for varying yield potentials are given in FAO paper 29. These are summarised below:

Tolerance	Irrigation Water Salinity (dS/m) Yield Potential %			
	100	90	75	50
Sensitive	0.7	1.0	1.7	2.8
Moderatively sensitive	1.8	2.5	3.9	6.0

Considering these allowable salinities and the estimated salinities and quantities of drainage water given in Table 4.6, the following conclusions may be drawn:

Initial Stage - No restriction on re-use, since the quality will be similar to the river water.

Intermediate Stage - Moderately sensitive crops would suffer virtually no yield loss using undiluted drainage water. More sensitive crops could suffer substantial yield loss (>25%) if the drainage water is used undiluted.

Final Stage - Moderately sensitive crops could suffer some yield loss (up to 10%) and more sensitive crops could suffer large losses (up to 50%) if the drainage water were used undiluted.

However, in all cases it is probable that in most months, the drainage water will be diluted either by rainfall or river water flowing down the Bokore. Thus, the re-use is likely to be acceptable at least in the initial and intermediate stages. The quality of the drainage water should be monitored from the beginning of the project so that any deterioration in quality can be identified as soon as possible.

b) Bokore Canal Capacity

The Bokore canal has insufficient capacity at present for the peak design drainage discharge of 8.1 m^3/s. The estimated capacity of the existing channel is about 3.0 m^3/s. In order to increase the capacity it would be necessary to remodel the canal and an outline design has been prepared for cost estimating purposes.

A longitudinal section of the canal showing the outline design is given on Figure 4.1. Figure 4.2 indicates the remodelling that would be required at each of the five locations where cross section survey data is available. It would also be necessary to remove or repair damaged gates at existing regulators, which would otherwise obstruct flow through these structures during periods of high discharge. Site inspections have shown that some of the gates at the regulators upstream and downstream of the tail pool are not operational and are stuck in a closed or partially closed position. Gates at the cross regulators at km 7.7 and km 13.0 are also not operational and would require attention.

The estimated cost of the works is US$ 300 000 and this has been included in the tender documents as a Provisional Sum. This includes bush clearance, earthworks and removal of damaged gates. No provision is made for repair or replacement of gates, which would be required for regulation of the canal.

4.9 Regional Outfall Drain

It is proposed to discharge drainage water into the Bokore canal, as discussed in Section 4.8 b). If field drains are installed in the project area in the future, the drainage water is likely to be saline and unsuitable for disposal into an irrigation canal. Alternative methods of disposal must therefore be considered.

The possibility of ponding water in local depressions to the south of the project area has been investigated but was found to be not practicable because the local depressions are too small for the amount of water involved.

The only other alternative is disposal into an existing old river channel some 15 km to the south of the area via a new disposal drain. A preliminary design for the disposal drain has been prepared for cost estimating purposes based on the 1978 1:25 000 scale maps and the more recent, but incomplete, 1:10 000 scale maps. The route selected is shown on Figure 4.3. Drainage water would be pumped from the Primary drain across the Sisab canal into the disposal drain. The route follows canal command boundaries as far as practicable to minimise interference with existing agriculture and also to minimise the the number of canal crossings which would be required. However, should the drain be constructed, further detailed investigations would be required to select the best route taking into account existing features and topography.

It is assumed that all the areas within the existing irrigation canal commands between the Farahaane project and the drain outfall would be drained by the new drain in the future. These areas, which total about 4 200 ha, are shown on Figure 4.3 together with the estimated peak discharge for each area based on a run-off rate of 1.3 l/s/ha. The design discharge of the drain would be 13.6 m^3/s at its outfall into the old river channel. The longitudinal section of the drain based on the preliminary design is shown on Figure 4.4 together with the design discharge and other hydraulic data. Typical cross sections are shown on Figure 4.5. The design includes 6 road crossings which are assumed to be box culverts.

Figure 4.1

Bokore Canal Longitudinal Section

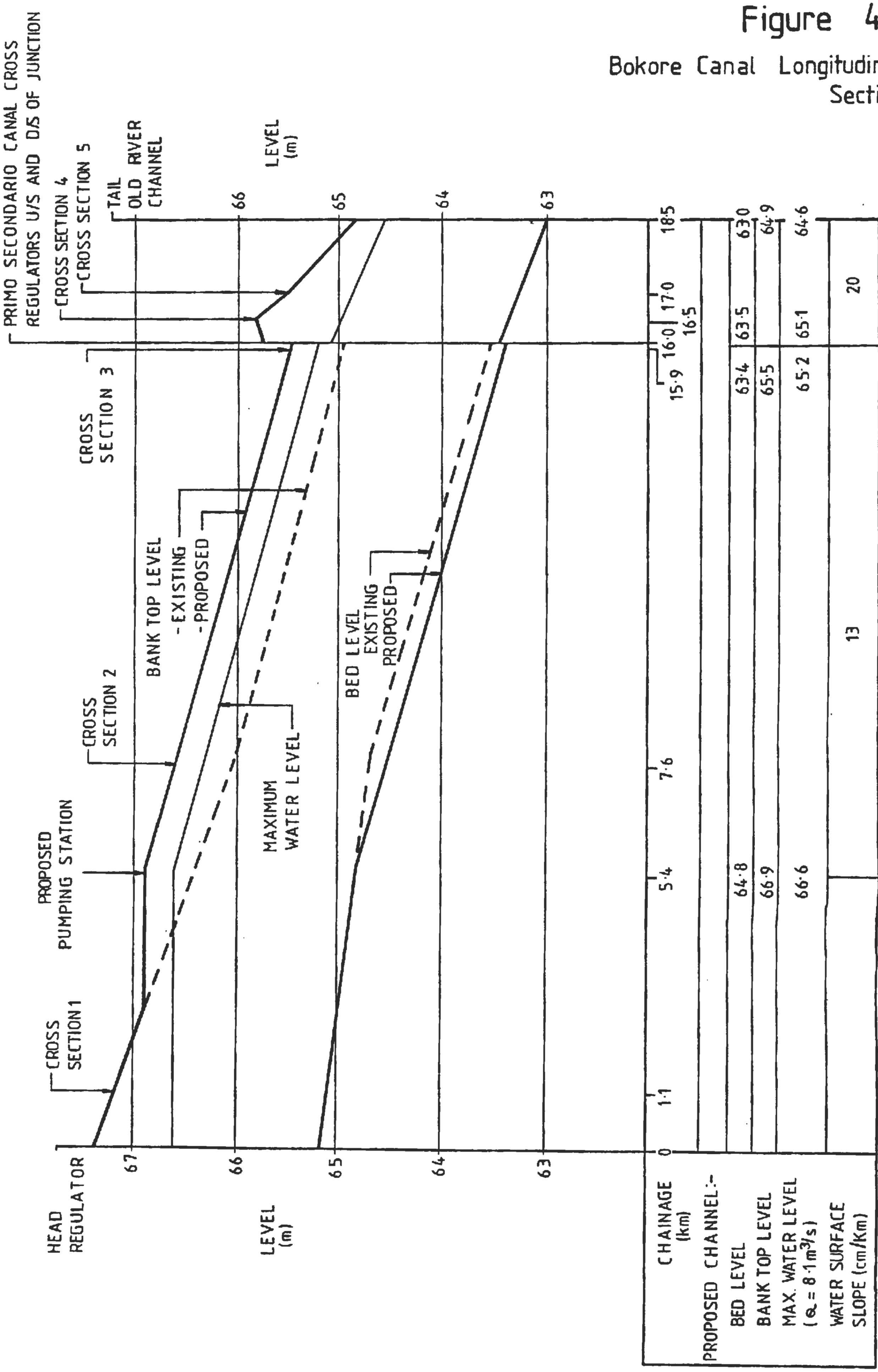


Figure 4.2
Bokore Canal Cross Sections

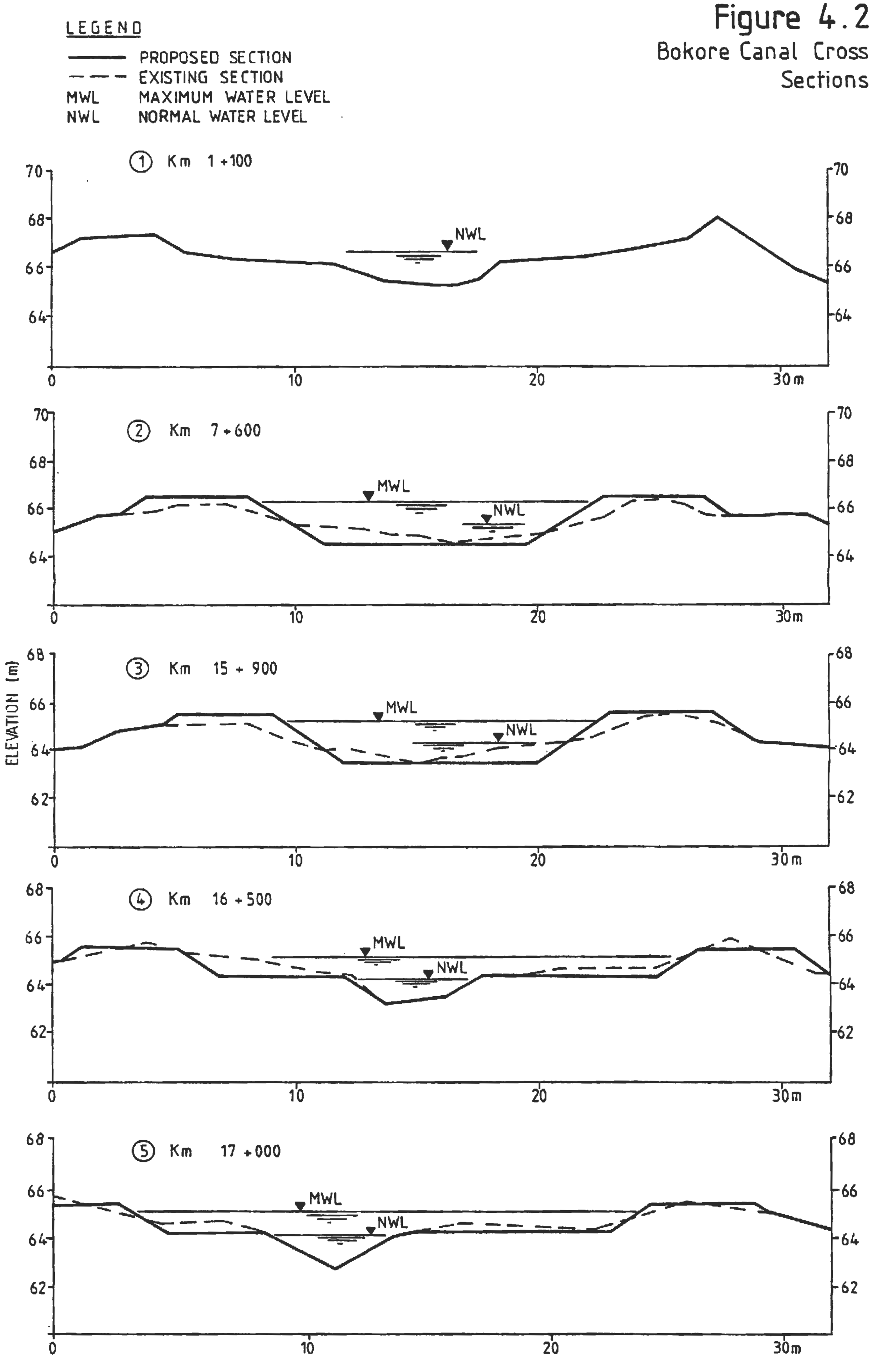


Figure 4.3
Disposal Drain

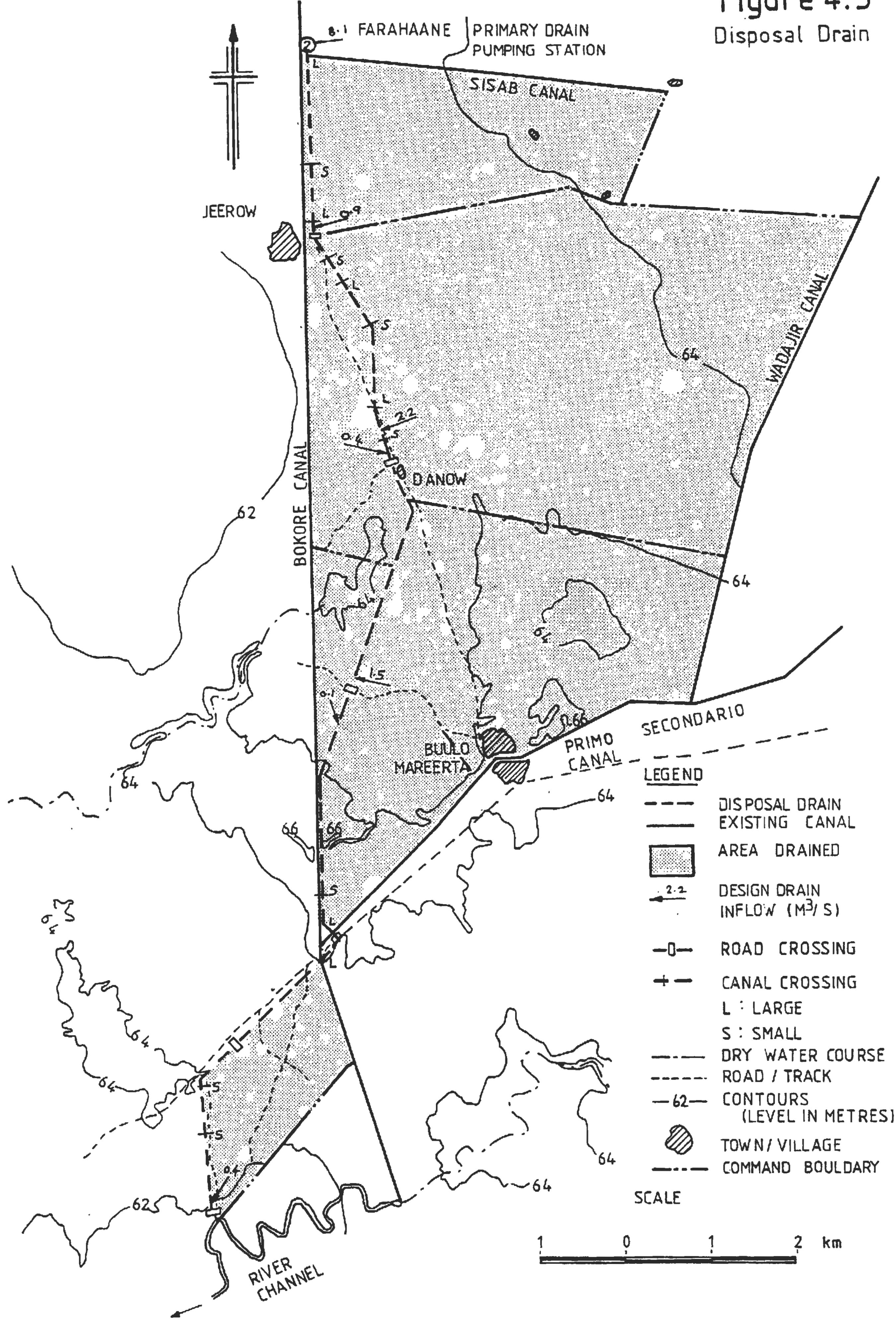


Figure 4.4
Disposal Drain Longitudinal Section

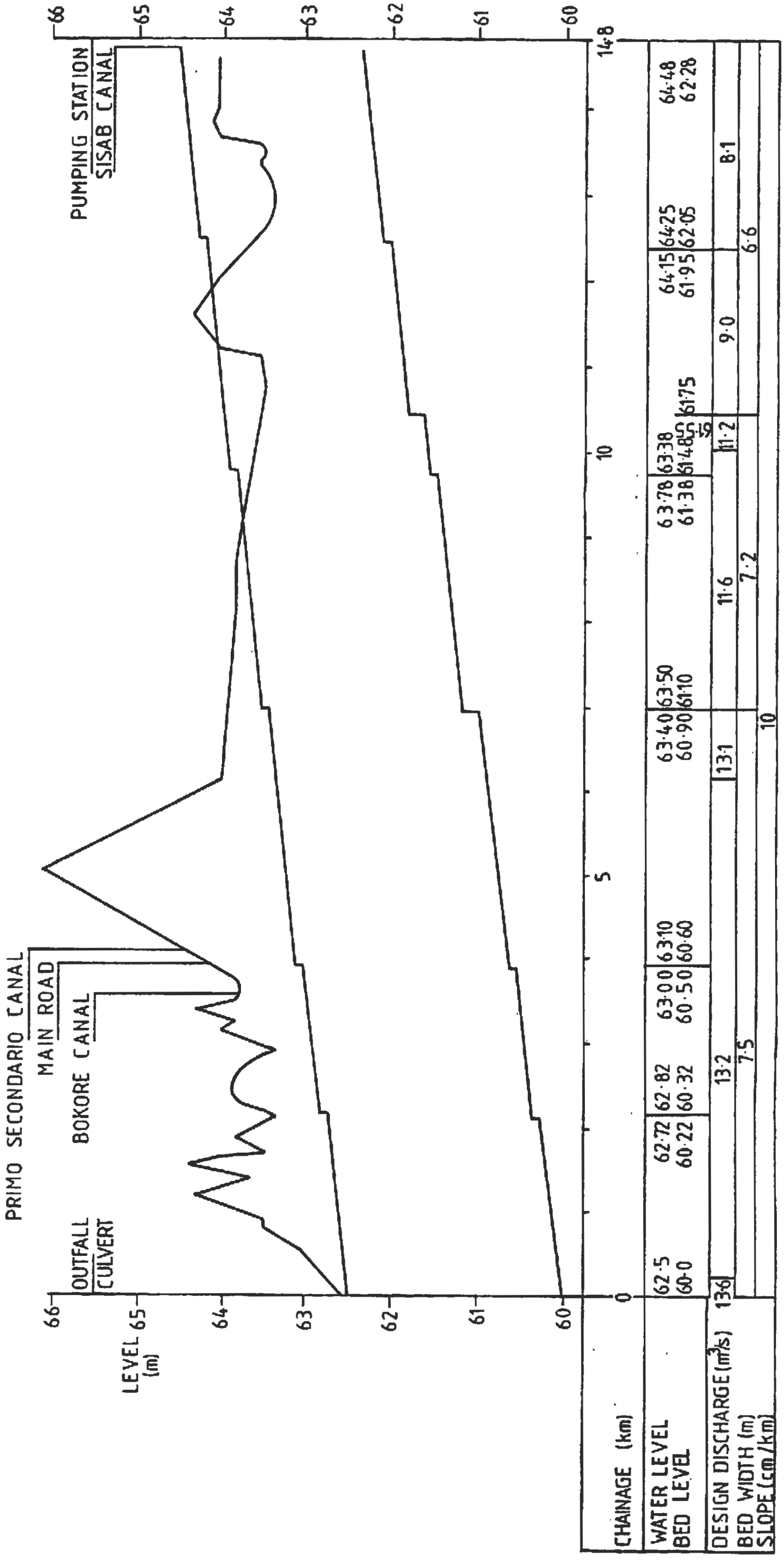
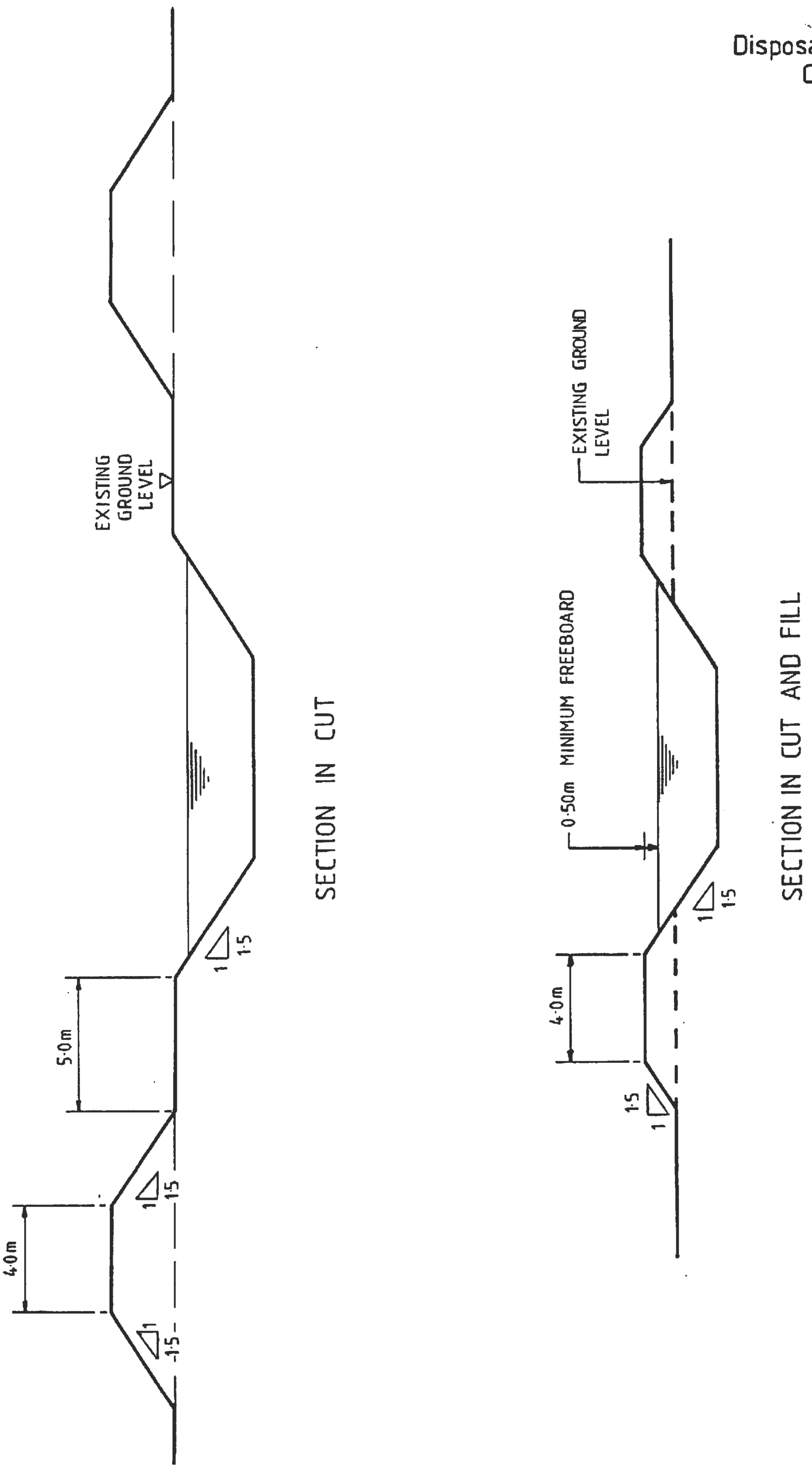


Figure 4.5

Disposal Drain Typical Cross Sections



Canal crossing would either be inverted siphons under the drain or pipe aqueducts, depending on the relative elevations of the canal and drain.

The estimated cost of the drain based on the preliminary design is US$ 3 million at current prices. This includes earthworks, 6 box culverts and 13 canal crossings, as indicated on Figure 4.3. It does not, however, include any costs for land acquisition. The total area of land required would be about 60 ha, some of which is cultivated at present.

As described in Section 4.8 a) above, the Regional Outfall Drain would not be needed until buried field drains were installed, and the exact requirements would need to be re-appraised at that time.

4.10 Flood Protection

Due to upstream overbank spillage and flood relief measures (eg Duduble and Jowhar) flooding in the project area tends to be less serious than elsewhere on the Shebelle. The last major flood was reported to be in the Gu season 1978 when large areas of land between Genale and Qorioley were affected. There are existing flood bunds along parts of the project area, although they are not continuous, and are often in poor condition.

Rehabilitation of these existing bunds would be very difficult, involving access problems and the possible destruction of numerous fruit trees. It was considered preferable therefore, to utilise the 4 m wide right hand bank of the Gayweerow Primary Canal and secondary canal G5 as a flood bund. The design bund top levels have been based on the estimated peak floods (MMP, 1978) with a freeboard of 0.40 m. Details are given in Table 4.7.

Table 4.7

Flood Bund Levels

	Km	Bund Level
Head Gayweerow Primary Canal	0.00	69.60
Tail Gayweerow Primary Canal	7.41	68.19
Head Secondary Canal G5	7.41	68.19
Tail Secondary Canal G5	8.35	68.01
Qorioley Headworks	8.39	68.00

A new bund is required between the tail of secondary canal G5 and Qorioley headworks. This will have a top width of 4 m and side slopes of 1 vertical to 1.5 horizontal. This bund will join the tail of secondary canal G5, thereby providing direct access from Qorioley to Gayweerow.

CHAPTER 5

ROADS

5.1 Introduction

The project is already relatively well served with roads. The metalled road from Genale to Qorioley runs through the northern part of the area, and there is a network of earth roads inter-connecting the villages. There are also numerous minor tracks giving access to the fields.

The following roads have been provided in the Contract:

- gravel roads
- access roads
- inspection roads

5.2 Gravel Roads

These will follow the alignments of the existing major earth roads in the project area. A short length of gravel road has also been allowed for at the project headquarters. Details of the roads are given below:

Table 5.2

Gravel Road Details

Road	Length (km)
Qorioley Barrage - Farahaane - Sisab Canal	11.7
Falkeerow - Farahaane - Haduman	8.7
Bulo Sheikh - Madhulow	5.0
Project Headquarters	0.5
	25.9

The roads comprise a compacted earth sub-grade (the existing earth road or natural ground) with a layer of selected fill and a coral sub-base and road base each of 0.15 m thickness. The thickness of the selected fill layer should be sufficient to achieve a finished road level a minimum of 0.6 m above adjacent ground level. The nominal surfaced width is 6 m with 1 in 4 sloping shoulders. It may be necessary, however, to reduce this width in areas where available space is limited - eg through existing villages.

A 3 m wide reservation should be left on one side of the road as a cattle track to avoid damage to the gravel surfacing.

5.3 Access Roads

Access roads of compacted earth have been provided along one side of primary and secondary drains. The roads are 4 m wide and a minimum of 0.15 m high. Where excess material from drain excavation is to be disposed of, the access road can be raised to a maximum height of 1.0 m. The minimum height, however, should be retained at drain culverts to minimise pipe lengths.

5.4 Inspection Roads

Inspection roads have been provided along canal banks - on one side for secondary canals and both sides for primary canals. These roads are intended for inspection and maintenance and should not be used by general traffic. The road width is 4 m in both cases.

5.5 Design Criteria

The adopted design criteria for roads are summarised in Table 5.2.

Table 5.2

Road Design Criteria

	Gravel road	Access road	Inspection road
Road width (m)	6.0	4.0	4.0
Height above GL (m)	0.6	0.15-1.0	On canal bank
Minimum centre line radius (m)	10.0	10.0	10.0
Maximum gradient	1 in 20	1 in 10	1 in 10

CHAPTER 6

BUILDINGS

6.1 General

To accommodate the increased staffing, administrative operations and vehicle and plant maintenance, additional buildings are to be provided in the Project Headquarters, namely:-

i) 4 nr houses
ii) administration building
iii) workshop

In addition operators quarters have been provided at seven sites around the project area:

i)	Gayweerow and Farahaane primary canal headworks and tail groups	(4 Nr)
ii)	Gayweerow primary canal cross regulator, km 1.90	(1 Nr)
iii)	Secondary canal G3 cross regulator, km 1.80	(1 Nr)
iv)	Drainage pump station	(1 Nr)

Buildings services (electricity, water, sewerage disposal) have been provided for the buildings at the project headquarters.

6.2 Building Layouts

Building designs have been based on those prepared by Weidleplan for buildings already constructed in the Project Headquarters.

The proposed house design is similar to the existing House Type D.

The proposed workshop is similar to the existing workshop but the roof on the office areas has been lowered to improve structural stability of the gable walls and to give cost savings.

The proposed administration building is a new design but using roof trusses similar to those on the existing administration building. The building will incorporate a conference room, five offices, store, kitchen and toilets.

The operator's quarters is also a new design incorporating a small office and living room.

The approximate floor areas of the buildings are given in Table 6.1.

Table 6.1

Building	Floor Area (m^2)
House	160
Administration building	250
Workshop	250
Operator's quarters	24

6.3 Structural Design

a) British Standards

The structural design of the buildings is in accordance with the following British Standards:-

BS 6399 :		Part 1	- Design Loading for Buildings
CP3 :	Chapter V:	Part 2	- Wind Loading
BS 8110 :			- Structural Use of Concrete
BS 5628 :		Part 1	- Structural Use of Unreinforced Masonry
BS 5268 :		Part 2	- Structural Use of Timber
BS 5268 :		Part 3	- Code of Practice for Trussed Rafter Roofs

b) Foundation Design

The soils in the Project Area are expansive clays. Foundations for all the buildings are designed to prevent building movement due to swelling and shrinking of the expansive clays with varying moisture content. This is achieved by replacing the clay in the zone of varying moisture content beneath the building with a stable material. The design has been proven elsewhere in Somalia on expansive clay soils.

Treatment of the ground beneath the buildings with insecticide, to reduce the probability of termite entry, has been specified.

c) Wall Design

The walls will be constructed of 200 mm thick unreinforced structural masonry designed to resist dead, live and wind loads.

d) Roof Construction

The roofs of the houses, administration building and workshop will be of asbestos cement sheeting bearing onto timber trussed rafters. The trussed rafters have been designed to resist dead loads, maintenance loads, service loads and wind. Joints are nailed plywood gussets, which avoids the need to obtain specialised connectors.

Treatment of the roof trusses, and other non-structural timber, against termite and other wood destroying insects and organisms has been specified.

For the operator's quarters a flat reinforced concrete roof has been provided which is cooler and more appropriate for this type of building.

6.4 Building Services

a) Electrical

The electrical services briefly consist of a new power generation unit, together with small power and lighting equipment for the new houses, workshop and administration building. An extension to the existing overhead line distribution system is also included.

The generating set incorporates a diesel engine driven alternator, starting batteries, exhaust system and control panel all mounted on a common baseplate which also has an integral day fuel storage tank. The set is located adjacent to the existing generator building and is mounted on a suitably strengthened concrete plinth. A canopy over the unit provides an effective sun screen. Power is supplied from the generator to a new distribution panel located in the generator building - this panel will replace the existing distribution board. The two existing generators are also connected to the new panel, and an interlock arrangement ensures that only one of three connected can actually supply the load. Power is then taken to the new workshop via an underground line, and to the new and existing administration buildings and housing via the the overhead line system extended as necessary. The new generator is rated at 80 KVA, which is of sufficient size to supply all new and existing loads, whilst also allowing 25% for future expansion.

A new bulk fuel storage tank is sited next to the generator. The bulk fuel storage tank has a capacity of 7 500 litres which is sufficient fuel to run the new generator for one month. The bulk fuel tank has fabricated steel supports and is mounted on a concrete plinth, fuel being fed to the generator by gravity. The fuel tank is surrounded by a bund in case of spillage.

The bulk fuel tank has an earth ramp next to it to facilitate fuel transfer from a tanker to the bulk fuel tank by gravity. The earth ramp has been placed as close to the bulk fuel tank as it is practical; to facilitate this, a retaining wall has been incorporated into the bund.

Within the new buildings, a consumer unit (houses) or three phase distribution board (workshop and administration buildings) supplies lighting and power outlets. The houses have lighting, socket outlets and ceiling fans in all major rooms, and the kitchen also has an extractor fan and the facility for connection of a small cooker. The administration building has air conditioning in three of the offices with ceiling fans in the others, plus lighting and socket outlets throughout. Finally, in addition to lighting and power outlets, the workshop has two three phase sockets for the connection of small machinery. A new consumer unit is also provided for the existing generator building. This is mounted on the main distribution panel and supplies the existing building services loads within the building.

b) Water Supply, Sewerage and Waste

The existing water supply system in the housing area has been extended to take water to the new houses. Water supply for the new administration building and workshop has been provided by connecting into the water mains running adjacent to these buildings. Each extension of the existing water distribution system has an isolating valve fitted.

Each building is provided with its own independant sewage and waste system, consisting of all pipework, manholes for access, a septic tank and a soakaway.

The plumbing system within each building has been kept as simple and straightforward as possible, keeping the internal pipe runs to a minimum. The potable water enters the buildings at low level and goes straight to the relevant sanitary appliance thus eliminating the need for a header tank or pipe runs within the building. There is only a cold water supply to each sanitary appliance - no hot water is provided.

Where a water supply is required at more than one place within a building, the supply enters the building at a point next to where it is required, similarily the waste pipework is taken straight out of the building.

6.5 Gravel Roads

Approximately 0.5 km of gravel road have been provided within the project headquarters area to facilitate all weather access. The surfaced width will be 6 m as described in Chapter 5.

CHAPTER 7

GENERAL STRUCTURAL DESIGN CRITERIA

7.1 Loading

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

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

7.2 Stability

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

A factor of safety of 5 (based on the exit gradient) has been adopted against piping. Lower values have been allowed where the extreme loading case is considered to be unlikely to occur, with an absolute minimum of 2.5.

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

7.3 Soil Properties

The following soil properties have been used in the design. They reflect the worst conditions likely to be met in the project area.

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

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

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

7.4 Concrete and Reinforcement

Reinforced concrete has been designed generally in accordance with BS 8110 for concrete grade 20 (characteristic strength 20 N/mm^2) and mild steel reinforcement (yield strength 250 N/mm^2).

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

The following properties of concrete have been assumed:

Weight of reinforced concrete	23.5 kN/m^3
Weight of mass concrete	22.0 kN/m^3
Modulus of elasticity (for deflections)	23×10^3 N/mm^2
Coefficient of linear expansion	11×60^{-6}/^oC
Coefficient of shrinkage	300×10^{-6}

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

Cover to reinforcement is 50 mm except for some small relatively unimportant members, and laps in bars have been set at a minimum of 40 x bar diameter.

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

The following minimum reinforcement percentages have been used:

(i) Beams and Slabs

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

(ii) Columns

Total vertical steel	0.4% total area
Links	6 mm diameter at 12 x diameter of longitudinal bars

(iii) Walls

Total vertical steel	0.4% total area
Total horizontal steel	0.3% total area

The classes of concrete used are as follows:

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

Sulphate resisting cement has been adopted for all drain structures and concrete below ground level.

7.5 Pipes and Pipe Bedding

Culverts, underpasses etc have been based on the use of spigot and socket concrete pipes with internal diameters of 0.225, 0.30, 0.375, 0.45, 0.60, 0.75, 0.90, 1.05 and 1.20 m.

Two classes of pipe have been specified: Class L and Class M; and two classes of bedding: Class A1 (mass concrete) and Class B (granular).

Table 7.1 gives the ranges of depths of cover appropriate to each size and class of pipe. Generally granular bedding has not been used for gated structures or structures where there is a large head loss through the pipe. This is because the granular bed presents a low resistance seepage path and any significant flow through it could cause piping and subsequent failure at the downstream end. Otherwise the granular bed is preferred since it is much cheaper than the concrete alternatives.

Table 7.1

Maximum Depths of Cover to Pipes

Dia (m)	Pipe Class	Bedding Type B	Bedding Type A1
Single pipes:			
0.225	L	7.6	7.6
	M	7.6	7.6
0.30	L	5.5	7.6
	M	7.6	7.6
0.375	L	-	3.1
	M	4.4	7.6
0.45	L	-	-
	M	4.6	7.6
0.60	L	-	-
	M	4.6	7.6
0.75	L	-	4.5
	M	4.6	7.6
0.90	L	-	3.5
	M	4.1	7.3
1.05	L	-	3.6
	M	4.1	7.3
1.20	L	-	3.3
	M	4.1	6.7
Multiple pipes:			
1.05	L	-	3.2
	M	3.6	5.1
1.20	L	-	3.1
	M	3.5	5.1

Note: minimum cover to be 0.9 m throughout.

CHAPTER 8

IMPLEMENTATION

8.1 Tender Documents

Draft tender documents have been produced for the works and have been submitted with this Design Report (March 1988). The final documents will be prepared following MOA approval. Some relevant features of the documents are given below:-

a) Eligibility of Contractors

The documents are suitable for international competitive bidding. It is not yet certain whether pre or post-qualification will be applicable, but the documents have been prepared assuming post-qualification of contractors which will keep the tendering period to a minimum. However, if pre-qualification is required modifications can easily be made prior to submission of the final documents.

b) Number of Contracts

The overall scope of the works is not excessively large, and it is important to maximise the size of the contract thus attracting international contractors to bid. Hence all the construction components have been included in a single contract.

c) Form of Tender Documents

The Tender Documents follow the World Bank guidelines and include:

Instructions for Tendering
Conditions of Contract - General (FIDIC)
- Particular
Specification
Bill of Quantities
Tender Form

d) Definitions

'The Employer' is the Ministry of Agriculture, Somalia.

'The Engineer' is not named, as it is not known at this stage who will undertake supervision of construction.

e) Currency of Contract

The currency of the Contract is Somali Shillings. The Contractor will however be able to take a proportion of his payments in one foreign currency, the proportions and foreign currency being nominated in his Tender. The exchange rates will be the current selling rate of the Central Bank of Somalia.

f) <u>Price Variation</u>

A price variation clause has been included in the Particular Conditions of Contract. This will take account of price variations in both local and foreign currency, based on published price indexes both in Somalia and the country whose foreign currency is nominated by the Tenderer.

g) <u>Engineer's Requirements</u>

Although the Engineer is not nominated at this stage certain necessary facilities for the Engineer will be provided under the Contract. These include

- office
- laboratory
- furniture, fixtures and equipment for office and laboratory
- surveying instruments
- boat for river investigations

Vehicles and housing for the Engineer have not been included, as it is assumed these would be covered under separate funding arrangements. The construction of houses for the Engineer may not be necessary as suitable rented accommodation could probably be obtained in Shalambod or Merka.

h) <u>Contract Period</u>

A contract period of 36 months is specified, followed by a 12 month maintenance period.

8.2 Tendering Procedure

Following approval of the draft tender documents, 50 copies of the final documents will be delivered to the MOA in Mogadishu. The tender documents would be issued by, and returned to, the MOA. Any queries or amendments would be dealt with by the Consultant, with formal addenda to the documents being issued where necessary.

Following the opening of the tenders in Mogadishu, the Consultants would carry out the tender evaluation. This would be done as specified in the Agreement for Consultancy Services (September 1987) - namely an arithmetic check on the 6 lowest tenders and a detailed evaluation of the 3 lowest fully responsive tenders. The results would be summarised in a concise Tender Evaluation Report giving recommendations for award.

8.3 Implementation Schedule

Implementation schedules have been prepared for the two cases of pre and post-qualification of contractors (Figures 8.1 and 8.2). As can be seen from the Figures, a full prequalification phase involving the preparation of draft and final prequalification documents, would add some 5 months to the overall implementation period.

Figure 8.1

Implementation Schedule

Post – Qualification

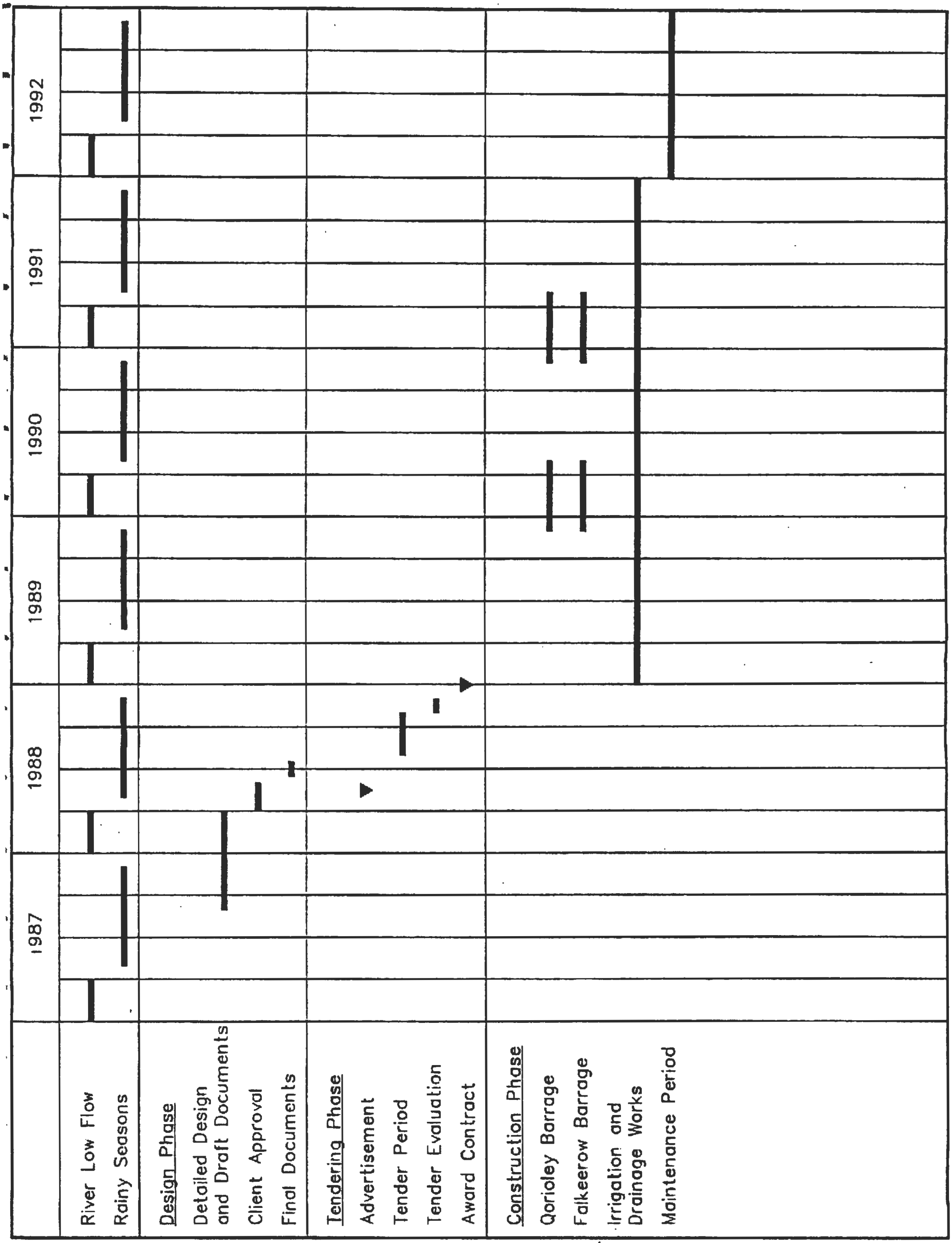
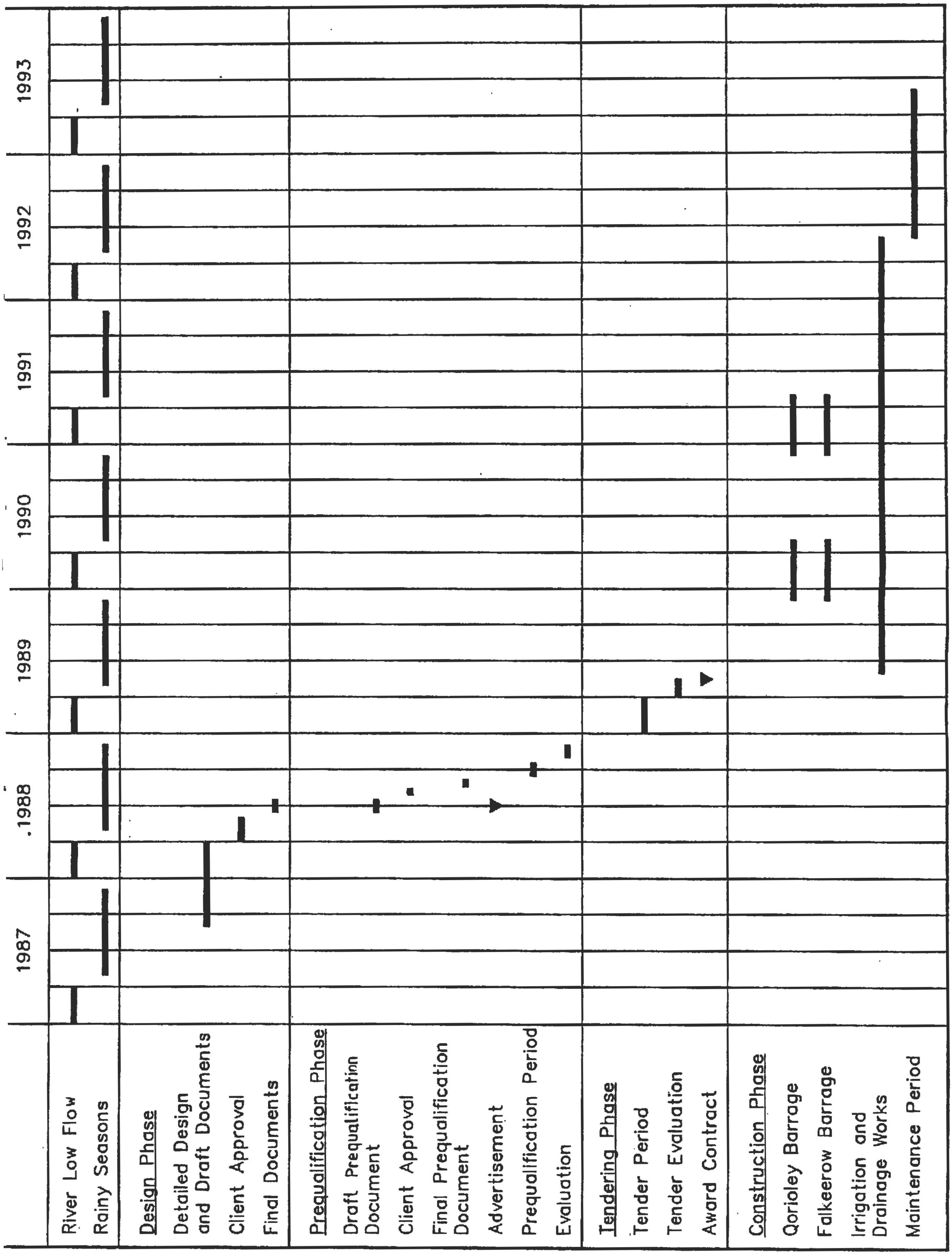


Figure 8.2

Implementation Schedule
Pre – Qualification



8.4 Cadastral Survey

In order that the detailed infield layout can be carried out, it will be necessary for the cadastral survey to be extended over the remaining area of about 5 700 ha gross.

The survey is likely to take at least six months and will require a considerable input in staff and facilities. Ideally the survey should be completed before the Contractor starts work, although this may not be essential as the construction works could be programmed to follow the areas where the cadastral survey had been completed.

Full details of the requirements are given in the Topographic and Cadastral Survey Report (MMP, 1987).

8.5 Land Acquisition and Compensation

The introduction of a drainage system, remodelling of existing canals and construction of some new canals will mean that a considerable area of agricultural land will have to be purchased. Based on the proposed irrigation and drainage layout and a survey of the existing canals, an estimate of the area to be purchased is summarised in Table 8.1. Further details are given in Appendix D.

Table 8.1

Land Acquisition Requirements

Canal or Drain	Area (ha)
Gayweerow Primary Canal	16.6
Farahaane Primary Canal	4.6
Secondary Canals	26.6
Tertiary/Quaternary Canals	41.0
Primary Drain	16.1
Secondary Drains	114.2
Tertiary/Quaternary Drains	225.0
	444.1

It is assumed that the gravel roads would follow the alignment of existing earth roads and thus require no land acquisition. No allowance has been made for remodelled tertiary/quaternary canals as the total width of an existing and remodelled channel will be similar.

Table 8.1 shows that the majority of the land to be acquired is needed for the drainage system. Of the total of 444 ha some 75% is accounted for by the drainage system and 50% (225 ha) is required for the tertiary and quaternary drains.

Assuming a cost of irrigable land of So Sh 50 000 per hectare, then the total cost of land acquisition will be So Sh 22.2 million.

In addition to land acquisition it will be necessary to destroy some fruit trees. The majority of these will along the line of the new Gayweerow primary canal close to the river, but there may also be significant numbers along existing canal lines which are to be remodelled. Assuming 250 mature trees have to be destroyed at an average cost of SO Sh 20 000 per tree, then the total cost will be So Sh 5 million.

The construction of the Gayweerow primary canal will also necessitate the destruction of an existing residential compound close to the river by the Gayweerow - Haduman river ferry crossing. The cost of replacing the two buildings (an 8 m x 4 m hut and a 5 m diameter Muduh) is estimated at So Sh 110 000.

A summary of the costs of land acquisition and compensation for destruction of trees and houses is given in Table 8.2.

Table 8.2

Land Acquisition and Compensation Costs

Item	Amount	Rate (So Sh)	Total (So Sh)
Land acquisition	444.1 ha	50 000	22 205 000
Compensation for destruction of trees	250 Nr	20 000	5 000 000
Compensation for destruction of buildings	50 m^2	2 200	110 000
			27 315 000

8.6 Maintenance of Irrigation Supplies During Construction

Irrigation is at present carried out throughout the project area and it is a requirement of the tender documents that the contractor should maintain the existing irrigation flows whilst the rehabilitation works are in progress.

Should the Contractor fail to meet this requirement he is required, under Clause 81 of the Conditions of Contract, to compensate the consumer to the extent of the crop loss.

Contractors are required, under Clause 8 of the Instructions of Tendering, to submit with their tender details of their proposed methods of maintaining flows in the channels. These will be examined by the Consultant at the tender evaluation stage.

Possible methods that could be adopted by the Contractor include:

a) the use of portable pumps and pipelines to temporarily by-pass canals that are being remodelled;

b) construction of temporary canal aqueducts or drain underpasses where existing canals are cut by new drains;

c) construction of new structures adjacent to canals with the canals re-routed after completion;

d) construction of by-pass channels so that new structures in existing channels an be constructed in the dry;

e) programming the works so that as much work as possible is undertaken in the January/February low flow period when irrigation is not normally carried out.

APPENDIX A

BARRAGE SURVEY AND INVESTIGATIONS

A.1 Introduction

The survey work at Falkeerow and Qorioley Barrages was carried out in October 1987 and January 1988 and included barrage surveys, investigations into the structural integrity of the existing structures and a river survey. Due to high river levels only the barrage superstructures and river banks were surveyed in October; the remaining survey work being carried out in January when the water level had fallen.

A.2 Barrage Survey

There are no drawings of either barrage, and thus the first task of the investigations was to produce accurate drawings of the existing barrages. This was done by a series of comprehensive measurements and level surveys from which Drawings 164701/71 and 81 (in the Album of Drawings) showing the existing situation were produced.

A.3 Structural Investigations

Measurements and levels were taken of the existing barrages and a visual inspection was made of the gates and structural condition. Many photographs were taken for use in the design office.

A concrete core was taken at each barrage for compressive strength tests. At Qorioley a vertical core was taken from the left bank upstream abutment and at Falkeerow a horizontal core was taken from the right bank downstream parapet wall. Also the extent of concrete carbonation was measured on the cores using the phenolphthalein test. In this test the clear indicator is sprayed from the top of the core downwards and the depth of carbonation is indicated when there is a rapid development of a pink colouration. At this depth the concrete is still sufficiently alkaline and should, in the absence of chloride, provide protection to the reinforcement. Core dimensions and test results are tabulated in Table A.1.

Table A.1

Concrete Core Details

Barrage	Core Dimensions Height (mm)	Diameter (mm)	Compressive Strength (N/mm^2)	Depth of Carbonation (mm)
Qorioley	125	151	27.9	35
Falkeerow	160	151	8.9	>160

There was no sign of honeycombing or segregation in either concrete core.

To see how representative the concrete cores were to other parts of the barrage, a PUNDIT (Portable Ultrasonic Non-Destructive Digital Indicating Tester) was used to measure the speed of ultrasonic waves through the cores and then across some upstream piers approximately 200 mm below the deck level. Emitting and receiving tranducers were placed directly opposite each other on either side of the piers and the time for the pulse to pass through the concrete was measured by the PUNDIT. Knowing the path length this was converted to velocity. The PUNDIT test results are tabulated in Table A.2.

Table A.2

PUNDIT Results

Barrage	Sample	Pulse Velocity (m/s)
Qorioley	Core	3 952
	Pier 4/5	3 292
	Pier 5/6	3 641
Falkeerow	Core	3 100
	Pier 1/2	4 054
	Pier 3/4	3 841
	Pier 6/7	3 948

There is no unique relationship between pulse velocity and strength as it is influenced by concrete constituents and curing conditions so evaluation of the quality of concrete in structures is made on a comparative basis using concrete of the same origin.

Although the concrete core taken from Falkeerow barrage has a low compressive test, the PUNDIT readings have shown that the concrete in the piers is of a better quality. At Qorioley the concrete in the piers which were tested was a slightly poorer quality than the concrete core, although the PUNDIT readings were in the same order indicating that there was no serious honeycombing or degradation of the concrete. The concrete can be classified as structural lightweight aggregate concrete; the lower density being due to the use of coral as the aggregate. It is concluded therefore that generally the structural concrete in the barrages is of reasonable quality, except where visually obvious deterioration has occurred.

Undisturbed soil samples from sample pits were taken at foundation level and on the river banks. The samples taken from the river banks were a silty clay and the samples from the foundation level of both barrages were medium to hard grey clay. A liquid and plastic limit test, a particle size analysis, a consolidation test and an undrained compression test were performed on the samples according to BS 1377, 1975. Test results are shown in Figures A.1 to A.7.

Figure A1

UNDRAINED COMPRESSION TEST RESULTS

CONTRACT FARAHAANE PROJECT

Borehole	Depth (m)	Sample	Moisture Content (%)	Bulk Density (Mg/m^3)	Lateral Pressure (kN/m^2)	Deviator Stress (kN/m^2)	Apparent Cohesion (kN/m^2)	Angle of Shearing Resistance (degrees)	Remarks
QORIOLEY	66.50	1	40	1.75	50 100 200	35 45 45	17	1.5	Very soft to soft reddy brown sl organic sandy silty CLAY with fibrous roots
FALKERROW	64.30	3	38	1.79	50 100 200	84 96 98	40	2.5	Soft to Firm reddy brown sl organic silty CLAY

Figure A2

PARTICLE SIZE DISTRIBUTION

CONTRACT FARAHAANE PROJECT

Borehole	Remarks Very soft to soft reddy brown slightly organic sandy silty CLAY with fibrous roots
Depth (m)	
Sample 1	Method WET SIEVE & HYDROMETER

Particle Size (mm)	Percentage Passing	Particle Size (mm)	Percentage Passing
2.00	100	0.012	56
0.425	99	0.0087	48
0.212	98	0.0063	45
0.063	92	0.0043	39
0.059	83	0.0037	36
0.043	79	0.0029	34
0.031	74	0.0023	32
0.022	68	0.0015	29
0.016	62		

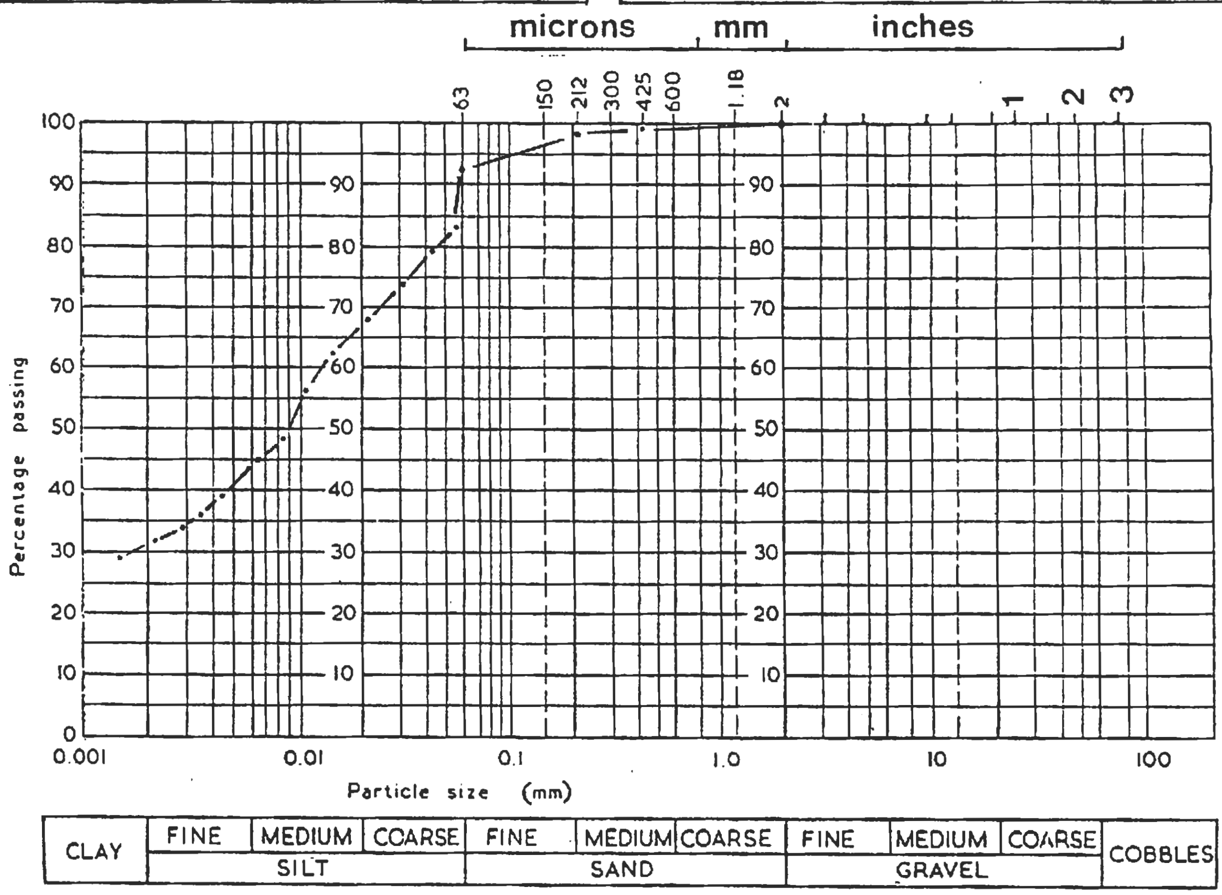


Figure A3

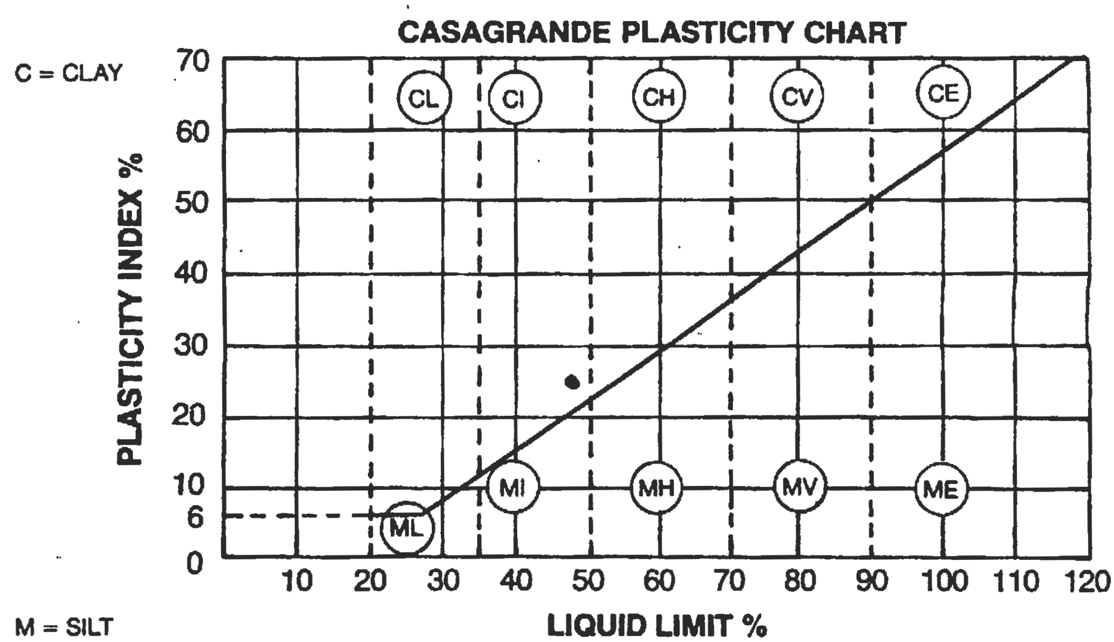
LIQUID AND PLASTIC LIMIT TEST RESULTS

CONTRACT FARAHAANE PROJ.	Natural Moisture Content	40	%
Borehole	Liquid Limit	48	%
Depth (metres)	Plastic Limit	23	%
Sample 1	Plasticity Index	25	%
METHOD	Liquidity Index	0.68	
Clayton & Jukes Single Point Method using the Cone Penetrometer	Clay Content	30.00	%
	Activity	0.83	

SAMPLE PREPARATION

Curing time (hours)	24	Sample retained 0.425 mm	1	%
		Method of preparation	OVEN DRIED	

Remarks Very soft to soft reddy brown sl organic sandy silty CLAY with fibrous roots

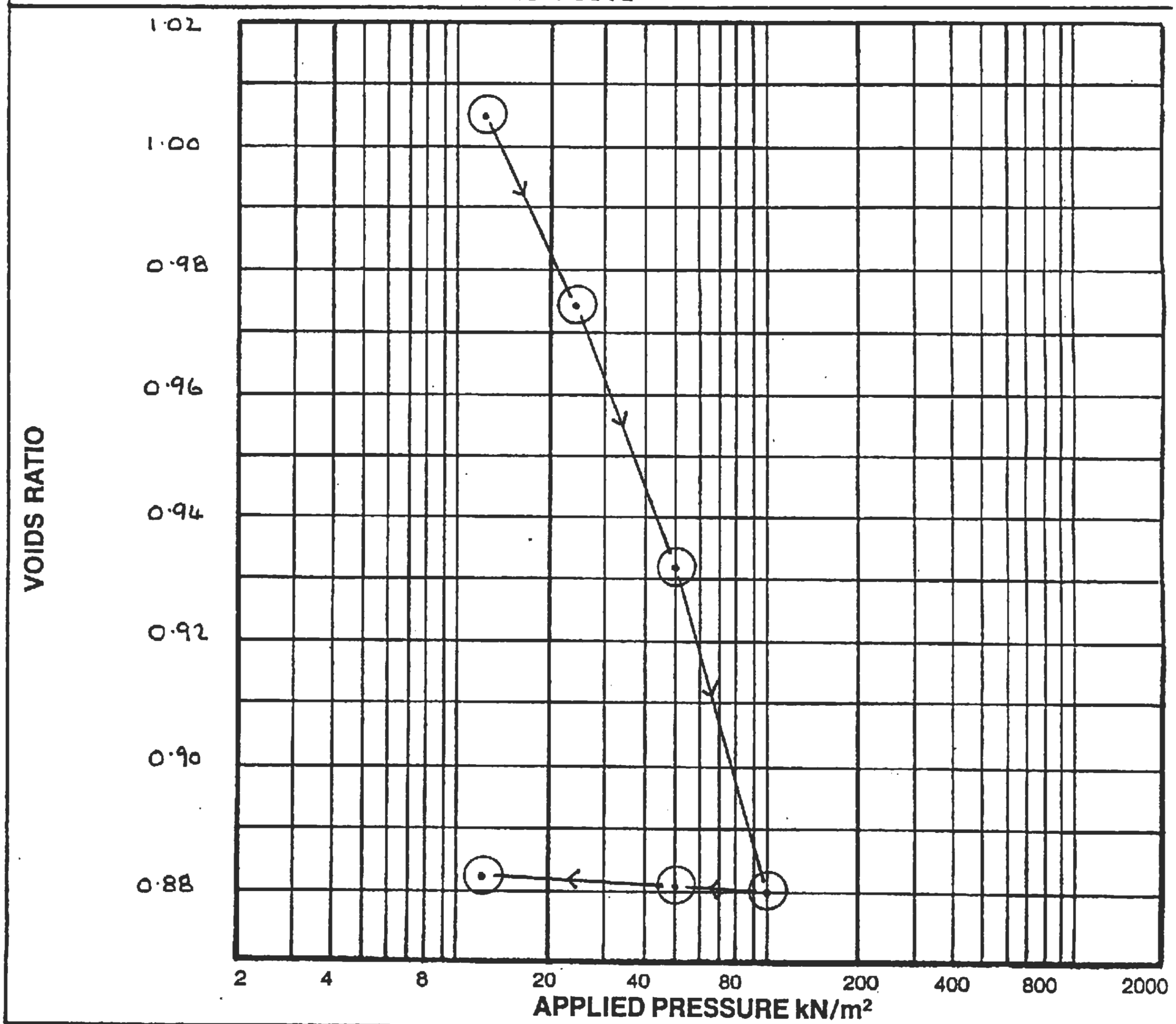


CONSOLIDATION TEST

CONTRACT FARAHAANE PROJECT **RESULTS** Figure A4

Borehole	Depth (metres)	Pressure	Coefficient of Consolidtion cv (m2/yr)	Coefficient of Compressibility mv (m2/MN)
Sample 1		0.0-12.5	0.724	4.088
Initial Voids Ratio	1.113	12.5-25.0	0.841	1.260
Specific Gravity	2.71 ASSUMED	25.0-50.0	0.698	0.852
Natural Moisture Content	37 %	50.0-100.0	0.905	0.531
Bulk Density	1.75 Mg/m³			
Dry Density	1.28 Mg/m³			
Degree of Saturation	89 %			
Swelling Pressure	0.0 kN/m²			

Remarks: Very soft to soft reddy brown sl organic sandy silty CLAY with fibrous roots



PARTICLE SIZE DISTRIBUTION

CONTRACT FARAHAANE PROJECT

Borehole	Remarks: Soft to firm reddy brown slightly organic silty CLAY
Depth (m)	
Sample: 3	Method: WET SIEVE & HYDROMETER

Particle Size (~~ins~~) (mm)	Percentage Passing	Particle Size (mm)	Percentage Passing
0.150	100	0.0055	78
0.063	99	0.0038	70
0.054	98	0.0033	66
0.038	97	0.0026	62
0.027	97	0.0021	58
0.014	93	0.0014	52
0.010	89		
0.0078	84		

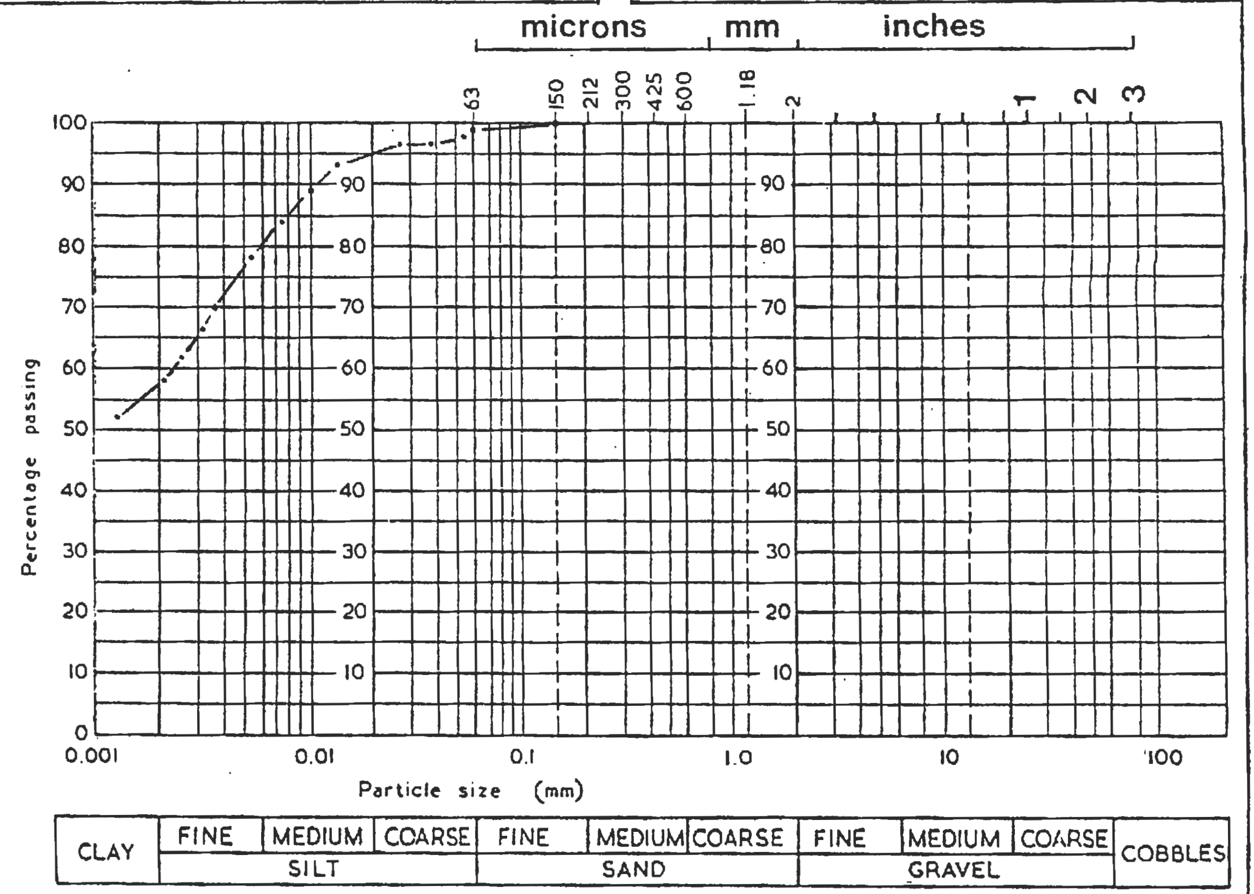


Figure A6

LIQUID AND PLASTIC LIMIT TEST RESULTS

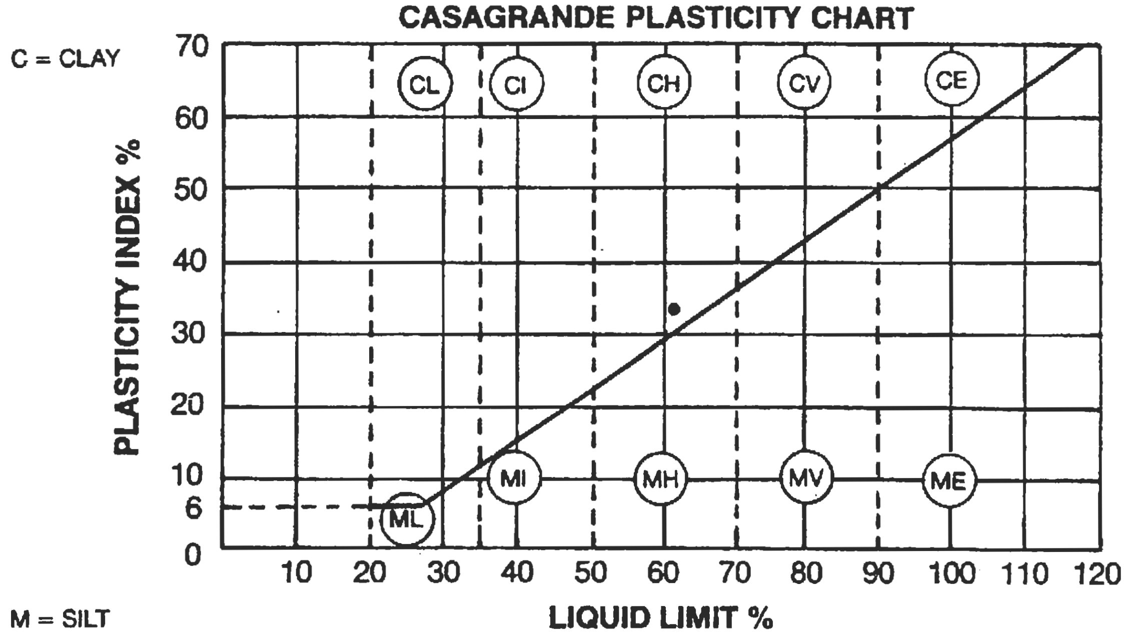
CONTRACT	FARAHAANE PROJ.	Natural Moisture Content	38	%
Borehole		Liquid Limit	61	%
Depth (metres)		Plastic Limit	28	%
Sample	3	Plasticity Index	33	%
METHOD Clayton & Jukes Single Point Method using the Cone Penetrometer		Liquidity Index	0.30	
		Clay Content	57.00	%
		Activity	0.58	

SAMPLE PREPARATION

Curing time (hours)	24	Sample retained 0.425 mm	0	%
		Method of preparation	OVEN DRIED	

Remarks
Soft to Firm reddy brown sl organic silty CLAY

CASAGRANDE PLASTICITY CHART



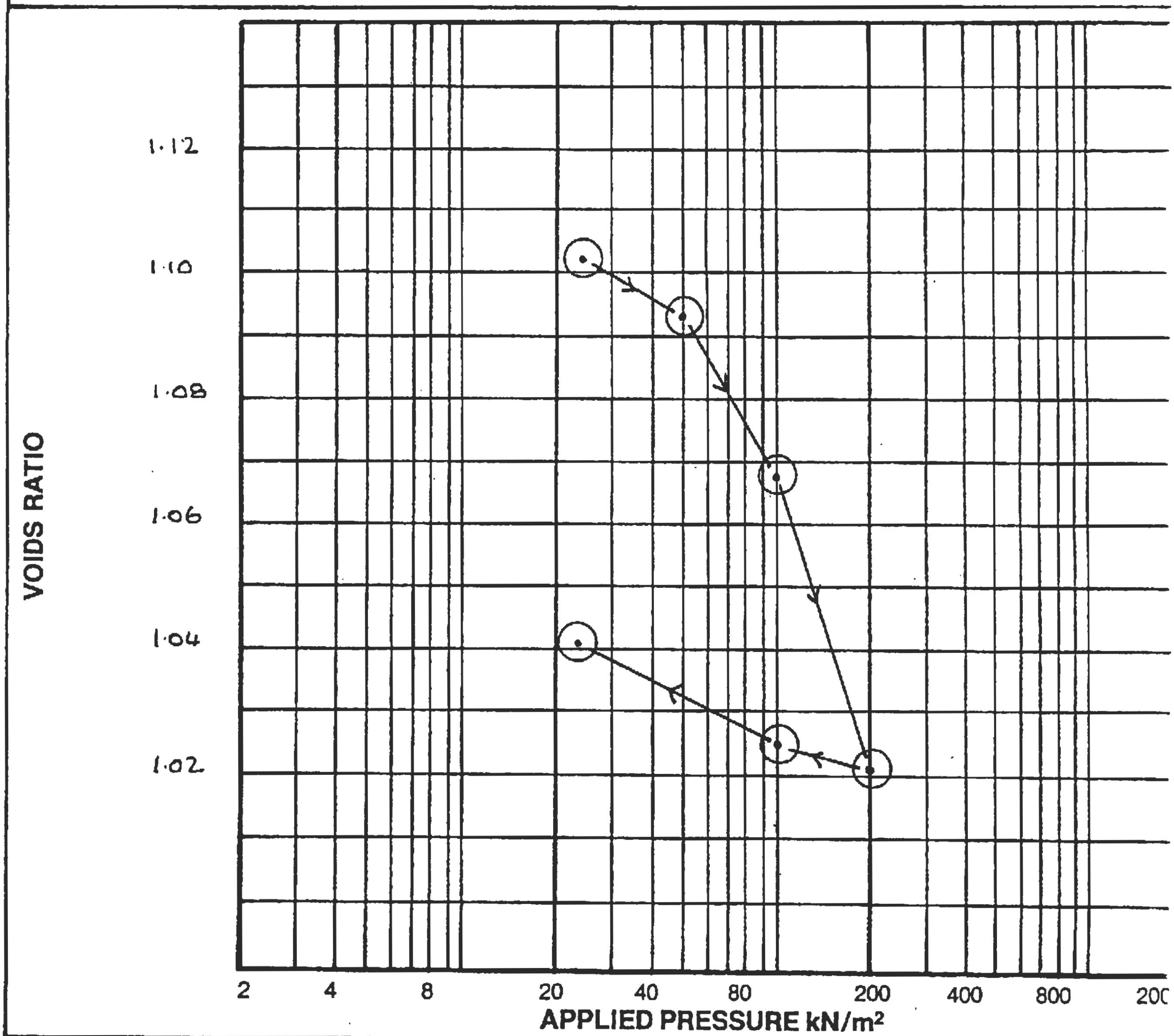
CONSOLIDATION TES
RESULTS Figure A7

CONTRACT FARAHAANE PROJECT

Borehole	Depth (metres)
Sample 3	
Initial Voids Ratio	1.106
Specific Gravity	2.73 ASSUMED
Natural Moisture Content	37 %
Bulk Density	1.78 Mg/m³
Dry Density	1.30 Mg/m³
Degree of Saturation	92 %
Swelling Pressure	12.5 kN/m²

Pressure	Coefficient of Consolidtion cv (m²/yr)	Coefficient of Compressibility mv (m²/MN)
12.5-25.0	2.500	0.156
25.0-50.0	0.288	0.158
50.0-100.0	0.714	0.245
100.0-200.0	0.718	0.228

Remarks: Soft to Firm reddy brown sl organic silty CLAY



A.4 Scour Hole Investigation

The position of the river bank and hence the extent of the scour hole, in relation to the barrages, were determined by tacheometry. Even in January a boat had to be used to obtain the level of the river bed. River cross-sections were taken of the upstream and downstream scour hole every 10 m in the vicinity of the barrages and then every 30 to 40 m until the end of any signs of scour or bank instability and the river had regained its natural section. The cross-sections were used to contour the scour holes and are shown in Drawing Nrs 164701/78 and 82 in the Album of Drawings.

A diver was employed to determine the extent of undermining of the barrage aprons. This was done by probing and the results can be seen on the Drawings.

APPENDIX B

SETTLING BASIN DESIGN

B.1 Introduction

The settling basins at the head of the Farahaane and Gayweerow primary canals have been designed for mechanical clearance by either drag line or long reach hydraulic excavator. We have also investigated the possibility of clearance by scour sluices, as described in this Appendix. The Gayweerow site only is discussed since this is much larger than Farahaane (5.9 m^3/s compared to 2 m^3/s), but the criteria and procedure would be similar.

B.2 Design of Settling Basin

The layout and location of the basin is shown in Figure B.1. The length of the basin is largely dictated by the bends in the river downstream of the barrage. We have calculated the size of the basin on a maximum settling velocity of 0.2 m/s and the sediment samples taken downstream of the Qorioley and Falkeerow barrage during the 1987 field survey (see Appendix).

With the basin dimensions in Figure B.1, and for depths of flow in the basin of 1.0, 1.25, 1.5, 1.75 and 2.0 m, we have calculated the scour discharge for the following parameters:

Slope vs discharge
Velocity vs discharge
Bed shear vs discharge
Rate of sediment removal vs discharge.

The results are summarised in Figure B.2 and Table B.1.

The main objective is to select a basin capacity which will satisfy the following requirements:

a) Scour the sediment without scouring the basins sides;
b) Remove the sediment within the available scour period.

The difficulty in assessing the scour parameters is that most of the information relates to permissible bed shear and velocities in unlined canals. In this respect therefore, the sediment scour velocity should be larger than the permissible velocity, however, the permissible velocities refer to virgin soils which are generally more compact than the sediment deposited in the canal, especially if the sediment is flushed relatively frequently. In the case of preventing scour to the sides of the channel, the scouring velocities and tractive forces should be less than the permissible values. There is, however, the option to pitch or pave sides of the basin to prevent bank scour, in which case the cost has to be added to the scour basin costs. With regard to the rate of scour, we have to rely on the theoretical calculations based on the sediment particle sizes and concentrations. For these calculations we have used Bagnolds total

Figure B.1
Layout of Scouring Basin

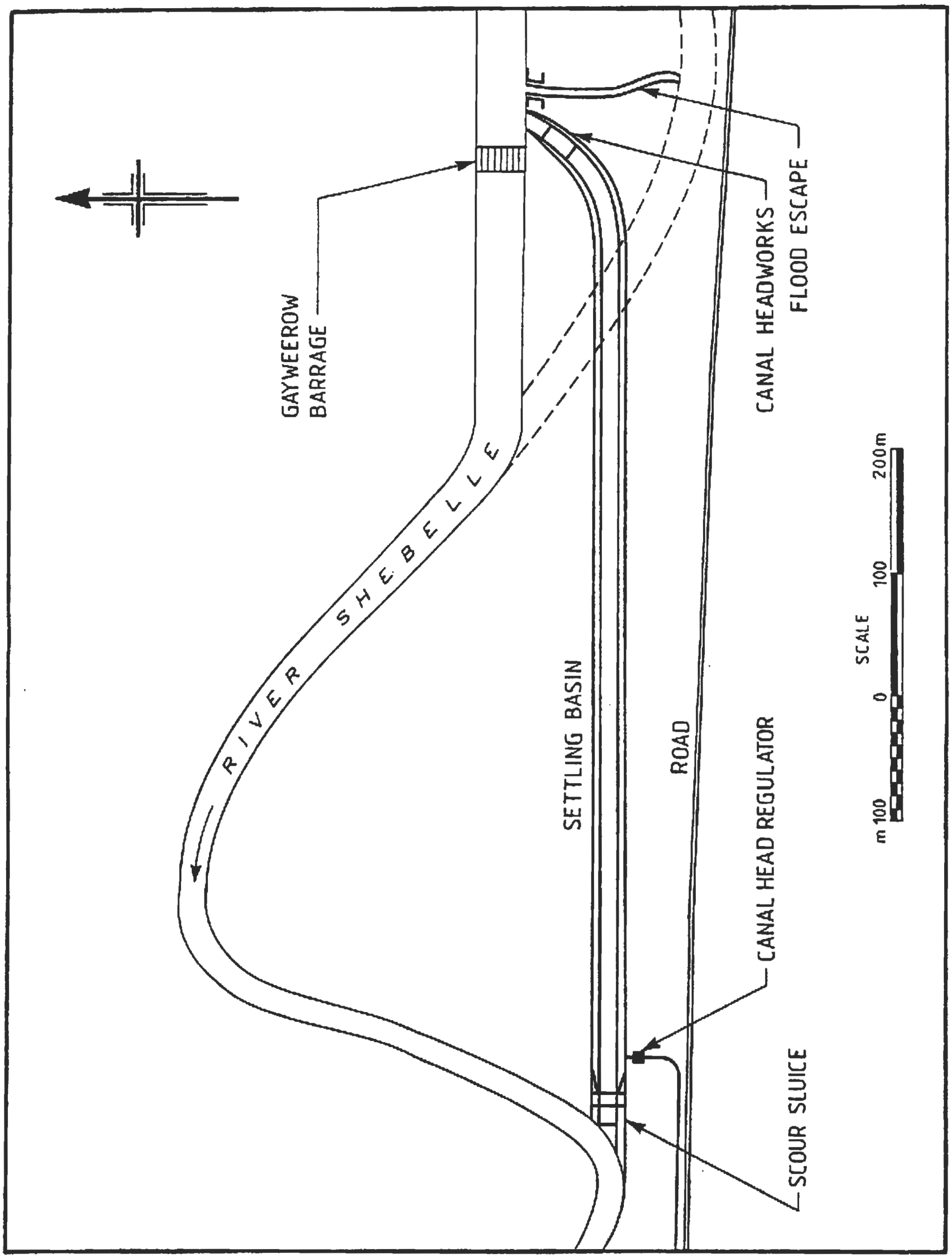
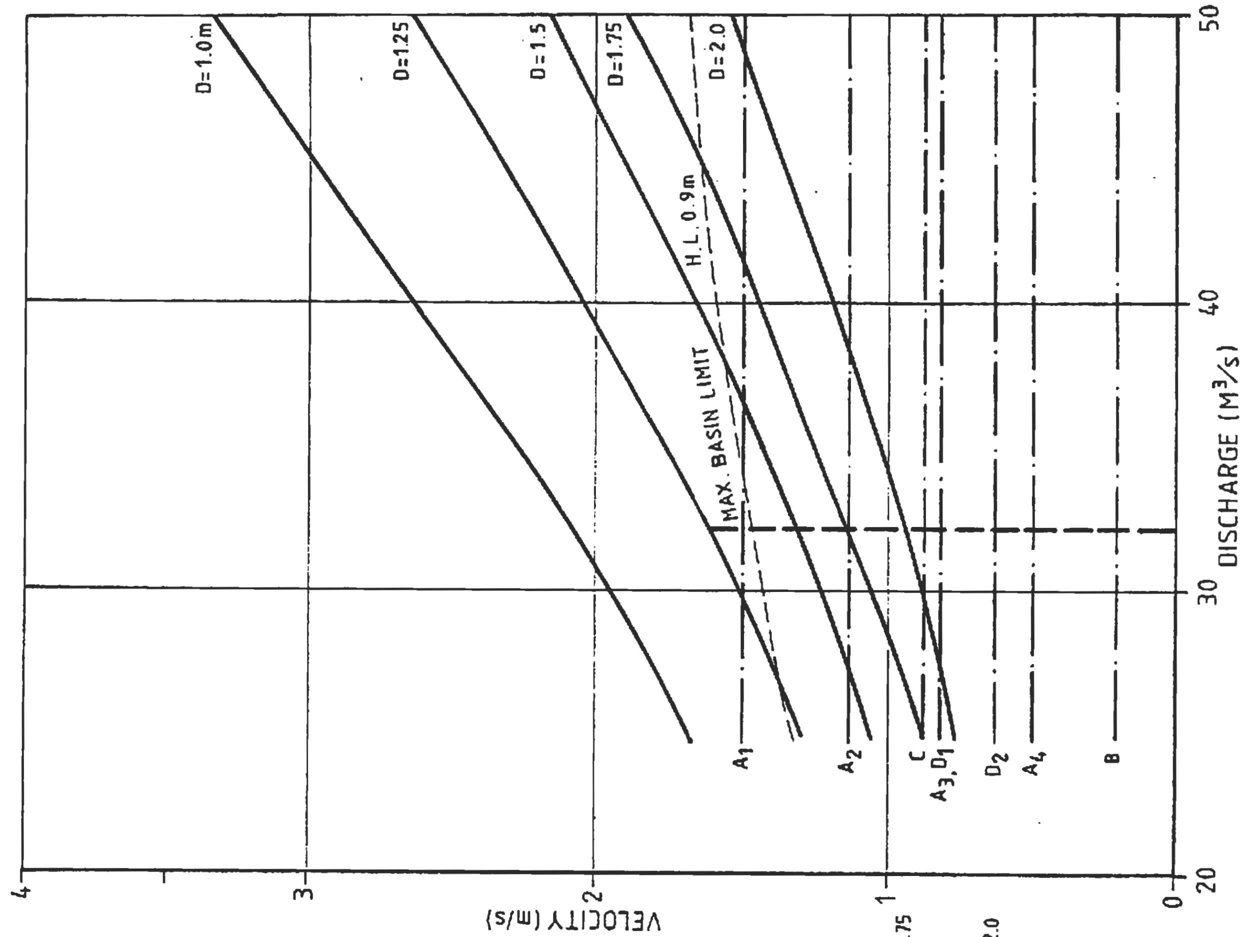
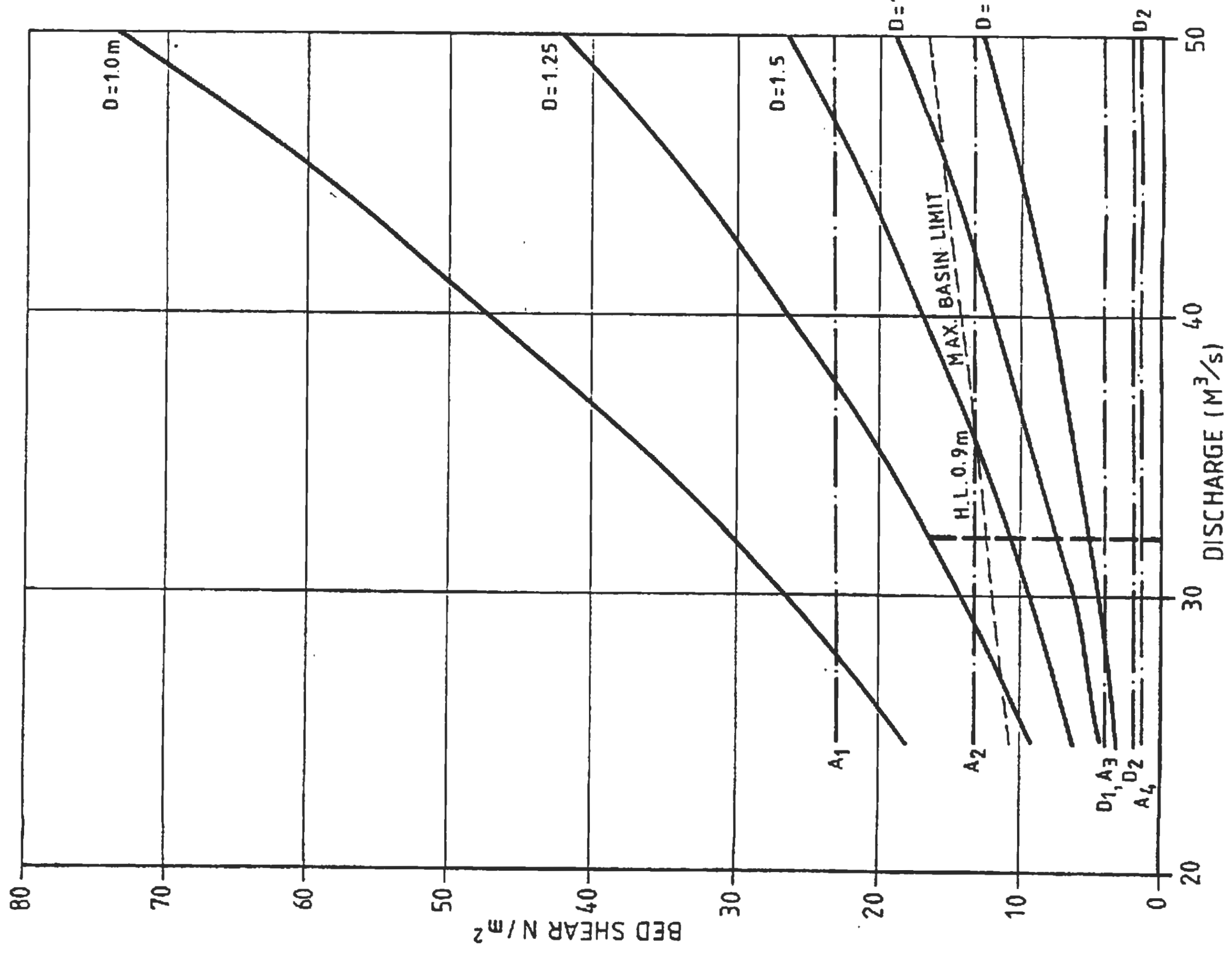


Figure B.2

Velocity and Bed Shear Against Discharge





KEY TO FIGURE B.2

LINE	DESCRIPTION	PERMISSIBLE VALUE VELOCITY (M/S)	BED SHEAR (N/M^2)
A1	STIFF CLAYS & VERY COOLIDAL ALLUVIAL SILTS COLLOIDAL (COLLOIDAL SILTY WATER)	1.50	22.50
A2	STIFF CLAYS & VERY COLLOIDAL ALLUVIAL SILTS COLLOIDAL (CLEAR WATER)	1.14	12.5
A3	FINE SAND (COLLOIDAL) (COLLOIDAL WATER)	0.76	3.6
A4	FINE SAND (COLLOIDAL) (CLEAR WATER)	0.45	1.3
B	SILTY SAND (GRAIN SIZE 0.1 MM)	0.24	
C	1987 CLAY SAMPLES FROM BARRAGES SANDY CLAY (VOIDS RATIO 1.0)	0.85	
	SHEBELLE CHANNEL	ESTIMATED VALUES	
D1	AVERAGE DISCHARGE 84 M^3/S	0.77	3.1
D2	AVERAGE DISCHARGE 48 M^3/S	0.63	2.3

REF:-	OPEN CHANNEL HYDRAULICS - VEN T CHOW
A	TABLE 7-3 MAX PERMISSIBLE VALUES FORTIER AND SCHOBY
B	FIGURE 7-3 PERMISSIBLE VELOCITIES NON COHESIVE SOILS - US & USSR DATA
C	FIGURE 7-4 PERMISSIBLE VELOCITIES COHESIVE SOILS - USSR DATA

load theory. The head loss across the barrage has been obtained from the difference in the pond levels between the various barrages as recorded in the Genale Bulo-Marerta Feasibility Report, MMP 1978. From these levels we estimate a head loss of 0.9 m across the settling basin. This is not related to the peak flood but average pond levels during the flood seasons. The Gayweerow barrage pond level is at the same bank top level as a cross section of the previous river channel before the construction of the barrage. The indication from this is that at the peak floods we would not anticipate much head loss at the barrage.

Using a cross section of the previous river channel at Gayweerow barrage, we have estimated the existing capacities, velocities and bed shear in the Shebelle. Velocities and bed shear values were estimated for bank full capacity for a value of Mannings 'n' of 0.0275 is approximately 84 m^3/s and for a water level of 1.0 m below bank top level the capacity is approximately 48 m^3/s. The velocity and bed shear (tractive forces) are shown on the lines D_1 and D_2, Figure B.2.

We have also plotted the values of the permissible channel velocities (line C, Figure B.2) based on soil samples taken at the Falkeerow and Qorioley barrages in the 1987 field survey. We have also plotted permissible channel velocities and bed shear values in Figure B.2. From the collected information and rate of scouring, we have selected a preliminary scour capacity of 32 m^3/s, and have studied this in relation to scour depth falls across the basin, water depth velocities and bed shear. For comparison purposes, we have also considered a scouring capacity of 25 m^3/s. The results are shown in Table B.1. The various conclusions from this analysis of basin parameters and their possible implications are discussed below.

(a) Scour of Banks of Basin

We consider bed shear the best indication for scour. The maximum velocity, 1.46 m/s, exceeds the permissible velocity of 0.85 m/s for the typical sandy clays in the project area, which have a voids ratio of 1.00. The maximum bed shear of 12.4 N/m^2 compares with the river bed shear of 3.1 N/m^2. We consider therefore, that it will be necessary to pitch the sides of the basin in the scour zone.

(b) Settling Velocities

We have a maximum through velocity of 0.2 m/s. On the Shield curve this is equivalent to the threshold of movement of a sediment particle size of 0.063 mm.

(c) Bed Shear

The scouring velocities and bed shear should be more than adequate at maximum head loss across the barrage. The lower the head losses ie 0.76 and 0.49 m (Table B.1), the bed shear falls to 10.8 and 7.2 N/m^2 and velocities to 1.37 and 1.15 m/s. These compare with estimated river values of 0.77 m/s and 3.1 N/m^2. We consider these values are adequate to scour the sediment in the basin, however, the rate of scouring will be reduced. This is discussed under Section (d).

Table B.1

Settling Basin Scour Parameters

Capacity (m^3/s)	Head Across Works (m)	Depth of Flow (m)	Velocity (m/s)	Bed Shear (N/m^2)	Time for Removal Daily (hrs)	Time for Removal Weekly (hrs)
)	0.90	1.42	1.46	12.40	1	7
32)	0.76	1.50	1.37	10.80	2	15
)	0.49	1.75	1.15	7.20	4	28
)	0.90	1.23	1.34	10.80	2	15
25)	0.50	1.50	1.07	6.50	5	34
)	0.38	1.75	0.90	4.05	8	60

(d) Rate of Scouring

As described above, the rate of scouring is based on Bagnolds Total Load theory. The amounts of sediment deposited in the basin are based on sediment concentrations taken during the 1987 field survey. This may not be the maximum sediment concentration, but not all this sediment will be deposited in the basin, as a large proportion of the wash load concentration will pass through the basin.

The general conclusion from Table B.1 is that at high floods with lower head losses it may be necessary to scour every night, at lower floods, however, and probably for most of the time, it will probably only be necessary to scour once a week.

There is a variation in the depth of scour at different heads across the basin and it may be advisable to scour relatively frequently to prevent residual sediment deposits building up in the basin.

(e) Scour Operation

The scour operation depends to a large extent on the type of gates proposed. For the Qorioley and Falkeerow barrages, the proposed gate travel time is approximately 15 cms/min. For the existing worm gear gates at the Gayweerow barrage, the travel time is probably 2.5 cms/min. In order to operate the scouring facilities it would be necessary to regulate the Gayweerow barrage gates so that flow through the barrage is reduced by the scouring discharge of 32 m^2/s. It is estimated that this would take approximately 1.5 to 2 hours assuming 3 gates are closed concurrently. Thus the total time for shutting and opening would be 3 to 4 hours, assuming 3 gatemen are employed on the barrage. With an 8 hour night shutdown of the canals this would leave 4 to 5 hours scouring time. The operation also requires one man operating the headworks and basin scour gates, which would need to be provided with counterweights to reduce the operating time of the gates.

The scouring procedure should be as follows:

(i) The canal head work gates will probably be part open prior to the scour operation. These should be closed and the canal head regulator gates (near the scour sluices) closed and scour sluice gates opened to balance 2 water levels, in the river and basin;

(ii) After the above the main scour operation commences;

(iii) The barrage gates are then closed in increments in phase with the opening of the headworks gates to maintain a constant barrage pond level.

On the completion of the scouring mode, more or less the reverse process takes place.

The scour operation is also complicated by the fact that the flow time intervals from the barrage to the basin outfall, via the river and basin, are different. These time intervals are approximately 60 minutes and 12 minutes respectively. This is likely to give flow variations in the river, which will make the head pond of the downstream barrage more difficult to control.

B.3 Estimate of Cost

We have made a comparative estimate of cost of a scouring basin, of 32 m^3/s capacity. This is compared with the project basin with mechanical removal of sediment. These costs are shown in Table B.2.

Table B.2

Additional Cost of Providing Scouring Facilities at Gayweerow

	$
Headworks	250 000
Scour Sluice	200 000
Lining to Settling Basin	600 000
	$1 050 000

B.4 Conclusion

This note shows that provision of a settling basin with scouring facilities may provide a feasible alternative to a settling basin with mechanical removal of sediment. However, we have certain reservations, which are discussed overleaf.

1) It is essential to obtain more information on the river and sediment during the next Gu and Der floods. This should consist of the following:

   a) Daily head losses across the Gayweerow and Qorioley barrages;

   b) Corresponding daily gate openings at the Gayweerow barrage;

   c) In conjunction with the above, daily sediment samples downstream of the Gayweerow barrage;

   d) Three cross-sections each upstream and downstream of Gayweerow barrage, with water levels recorded at representative high, medium and low floods.

2) The crucial criteria is the rate of scouring of sediment. Inevitably it is necessary to rely on theoretical calculations. In employing this method of calculation, the error factor can be quite high. The rate of scour could either be underestimated or overestimated.

   Inevitably in this situation it is advisable to err on the safe side.

3) The difficulty of operating the 'scour mode' for the basin should not be underestimated, especially at the Gayweerow barrage. The reasons for this are:

   a) It takes place virtually all night, with a period of scouring between gate adjustments;

   b) The time required to operate the existing Gayweerow gates with their very slow travel time will make the scour operation very difficult;

   c) At Gayweerow the scouring discharge of 32 m^3/s represents a large proportion of the river flow. The removal of such a large proportion of the river flow inevitably presents difficulties, especially combined with the different times of flows between the barrage and the basin outfall to the river.

4) The estimated capital cost of the 'scour clearance' is considerably more than that for 'mechanical clearance'. The estimated difference in cost at Gayweerow is about $1 million.

In view of the above, we consider that at this stage it is not advisable to proceed with the scour clearance basins unless the data collection is carried out and proves to be positive.

APPENDIX C

SEDIMENT INVESTIGATIONS

During the 1987 field survey, sediment investigations were carried out in order to obtain the sediment concentrations and particle size distribution of the sediment transported in the river and also the particle size distribution in the bed sediment of the canals.

This investigation consisted of bed sediment samples from 6 canals, and 4 sediment samples from the boil downstream of the Gayweerow and Qorioley barrages, the barrage samples giving the total sediment loads at these locations. The samples were divided into wash load (particle size less than 0.063 mm) and bed material load (particle sizes greater than 0.063 mm).

The particle size distribution from the canal samples are shown in Figures C.1 to C.5. It can be seen from these results that virtually all the sediment in the canal bed consists of wash load. The normal conclusion that would be drawn from this, without any other evidence, is that there should not be a serious sediment problem.

The possible reasons for the sedimentation in the canals is discussed later.

In the case of the total sediment samples taken from downstream of the barrages, the particle size distribution is shown in Figures C.6 to C.9. The sediment concentration of the river samples are shown in Table A1. Samples in Figure C.8 and C.9 were taken at the two barrages on the same day during a falling flood. In both cases the samples consisted entirely of wash load. The samples in Figure C.6 and C.7, were taken from the the 2 barrages on the same day when there was a relatively high flood in the river. From the inspection of the particle size distribution and Table C.1, it can be seen that the Gayweerow barrage sample had 27% bed material load and the Qorioley barrage sample had 45% bed material load.

These field samples were, of necessity, taken during the 1987 Der floods. Samples were taken during the 1977 'Der' flood during the MMP Feasibility Study for the Genale-Bulo Marerta Project. The sediment concentrations of these samples are shown in Table C.1.

CANAL BED SAMPLE
(Particle size distribution)

Location :- WADAJIR CANAL AT ROAD BRIDGE Km 0.70

Date :- 20.10.87

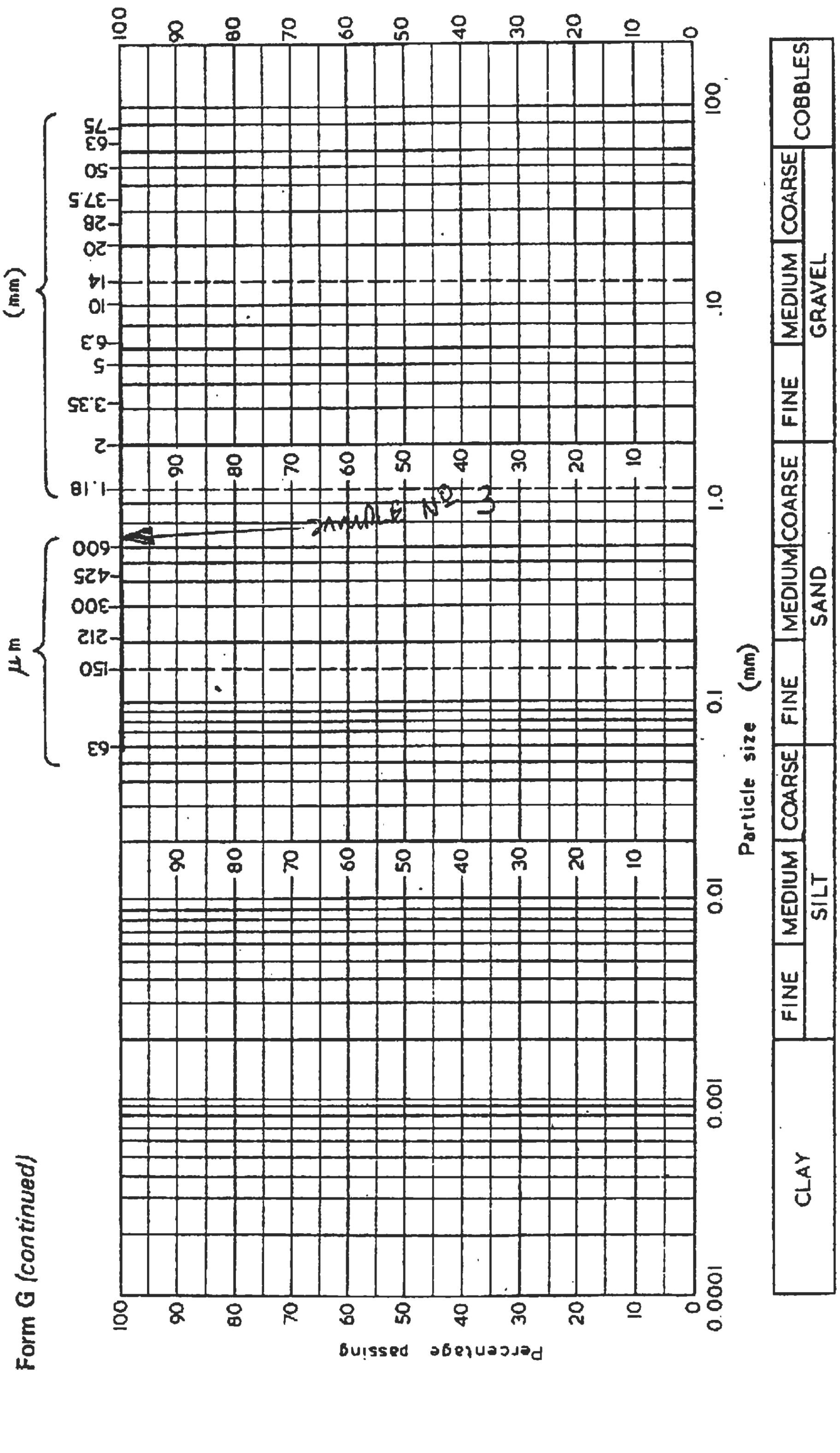


Figure C.2

CANAL BED SAMPLE
(Particle size distribution)

Location :- BAR GOOYE CANAL Km. 0.00

Date :- 20.10.87

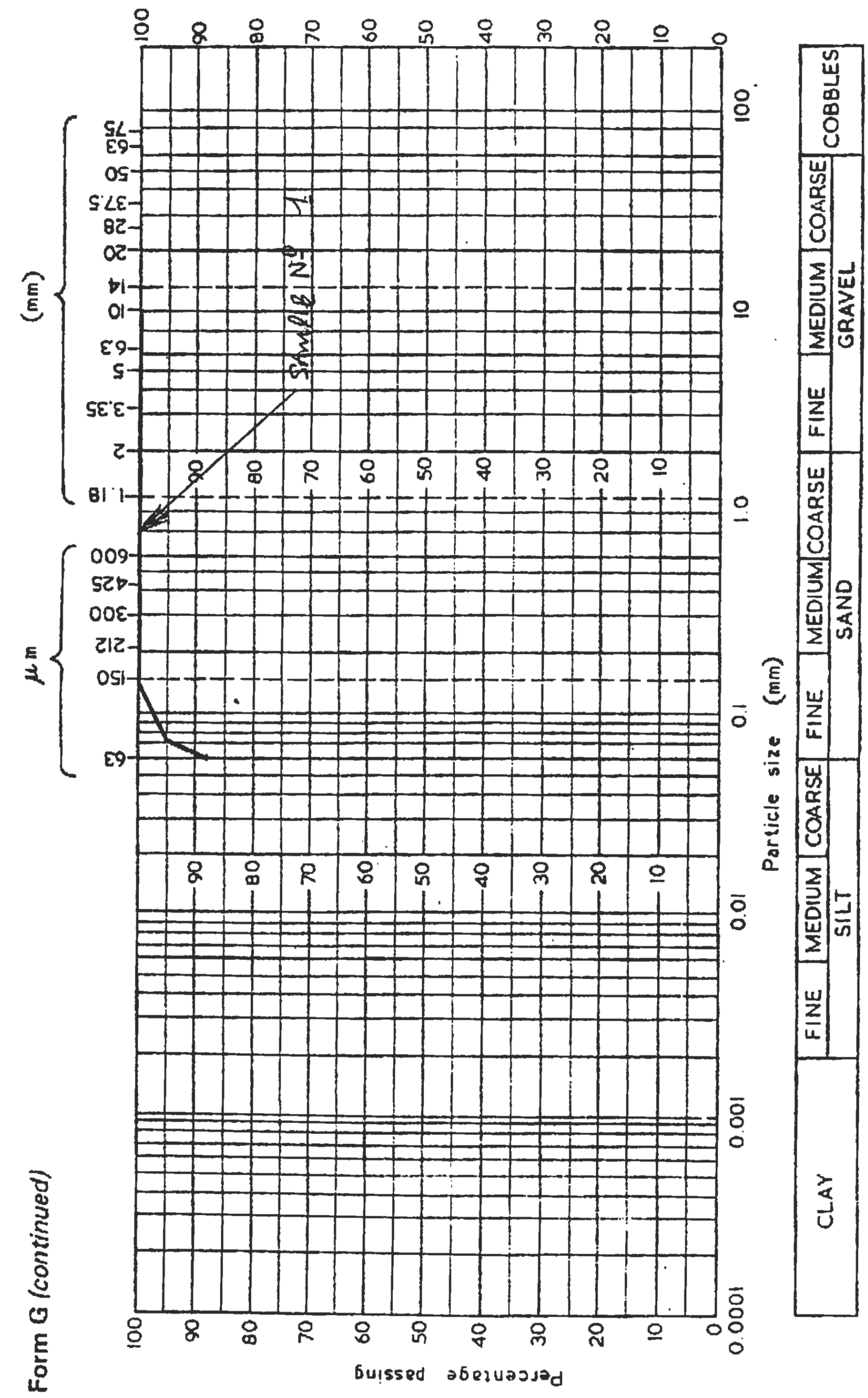
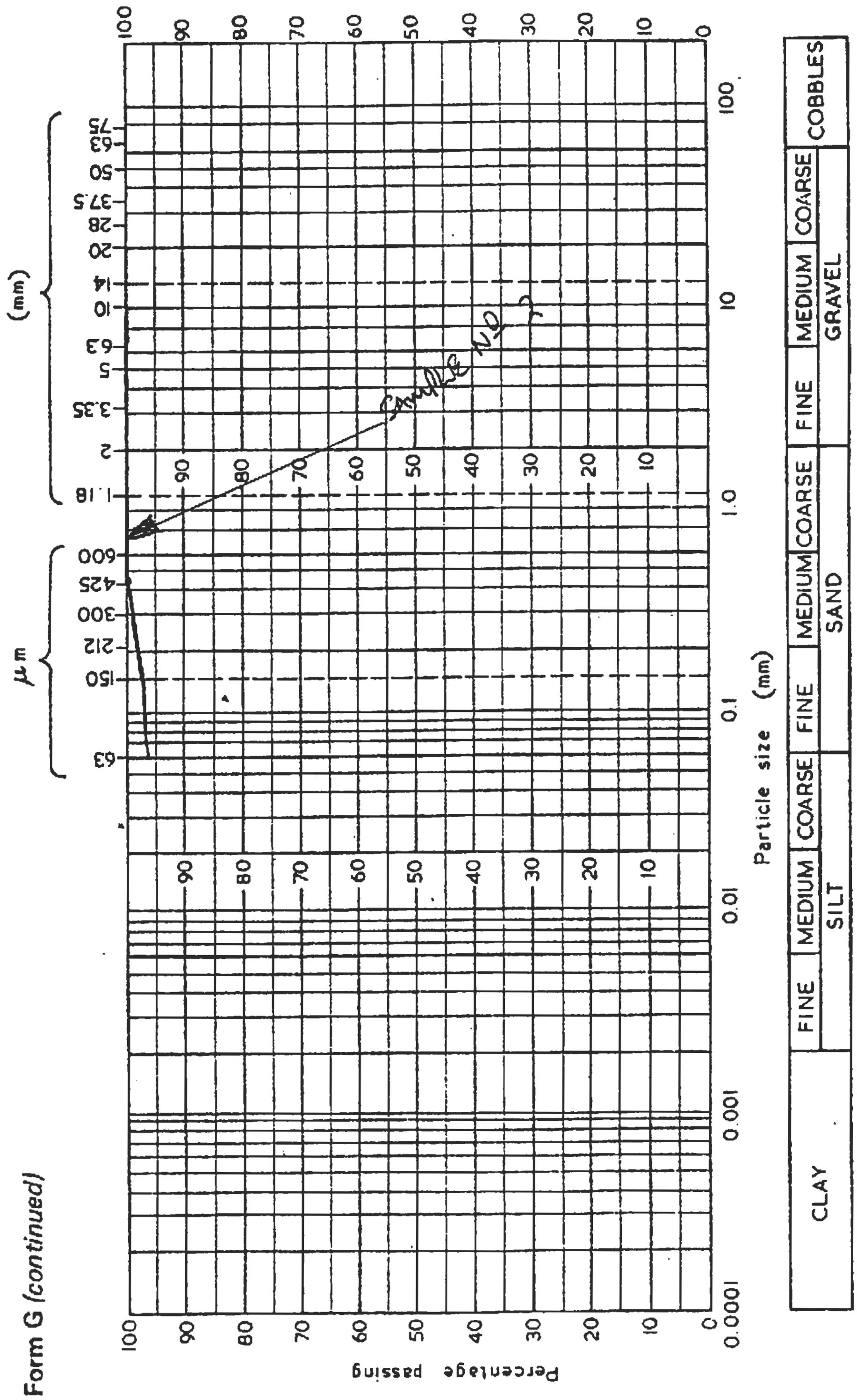


Figure C.3

CANAL BED SAMPLE
(Particle size distribution)

Location :– U/S FARAHAANE CANAL HEAD REGULATOR

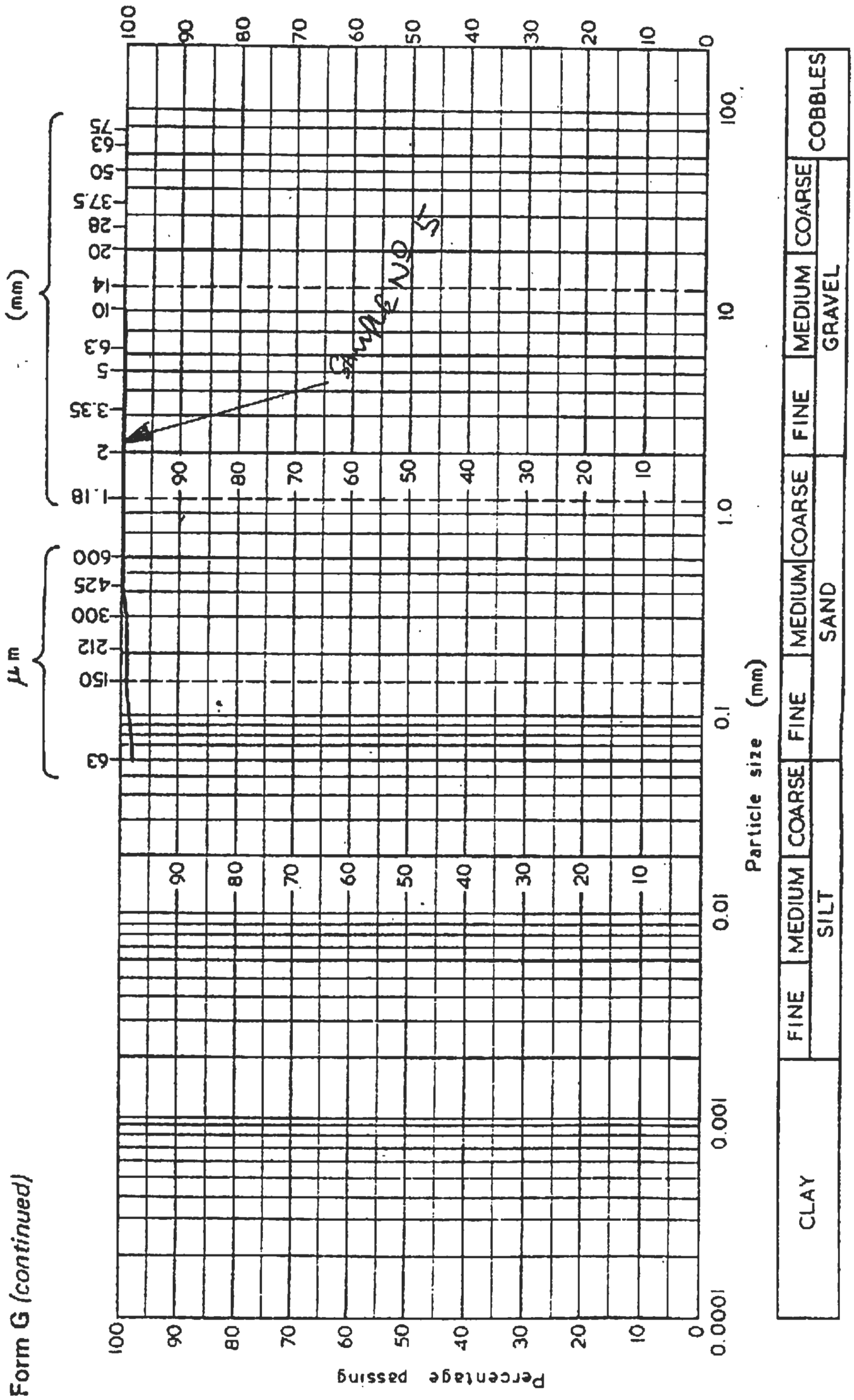
Date :– 22.10.87



CANAL BED SAMPLE
(Particle size distribution)

Location :- FARAHAANE CANAL Km 0.50

Date :- 22.10.87



CANAL BED SAMPLE
(Particle size distribution)

Location :- FARAHAANE CANAL D/S OF BRIDGE, Km 3.00

Date :- 22.10.82

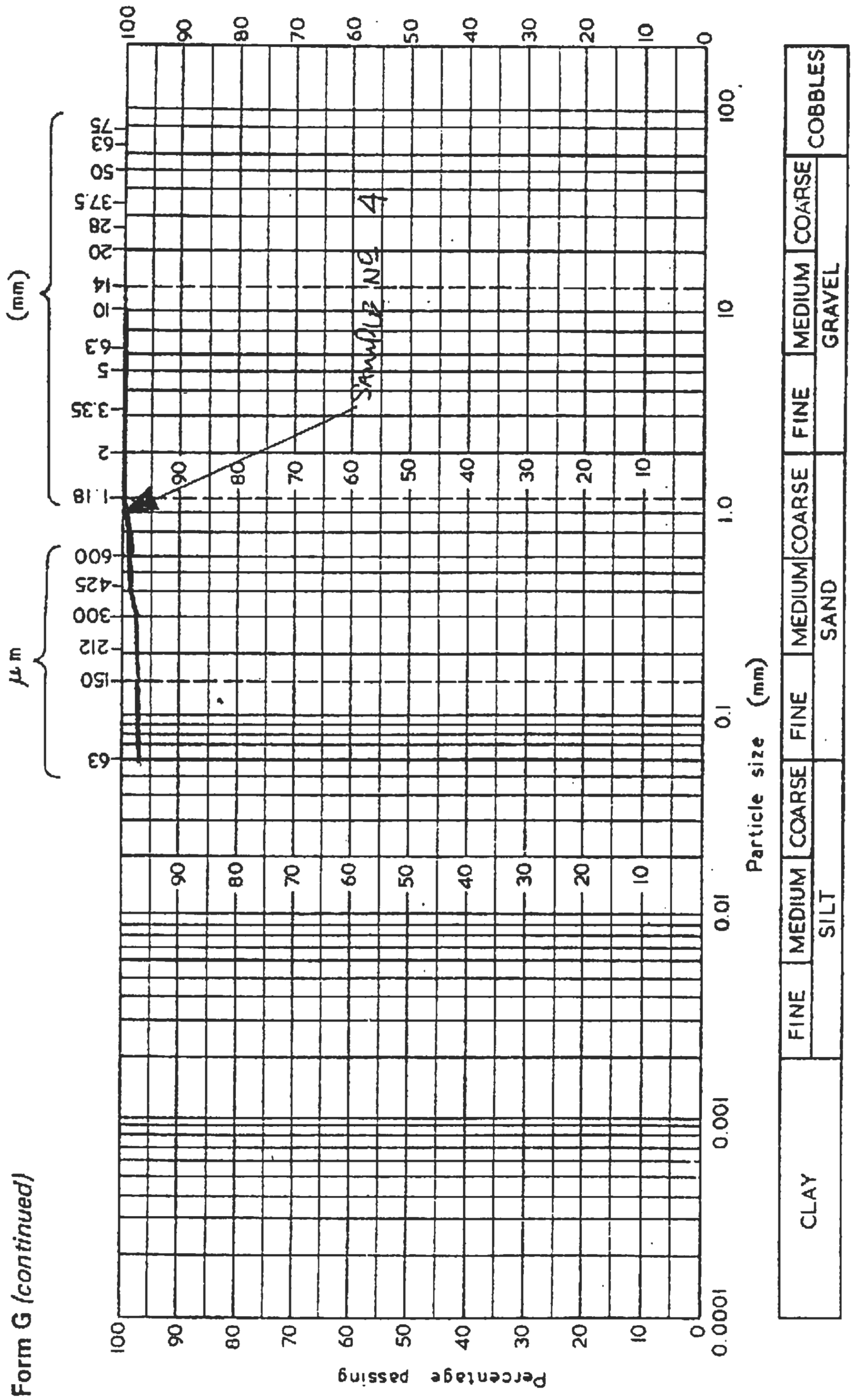


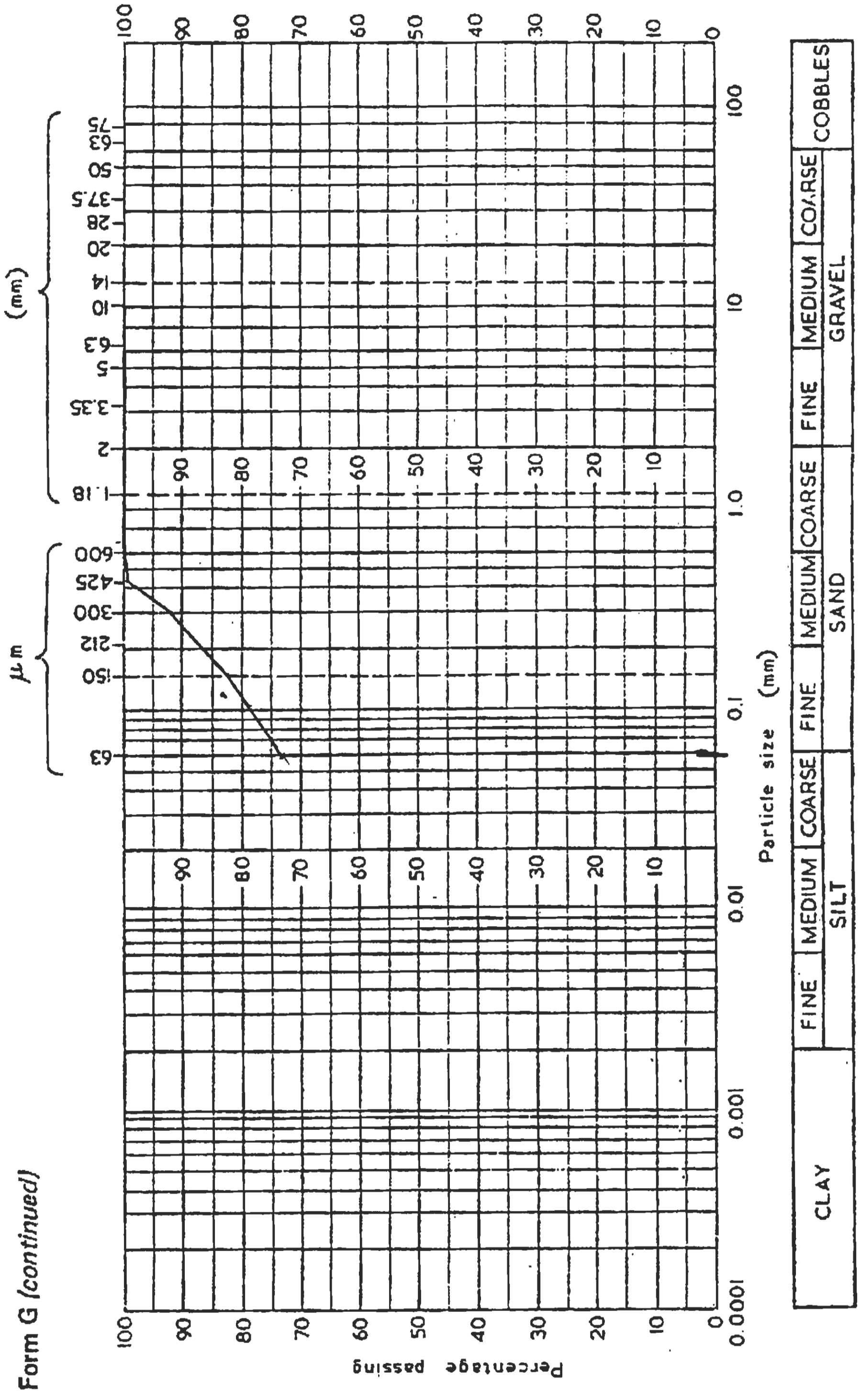
Figure C.6

SHEBELLI TOTAL LOAD SAMPLE
(Particle size distribution)

Location :- GAYWEEROW BARRAGE

Sediment Concentration :- 6325 ppm

Date :- 25.10.87



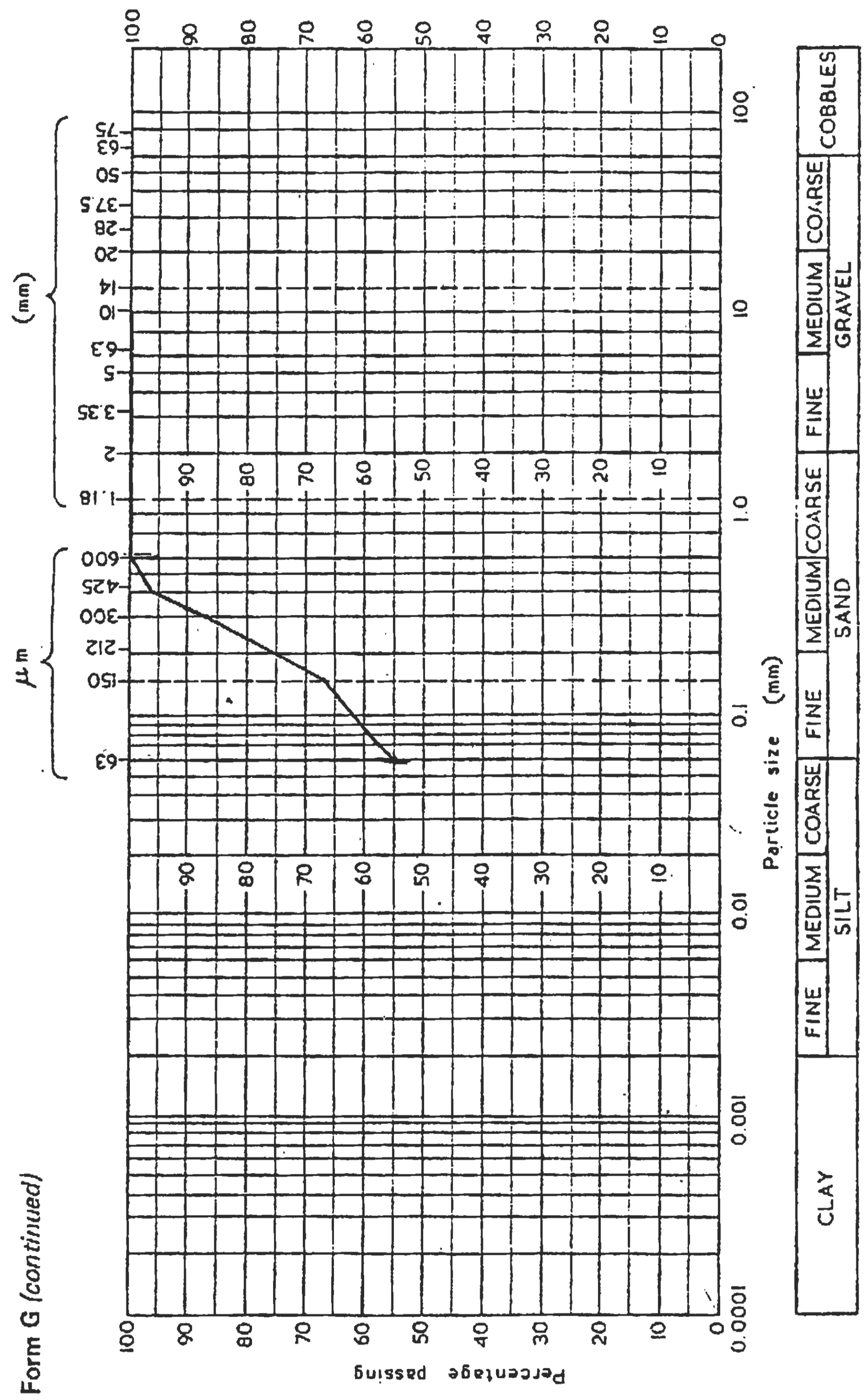
Form G (continued)

SHEBELLI TOTAL LOAD SAMPLE
(Particle size distribution)

Location :- QORIOLEY BARRAGE

Sediment Concentration :- 5482 ppm

Date :- 25.10.87



Form G (continued)

SHEBELLI TOTAL LOAD SAMPLE
(Particle size distribution)

Location :- GAYWEEROW BARRAGE

Sediment Concentration :- 2469 ppm

Date :- 11.11.87

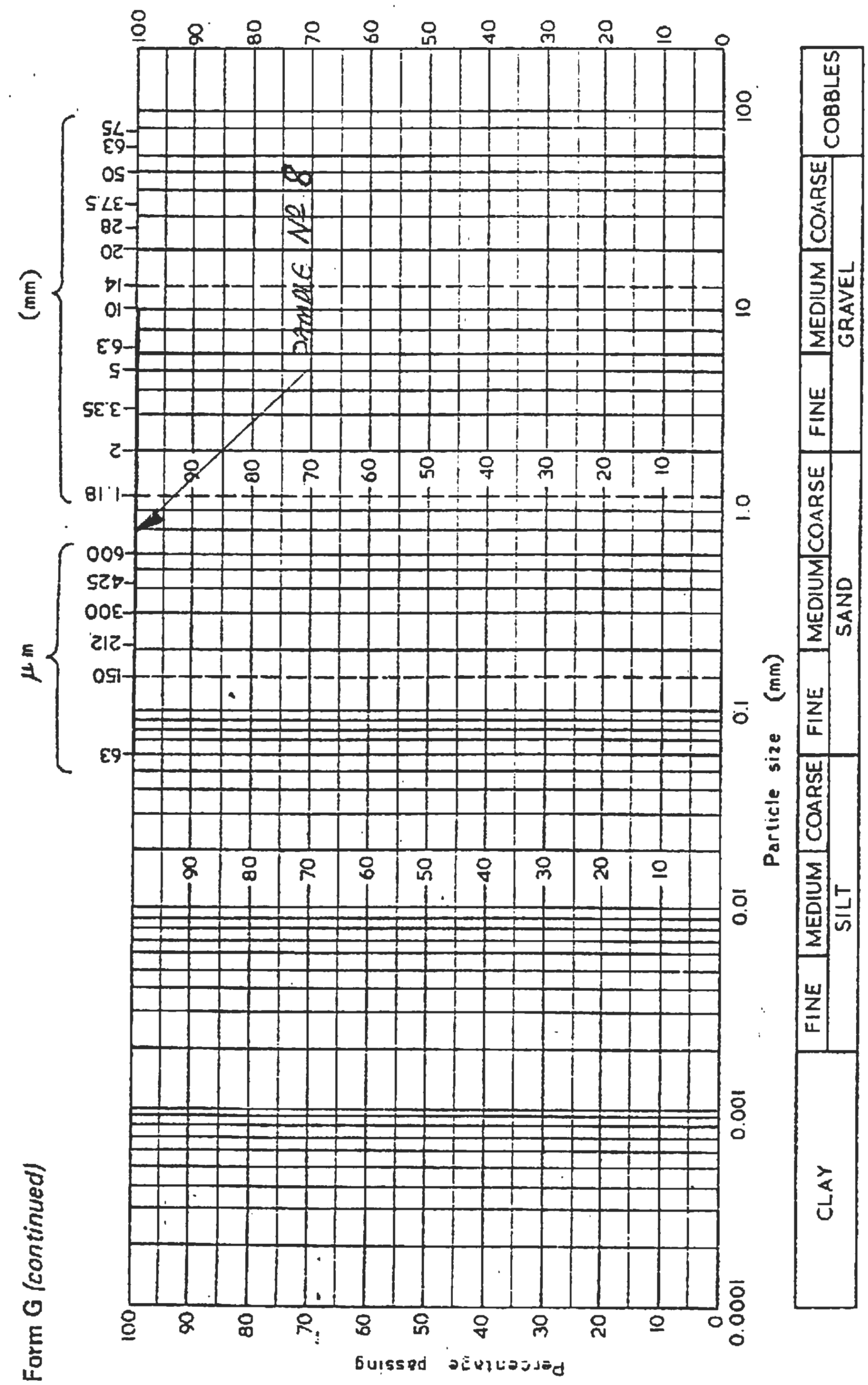


Figure C.9

SHEBELLI TOTAL LOAD SAMPLE
(Particle size distribution)

Location :- QORIOLEY BARRAGE

Sediment Concentration :- 2632 ppm

Date :- 11.11.87

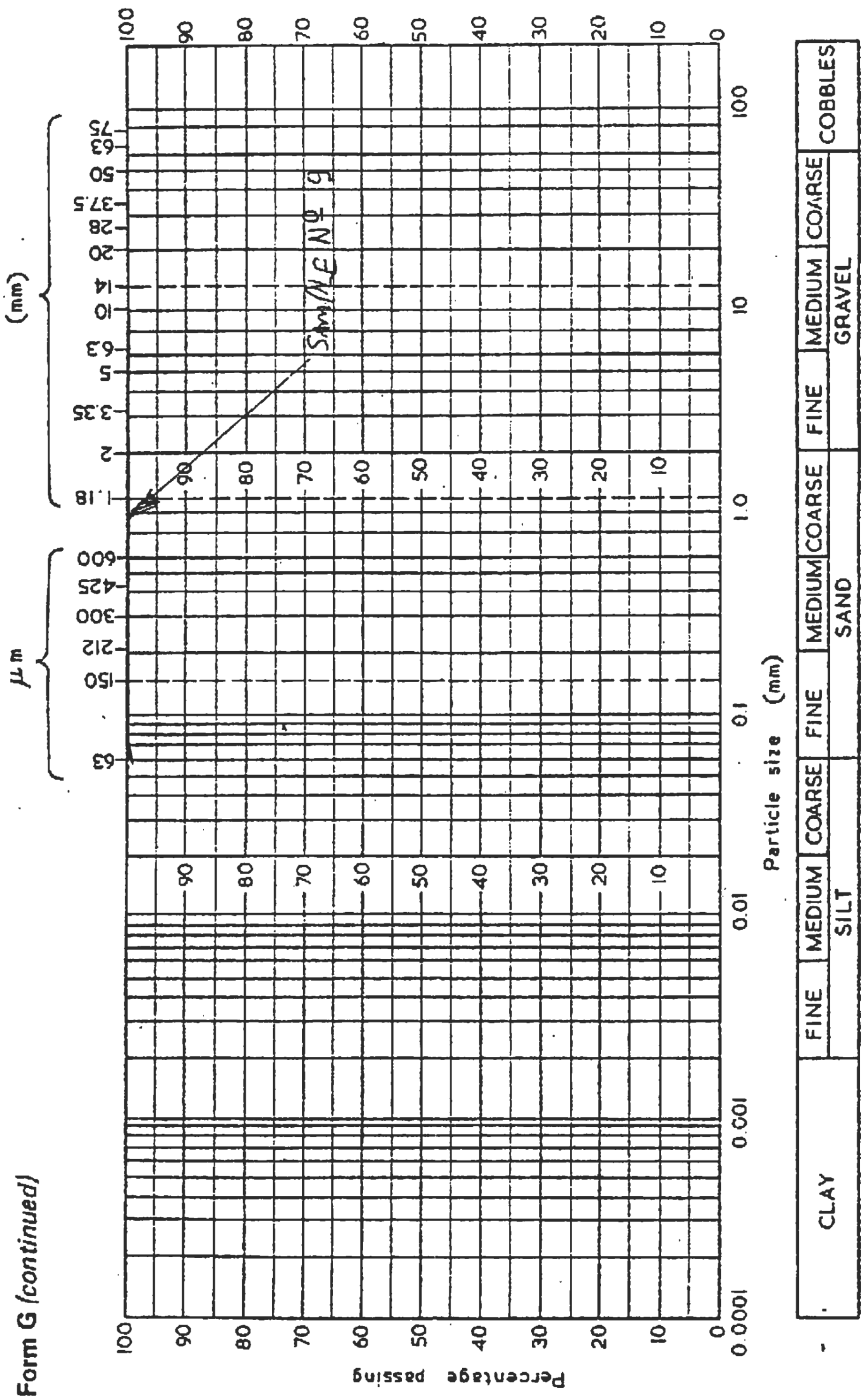


Table C.1

Shebelle Sediment Concentrations

1987 Sediment Concentrations

Date	Sediment Concentration (ppm)	Remarks
25 Oct	6 325	Gayweerow Barrage (High Flood)
25 Oct	5 482	Qorioley Barrage (High Flood)
11 Nov	2 469	Gayweerow Barrage (Falling Flood)
11 Nov	2 632	Qorioley Barrage (Falling Flood)

Sediment Concentrations at Majabto (1977/78)

1977		
19 July	2 179	
25 July	3 357	
01 Aug	3 604	
08 Aug	2 582	
15 Aug	2 661	
22 Aug	3 464	main body of der flood
29 Aug	3 841	
05 Sep	2 776	
13 Sep	2 544	
21 Sep	3 340	
26 Sep	2 768	
04 Oct	2 793	
12 Oct	2 078	
18 Oct	2 324	
24 Oct	6 569	period just after heavy
31 Oct	7 328	local rains
07 Nov	2 260	
14 Nov	1 309	
22 Nov	1 191	
28 Nov	831	
05 Dec	759	
15 Dec	760	
21 Dec	1 344	
29 Dec	No record	
1978		falling water flow
07 Jan	839	
17 Jan	444	
22 Jan	300	
30 Jan	48	practically no flow in the
05 Feb	56	river

If the relationship between the sediment concentrations and particle size distribution recorded in the 1987 sediment survey is used to assess the 1977 sediment sample concentrations, then most of the samples taken at that time would consist of wash load. The 1977 series of samples were recorded at weekly time intervals and do not necessarily coincide with flood peaks, when we can anticipate higher discharges and bed material load concentrations.

Because of the very high proportion of wash load recorded in the bed of the canals, with an absence of bed material load, it is necessary to consider to what extent there is a sediment problem and how best to solve it.

What we consider could be happening at present, is that most of the bed material load is carried down the Shebelle during a rising flood when the discharges and mechanics of flow are capable of transporting the bed material load. In the Shebelle, where there are numerous barrages which pond the water level up to the bankfull level, the natural sediment transport regime probably only occurs during high discharges. On the flood recession, as the discharge falls, the velocities upstream of the barrages are reduced, and the reach of the river channel upstream of the barrages act as settling basins where the bed material load is deposited in the river bed. In the succeeding flood, in addition to the bed material load being carried by the flood, the bed material load deposited during the previous flood could be picked up. We could therefore have the situation where the bed material load, which is to a large extent discharge orientated, occurs only at higher floods and in high concentrations.

At present the control of the river and canals is extremely difficult, because of the damage to the barrage gates, and lack of gate opening indicators and accurate water level recorders. This makes it is extremely difficult to control the upstream water levels within reasonable limits. With the low afflux canal intake regulators this can result in excessive fluctuations of discharge in the canals. These fluctuations can increase the amount of sediment deposited in the canals. In addition to this if the canals are shut down at night with a falling canal discharge, more wash load is likely to fall out in the canals. At present, with a system of individual canals, the cultivators can shut down their canals during floods, which would explain the lack of bed material load. With larger feeder canals this type of operation becomes more difficult.

Up to now, because of the poor state of the various barrage gates and insensitive recording systems it has been very difficult to assess the bed load sediment concentrations in the river, in relation to river discharge.

In the Farahaane Contract we have installed accurate gate opening indicators at Falkeerow and Qorioley barrages. This will enable the response time of the fluctuation in upstream pond levels to be gauged more accurately, and should assist in controlling the upstream barrage pond levels. The difficulties of controlling the upstream pond levels should not be underestimated, especially if it is considered in relation to the available skilled staff for operating the barrages.

APPENDIX D

ESTIMATE OF LAND ACQUISITION REQUIREMENTS

a) Primary and Secondary Canals

Canal	Chainage (m)	Existing Canal Width (m)	Required Canal Width (m)	Land Acquisition Area (ha)
Farahaane Primary	050	-	21	0.01
	270	-	40	0.88
	605	-	23	0.77
	805	-	22	0.44
	1100	16	19	0.08
	1450	15	18	0.11
	2000	20	21	0.05
	2720	14	19	0.36
	3105	16	19	0.11
	3405	8	17	0.27
	3970	4	16	0.68
	4590	8	16	0.50
	4950	7	16	0.32
				4.58
Gayweerow Primary	260	-	25	0.65
	590	-	50	1.65
	650	-	25	0.15
	1450	-	24	1.92
	2000	-	26	1.43
	2475	-	22	1.04
	3250	-	22	1.71
	3735	-	22	1.07
	4020	-	21	0.60
	4255	-	22	0.52
	4790	-	22	1.18
	5665	-	17	1.49
	6315	-	18	1.17
	7000	-	18	1.23
	7410	-	18	0.74
				16.55

Canal	Chainage (m)	Existing Canal Width (m)	Required Canal Width (m)	Land Acquisition Area (ha)
F1	450	8	11	0.14
	1215	9	11	0.15
	1545	10	11	0.03
	1890	10	12	0.07
	2525	9	12	0.19
	2920	8	13	0.20
	3195	9	11	0.06
	3385	9	11	0.04
	3550	8	12	0.07
	3755	9	11	0.04
	3990	9	11	0.05
	4540	11	13	0.11
	5330	10	12	0.16
	5755	8	12	0.17
				1.48
F2	280	11	13	0.06
	960	7	11	0.27
	1885	8	11	0.28
	2560	6	11	0.34
	2730	1	10	0.15
	2940	1	9	0.17
	3410	8	11	0.14
	3880	10	12	0.09
	4080	8	10	0.04
				1.54
F3	1050	-	11	1.16
	1575	9	11	0.10
	2150	9	12	0.17
	3180	14	15	0.17
				1.60

Canal	Chainage (m)	Existing Canal Width (m)	Required Canal Width (m)	Land Acquisition Area (ha)
G1	760	-	15	1.14
	1245	-	14	0.68
	1585	-	12	0.41
	2060	-	13	0.62
	2635	-	13	0.75
	2840	-	11	0.23
	3045	-	11	0.23
	3695	-	11	0.72
	4880	-	17	2.01
	5730	-	17	1.44
	6205	-	11	0.52
	6480	-	11	0.31
				9.06
G2	325	7	13	0.19
	890	7	12	0.28
	1190	7	11	0.12
	1250	7	10	0.02
	1485	9	11	0.05
	1945	9	12	0.14
	2820	8	10	0.17
	3730	11	12	0.09
	4000	14	12	-
				1.06
G3	105	7	14	0.07
	450	9	14	0.14
	950	10	13	0.15
	1440	8	12	0.20
	1695	10	13	0.08
	1750	9	14	0.03
	1800	11	15	0.02
	2005	8	10	0.04
	2450	8	11	0.13
	2950	9	12	0.15
	3450	8	12	0.20
	3960	8	12	0.20
	4460	7	11	0.20
	4975	7	16	0.46
	5410	7	14	0.30
	5870	6	12	0.28
	6170	4	13	0.27
				2.92

Canal	Chainage (m)	Existing Canal Width (m)	Required Canal Width (m)	Land Acquisition Area (ha)
G3-1	320	2	10	0.26
	845	7	9	0.10
	1365	7	10	0.16
	1920	11	11	-
	2420	8	11	0.15
	2920	8	11	0.15
	3420	7	11	0.20
	3670	7	11	0.10
				1.12
G3-2	645	9	13	0.26
	1225	8	16	0.46
	1750	9	19	0.52
	2250	11	18	0.35
	2750	10	19	0.45
	3250	10	21	0.55
	3750	8	21	0.65
	4375	9	17	0.50
	5125	11	21	0.75
	5700	10	15	0.29
	6075	10	12	0.08
	6315	-	12	0.29
	6390	-	12	0.09
	6410	-	11	0.02
	6420	-	11	0.01
				5.27
G4	285	6	9	0.08
	770	9	11	0.09
	1030	6	10	0.10
	1285	12	12	-
	1530	-	10	0.24
				0.27
G5	745	-	12	0.89
	1490	-	12	0.89
	1940	-	12	0.54
				2.32

b) **Tertiary and Quaternary Canals**

Estimated total length of new tertiary and quaternary canals	=	95 km
Average canal width	=	4.35 m
Hence area of land required	=	41 ha

Assume no additional land required for remodelled tertiary and quaternary drains.

c) **Primary and Secondary Drains**

Drain	Chainage (m)	Required Drain Width (m)	Land Acquisition Area (ha)
Primary	500	37	1.84
	1050	36	1.96
	1500	35	1.58
	2100	33	2.00
	3100	34	3.37
	4900	30	5.34
			16.09
D1	600	26	1.57
	960	26	0.94
	1280	26	0.83
	1800	26	1.37
	1950	26	0.39
	2200	25	0.62
	2900	24	1.71
	4450	25	3.93
	5000	24	1.33
	5600	23	1.41
	5950	24	0.83
	6500	25	1.35
	8100	24	3.78
	8900	23	1.83
	9900	23	2.26
	10500	22	1.30
			25.45
D1-1	1250	22	2.73
	1350	21	0.21
	2100	21	1.56
	2500	22	0.87
			5.37

Drain	Chainage (m)	Required Drain Width (m)	Land Acquisition Area (ha)
D5	580	27	1.54
	860	26	0.74
	1540	27	1.81
	1700	25	0.40
	2600	26	2.37
	2900	24	0.73
	3200	23	0.69
	3500	23	0.68
	5000	23	3.47
	5760	22	1.64
	6250	21	1.03
	6600	22	0.77
			15.87
D5-1	600	24	1.42
	1870	24	3.10
	2850	24	2.34
	2900	24	0.12
	3600	21	1.50
	3800	21	0.43
	4200	20	0.82
			9.73
D5-2	800	21	1.69

d) Tertiary and Quaternary Drains

Estimated total length of new tertiary and quaternary drains	=	450 km
Average drain width	=	5.0 m
Hence area of land required	=	225 ha

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