SOMALI DEMOCRATIC REPUBLIC
MINISTRY OF NATIONAL PLANNING

JOWHAR SUGAR ESTATE

Feasibility Study for Rehabilitation

Final Report

ANNEX I
Irrigation and Drainage

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GLOSSARY

Abbreviations used in the text :

SoSh	Somali Shilling.
DM	Deutschmark
$	United States Dollar
JOSR	Jowhar Offstream Storage Reservoir
DOSR	Duduble Offstream Storage Reservoir

Units :

The metric (SI) system of units has been adopted throughout. Some of the units used are explained below.

Mm^3	Million cubic metres
t/d cane	Tonnes of cane per day
t/h cane	Tonnes of cane per hour

Glossary of terms :

Main canal	The canal served by a river intake.
Secondary canal	A canal offtaking from a main canal.
Tertiary canal	A canal offtaking from a secondary canal and serving a field unit.
Quaternary canal	A canal offtaking from a tertiary canal (existing field layout).
Aquiole	A small temporary channel offtaking from a quaternary canal, from which each part of the field unit is irrigated.
Fascia	The strip of land within a field unit between two quaternary canals.
Drainageway	A small temporary drainage channel serving a fascia.
Collector drain	The drainage channel serving a field unit.
Branch drain	A drainage channel into which collector drains flow.
Main drain	A drainage channel into which branch drains and collector drains flow.
Field drain	A buried drainage pipe designed to remove subsurface water.
Header channel	A branch of a tertiary canal, serving part of a field unit, from which irrigation water is siphoned into the furrows (proposed field layout).

Anaerobic	Living in the absence of free oxygen (gaseous or dissolved).
Glycolysis	A complex system of converting glucose into pyruvic acid which is important in providing energy for short periods when oxygen is deficient.
Land levelling	Earth movement to achieve an even land surface to facilitate irrigation.
Land planing	Smoothing of the land levelled surface to remove irregularities.

ANNEX I - IRRIGATION AND DRAINAGE

This Annex is divided into five sections:-

Section A	Water Resources
Section B	Irrigation
Section C	Drainage
Section D	Operation and Maintenance
Section E	Costs

In Section A we examine the problems of water availability and water quality, both of which are significant in terms of reduced cane yields. Possible solutions to the problem of dry season water shortages are discussed.

Sections B and C deal with the existing irrigation and drainage systems and describe the alternative ways in which improvements can be made. The proposed rehabilitation measures are discussed in detail. Details of the investigative work carried out on the Estate during the fieldwork phase are presented.

Section D discusses the requirements for operation and maintenance of the rehabilitated scheme and presents details of staffing and plant. In Section E we present the cost estimates of the proposed rehabilitation measures, and recurrent costs for operation and maintenance.

SECTION A

WATER RESOURCES

CHAPTER 1

WATER QUALITY

1.1 Introduction

The water quality of the Shabelle river has been monitored intermittently for many years, not only at Jowhar, but also at other projects which make use of the water (e.g. Afgoi). The two parameters of interest to irrigated agriculture are chemical quality and suspended sediment. These are discussed separately below.

1.2 Chemical Quality

1.2.1 General

Chemical analyses of the Shabelle water were presented in the Drainage and Reclamation Study report (MMP, 1976). The data referred to samples taken at Afgoi in 1966, plus two samples taken at Jowhar in 1976. It was concluded that the water can be classified as C2-S1 to C3-S1 using the USDA classification. This indicates that there is a medium to high salinity hazard. It was also concluded that there would be no boron toxicity risk and little likelihood of a bicarbonate hazard.

1.2.2 Salinity

The greatest risk to irrigated cane production therefore comes from the high salinity of the water. Generally salinity levels are relatively low from June to November. As the der season flood recedes in December salinity levels start to rise and are frequently high during the dry season months of January to March. Exceptionally high values can be experienced with the early part of the gu season flood in April to May. This pattern is clearly indicated in the average monthly values for 14 years, in the period 1965 to 1983, presented in Table I.1.1. It should be noted that average monthly values of salinity do not mask large day-to-day variations. In general fairly constant values are recorded through the month, with changes occurring gradually except at the start of the gu season flood.

In fact there is a clear correlation between average monthly river flow and salinity values. In the months January, February and March if the average river flow is more than 10 m^3/s average salinity values rarely exceed 1 250 micromhos/cm. Conversely, when average flow in these months is less than 10 m^3/s the average salinity level is very rarely less than 1 500 micromhos/cm. This phenomenon is clearly illustrated by the data for 1980, probably the worst year for salinity on record. The pattern was as follows :

	Jan	Feb	Mar	Apr	May	Jun	Jul
Flow (% average)	50	34	16	13	94	42	76
Salinity (% average)	135	132	171	258	179	161	147

Source: Table I.1.1.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Salinity Records for the Shabelle River (micromhos/cm)(1)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1965	580	700	1 310	1 880	1 350	1 060	1 510	1 780	450	610	1 010	1 070
1966	1 100	1 690	1 240	610	1 070	850	470	400	480	480	960	880
1968	828	1 066	1 076	641	780	620	520	454	465	403	742	845
1973	-	-	-	3 350	1 990	850	830	370	360	470	860	930
1974	1 780	2 680	2 870	1 090	1 270	990	420	350	410	440	560	860
1975	1 470	2 390	2 850	2 690	1 170	1 600	900	630	540	420	630	1 180
1976	1 260	1 850	2 330	2 470	1 410	1 440	1 380	400	360	410	730	560
1977	1 170	900	690	1 400	1 480	960	720	620	400	450	570	639
1978	1 230	1 794	1 115	861	880	937	1 013	510	563	477	672	915
1979	1 280	888	633	881	887	1 010	1 038	853	585	603	457	1 106
1980	1 858	2 463	3 163	4 275	1 969	1 593	1 314	615	409	428	362	826
1981	1 656	2 273	-	997	807	837	1 353	663	497	463	569	1 431
1982	2 784	3 599	3 288	1 798	599	507	433	390	374	325	550	611
1983	613	990	960	806	417	-	-	-	-	-	-	-
Average	1 355	1 791	1 794	1 696	1 149	1 020	915	618	438	460	667	911
Average for 1978-1983	1 570	2 001	1 830	1 603	926	977	1 030	606	486	459	522	978

Notes : (1) Salinity records for Jowhar except 1965 to 1968 (Afgoi).

This pattern tends to be repeated throughout the year - whenever river flow is below average, salinity levels are above average. This general trend does not apply to the first flush of the gu season flood in April or May when salinity levels can be even higher, sometimes in excess of 6 000 micromhos/cm. However, such high levels rarely last for more than a few days.

Over the period for which records are available the average annual salinity value is about 1 100 micromhos/cm, monthly averages being in excess of this from January to May. For much of the year the salinity hazard is classified as high (more than 750 micromhos/cm), and only for four months does the hazard reduce to medium (250 to 750 micromhos/cm).

1.2.3 Irrigation with Saline Water

The use of saline water for irrigation can have adverse effects on both crop growth and soil permeability, the extent depending on the concentration of dissolved salts and, in particular, the concentration of sodium.

In the absence of detailed chemical analyses, the concentration of dissolved salts can be estimated by measurement of the electrical conductivity (EC) of the water. If water with a high EC is used for irrigation the quantity of water available to the plant is reduced and reduced yields will result.

Experiment on NCo 293 and NCo 310 sugar cane (Bernstein et al, 1966) showed that both varieties and similar salt tolerance at low salinities, but NCo 310 was less tolerant at high salinities. Yields decreased by 10% for an EC of 3 000 micromhos/cm, and by about 25% for an EC of 5 000. At Jowhar, irrigation will not normally take place when salinity levels exceed 3 000 micromhos/cm, and most of the time salinity will be much lower, so yield depression as a result of salinity should be relatively small (see also Annex II).

Using water with an EC of 3 000 micromhos/cm would necessitate a leaching requirement of 0.25. This will be achieved through the deep percolation losses which have been allowed for in the field irrigation efficiency of 65%.

The quantity of sodium in irrigation water is usually expressed as an adjusted SAR value which takes into account the ratio of sodium to calcium and magnesium. In order to estimate SAR values, daily records of hardness, sodium chloride and salinity (EC), as recorded at the Estate, were examined. During the dry season months of January to April salinity and SAR tend to be high. However, after the start of the gu flood both salinity and SAR fall, the drop being more pronounced in the case of SAR because of an appreciable drop in sodium content compared with calcium and magnesium content.

Using the data available from the Estate records it was not possible to calculate adjusted SAR values, but more detailed analysis of other data showed that adjusted SAR was approximately twice the corresponding SAR value.

Adjusted SAR values are used in both identifying possible soil permeability problems and toxic effects on plants. FAO (Paper Nr 29, 1976) suggests that severe problems of deteriorating soil permeability will occur in montmorillonite clays if the adjusted SAR value exceeds 9.0. An approximate relationship between EC and equivalent SAR, with poor correlation, is:

$$\text{SAR} = 2.9 + 1.3\,\text{EC} \quad (\text{EC in mmhos/cm})$$

This would indicate that an EC of 1.2 mmhos/cm would give an SAR of 4.5 and adjusted SAR of 9. The EC of the Shabelle water is less than 1.2 mmhos/cm for 8 months of the year and therefore the higher EC and SAR values can be tolerated for the remainder of the year because the soil profile does have the capability to attenuate changes in water quality.

As far as toxicity to the sugar cane is concerned there is unlikely to be a major problem. Some yield reduction can be expected when high salinity levels continue for extended periods, as they did in 1980, but generally the affect will be small.

1.2.4 Existing Estate Practice

Estate practice has varied over the years. In 1976 the policy was said to be to cease all irrigation when salinity levels rose above 2 250 micromhos/cm. However, in more recent years this upper level seems to have increased to 2 500 or even 3 000 micromhos/cm.

Our conclusion is that, in general, it is better to use saline water rather than to use no water at all. In view of the fact that acceptable salinity levels are experienced for 8 months of the year, the use of water with an EC value of up to 3 000 micromhos/cm is an appropriate policy.

1.3 Sediment

1.3.1 General

Suspended sediment concentrations in the Shabelle River have been monitored sporadically for 15 years or more, but there has been no long-term programme of data collection. Even so, it is clear that sediment levels increased very significantly during the mid-1970s and are now much more of a problem than was the case before 1975. It is likely that this increase was a direct result of the 1975 to 1977 drought during which there must have been large-scale loss of natural vegetation in the catchment area. If this is indeed the cause, then it is possible that there will be a gradual recovery which may result in sediment concentrations being reduced to their former levels.

The longest period of record for sediment concentration was obtained for the monitoring programme set up following commissioning of the Jowhar Offstream Storage Reservoir (JOSR). Samples were taken during the period 1980 to 1981, but, unfortunately, not continuously. Table I.1.2 summarises the very limited data available.

It should be noted that the averages presented in Table I.1.2 are, in many cases, based on only a few readings, sometimes as few as two in a month. The most comprehensive record is that for Sabuun barrage during May 1980 to September 1981. In this period samples were taken during 11 of the 24 months, and the averages presented are based on at least five readings in any month. Some months had readings for every day.

TABLE I.1.2

Suspended Sediment Concentration (ppm)

Month	1968(1)	1976(2)	1980(3)	1981(3)	1983(4)
Jan	-	-	-	-	-
Feb	-	-	-	-	-
Mar	-	-	-	-	-
Apr	1 016	3 095	-	1 464	-
May	213	5 767	11 255	673	-
Jun	298	1 450	7 423	2 098	300
Jul	-	-	-	-	160
Aug	-	-	4 573	3 646	840
Sep	345	-	4 556	2 600	100
Oct	-	-	2 279	-	100
Nov	-	-	2 032	-	-
Dec	-	-	-	-	-

Notes : (1) Averages of very limited records for Beled Weyn, Bulo Burti, Mahaddey Weyn and Jowhar.
(2) Values based on a total of seven samples taken at Afgoi.
(3) Records for Sabuun barrage.
(4) Samples taken during fieldwork for this study, and by the Estate staff after June.

The limited records of sediment show no obvious correlation with flow, but the following tentative conclusions can be drawn:

- sediment concentrations will be lowest during the dry season when river flows are generally low;

- peak concentrations are experienced in the gu season flood, but the higher discharges do not necessarily mean higher sediment concentration (in fact the opposite seems to be true in May and June);

- in the gu season higher peak concentrations are experienced when the previous dry season has been drier than average.

These phenomena can be explained by the following argument. When a drought is experienced in the catchment from December to March/April, increased erosion occurs when the rains commence in April/May. However, as the rainy season continues, vegetation re-establishes itself, and soil erosion is reduced even though river flows remain high - hence reduced sediment concentrations in the latter part of the gu season.

It is interesting to note that sediment levels recorded during the fieldwork were very low - around 300 ppm and even lower values have been recorded subsequently by Estate staff (Table I.1.2). This may be partly explained by the fact that flows in the river from November 1982 to March 1983 were well above average, indicating that the catchment area was not experiencing its usual drought conditions. The same is true for the similar low concentration recorded in 1968.

None of this helps us to predict long-term trends in sediment concentration but it is unlikely that the very high levels experienced in 1980 and 1981 will become the norm. In the short term, it is an unfortunate fact that a drought in January to March has three major drawbacks - shortage of water, high salinity of the water, and the likelihood of high sediment concentration in the following gu season.

1.3.2 Sediment Size

No data are available for particle size analysis of the sediment carried in the Shabelle water. The concentrations recorded during the fieldwork were too low to give representative results, although a visual examination of the sediment showed it to be very fine.

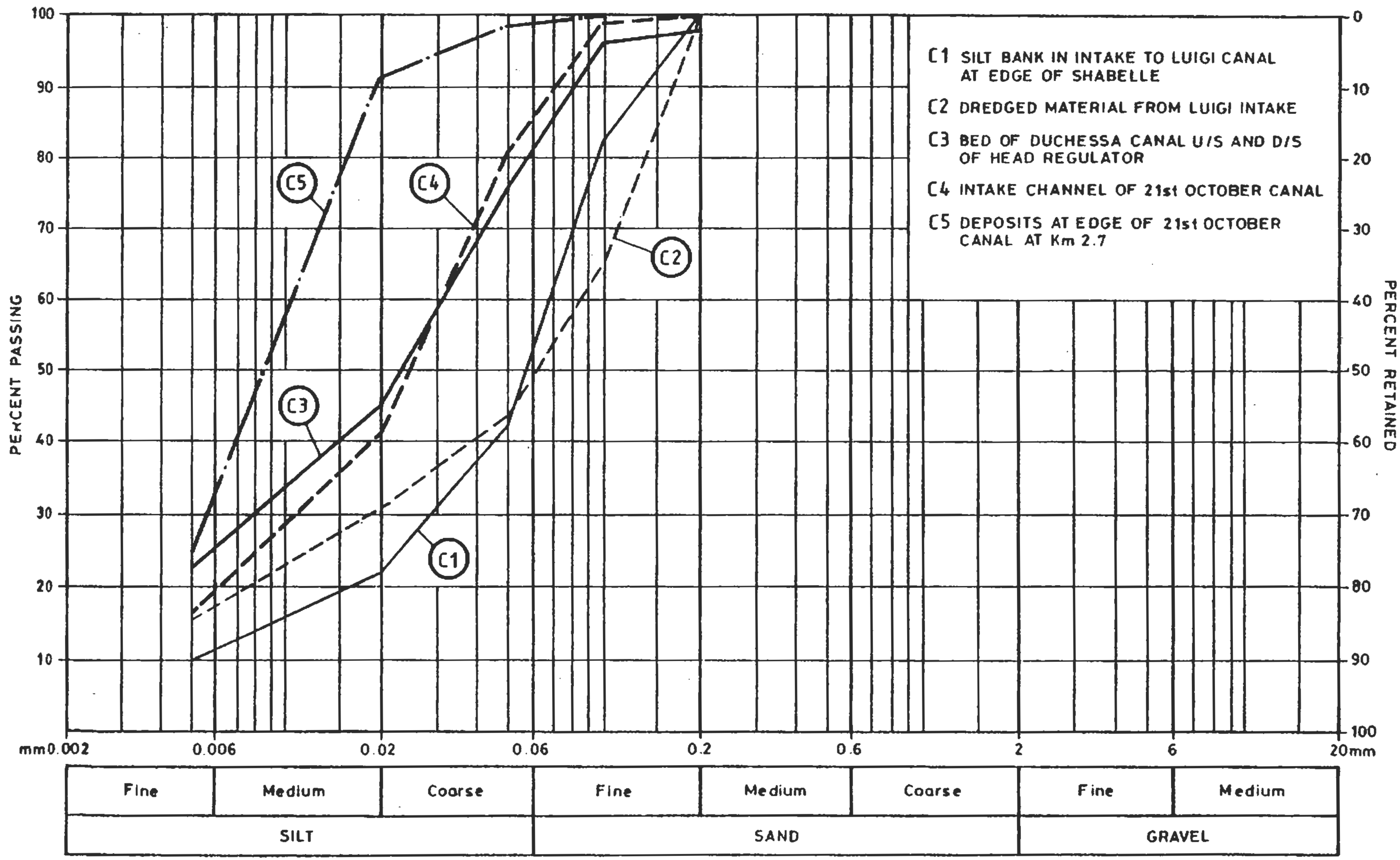
However, samples were taken of the material deposited in the two intake channels and in the canal system. The results of the particle size analysis of these samples are given in Figure I.1.1, and these data have been used to determine the requirements for sediment removal works (Chapter 5).

Figure I.1.1

PARTICLE SIZE DISTRIBUTION

PROJECT JOWHAR SUGAR ESTATE
SAMPLE No. AS SHOWN
DATE OF SAMPLING JUNE '83

LOCATION AS SHOWN
DESCRIPTION CHANNEL BED SAMPLES



CHAPTER 2

WATER AVAILABILITY

2.1 Introduction

During the dry season months of January to March river flows are at their lowest and there is often insufficient water to meet the irrigation needs of the Estate. In the past 30 years the river has in fact dried up completely in February on three occasions.

Clearly this places a constraint on the production of sugar cane at Jowhar. Sugar cane is a perennial crop and, in Somalia, the peak irrigation requirements tend to coincide with the period of lowest water availability. In this annex we examine the question of water availability and estimate the statistical probability of the Estate's requirements being met. The effect of water shortages on cane production is dealt with in Annex II.

Of prime importance in the investigation of water resources available from the Shabelle are the river discharge records of Mahaddey Weyn which lies at the head of all the irrigated reaches on the river. These records offer the best flow index for any analysis of water resources if the full consequences of any irrigation development are to be seen in relation to the complete river and all other existing or proposed irrigated agriculture.

2.2 Analysis of Flow Records

Water level measurements have been made on the river Shabelle at Mahaddey Weyn since April 1951, and at Sabuun above the offtake to the Jowhar Offstream Storage Reservoir (JOSR) since 1973. The record at Mahaddey Weyn is incomplete, particularly during the mid 1970s. Even when both station records are combined significant gaps remain.

Unfortunately both gauging stations are sited at sections where the river bed is unstable, with consequent adverse effects on the accuracy of rating relationships. In the absence of frequent stage discharge measurements to update rating relationships, discharge estimates from water level records at such stations are of uncertain reliability. Table I.2.1 presents the monthly flow records for the period 1951 to 1983 for both Mahaddey Weyn and Sabuun, and includes summary statistics.

Previous studies on the operation of JOSR had used 5 day mean discharges for the 12 year period April 1961 to March 1972. To make as much use as possible of the available data during the flow analysis for this present study, the full record (Table I.2.1) has been divided into three periods of unbroken 5 day records: 1954 to 1960 (6 years), 1961 to 1975 (14 years), and 1981 to 1983 (2 years). The record thus obtained has been used for the reservoir operation studies and is presented in Table I.2.2 together with summary statistics.

In order to determine the magnitude and lengths of periods of water shortage it is necessary to estimate monthly flows for various probability levels. Two approaches have been adopted to achieve this, as described below.

Using the reduced data set (Table I.2.2) a normal distribution was fitted to the independent monthly records to estimate the maximum flow in each month for three exceedence probabilities (50%, 75% and 80%). The resulting monthly values are

Table Nr I.2.1

FLOW OF SHEBELLI AT MAHADDEY WEIN DISCHARGE (MCM)

YEAR	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	JAN.	FEB.	MAR.	TOTAL
1951/52	315.0	346.0	276.0	67.0	147.0	204.0	123.0	336.0	242.0	29.0	13.0	14.0	2112.0
1952/53	13.0	208.0	58.0	28.0	63.0	286.0	211.0	110.0	12.0	*****	*****	*****	*****
1953/54	*****	*****	*****	*****	*****	*****	*****	*****	*****	31.0	28.0	30.0	*****
1954/55	189.0	161.0	69.0	11.0	172.0	335.0	347.0	179.0	90.0	15.0	7.0	21.0	1596.0
1955/56	22.0	103.0	35.0	7.0	54.0	242.0	338.0	103.0	18.0	16.0	10.0	26.0	974.0
1956/57	49.0	318.0	74.0	103.0	251.0	331.0	317.0	317.0	81.0	36.0	16.0	41.0	1939.0
1957/58	140.0	343.0	265.0	125.0	283.0	311.0	132.0	134.0	105.0	25.0	40.0	112.0	2015.0
1958/59	55.0	170.0	30.0	38.0	270.0	336.0	347.0	179.0	53.0	26.0	18.0	15.0	1537.0
1959/60	13.0	192.0	61.0	38.0	165.0	300.0	311.0	267.0	62.0	167.0	61.0	30.0	1687.0
1960/61	54.0	159.0	97.0	49.0	99.0	206.0	*****	*****	*****	30.0	29.0	16.0	*****
1961/62	22.0	117.0	44.0	108.0	305.0	335.0	346.0	336.0	347.0	42.0	8.0	11.0	2021.0
1962/63	30.0	147.0	40.0	24.0	71.0	176.0	217.0	326.0	186.0	17.0	8.0	9.0	1251.0
1963/64	131.0	344.0	264.0	136.0	260.0	338.0	249.0	122.0	216.0	99.0	43.0	20.0	2222.0
1964/65	65.0	95.0	48.0	81.0	240.0	334.0	341.0	230.0	71.0	138.0	40.0	20.0	1703.0
1965/66	18.0	126.0	38.0	13.0	46.0	166.0	219.0	278.0	121.0	32.0	10.0	78.0	1145.0
1966/67	90.0	227.0	116.0	108.0	165.0	281.0	257.0	201.0	52.0	11.0	2.0	1.0	1511.0
1967/68	89.0	270.0	195.0	79.0	257.0	351.0	352.0	305.0	331.0	103.0	48.0	191.0	2571.0
1968/69	204.0	371.0	312.0	222.0	281.0	334.0	310.0	180.0	184.0	76.0	58.0	268.0	2800.0
1969/70	283.0	306.0	163.0	143.0	298.0	347.0	265.0	136.0	47.0	26.0	57.0	119.0	2190.0
1970/71	257.0	368.0	113.0	51.0	265.0	363.0	372.0	261.0	71.0	37.0	25.0	11.0	2204.0
1971/72	38.0	252.0	143.0	221.0	306.0	374.0	316.0	225.0	124.0	44.0	50.0	56.0	2199.0
1972/73	87.0	361.0	221.0	207.0	336.0	367.0	335.0	212.0	80.0	35.0	18.0	8.0	2267.0
1973/74	7.8	138.5	39.0	45.0	232.7	396.2	308.2	99.8	21.6	4.0	0.0	0.0	1332.9
1974/75	*****	158.0	177.0	189.0	274.0	333.0	240.0	63.0	26.0	13.0	0.0	0.0	*****
1975/76	35.0	163.0	133.0	131.0	371.0	*****	*****	*****	*****	*****	*****	*****	*****
1976/77	*****	337.0	*****	235.0	302.0	354.0	261.0	208.0	*****	*****	*****	35.0	*****
1977/78	199.0	373.0	232.0	177.0	277.0	307.0	363.0	374.0	348.0	*****	70.0	*****	*****
1978/79	235.0	245.0	*****	105.0	66.0	134.0	271.0	235.0	135.0	152.0	184.0	121.0	*****
1979/80	270.0	185.0	257.0	177.0	269.0	*****	*****	*****	*****	25.0	11.0	8.0	*****
1980/81	16.0	230.0	59.0	78.2	206.0	225.0	149.0	79.3	34.5	5.0	0.0	92.0	1174.0
1981/82	372.2	394.3	176.0	67.7	236.8	369.1	396.2	170.3	56.2	27.6	25.0	26.6	2318.0
1982/83	150.9	337.6	256.6	126.7	275.5	326.8	323.0	404.7	244.8	130.0	65.7	50.0	2692.3
1983/84	66.0	289.0	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****	*****
MEAN	119.2	244.8	139.4	103.1	221.4	301.8	286.3	216.8	124.4	49.7	32.6	49.3	1889.6
CV	0.58	0.39	0.64	0.66	0.41	0.23	0.25	0.43	0.83	0.95	1.11	1.26	0.27
SKEW	0.98	0.03	0.43	-0.43	-0.74	-1.09	-0.82	0.22	1.07	1.41	2.72	2.16	-0.09

SUM OF MONTHLY MEANS = 1888.8

NOTE: ***** INDICATES A MISSING VALUE

Table Nr I.2.1

UNREGULATED RIVERFLOWS - SIMULATION PERIOD DISCHARGE (MCM)

YEAR	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	JAN.	FEB.	MAR.	TOTAL
1954/55	188.6	161.1	68.5	11.1	171.9	334.9	347.1	179.1	90.5	16.4	7.2	20.6	1597.0
1955/56	22.3	103.3	35.3	7.2	54.5	242.2	337.7	103.2	13.3	16.3	9.8	25.8	975.8
1956/57	45.3	318.4	73.7	107.7	250.9	330.9	316.7	311.7	81.2	35.6	15.8	40.8	1932.2
1957/58	140.3	343.5	264.8	124.8	282.6	311.2	131.5	133.9	105.4	24.6	40.4	111.8	2014.8
1958/59	54.8	169.9	29.6	38.1	270.2	335.9	347.1	178.8	52.8	25.5	18.0	14.8	1535.5
1959/60	13.5	192.4	61.3	38.2	185.0	299.9	311.5	267.5	61.5	166.6	60.7	30.3	1683.3
1961/62	21.9	116.7	44.2	107.8	305.3	335.1	345.6	335.9	347.1	42.3	8.1	10.6	2020.6
1962/63	30.5	148.2	39.7	23.6	71.0	176.0	216.6	325.7	186.4	17.4	7.7	8.5	1251.4
1963/64	130.9	343.6	264.4	136.2	260.2	337.8	249.0	122.3	215.6	99.1	43.1	19.8	2222.0
1964/65	64.7	94.9	47.7	80.8	239.9	333.7	341.4	229.8	71.4	138.0	40.5	20.2	1703.2
1965/66	18.5	125.5	33.2	12.9	46.4	165.9	219.1	278.4	121.3	31.9	10.3	78.2	1146.9
1966/67	89.7	226.9	116.0	108.2	165.3	280.5	257.3	201.0	51.6	11.3	2.3	1.0	1511.0
1967/68	38.8	269.8	194.6	78.5	256.9	350.6	352.0	304.6	330.8	103.0	48.3	190.8	2568.8
1968/69	203.5	371.4	311.7	222.0	281.0	333.5	310.1	180.4	184.0	76.2	58.4	268.3	2800.6
1969/70	282.8	305.7	163.2	142.6	298.4	347.5	264.9	135.5	47.0	25.6	56.6	119.0	2189.0
1970/71	267.7	368.1	112.7	50.7	265.4	363.0	371.6	261.1	70.7	37.4	25.0	11.1	2204.4
1971/72	88.2	252.1	143.3	220.8	305.7	374.4	315.7	225.2	123.5	43.8	50.4	56.3	2199.3
1972/73	87.0	360.7	221.5	207.2	335.5	366.8	334.9	212.0	79.6	34.6	18.3	8.0	2266.1
1973/74	7.8	138.5	39.0	45.0	232.7	386.2	308.2	99.8	21.6	3.9	0.0	0.0	1332.6
1974/75	0.0	157.7	177.1	188.5	273.6	333.1	240.4	62.6	26.2	12.8	0.0	0.0	1472.1
1981/82	372.2	394.3	170.0	67.7	236.8	369.1	396.2	170.3	56.2	27.6	25.0	26.6	2318.1
1982/83	150.9	337.6	256.6	126.7	275.5	326.8	323.0	404.7	244.8	129.7	63.3	50.3	2690.0
MEAN	107.9	240.9	133.2	97.6	230.2	319.8	301.7	214.7	117.6	50.9	27.7	50.6	1892.7
CV	0.94	0.43	0.67	0.70	0.36	0.18	0.21	0.41	0.81	0.91	0.79	1.34	0.27
SKEW	1.19	0.04	0.54	0.48	-1.26	-1.73	-1.08	0.28	1.30	1.33	0.33	2.16	-0.01

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Water Availability (Mm^3)(1)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Period 1951 - 1983 (independent monthly values)												
exceedance probability 50%	31	21*	26*	88	245	125	107	261	332	311	212	86
exceedance probability 75%	21*	9*	11*	33	161	58	48	164	259	242	144	53
exceedance probability 80%	16*	8*	9*	24	159	48	38	119	225	219	132	50
Periods 1954-60, 1961-75, 1981-83 (independent monthly values)												
exceedance probability 50%	33	22*	23*	88	240	114	94	259	334	316	206	80
exceedance probability 75%	17*	8*	10*	22	148	48	38	184	311	257	135	53
exceedance probability 80%	16*	8*	8*	21	136	43	36	171	297	248	132	51
Periods 1954-60, 1961-75, 1981-83 (homogeneous monthly sequence)												
exceedance probability 50%	32	31	23*	95	331	118	102	265	319	293	220	145
exceedance probability 75%	25*	15*	10*	88	176	109	85	147	271	327	172	82
exceedance probability 80%	24*	10*	8*	54	201	99	91	151	308	258	182	62
Estimated requirements(2)	14	13	16	9	8	9	9	11	14	8	8	10

Notes (1) Based on flow records for Maheddey Weyn (34 km upstream of Jowhar) and Sabuun.

(2) For Jowhar Estate, based on 5 300 ha net cane and using the latest estimates of crop water requirements (see Annex II).

* Water availability inadequate for requirements assuming the Estate can abstract a maximim of 50% of river discharge.

given in Table I.2.3. These values can be compared with similar values calculated using the full data set. Both sets of results show that water shortages can be expected during the period January to March as frequently as one year in two, assuming 5 300 ha of cane and continued enforcement of the policy that the Estate can only take 50% of available flow.

The monthly values obtained by the above-described method are independent of one another. However river flows exhibit persistence and flows at the end of the dry season are related to the flows which occurred in the preceeding high flow season. In assessing the impact of water shortages lasting more than one month it is therefore necessary to adopt a different approach. The last month in the dry season is assumed to be critical, in this case March, and the flow in that month assessed for a given exceedence probability as above. Flows in the preceeding months are then estimated such that the 12 month sequence has the same probability of exceedence as the critical month. Comparison of the monthly values derived by this method with those derived above shows that the two sets are identical only for the critical month March. In the months of January and February the flow available to the Estate is increased but shortages will still occur. The results of all the analyses are summarised in Table I.2.3.

2.3 Other Demands

In addition to the Estate demands given in Table I.2.3 consideration must be given to demands for users downstream of the Estate. The recently completed JOSR is sited to the south of the Estate, with its river intake at Sabuun upstream of the Estate, and its outlet downstream of the Estate. The reservoir presently serves to augment downstream demands only.

The demands for water downstream of Jowhar were estimated for the Genale - Bulo Marerta Study (MMP, 1978). During this present study attempts were made to up-date these estimates. Consultations with Ministry of Agriculture staff, both in Mogadishu and in Jowhar, yielded no additional information. Some data on irrigated areas were obtained from the staff of the Food Security Programme in Mogadishu, who were at the time (June 1983) engaged in assessing the extent of both rainfed and irrigated cropping along the Shabelle. Unfortunately these data proved to be of insufficient accuracy to be of use. For the reservoir analyses described in Chapter 3 we have therefore used the 1978 data. It is likely that there has been some expansion of irrigated cropping in the last five years, but all or most of this expansion will be for annual crops which are not grown in the dry season and therefore do not have a water demand in the critical dry. The adopted downstream demands are given in Table I.2.4.

TABLE I.2.4

Downstream Demands (Mm^3)

J	F	M	A	M	J	J	A	S	O	N	D
31.3	18.9	20.9	13.7	38.8	63.0	60.8	21.4	32.1	68.6	77.8	66.2

It should be noted that no allowance has been made for irrigation demands from the river between Mahaddey Weyn and Jowhar. The extent of irrigation in this reach is very limited and, again, only annual crops are grown.

2.4 Conclusions

It is clear from the available data that water shortages are experienced frequently, but the severity of these would be reduced significantly if the Estate could abstract all available river flow during dry periods. The situation is summarised in Table I.2.5.

TABLE I.2.5

Frequency of Water Shortages(1)

Frequency	Month	50% river flow 5 300 ha(2)	50% river flow 7 950 ha(2)	100% river flow 5 300 ha (2)	100% river flow 7 950 ha(2)
1 year in 2	Jan	0(0)	20(18)	0(0)	0(0)
	Feb	0(15)	23(45)	0(0)	0(0)
	Mar	28	52	0	4
1 year in 4	Jan	11(39)	38(58)	0(0)	0(15)
	Feb	42(69)	63(80)	0(38)	25(60)
	Mar	69	79	38	58
1 year in 5	Jan	14(43)	40(60)	0(0)	0(20)
	Feb	62(69)	75(80)	23(38)	50(60)
	Mar	75	83	50	67

Notes : (1) Figures in the table are percent deficits based on the homogeneous monthly sequences in Table I.2.3. Figures in brackets are for individual months, also from Table I.2.3.

(2) Cane area.

For a cane area of 5 300 ha (the recommended area), if the Estate could abstract all the river flow when necessary, severe shortages (more than 50% deficit) would occur less frequently than 1 year in 5. If the current legislation limiting the Estate's share of river flow to 50% is maintained, severe deficits can be expected about 1 year in 4.

Table I.2.5 also shows the water shortage statistics for full development of the Estate (7 950 ha). Under such circumstances severe deficits would be experienced one year in two, unless the Estate could take all the river flow in which case the frequency would reduce to 1 year in 4.

The effect of water shortages on cane yields has been examined and the results are presented in Annex II. The general conclusion is that, even when only 50% of the river flow is available to the Estate, the drop in long term average yield is only about 10%, provided that water is always available for plant cane.

If the legislation were to be changed to allow the Estate to take more than 50% of the river flow when necessary, the yield potential would increase and the risks of severe deficits reduced. Since there are no perennial crops grown between the Estate and the outfall from the JOSR, there would seem to be little justification for continuing to restrict the Estate's abstraction rights during the dry season. This policy should therefore be reviewed. A possible alternative would be to restrict the Estate's abstraction to leave a minimum of say

2 Mm^3/month downstream at Jowhar. This would be much easier to administer and is more logical because water demands downstream of the Estate are likely to be constant during the dry season.

CHAPTER 3

WATER STORAGE

3.1 Introduction

Various alternative solutions to the problem of water shortages have been examined and these are discussed below.

There are no suitable on-river reservoir sites in the Somali catchment of the Shabelle (MMP, 1969), but there is potential for offstream storage and this has already been exploited by the construction of the reservoir to the south of the Jowhar Sugar Estate. The Jowhar Offstream Storage Reservoir (JOSR) was commissioned in 1980 and has been operational for the last four years. This reservoir is located in a natural depression and makes full use of the favourable topography. The normal pattern of operation is to fill the reservoir during the two flood seasons (May to June and August to November) and make releases in the dry season (January to March). The reservoir has generally been successful in reducing the frequency of water shortages for downstream users although operation of the reservoir has been less than optimum (see Section 3.4.1).

There are other sites for offstream storage, although none is as favourable as the JOSR site. The most promising is some 50 km to the north near the village of Duduble (see Figure I.3.1). Duduble reservoir area was mapped using aerial photography for this present study and 1 : 5 000 photo mosaics with 0.5 m contours were produced. From this a 1 : 25 000 contour map has been prepared and this has been used to determine reservoir characteristics. Details are given in subsequent sections.

Since a new reservoir at Duduble would inevitably be expensive and might not be economically justifiable, the alternative of pumping water from JOSR has also been examined. Current Ministry of Agriculture policy does not allow the Estate to remove water from JOSR, although this possibility was planned for when the reservoir was designed. Nevertheless, there is potential for increasing the storage volume of JOSR and this increased volume could be reserved for use by the Estate. This alternative has therefore been examined and the conclusions are presented later in this chapter.

A partial solution to the problems of water shortage has also been examined, and this is the provision of limited storage on the Estate for the irrigation of plant cane during drought conditions. The reasoning being, that whereas mature cane can be deprived of water for long periods without drastic results (cane yields will fall), it is vital that each year's new plant cane receives adequate irrigation in order that it survives. The irrigation requirements for plant cane during a three month drought have been calculated and are presented in Annex II. The water storage requirements and means of providing them are discussed below.

3.2 Storage Reservoirs for Plant Cane

Two storage basins exist within the Estate and their location is shown in Figure I.3.2. They were constructed in the 1960s, but no records are available of their use. It is understood, however, that they were normally filled by gravity and used for irrigation by pumping from them during drought periods.

Shortly after commissioning them, however, the basins were abandoned as a means of storage as it was thought that peripheral seepage from the basins contributed to the salinity and waterlogging problems on the adjacent fields. The serious water shortage early in 1975 resulted in renewed interest in the basins and both were filled in July and August 1975. In more recent years these basins have had surplus irrigation water discharged into them.

Approximate dimensions of the two basins are given below:

Basin Nr	Served by	Plan area (ha)	Average bed level (m)	Average bank top level (m)
1	Luigi canal	70	101.8	104.5
2	Canal S3	160	100.5	103.5

In the proposed rehabilitation measures described later in this annex, reservoir Nr 1 will be used for night storage for canal S2 and also for canal S1 when this cannot be served from the Luigi canal. The 12 hour storage volume required for both canals is 185 000 m^3 (assuming cane on the whole commanded area - only 110 000 m^3 for the proposed cane development). Thus only a maximum of 0.26 m storage depth is actually required for overnight storage. The total volume which can be stored, assuming gravity filling from minimum river level, is 1.05 Mm^3. Allowing for dead storage there is thus a volume of some 800 000 m^3 gross for plant cane.

Reservoir Nr 2 can be used solely for plant cane water storage. It can be filled from canal S3 by gravity up to a level of 102.2 m, or by pump up to a level of 103.0 m. A maximum gross storage volume of about 4.0 Mm^3 is available, of which 2.7 Mm^3 can be filled by gravity.

The total volume of water required to irrigate plant cane during a three month total drought has been estimated at 1.4 Mm^3. Allowing for seepage and evaporation losses over a three month period the gross volume required is some 3.5 Mm^3. This volume is available from the two reservoirs, but there are problems associated with its use, as described below.

The plant cane can be located virtually anywhere in the Estate since every 4 to 5 years the old cane is uprooted and new stock is planted. The water stored in the basins must therefore be transferable to any part of the Estate. In the case of basin Nr 2 this involves pumping two or even three times, and there will inevitably be additional distribution losses which have not yet been allowed for in the calculations. This problem can be partially overcome by ensuring that the dry season's new planting areas are located as near to the basins as possible. The planting in the following season, when water shortages should not occur, can be in areas remote from the basins.

Nevertheless, since it is vital that there is sufficient water for plant cane, it is proposed that a further reservoir can be constructed near the head of the 21st October canal (Figure I.5.2). This would have a surface area of about 35 ha and could store up to 700 000 m^3. Water could be diverted to any part of the Estate from this reservoir, some of it by gravity.

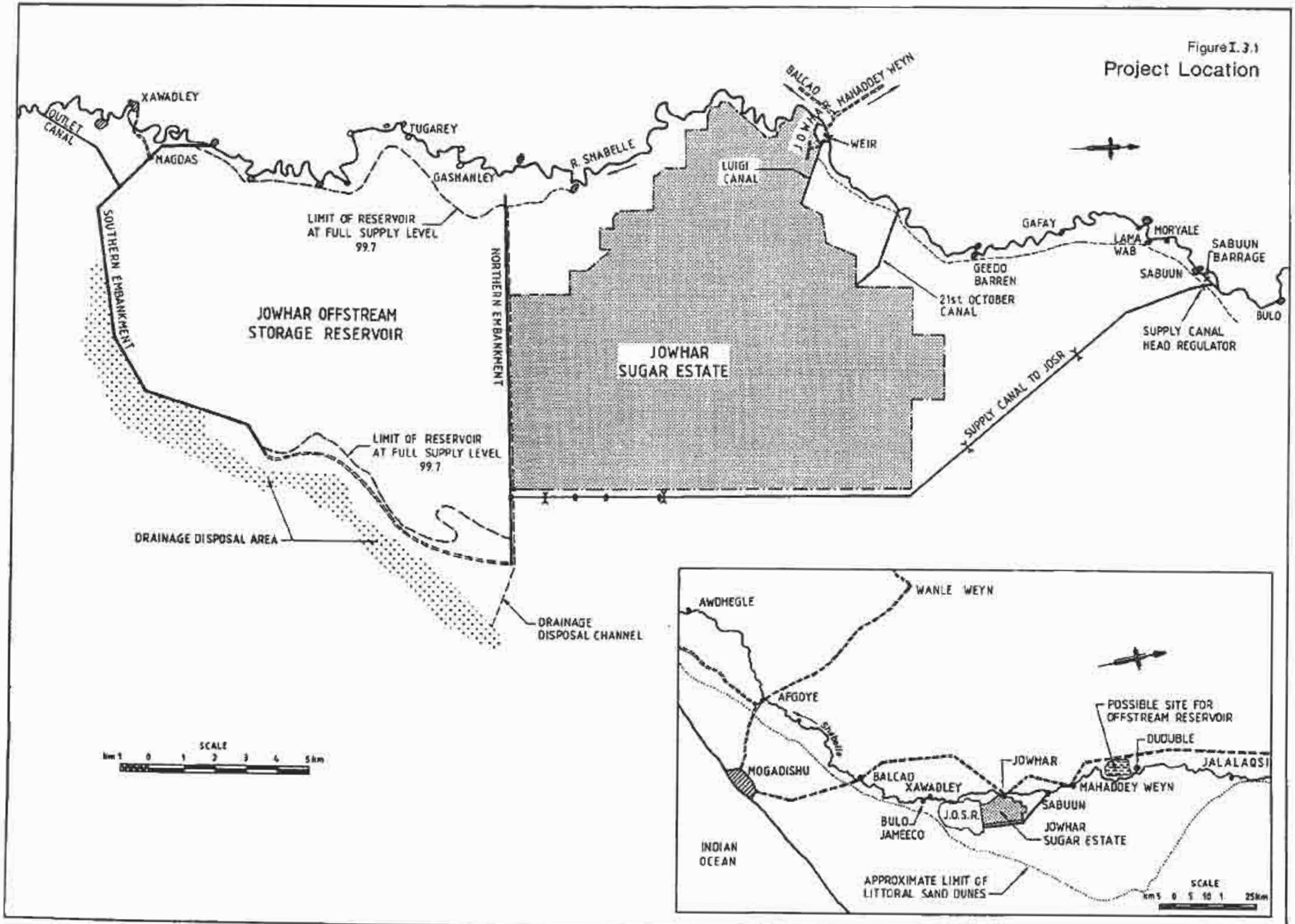
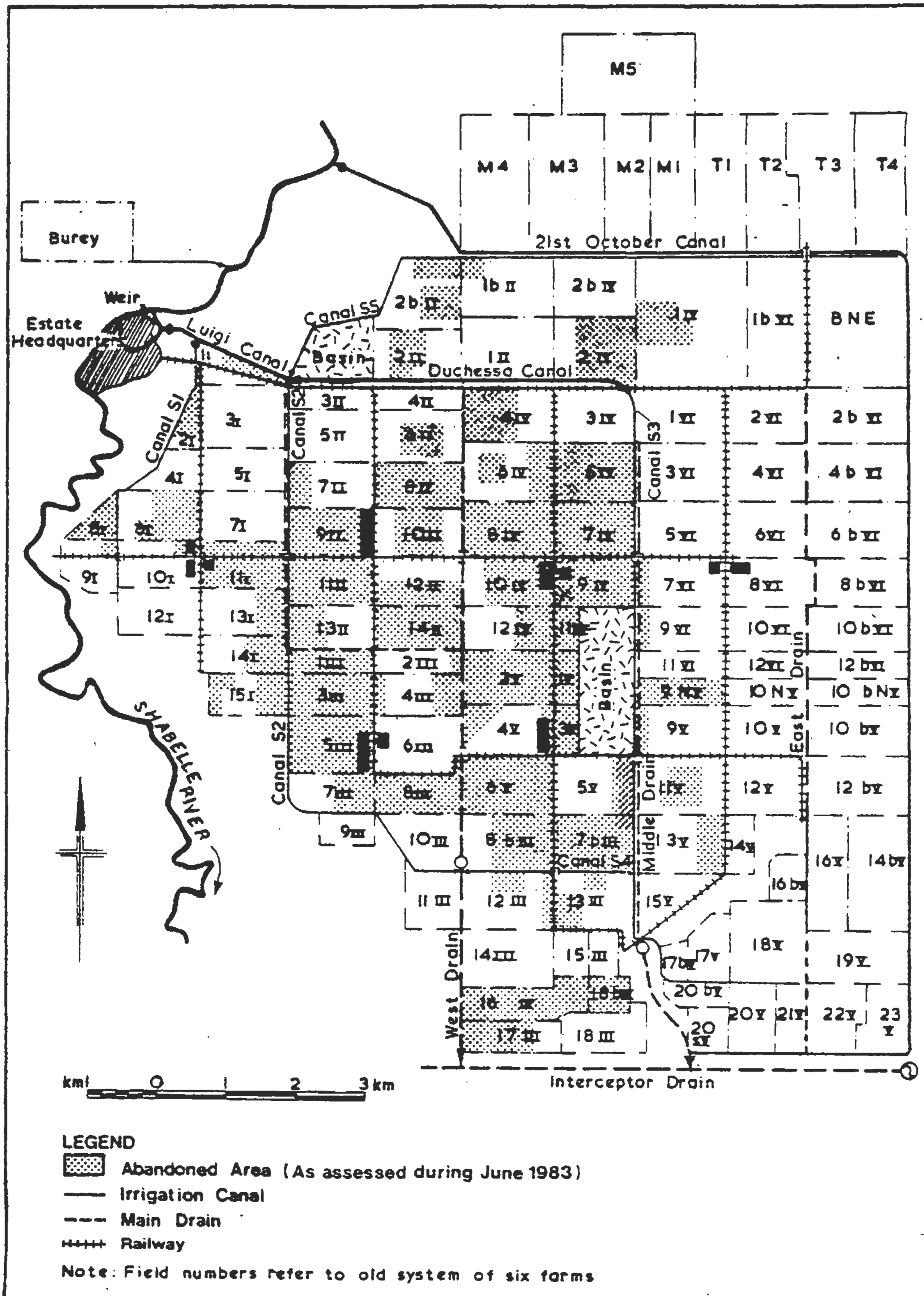


Figure I.3.1
Project Location

Figure 1.5.2

Jowhar Sugar Estate
Existing Layout



Between them these three reservoirs can provide a gross storage volume of up to 5.5 Mm^3. Assuming that pumped filling of the reservoirs is undesirable, the total volume reduces to some 4.2 Mm^3. In the event of a drought where the river dried up completely for three months (about a 1 : 20 year event) this water could be used to irrigate the plant cane. Water in basin Nr 3 (the new reservoir) would be used first since its water would be the easiest to evacuate to any part of the Estate. Basin Nr 1 would be next, with water only being pumped from basin Nr 2 when the other two were emptied. Under these circumstances, over the three month period, the seepage and evaporation losses would be some 1.8 Mm^3. This would leave 2.4 Mm^3 to meet the estimated gross requirement of 1.4 Mm^3. This should be adequate allowing for the inevitably high distribution losses that will occur when small flows are discharged in the canal system over long distances.

During the period when water is stored in the reservoirs the salinity will increase progressively as water evaporates. Salinity values during August to November are typically 500 micromhos/cm, rising to 950 micromhos/cm in December. Water stored for three months in basin Nr 2 (the last basin to be emptied) could thus end up with a salinity of up to 1 500 micromhos/cm if virtually all the water were to be used. For most of the time, however, it can be expected that salinity levels would be less than 1 000 micromhos/cm.

In the past it was feared that seepage from the reservoirs caused the local watertable to rise and resulted in the abandonment of adjacent fields. There is some evidence to support this in the vicinity of basin Nr 2. However, during the fieldwork, the soils in the bed of one of the basins were examined and it was concluded that infiltration rates would be very low. Seepage losses from the reservoirs can therefore be expected to be acceptably low. Nevertheless, it is proposed that cut-off drains are excavated around the reservoirs, and that these are connected into the main drainage system. Material excavated from these drains can be used to form new reservoir embankments or augment existing embankments.

3.3 Duduble Reservoir

3.3.1 General

The proposed Duduble reservoir is located some 40 km north of Jowhar town on the right bank of the river. It was identified as a possible reservoir site during the Shabelle River Study (MMP, 1969), although it was pointed out then that the reservoir topography did not appear ideal, and that the site was best suited for flood relief works being a natural spillage area.

Subsequently, in 1978, designs were prepared for a flood relief regulator and channel at Duduble (MMP, 1978). It is understood that these works are now about to be constructed under Chinese aid.

In the wake of the recent successful use of the Jowhar reservoir, interest in the Duduble reservoir has been renewed and an aerial survey was commissioned in 1983 as part of this present study. From the 1 : 5 000 air photomaps produced a 1 : 25 000 contour map has been prepared (Drawing Nr 12700/10). The survey confirms that the topography of the site is not ideal for a reservoir, but there is one fairly large depression which can be incorporated into a substantial reservoir.

3.3.2 Reservoir Details

Several possible alignments of the embankment have been examined, the aim being to maximise the storage volume whilst keeping the embankment height less than 5.0 m. Embankment heights in excess of 5.0 m will require much more exacting construction techniques and selection of fill material.

The embankment alignment selected makes best use of natural high spots; it is shown on Drawing Nr 12700/10.

The reservoir stage/storage/area relationship is shown in Figure I.3.3. A reservoir volume of 150 Mm^3 can be achieved with a maximum water level of 114.8 m, which would require a maximum embankment height of 5.0 m, allowing 1.5 m freeboard. The reservoir surface area for this volume would be about 80 km^2, giving a mean storage depth of 1.9 m.

In comparison the JOSR achieved a volume of 200 Mm^3 with a maximum embankment height of 4.3 m and a surface area of 110 km^2 (mean storage depth 1.8 m). To achieve 200 Mm^3 storage volume at Duduble would require a maximum water level of 115.3 m which would necessitate some 2.0 km of embankment higher than 5.0 m (maximum height 5.5 m).

With a maximum storage level of 114.8 m a total length of embankment of 33 km would be required to contain the potential 150 Mm^3 stored volume. Approximately 1 150 000 m^3 of earth would be required to form this embankment.

In addition to the embankment, inlet and outlet structures are required. Outline designs have been prepared for these in order to obtain preliminary cost estimates. Possible sites for these structures are indicated on Drawing Nr 12700/10. It has been assumed that water would only be diverted into the reservoir when the river flow exceeds 100 m^3/s (this was the criterion assumed for the Duduble flood relief channel during the planning stage of JOSR). At the selected site for the intake structure, some 7 km up-river from Duduble village, the river water level is estimated to be 116.0 m for a flow of 100 m^3/s and 116.8 m for a flow of 150 m^3/s. These levels are more than adequate to enable filling of the reservoir to take place for all river flows in excess of 100 m^3/s.

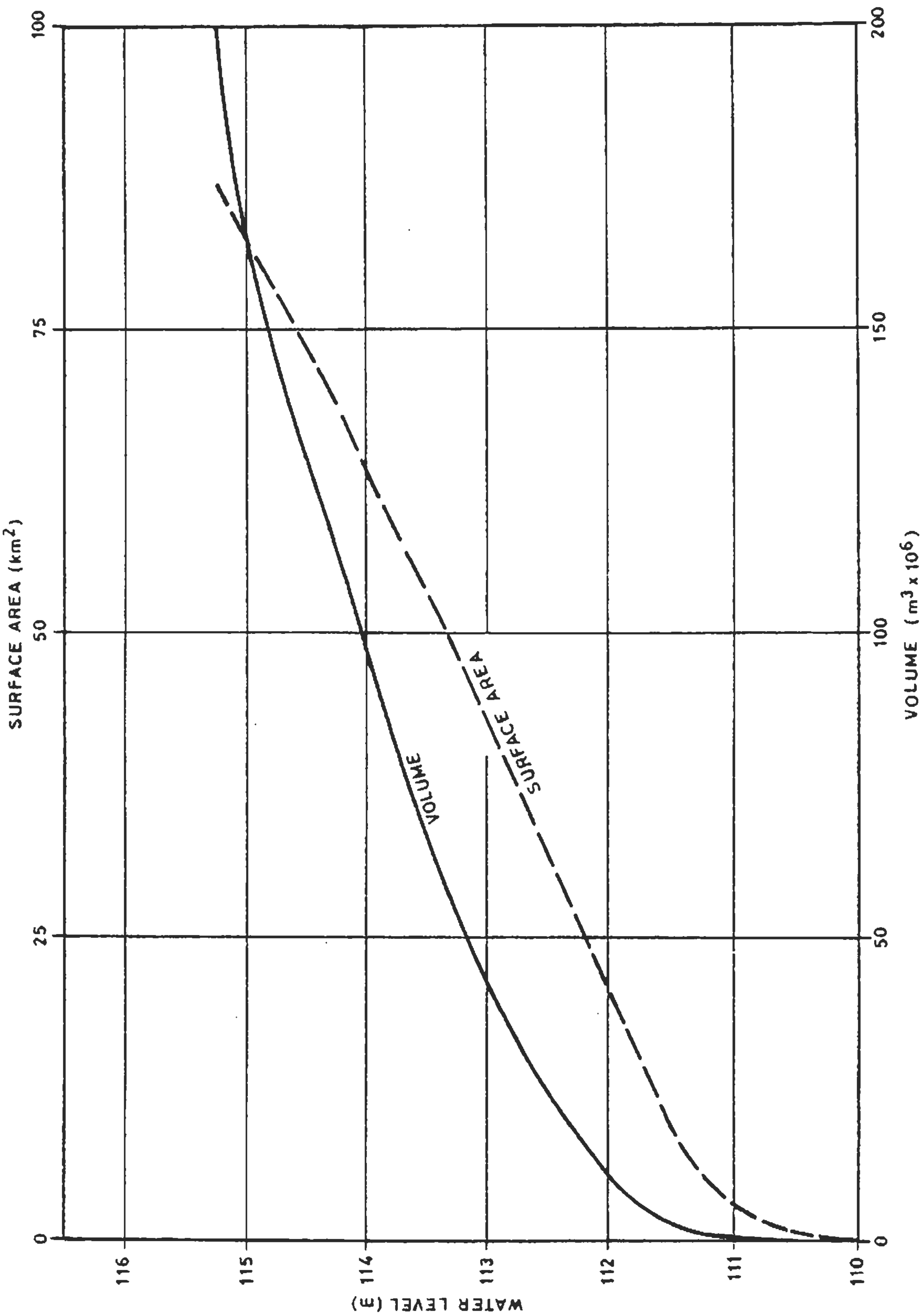
3.3.3 Inlet and Outlet Structures

The inlet structure has been assumed to have a capacity of 40 m^3/s for a river flow of 140 m^3/s. A similar structure to the JOSR supply canal head regulator would be required, with four 4.0 m wide gated bays. This structure could be combined with the proposed flood relief head regulator if both schemes were to go ahead. In this case floods would be routed through the reservoir, spilling through an escape structure in the western embankment, thus saving the cost of the flood channel and also ensuring that maximum use is made of the reservoir. This should be given serious consideration in view of the fact that work on the flood relief regulator and channel may start very soon.

The outlet structure would be required to discharge water into the river during periods of low flow. Minimum reservoir level would be about 111.5 m, allowing for some dead storage, compared with a minimum river level of about 109.0 m. A twin-bayed structure with two 4.0 m wide gates would be required to pass a minimum flow of 10 m^3/s from the reservoir to the river, with the reservoir level at its lowest.

Figure I. 3.3

Dudubie Reservoir Characteristics



3.3.4 Hydrology

(a) General

In previous studies (MMP, 1978) a water resources model of the river system downstream of Mahaddey Weyn has been built up. The existing situation is shown schematically in Figure I.3.4. The river gauge at Mahaddey Weyn lies at the head of the irrigated reaches of the river. The composite record for this station and Sabuun (Chapter 2) has been used as input to the model. The first control below Mahaddey Weyn occurs at Duduble where a flood relief channel with a maximum capacity of 40 m^3/s is planned. The relief channel is designed to offtake as soon as river discharges exceed 100 m^3/s, the estimated maximum flow which can be contained within bank below Sabuun.

The existing JOSR can abstract up to 50 m^3/s from the river for storage. This abstraction is only permitted when river flow past the offtake is sufficient to meet the total demand from all downstream users including the Estate. The Estate requirements are subtracted from the river flow subject to the rule that the Estate may only abstract half the river flow. If river flow below the Estate is insufficient to meet downstream requirements the shortfall can be offset by releases from JOSR. Releases are determined by available storage (the maximum live storage of JOSR is 200 Mm^3) and the maximum capacity of the outlet which is 25 m^3/s. Under present conditions, the Estate is entirely dependent on river flows and in the dry season, particularly January to March, experiences shortages.

(b) Reservoir Operation

The computer program developed for JOSR (MMP, 1978) has been modified to simulate the operation of the proposed DOSR. It is possible to model the two storage reservoirs separately on the assumption that DOSR will be operated in such a way as to avoid affecting the operation of JOSR.

Abstractions would be made from the river during the flood seasons and releases made when river flows drop below twice the Estate's requirements. The DOSR would therefore have no effect on users downstream of the Estate who are served by releases from JOSR. Figure I.3.1 shows the location of the proposed DOSR in relation to the Estate and JOSR.

The computer program models the reservoir system and operating rules, storing and allocating water to meet the monthly demands on a 5 day basis. However, in order to reduce errors when estimating reservoir losses, calculation of the stored volume and hence surface area of the reservoir has been made on a daily basis.

A summary of the reservoir details and operating rules is given in Table I.3.1. The three inflow sequences used in all the operation runs are tabulated in Table I.2.2. For each set of reservoir parameters a run has been made with each of the three inflow sequences, starting each run with the reservoir empty. A range of reservoir sizes, from 68 to 200 Mm^3 gross storage have been examined. Evaporation and seepage losses have been calculated as in the JOSR model. However, as less is known about soil conditions at the DOSR site than at JOSR, two seepage rates of 2 and 5 mm/d have been used in the operation runs.

Initially it has been assumed that DOSR would be incorporated in the proposed flood protection scheme at Duduble. Thus the supply canal capacity has been set at 40 m^3/s and the invert level of the intake fixed to permit abstractions only when river flows exceed 100 m^3/s.

TABLE I.3.1

Duduble Offstream Storage Reservoir
Summary Details

Gross storage	:	68 - 200 Mm^3
Dead storage	:	10 Mm^3
Inlet capacity	:	40 m^3/s
Inlet canal losses	:	1 m^3/s
Outlet canal capacity	:	10 m^3/s
Outlet canal losses	:	01 m^3/s
Seepage losses	:	2 or 5 mm/d
Inflow sequence	:	Table I.2.2

	J	F	M	A	M	J	J	A	S	O	N	D
Evapn. (mm/d)	6.6	7.4	7.6	6.5	6.0	5.4	5.3	6.0	6.7	6.1	6.2	6.1
Estate demand 5 300 ha (m^3/s)	5.09	5.51	6.04	3.29	3.02	3.39	3.23	3.98	5.57	2.81	3.18	3.92

Table I.3.2 showing the water year 1962/63 is included merely as an illustration of how the operational study is performed. During 1962 river flows during the gu season never exceeded 100 m^3/s and no abstraction was made to DOSR until the der flood. The letter 'F' indicates all 5 day periods in which the Estate's requirements would not have been met.

The 5 day flows downstream of the DOSR outlet have been aggregated to give a 22 year record of monthly flows for each combination of reservoir size and seepage losses. Table I.3.3 shows the 22 year record for a 130 Mm^3 reservoir assuming seepage losses of 5 mm/d. These regulated flow records were analysed to give both independent and homogeneous monthly values for 50%, 75% and 80% exceedance probabilities. Table I.3.4 shows the results for the flows in Table I.3.3. Comparison of these figures with those for the unregulated flows for the same period, Table I.2.3, shows the effect of the 130 Mm^3 capacity DOSR in increasing the flows available in the dry season January to March.

To determine the reliability with which the Estate's demands are met month by month with regulation by each size of DOSR the number of failures over the 22 year period has been counted. Failure has been defined for this purpose as being

Figure I.3.4

Shabelle River
Schematic Flow Allocation

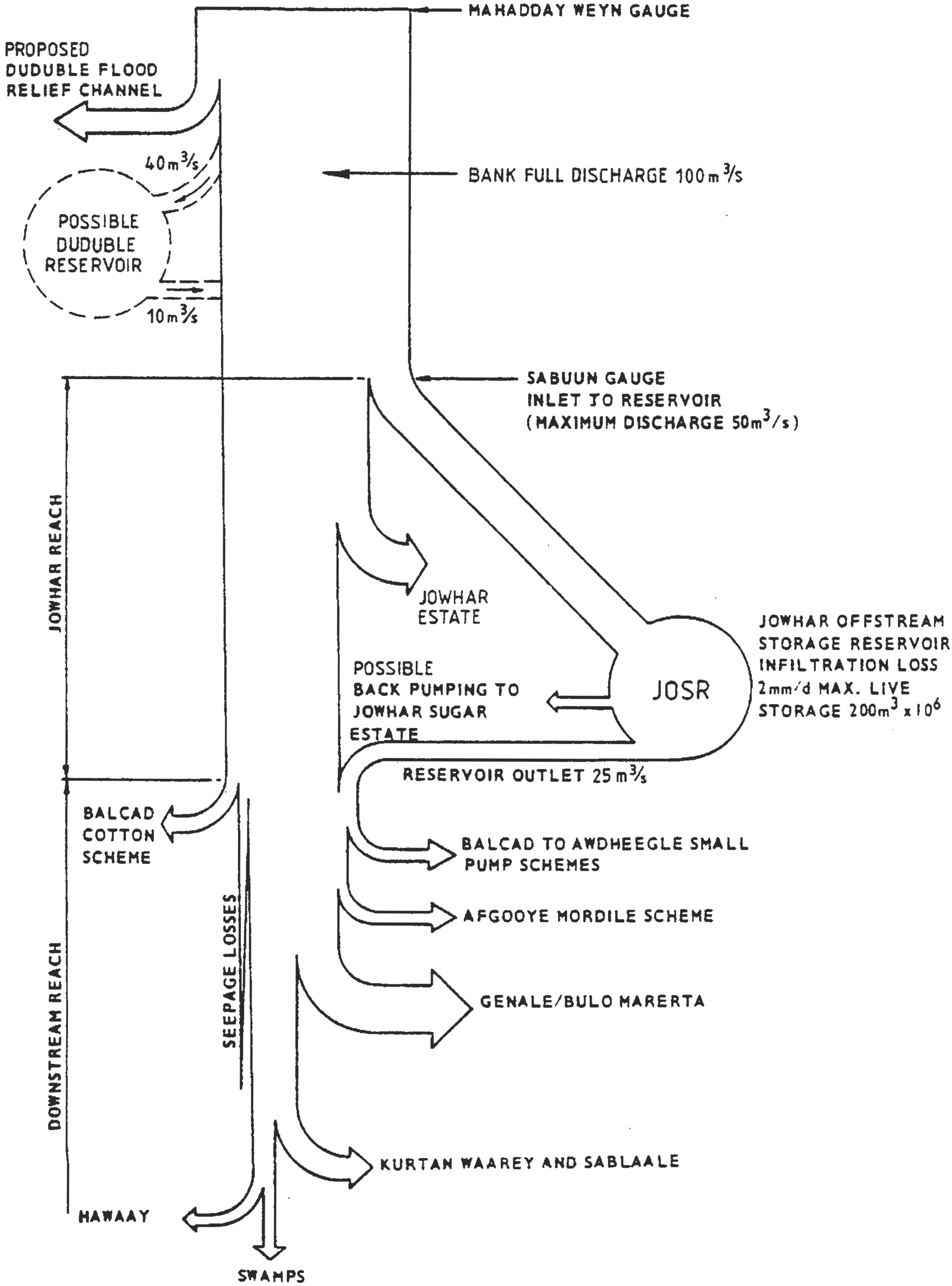


Table Nr I.3.2

YEAR 1962/63 Dudubie Reservoir - Sample Operation Run

MTH	PRD	MODE	U/S RIVER	RELIEF FLOW	SUPPLY CANAL	USE REQS	OUTLET CANAL	D/S OUTLET	STORED VOLUME (END PRD.)
APR	1	5	2.7			3.3	2.0	4.6	38.9
	2	5	6.2			3.3	.3	6.4	36.4
	3	5	5.2			3.3	.8	5.9	37.7
	4	5	4.7			3.3	1.0	5.6	31.0
	5	4	32.0			3.3		32.0	28.9
	6	4	19.8			3.3		19.8	26.3
MAY	1	4	10.6			3.0		10.6	25.0
	2	4	45.8			3.0		45.8	23.2
	3	4	59.9			3.0		59.9	21.5
	4	4	72.9			3.0		72.9	19.8
	5	4	72.2			3.0		72.2	18.3
	6	4	68.0			3.0		68.0	16.5
JUNE	1	4	60.2			3.4		60.2	15.3
	2	4	19.8			3.4		19.8	14.0
	3	5	2.7			3.4	2.1	4.7	12.0
	4	5	3.7			3.4	1.6	5.2	10.3
	5	5	2.7			3.4	2.1	2.8	10.0

RESERVOIR IS EMPTY ON DAY 1 OF PERIOD 5

MTH	PRD	MODE	U/S RIVER	RELIEF FLOW	SUPPLY CANAL	USE REQS	OUTLET CANAL	D/S OUTLET	STORED VOLUME (END PRD.)
	6	4	2.7			3.4		2.7	8.4
JULY	1	4	7.6			3.2		7.6	7.7
	2	4	4.3			3.2		4.3	6.9
	3	4	10.0			3.2		10.0	6.3
	4	4	10.0			3.2		10.0	5.6
	5	4	10.0			3.2		10.0	5.1
	6	4	10.6			3.2		10.6	4.4
AUG	1	4	10.6			4.0		10.6	3.9
	2	4	19.8			4.0		19.8	3.4
	3	4	21.5			4.0		21.5	3.0
	4	4	21.5			4.0		21.5	2.6
	5	4	27.3			4.0		27.3	2.2
	6	4	53.0			4.0		53.0	1.8
SEPT	1	4	71.6			5.6		71.6	1.5
	2	4	74.5			5.6		74.5	1.2
	3	4	64.2			5.6		64.2	1.0
	4	4	55.6			5.6		55.6	.7
	5	4	70.0			5.6		70.0	.5
	6	4	71.6			5.6		71.6	.3
OCT	1	4	73.3			2.8		73.3	.1
	2	0	65.4			2.8		65.4	.0
	3	0	62.4			2.9		62.4	.0
	4	0	74.1			2.6		74.1	.0
	5	1	105.0		5.0	2.5		100.0	.9
	6	4	101.0			2.8		101.0	.7
NOV	1	1	112.0		12.0	3.2		100.0	4.2
	2	1	128.4		28.4	3.2		100.0	13.3
	3	1	129.6		29.6	3.2		100.0	22.9
	4	1	128.4		28.4	3.2		100.0	31.7
	5	1	128.4		28.4	3.2		100.0	40.5
	6	1	127.2		27.2	3.2		100.0	45.9
DEC	1	1	112.0		12.0	3.9		100.0	50.9
	2	4	80.0			3.9		80.0	45.1
	3	4	59.2			3.9		59.2	45.6
	4	4	62.6			3.9		62.6	43.0
	5	4	55.0			3.9		55.0	40.6
	6	4	44.0			3.9		44.0	37.7
JAN	1	4	19.8			5.1		19.8	35.3
	2	5	8.0			5.1	1.2	9.1	32.5
	3	5	2.7			5.1	3.8	6.4	29.7
	4	5	2.7			5.1	3.6	6.4	25.0
	5	5	3.6			5.1	3.4	6.9	21.6
	6	5	3.8			5.1	3.3	7.0	17.9
FEB	1	5	3.3			5.5	3.7	7.4	14.7
	2	5	3.5			5.5	3.9	7.3	11.7
	3	5	3.5			5.5	3.9	4.4	10.0

RESERVOIR IS EMPTY ON DAY 4 OF PERIOD 3

MTH	PRD	MODE	U/S RIVER	RELIEF FLOW	SUPPLY CANAL	USE REQS	OUTLET CANAL	D/S OUTLET	STORED VOLUME (END PRD.)		
	4	4	2.7			5.5		2.7	8.7	*	F
	5	4	2.7			5.5		2.7	7.9	*	F
	6	4	2.7			5.5		2.7	7.2	*	F
MAR	1	4	2.7			6.0		2.7	6.4	*	F
	2	4	2.7			6.0		2.7	5.6	*	F
	3	4	2.7			6.0		2.7	4.9	*	F
	4	4	2.7			6.0		2.7	4.3	*	F
	5	4	2.7			6.0		2.7	3.7	*	F
	6	4	5.2			6.0		5.2	3.1	*	F

OTAL INFILTRATION LOSSES FOR YEAR ARE 46.13 MCM
OTAL EVAPORATION LOSSES FOR YEAR ARE 48.92 MCM

Table Nr I.3.3

FLOWS D/S OF DUDUELE - 130 MCM RESERVOIR, SEEPAGE 5MM/DAY DISCHARGE IN MCM

YEAR	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.	JAN.	FEB.	MAR.	TOTAL
1954/55	151.2	159.7	63.5	11.1	166.7	259.2	267.8	162.0	90.5	22.1	16.9	26.5	1402.3
1955/56	22.3	103.3	35.3	7.2	54.5	221.1	267.8	103.2	21.3	21.8	9.6	25.8	893.4
1956/57	48.3	255.6	73.7	104.2	250.2	259.2	266.4	249.1	81.2	35.6	21.2	46.8	1691.9
1957/58	139.1	267.8	235.5	124.8	244.4	259.2	131.5	133.9	105.4	26.7	41.7	111.8	1821.8
1958/59	54.6	166.8	29.6	38.1	240.3	259.2	267.9	166.1	52.8	26.7	22.3	23.6	1349.9
1959/60	15.3	178.5	61.3	38.2	135.0	244.6	265.0	223.9	61.5	166.0	59.3	31.3	1529.9
1961/62	21.9	116.7	44.2	107.2	258.6	259.2	267.8	259.2	267.8	43.8	17.4	21.5	1686.1
1962/63	32.1	148.2	41.2	23.6	71.0	176.0	214.4	259.2	131.3	24.2	11.3	8.5	1191.1
1963/64	119.7	267.8	220.9	136.2	243.4	259.2	238.7	122.3	215.6	99.1	42.0	26.1	1991.0
1964/65	65.5	94.9	47.7	80.8	218.9	259.2	267.8	200.8	71.4	138.0	40.5	26.3	1512.0
1965/66	21.1	125.8	38.2	12.9	46.4	145.9	206.3	243.8	121.3	32.1	10.3	78.2	1102.9
1966/67	89.7	215.4	116.0	105.2	145.3	251.9	219.1	195.8	51.6	14.4	2.3	1.0	1430.6
1967/68	88.5	240.6	171.6	79.5	230.8	259.2	267.8	257.0	265.8	103.0	46.5	190.8	2200.6
1968/69	133.7	269.4	259.2	222.0	253.0	259.4	267.6	177.0	164.0	76.2	58.4	234.8	2449.9
1969/70	253.3	254.0	150.2	142.8	252.8	259.2	239.5	135.5	47.0	27.6	56.6	119.0	1937.7
1970/71	245.6	271.3	112.7	50.7	235.6	260.0	268.5	212.5	70.7	37.5	25.0	21.7	1813.1
1971/72	90.5	245.5	143.3	220.8	267.5	270.7	267.0	204.3	123.5	43.8	45.6	55.4	1978.9
1972/73	87.0	270.9	190.4	199.3	267.8	267.2	270.0	207.5	79.6	34.6	22.5	20.2	1917.0
1973/74	10.3	138.5	39.0	45.0	219.0	293.8	257.5	99.8	22.9	15.6	13.3	3.3	1198.6
1974/75	0.0	146.0	175.8	193.8	249.3	259.2	213.7	62.6	26.2	20.0	7.0	0.0	1343.6
1981/82	268.7	287.2	154.2	67.7	210.5	269.5	289.0	167.7	56.2	28.7	26.1	29.8	1865.3
1982/83	142.0	274.6	219.0	126.7	244.1	259.2	264.6	301.0	231.1	129.7	63.3	50.5	2305.6
MEAN	93.0	204.6	122.2	96.8	208.0	251.0	249.4	188.4	110.4	53.1	30.0	52.4	1664.2
CV	0.34	0.32	0.61	0.70	0.33	0.11	0.14	0.33	0.71	0.84	0.64	1.16	0.24
SKEW	0.85	-0.31	0.34	0.48	-1.60	-2.33	-2.13	-0.23	0.89	1.42	0.37	1.96	0.06

Table Nr I.3.3

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Assessment of Water Availability with Regulation by DOSR (130 Mm^3)

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Independent monthly probabilities	50%	33	24	26	88	228	114	93	238	259	267	198	80
	75%	24	13	21	22	146	48	38	184	259	237	135	53
	80%	22	11*	18	22	136	44	36	166	251	218	132	51
Homogeneous monthly sequence	50%	33	35	26	107	238	115	94	244	226	238	193	139
	75%	26	17	21	57	169	102	77	126	259	251	168	75
	80%	22	15	18	78	156	82	89	116	258	253	170	62
Estate requirements 5 300 ha		14	13	16	9	8	9	9	11	14	8	8	10

Notes :
(1) * indicates that Estate requirements are not met in full.
(2) Assumes seepage loss of 5 mm/d
(3) This table is not directly comparable with Table Nr I.2.3 because the latter is based solely on monthly average flows. This table is based on 5 day flows.

a failure over more than 10 days in the month. The number of failures in any month in the 22 year period has been expressed as a probability of exceedance.

These probabilities for a range of reservoir sizes and 5 mm/day seepage are given in Table I.3.5. This procedure is necessary because of the rule that the Estate may only take half the unregulated river flow but is entitled to all the release from DOSR.

Table I.3.5 shows that improvements in reliability in dry season months occur with increasing reservoir size up to 130 Mm^3, after which little further improvement is shown. Inspection of the 5 day operation runs indicates that the principal reason for this is the restriction on abstractions from the river. By confining abstractions to periods of river flow over 100 m^3/s the number of days when abstractions can be made is restricted and the reservoir frequently fails to fill. The effect of revising this operating rule is described in (c) below.

TABLE I.3.5

Percentage Probabilities of Monthly Estate Requirements Being Met (Abstractions only for River Flows Greater than 100 m^3/s)

Gross Volume	68 Mm^3	95 Mm^3	130 Mm^3	170 Mm^3	200 Mm^3
Apr	68	77	91	91	91
May	100	100	100	100	100
Jun	95	95	95	100	100
Jul	86	86	86	86	86
Aug	100	100	100	100	100
Sep	100	100	100	100	100
Oct	100	100	100	100	100
Nov	100	100	100	100	100
Dec	100	100	100	100	100
Jan	95	95	95	95	95
Feb	68	77	77	77	77
Mar	45	73	77	77	77

(c) Effect of Revised Operating Rules

In view of the fact that the maximum demand downstream of Duduble during the flood season is 83 m^3/s it is possible to relax the restriction on abstractions to DOSR below the 100 m^3/s limit without affecting the operation of JOSR. The operation analysis has therefore been repeated adopting a lower limit of 80 m^3/s for comparison.

Table I.3.6 shows the probabilities of the Estate demands being met with different reservoir sizes for comparison with Table I.3.5. Again the results show that improvements occur as reservoir size increases up to 130 Mm^3. The Estate demand can be met successfully 1 year in 5 with regulation by a reservoir of the order of 95 Mm^3 capacity.

TABLE I.3.6

Percentage Probabilities of Monthly Estate Requirements Being Met (Abstractions for River Flows Greater than 80 m^3/s)

Gross volume	68 Mm^3	95 Mm^3	130 Mm^3	170 Mm^3	200 Mm^3
Apr	68	82	95	95	95
May	100	100	100	100	100
Jun	95	95	95	100	100
Jul	91	91	91	95	100
Aug	100	100	100	100	100
Sep	100	100	100	100	100
Oct	100	100	100	100	100
Nov	100	100	100	100	100
Dec	100	100	100	100	100
Jan	100	100	100	100	100
Feb	77	95	100	100	100
Mar	55	77	86	95	95

It can be concluded that there is a significant advantage in reducing the minimum flow at Duduble for which abstractions could take place. Adopting a minimum flow of 80 m^3/s would not significantly affect the operation of the Jowhar reservoir downstream assuming that JOSR is operated as planned (see Sections 3.4.1 and 3.5.3).

3.4 Pumping from JOSR

3.4.1 Present Operation

The JOSR has been in operation since 1980 and has generally been successful in reducing water shortages for downstream users. Proposed operating rules for the reservoir were drawn up in detail by Sir M. MacDonald & Partners and presented in the Operation and Maintenance Manual (1981).

For the period April 1981 to March 1982 full records of actual operation are available. This period has been modelled using the computer model and adopting the operation rules set out in the manual. The results are presented graphically in Figure I.3.5. On the same figure the actual operation has been plotted for comparison.

Figure I.3.5 shows the recorded discharge at Mahaddey Weyn. The flow available at the Sabuun barrage is lower than this for flows over 100 m^3/s due to the affects of flooding. The figure also shows the operation of supply and outlet canals, reservoir volume and flow below the Estate. Examination of the figure reveals a number of discrepancies between actual and theoretical operation. In particular the failure of the reservoir to fill in either gu or der seasons despite high flood discharges is striking. As shown in Table I.3.7 1981/82 was wetter than average, and the failure to fill the reservoir under these conditions suggests that serious shortages will affect downstream users in drier years if present practice continues.

TABLE I.3.7

Comparison of Average Discharge at Mahaddey Weyn with 1981/82

	1981									1982			
	A	M	J	J	A	S	O	N	D	J	F	M	Year
Average* Mm^3	119	245	139	103	221	302	286	217	124	50	33	49	1890
1981/82 Mm^3	372	394	176	68	237	369	396	170	56	28	25	27	2318
1981/82 flow as % of average	313	161	127	66	107	122	138	78	45	56	76	55	123

Note : * Source full record 1951 to 1983 Chapter 2.

The present practice of restricting inflows to the reservoir is in response to the demands of users in the Jowhar reach between Sabuun and Balad. Prior to regulation of the river at Sabuun these users obtained water by gravity to irrigate seasonal crops. If the offtake at Sabuun is operated according to the rules given in the Operation and Maintenance Manual river flows downstream of the barrage would fall below that necessary for users to obtain supply by gravity. During the feasibility studies for JOSR the Estate was taken as the only user in the Jowhar reach, and the operational rules were designed to provide as much storage in JOSR each flood season while protecting the interests of the Estate.

The conflict of interest which exists at present needs to be resolved urgently since it has an important bearing on future capital works on the Shebelle below Mahaddey Weyn.

3.4.2 Enlargement of JOSR to Allow Pumped Supply to the Estate

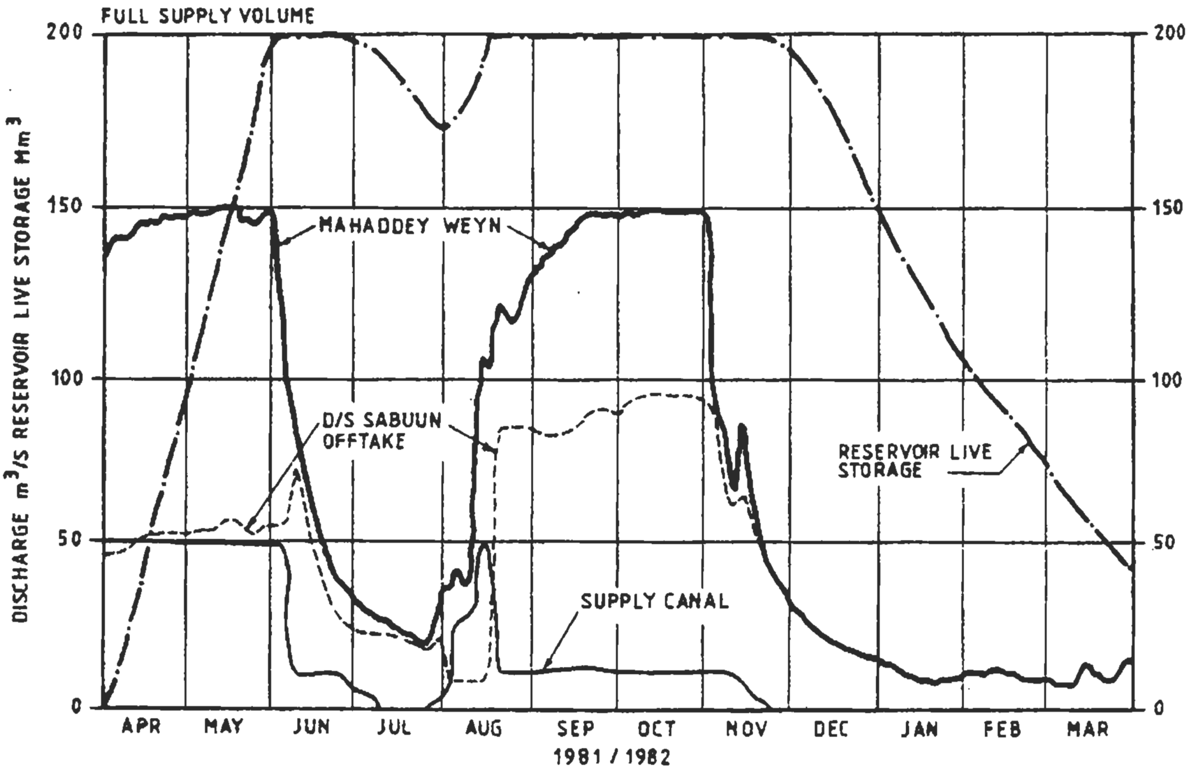
An alternative to building a new storage reservoir is to increase the use of the existing JOSR to include that part of the Estate's requirement which cannot be met from the Shabelle. To achieve this it would be necessary to increase the reservoir capacity to ensure that supplies to downstream users could be met at the same level of reliability. Storage volume in the reservoir can readily be increased by about 55 Mm^3 by raising the embankments by 0.5 m. Minor modifications to the outlet structure would also be required and the supply canal capacity would be increased to 60 m^3/s. Water would be pumped from the reservoir to the Estate at a maximum rate of 6 m^3/s.

Table I.3.8 summarises the criteria for the operation runs. It has been assumed that back pumping to the Estate has an equal priority with releases to meet downstream requirements. In all other aspects the model is unchanged from earlier studies, no attempt has been made to incorporate the demands of other users above Balad. Table I.3.9 illustrates the operation of JOSR on a 5-day basis for the water year 1973/74. The letter 'J' indicates a failure to meet the Estate needs and 'F' a failure to meet the downstream requirements.

Figure I .3.5

Comparison of Theoretical and Actual Operation of JOSR 1981/82

Theoretical Operation



Actual Operation

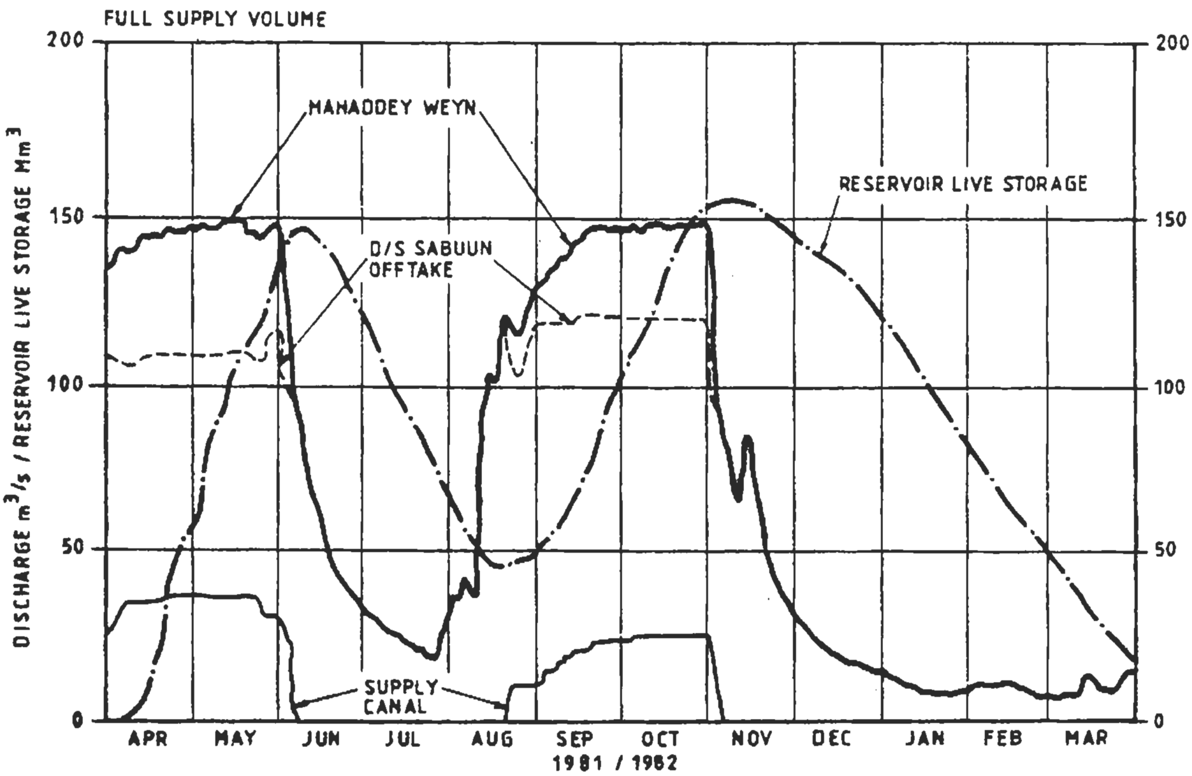


TABLE I.3.8

Jowhar Offstream Storage Reservoir with Increased Capacity
Summary Details

Gross storage	:	260	Mm^3
Dead storage	:	5	Mm^3
Inlet capacity	:	60	m^3/s
Intake canal losses	:	1	m^3/s
Outlet canal capacity	:	25	m^3/s
Outlet canal losses	:	0.1	m^3/s
Seepage losses	:	2	mm/d
Inflow sequence	:	Table I.2.2	

	J	F	M	A	M	J	J	A	S	O	N	D
Evaporation (mm/d)												
	6.6	7.4	7.6	6.5	6.0	5.4	5.3	6.0	6.7	6.1	6.2	6.1
Estate demand 5 300 ha (m^3/s)												
	5.09	5.51	6.04	3.29	3.02	3.39	3.23	3.98	5.57	2.81	3.18	3.92
Downstream requirements (m^3/s)												
	11.7	7.8	7.8	5.3	14.5	24.3	22.7	8.0	12.4	25.6	30.0	24.7

The 5-day flows downstream of the reservoir outlet have been aggregated and the monthly totals analysed to check that the back pumping had no adverse effect on the reliability of supplies to downstream users. The results show that reliability of downstream supply was not adversely affected by back pumping.

Further examination of the operation run shows that the Estate's requirements were not met in 5 months in 22 years despite back pumping up to 6 m^3/s. Failure occurred in July 1955, April 1961, March 1974, April 1974 and March 1975. This represents a probability of the Estate's needs being met of over 90% in all months.

3.5 Comparison of Major Storage Options

3.5.1 Performance

In order to reduce the probability of the Estate's demands not being met to one year in five or better, three schemes can be compared.

(i) enlarging JOSR to 260 Mm^3 and pumping supplies to the Estate when necessary;

(ii) constructing a 130 Mm^3 reservoir at Duduble with abstractions for river flows in excess of 100 m^3/s (this option only provides 77% reliability in February and March);

Table Nr I.3.9
Jowhar Reservoir - Sample Operation Run
with Pumped Supply to the Estate

1973/74

MTH	PRD	MODE	U/S BARRAGE	JSE REQS	D/S JSE	SUPPLY CANAL	D/S REQS	PUMP REQS	OUTLET CANAL	D/S OUTLET (END PRD.)	STORED VOLUME
APR	1	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	96.1
	2	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	90.2
	3	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	84.3
	4	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	78.5
	5	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	72.8
	6	5	3.0	3.3	1.5		5.3	1.8	3.9	5.3	67.1
MAY	1	4	30.0	3.0	27.0	.0	14.5			27.0	64.2
	2	4	73.0	3.0	70.0	.0	14.5			70.0	61.4
	3	1	38.0	3.0	14.5	20.5	14.5			14.5	66.6
	4	1	35.0	3.0	14.5	17.5	14.5			14.5	70.5
	5	1	57.0	3.0	14.5	39.5	14.5			14.5	83.4
	6	1	73.0	3.0	14.5	55.5	14.5			14.5	106.7
JUNE	1	1	87.0	3.4	24.3	59.3	24.3			24.3	127.7
	2	1	45.0	3.4	24.3	17.3	24.3			24.3	131.2
	3	5	24.0	3.4	20.6		24.3		3.8	24.3	126.2
	4	5	18.0	3.4	14.6		24.3		9.8	24.3	118.6
	5	5	17.0	3.4	13.6		24.3		10.8	24.3	110.7
	6	5	15.0	3.4	11.6		24.3		12.8	24.3	102.0
JULY	1	5	12.0	3.2	8.8		22.7		14.0	22.7	92.9
	2	5	10.0	3.2	6.8		22.7		16.0	22.7	83.0
	3	5	13.0	3.2	9.8		22.7		13.0	22.7	74.5
	4	5	15.0	3.2	11.8		22.7		11.0	22.7	67.0
	5	5	17.0	3.2	13.8		22.7		9.0	22.7	60.5
	6	3	31.0	3.2	22.7	5.1	22.7			22.7	59.5
AUG	1	1	73.0	4.0	9.0	60.0	8.0			9.0	80.8
	2	1	65.0	4.0	8.0	53.0	8.0			8.0	99.2
	3	1	57.0	4.0	8.0	45.0	8.0			8.0	114.2
	4	1	92.0	4.0	28.0	60.0	8.0			28.0	135.2
	5	1	100.0	4.0	36.0	60.0	8.0			36.0	155.9
	6	1	100.0	4.0	36.0	60.0	8.0			36.0	180.5
SEPT	1	1	105.0	5.6	39.4	60.0	12.4			39.4	201.0
	2	1	121.0	5.6	55.4	60.0	12.4			55.4	222.3
	3	1	113.0	5.6	47.4	60.0	12.4			47.4	245.7
	4	1	113.0	5.6	47.4	60.0	12.4			64.3	260.0

RESERVOIR IS FULL ON DAY 3 OF PERIOD 4

MTH	PRD	MODE	U/S BARRAGE	JSE REQS	D/S JSE	SUPPLY CANAL	D/S REQS	PUMP REQS	OUTLET CANAL	D/S OUTLET (END PRD.)	STORED VOLUME
	5	2	105.0	5.6	91.4	8.0	12.4			91.4	260.0
	6	2	100.0	5.6	86.4	8.0	12.4			86.4	260.0
OCT	1	2	100.0	2.8	89.6	7.6	25.6			89.6	260.0
	2	2	98.0	2.8	87.6	7.6	25.6			87.6	260.0
	3	2	73.0	2.8	62.6	7.6	25.6			62.6	260.0
	4	2	100.0	2.8	89.6	7.6	25.6			89.6	260.0
	5	2	105.0	2.8	94.6	7.6	25.6			94.6	260.0
	6	2	100.0	2.8	89.6	7.6	25.6			89.6	260.0
NOV	1	2	92.0	3.2	81.2	7.6	30.0			81.2	260.0
	2	2	53.0	3.2	42.2	7.6	30.0			42.2	260.0
	3	5	31.0	3.2	27.8		30.0		2.3	30.0	256.0
	4	5	22.0	3.2	18.8		30.0		11.3	30.0	247.9
	5	5	18.0	3.2	14.8		30.0		15.3	30.0	237.5
	6	5	15.0	3.2	11.8		30.0		18.3	30.0	225.5
DEC	1	5	12.0	3.9	8.1		24.7		16.7	24.7	214.0
	2	5	10.0	3.9	6.1		24.7		18.7	24.7	201.5
	3	5	9.0	3.9	5.1		24.7		19.7	24.7	188.6
	4	5	7.0	3.9	3.5		24.7	.4	21.3	24.7	174.9
	5	5	6.0	3.9	3.0		24.7	.9	21.8	24.7	160.9
	6	5	5.0	3.9	2.5		24.7	1.4	22.3	24.7	143.7
JAN	1	5	3.0	5.1	1.5		11.7	3.6	10.3	11.7	133.6
	2	5	3.0	5.1	1.5		11.7	3.6	10.3	11.7	123.7
	3	5	3.0	5.1	1.5		11.7	3.6	10.3	11.7	113.8
	4	5	.0	5.1	.0		11.7	5.1	11.8	11.7	102.8
	5	5	.0	5.1	.0		11.7	5.1	11.8	11.7	91.9
	6	5	.0	5.1	.0		11.7	5.1	11.8	11.7	79.0
FEB	1	5	.0	5.5	.0		7.8	5.5	7.9	7.8	69.6
	2	5	.0	5.5	.0		7.8	5.5	7.9	7.8	60.4
	3	5	.0	5.5	.0		7.8	5.5	7.9	7.8	51.4
	4	5	.0	5.5	.0		7.8	5.5	7.9	7.8	42.7
	5	5	.0	5.5	.0		7.8	5.5	7.9	7.8	34.2
	6	5	.0	5.5	.0		7.8	5.5	7.9	7.8	29.3
MAR	1	5	.0	6.0	.0		7.8	6.0	7.9	7.8	21.1
	2	5	.0	6.0	.0		7.8	6.0	7.9	7.8	13.5
	3	5	.0	6.0	.0		7.8	6.0	7.9	7.8	6.3
	4	5	.0	6.0	.0		7.8	6.0	7.9	2.7	5.0

RESERVOIR IS EMPTY ON DAY 1 OF PERIOD 4

MTH	PRD	MODE	U/S BARRAGE	JSE REQS	D/S JSE	SUPPLY CANAL	D/S REQS	PUMP REQS	OUTLET CANAL	D/S OUTLET (END PRD.)	STORED VOLUME			
	5	4	.0	6.0	.0	.0	7.8			.0	3.9	*	J	F
	6	4	.0	6.0	.0	.0	7.8			.0	3.2	*	J	F

TOTAL INFILTRATION LOSSES FOR YEAR ARE 59.73 MCM
TOTAL EVAPORATION LOSSES FOR YEAR ARE 176.02 MCM

(iii) constructing a 95 Mm^3 reservoir at Duduble with abstractions for river flows in excess of 80 m^3/s.

All the above options assume that the Estate only has the right to abstract 50% of the unregulated flow in the dry season. A change in this rate to, for example, the maintenance of a constant compensation flow of 2 Mm^3/month, would significantly alter dry season reliabilities.

3.5.2 Costs

Cost estimates have been prepared for the three options described above.

The 130 Mm^3 Duduble reservoir would cost some SoSh 110 million. This would be reduced to SoSh 93 million for a gross storage capacity of 95 Mm^3. Increasing the volume of JOSR by 55 Mm^3 and providing a pumped supply to the Estate would cost an estimated SoSh 62 million.

The use of JOSR is thus the option with the lowest capital cost. However there would be the additional recurrent cost of pumping. Average pumping costs have been estimated from an examination of the computer programme output. Pumped volumes and pumping heads for each 5-day period over the 22 year record have been aggregated to give an average annual power demand. From this the recurrent cost of fuel, oil, maintenance and replacement has been estimated at SoSh 1.1 million/year.

3.5.3 Conclusions

The following conclusions can be drawn.

(i) the enlargement of JOSR is the cheapest alternative for improving water supplies to the Estate, even taking into account recurrent costs;

(ii) significant improvements to water availability for the Estate can be achieved simply by changing the "50% rule" (i.e. allowing the Estate to take more than half the unregulated river flow);

(iii) before any further offstream storage is considered, it is essential that the present inefficient operation of JOSR is improved.

SECTION B

IRRIGATION

CHAPTER 4

PRESENT IRRIGATION SYSTEM

4.1 Irrigation Supply and Distribution System

4.1.1 General

The Estate currently obtains its irrigation water from two offtakes on the left bank of the Shabelle river. The Luigi di Savoia offtake, immediately upstream of the weir across the Shabelle, is the original supply constructed in the 1920s. It presently serves some 60% of the irrigable area through 6.35 km of main canal (Luigi di Savoia and Duchessa d'Aosta canals) and 26 km of secondary canal (S1, S2, S3 and S4 canals).

The 21st October offtake, located approximately 3.5 km up-river from the weir, was constructed in 1969 to 1971 as a flood escape. Subsequently it was provided with cross regulators and offtakes to serve the northern and eastern sectors of the Estate. The 21st October main canal is some 19.2 km long and serves its commanded area (approximately 40% of the total irrigable area) through directly offtaking tertiary canals.

Water levels in the river are controlled by the weir which is some 85 m long with an average crest level of about 103.8 m. The condition of the weir is not known since inspection during the study was impossible due to substantial river flows. The fact that the weir has survived for 60 years is indicative of sound design and construction; however, it is to be expected that the fabric of the structure will have deteriorated somewhat during its life.

4.1.2 The River Intakes

The river intake structures are similar in that they both have three vertical lift sluice gates, those on the Luigi regulator being 2.0 m wide and those on the 21st October structure somewhat wider at 2.3 m. The Luigi regulator is about 60 years old and is of brick arch construction which is generally sound. The 21st October regulator is much more recent, being constructed around 1969, and is of reinforced concrete. Sill level of the Luigi intake structure is about 102.0 m, some 1.8 m below weir crest level. The 21st October intake sill is a little higher at about 102.355 m.

Both intakes have short reaches of channel between the river and the head regulator structure. These intake channels trap some of the sediment carried in the river flow, but neither channel is large enough to act as an effective sediment trap and thus the water entering the canal systems is still highly charged with silt. It is reported that the intake for the Luigi canal has to be cleared out by dragline about once a month, and more frequently when the river sediment load is high. This compares with the much less frequent clearing of the 21st October canal intake channel of about three times a year.

The intake channels were surveyed in June 1983 during the fieldwork programme, and the results of this survey are plotted on Drawings Nr 12700/2 and 4. In view of the rate at which these channels silt up, especially the Luigi intake, the survey data are of limited use.

The Luigi intake channel is about 270 m long with a design bed width of 12 m. It suffers from severe sediment deposition for a number of reasons. The layout of the intake is illustrated in Figure I.4.1. It is immediately clear that the existing arrangement promotes the deposition of sediment in the slack-water zone at the mouth of the intake channel. The intake channel is on the inside of a slight bend in the river and, because of the angle of the channel to the river, the flow has to turn through 125^{o}. The end result is an increase in the sediment load in the water entering the intake channel. The coarser fractions settle out progressively in the intake channel, whilst the finer sediment is carried through into the canal system.

The 21st October canal head regulator is located less than 100 m from the edge of the river. The short intake channel upstream (about 90 m long) is totally ineffective as a sediment trap, so suspended sediment is carried through into the canal. However, the intake is located on the outside of a bend in the river, and is 3.5 km upstream of the weir, and therefore does not suffer from the same problems as the Luigi intake.

Under the current operating regime both intakes remain open 24 hours/day for most of the year. Generally the gates are only closed when the river salinity is unacceptably high or when there has been substantial rainfall rendering irrigation unnecessary.

4.1.3 The Canals

Figure I.4.2 shows the basic layout of the Estate and the alignments of the canals (a much more detailed layout at 1 : 25 000 scale is presented on Drawing Nr 12700/1). There are two canal systems, one based on each of the river intakes, although some fields can be served from either system. An inventory of canal structures, as assessed during June 1983, is given in Appendix A to this annex.

The Luigi di Savoia main canal serves secondary canals S1, S2 and S5, and continues as the Duchessa d'Aosta canal which in turn serves secondary canal S3. Secondary canal S4 links the tails of canals S2 and S3 and connects them to the West drain which is pumped to provide irrigation water to the southern end of the Estate. The Middle drain is also pumped at its southern end, and it is possible to discharge the drainage water into the tail end of canal S3. Canal S5 although connected to the Luigi canal has been abandoned and the fields it used to serve are now served from the 21st October canal. The 21st October canal serves its irrigated area through directly offtaking tertiary canals.

Irrigation water is passed into the secondary and tertiary canals through head regulator structures which generally have manually operated steel lifting gates to control the flow. Secondary canals have similarly gated cross regulators but in the case of these structures most of the gates are missing or broken. Under the present circumstances of silted-up and overgrown canals the cross regulators serve little purpose since the canals have to operate at high water levels just to get the required flow. There are no facilities for flow measurement anywhere in the distribution system.

Many of the canal structures in the Luigi system date from the 1920s and are of brick construction. In spite of their age most are reasonably sound but almost all cross regulators and about half the tertiary head regulators require new control gates. The structures on the 21st October canal are much more recent, dating from the early 1970s, and are generally sound. However, this canal also requires complete rehabilitation of control gates. The major problem which

Figure I. 4.1

Layout of Luigi Canal Intake

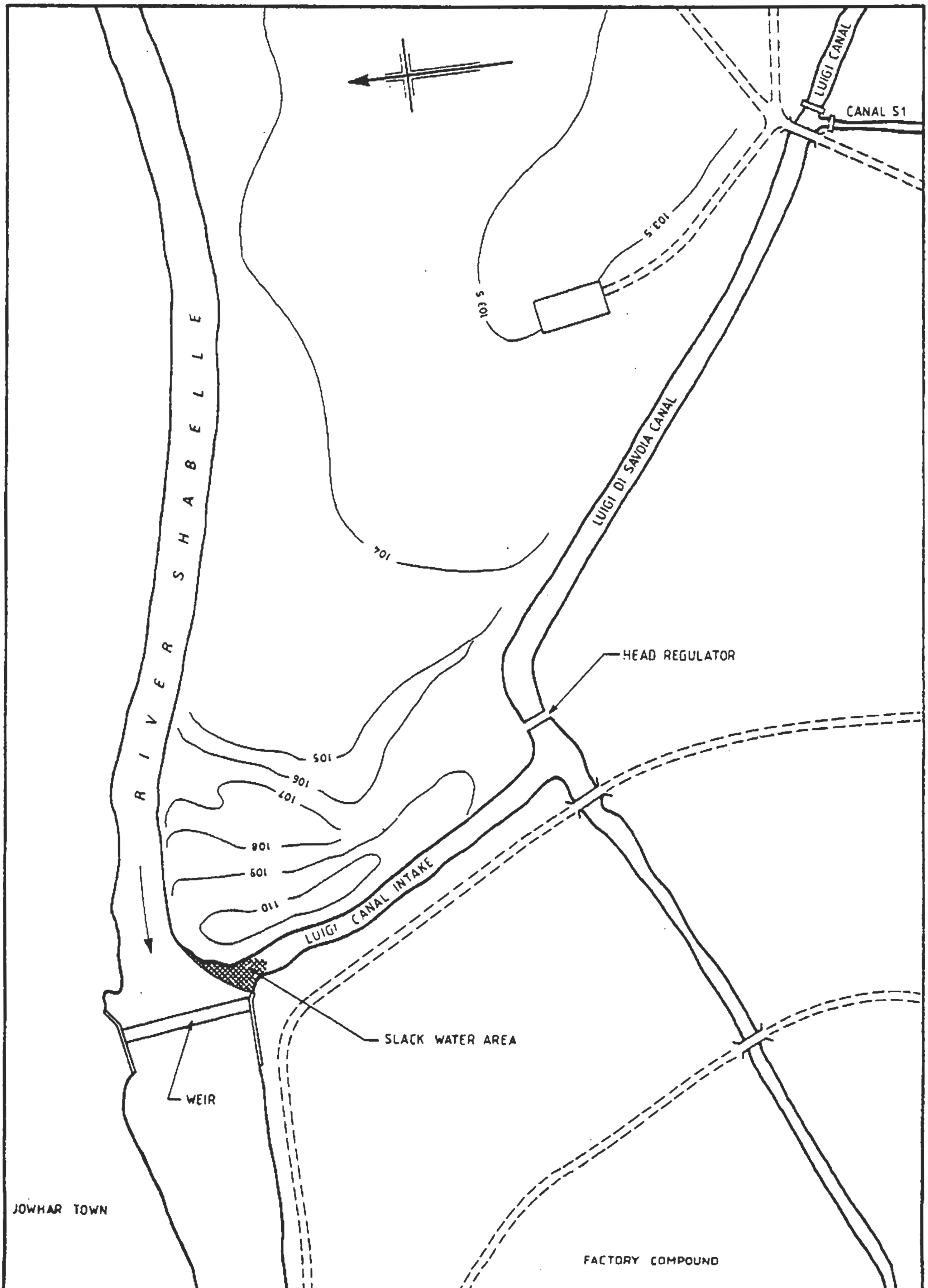
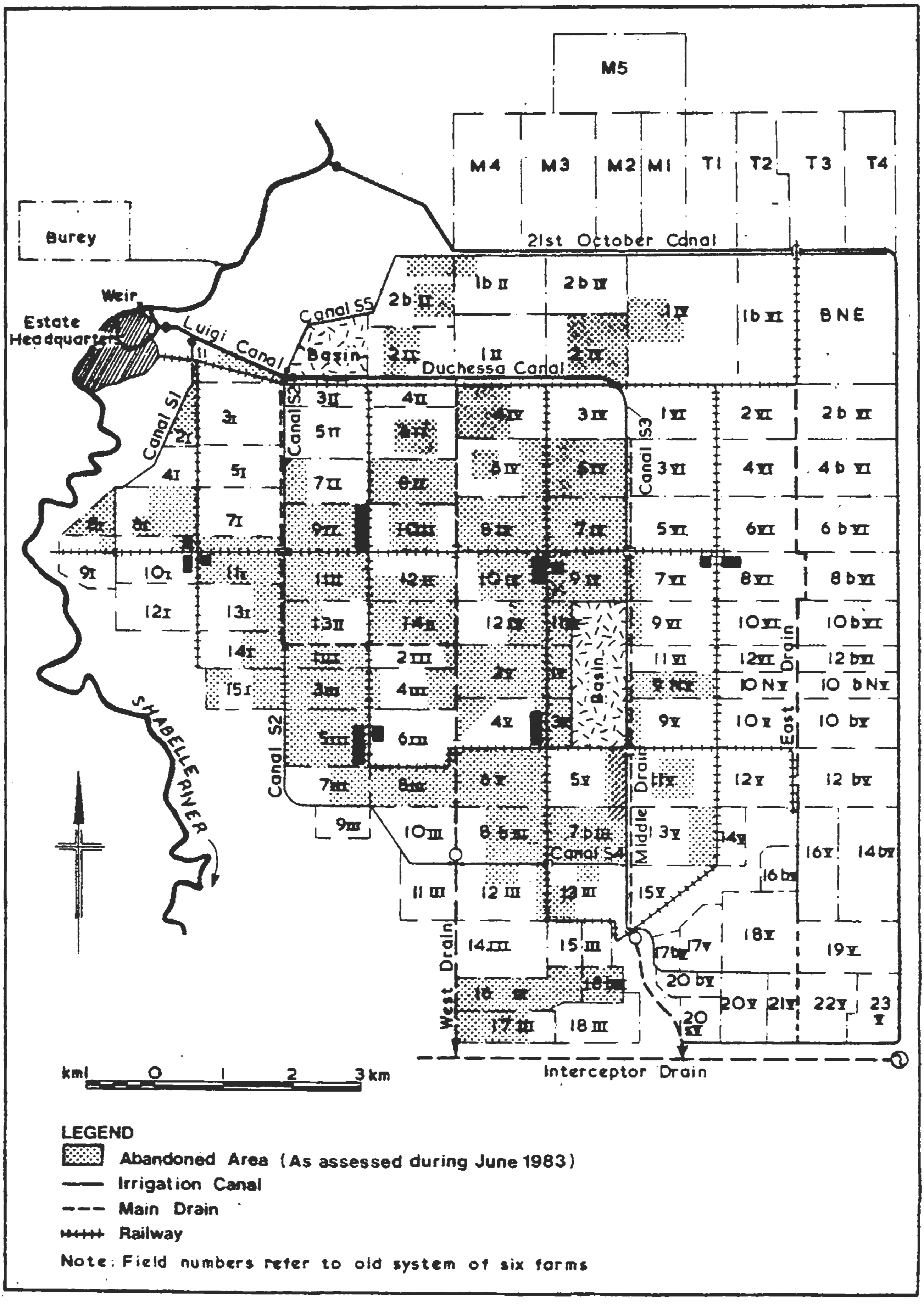


FIGURE 1.4.2

Jowhar Sugar Estate Existing Layout



afflicts the steel lifting gates throughout the Estate is corrosion. However, there are also many examples of loose and/or distorted frames and absence of lifting gear. It is clear that the standard of construction of the water control equipment is poor, particularly in that steel sections are too thin to resist corrosion and distortion.

Two serious problems which all the canals suffer from are siltation and weed growth. Siltation results from the totally inadequate provisions for removing sediment from the river water before it is passed into the canals. All canals on the Estate are affected by siltation, but the worst affected are those at the head of the Luigi system. Depths of silt in excess of 1.0 m on the beds of canal are not uncommon. During the fieldwork for this study samples were taken from sediment deposits and analysed for particle size. The results are presented in Figure I.1.1. The analyses show that the material deposited in canals is generally very fine, virtually no material being larger than 0.2 mm (find sand). Furthermore the sandy deposits are only found in the upper reaches of the canal systems; elsewhere the sediments are almost exclusively in the silt fractions (0.002 to 0.06 mm). The fact that such fine material settles out is indicative of the very slow flow in the canals (flat slopes) and the extensive weed growth which itself promotes sediment deposition.

Weed growth in canals and drains results in reduced channel capacity and makes inspection of the channels very difficult. Furthermore weed seeds deposited in the irrigation water inevitably increase the weed problem in the cane fields. The most severe problems are caused by reeds which grow very densely and at an alarming rate, regrowth being as quick as 2 to 3 weeks after cutting.

A further problem which affects some parts of the Estate is excessive command in canals, particularly tertiary canals. This tends to increase seepage losses from the canals and increases the risk of a serious breach occurring. The most common cause of excessive command in tertiary channels is the practice of serving up-hill, that is the secondary canal is at the lower end of the field unit.

4.2 Operation of the Canal System

4.2.1 General

The current operating regime is based on 24-hour flow throughout the year with the system being closed down only because of high river salinity or after substantial rainfall. Throughout the year river water level at the weir varies between 103.8 m at low flows to 105.3 m during high floods. As would be expected, the two main canal systems have been designed for a water level based on the low river level, so that irrigation is possible throughout the year. However, because these design water levels cannot easily command all of the Estate (the elevation of some fields being too high) the operating staff tend to run the canals with water levels higher than design whenever river levels permit. Furthermore the extensive silt deposits in canals require higher water levels to achieve design flows. Thus, when river levels are relatively high, canal water levels are similarly high and, as a result, many of the canal structures have inadequate freeboard or are submerged. Such a situation was observed for most of the period of fieldwork during May and June. At the end of June, when the river level started to fall, the full extent of canal siltation was revealed. In fact, because of an extensive silt bank at the mouth of the Luigi intake, a drop in river level of only 0.3 m resulted in no water entering the Luigi system.

The practice of maintaining flows in canals 24 hours per day would be acceptable if it was matched by 24-hour field irrigation. Unfortunately this is not often the case. Irrigation does take place at night, but not to the same extent as in the daytime, and it is poorly supervised and inefficiently executed. Thus during the night water tends to run to waste, frequently being discharged into abandoned fields and eventually finding its way into the drainage system. Wastage of water during the daytime could also be observed, even though at the time of the study there were many fields in need of irrigation. This is a direct result of the poor condition of the canals, it being impossible in some areas to pass sufficient water through the silted up and overgrown channel sections. Some of the 'wasted' water is in fact used for irrigation since it passes into the drains and is subsequently pumped back into the canal system lower down.

Most of the cross regulator structures become redundant when excessively high water levels are maintained in the canals. Thus, under the present operating conditions, the fact that many such regulators have no gates is not so important.

4.2.2 System Operation

The operation of the irrigation system has been adapted to suit the prevailing poor condition of the supply and distribution system. This generally means flow in all main and secondary canals whenever there is water available, whether it is needed or not.

Management of the supply and distribution system is the responsibility of the Irrigation Service of the Agricultural Department. It is understaffed and lacks the technical expertise required. Control of flow in tertiary and quaternary canals, and field irrigation therefrom, is the responsibility of the Farm Managers, and is executed reasonably well in view of the constraints imposed by the deterioration of the system.

The Irrigation Service is presently divided into two sections - irrigation and land preparation - under the overall control of the Irrigation Manager. In total there are some 80 staff members in the Irrigation Service comprising the manager, one chief of section (irrigation; chief of section post for the land preparation unit is vacant), two assistants, 12 canal guards (gate operators), several gangers, 40 plant operators, and a number of drivers, labourers, etc. Both the Irrigation Manager and the chief of irrigation section have been on the Estate for a substantial number of years. However, their experience in the field of irrigation is limited by what they have learned in-situ. In 1976 both men were surveyors and, indeed, during this present study the chief of the irrigation section was seconded to the team as a surveyor.

The irrigation section is in charge of the operation and maintenance of the canal systems as far as the tertiary canal head regulators, and for the maintenance of main drains. The land preparation section deals with land levelling, uprooting and ploughing.

Because there is no simple means of estimating flows in canals, no records are kept of channel discharges. In fact the only records of irrigation are those kept by each farm on a standard form. As well as records of irrigation applications, the standard forms include details of planting and harvesting dates, variety, mechanical and manual cultivation, fertiliser application, rainfall and cane yields. These records are regularly updated and, as far as can be assessed, provide an accurate history of operations. Unfortunately, the records of irrigation give only the dates of application - there being no way to measure the quantity applied.

The detailed field records were studied for Farms 1, 2 and 3 (new farm numbering system, see Appendix E) - a total of some 46 fields in the western half of the Estate. In the first 6 months of 1983 most fields which were growing cane had received at least one irrigation. No field had received more than four irrigations. Of the fields which had received two or more irrigations, 14 were for cane newly planted in 1982, and 6 were for a ratoon crop, preference clearly being given to the more recently planted crop. Some 6 fields or parts of fields received no irrigation at all although they are presently under cane.

The following conclusions can be drawn from these records :

- irrigation intervals are frequently too long;
- the distribution system in its present condition is not capable of providing irrigation water to all parts of the Estate.

From observations in the field it is also obvious that, quite often, irrigation applications are too large - this is evidenced by extensive waterlogged areas in some fields. It would therefore seem likely that, by reducing the depth of application and shortening the irrigation interval, a better irrigation regime could be established. The problems of not being able to get water to certain fields are symptomatic of the deficiencies in the distribution system, the principal cause being reduced conveyance of channels resulting from silting up and weed growth.

4.2.3 Maintenance

The major items of maintenance are silt removal and weed clearance. Reinstatement of earth roads after rain is also a significant part of the maintenance programme. On a much smaller scale are such items as gate painting and greasing and minor structural repairs, the latter being carried out by the Building Department.

Although the Estate has a large fleet of mechanical plant much of it is non-operational due to old age or shortage of spare parts. In June 1983 the following items of operational plant were available to the Irrigation Section:

- one dragline (link belt)
- two hydraulic excavators
- two bulldozers (one Fiat, one Caterpillar)
- one grader (Caterpillar 140G)

The dragline is used for desilting work on the Luigi and 21st October canal intake channels, and for similar work on the larger canals. The hydraulic excavators are used for desilting and reforming secondary canals. The bulldozers and grader are most often used for road maintenance.

Generally speaking these items of plant are appropriate for the maintenance work but are insufficient in number and, in the case of the dragline, have insufficient capacity for the large maintenance problems, particularly in view of the present state of canals. The quality of operating staff, as assessed by observation during the field work period, is quite good.

Maintenance work does not follow a routine programme - the shortage of plant and the large backlog of work makes this impossible. Maintenance is generally carried out in response to emergency situations, when canals are in danger of overtopping or where the channel is so silted/overgrown as to prevent sufficient irrigation water passing through.

Material removed from canals during the reforming/cleaning process is generally dumped on the canal banks. This mixture of wet silt and plant growth makes the road on the bank impassable and, even when the deposits dry out, it is necessary to bulldoze or grade the surface to restore access. Quite often this problem occurs because there is no alternative place to dump the dredged material.

On many canal banks the growth of weeds, reeds, bushes and even trees has been allowed to continue unchecked, except for the occasional passage of a bulldozer or hydraulic excavator. The dense growth makes access difficult and channel inspection impossible, as well as increasing water losses from the canals.

Hand cutting of weeds in canals is generally organised by the Farm Managers, but the rapid rate of regrowth makes it hardly worthwhile. Weeds are also cleared during the canal reforming process using excavators or draglines.

4.3 Field Irrigation and Drainage

4.3.1 Field Layout

The basic unit of the irrigation system is the field unit which is an area of land served by a gated outlet on the secondary canal. The field size varies considerably from 30 ha to 130 ha but generally 70 ha is an average size. Fields are usually rectangular, approximately 1 000 m wide and up to 700 m long. A typical field layout is shown in Figure I.4.3.

The field is divided into strips of land 100 m wide called fascias. Water is distributed across the top of the field by the tertiary canal and 0.3 m pipe outlets are provided for quaternary channels which supply individual fascias. Within the fascia, aquiole channels spaced every 25 m distribute water to the cane furrows. Quaternary channels are permanent canals, but aquioles are reformed regularly because inter-row cultivation destroys the existing channel formation.

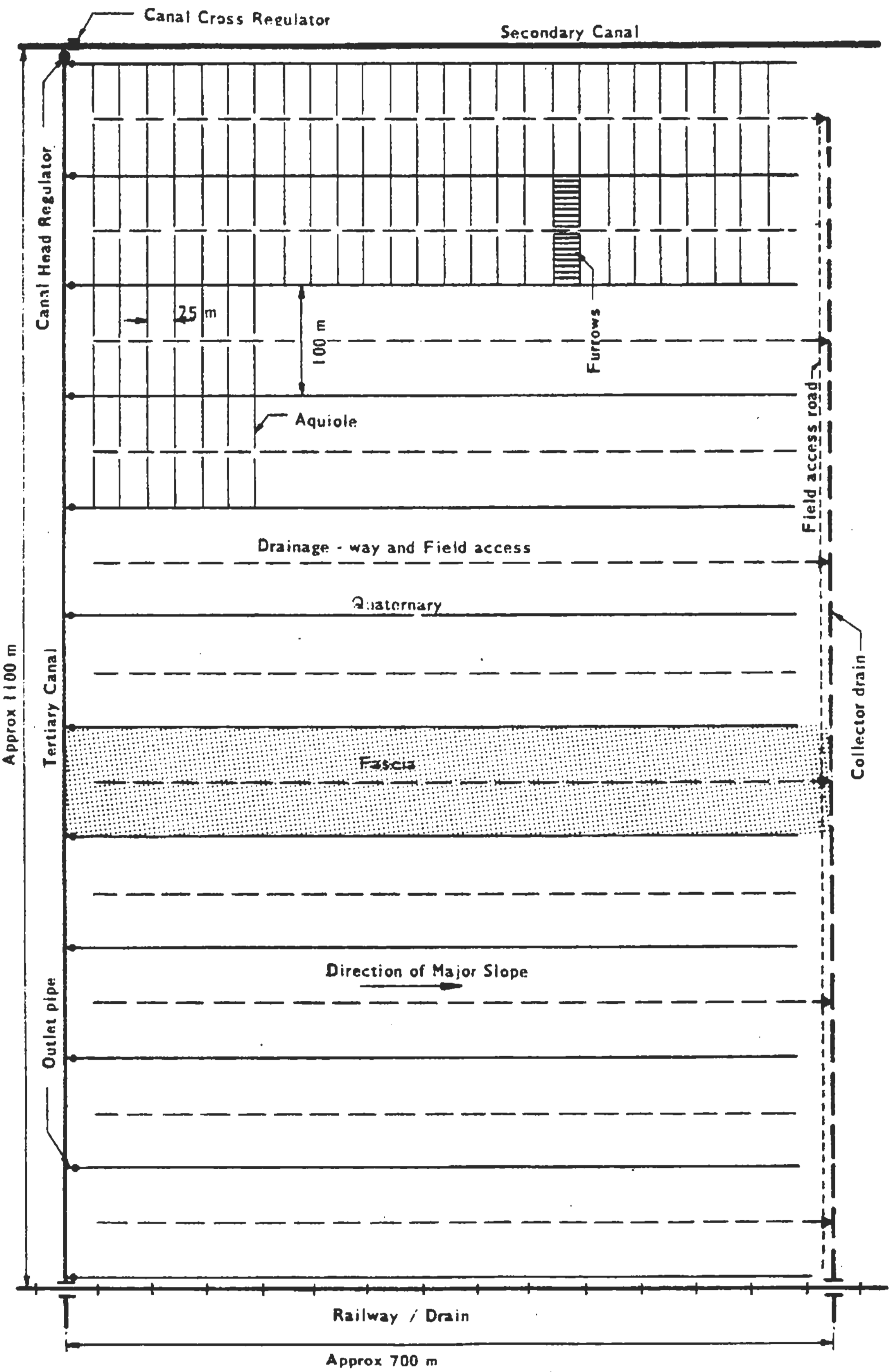
Most fields are provided with a collector drain along the bottom of the field which connects with the main disposal system. The drains are generally not deep enough to influence groundwater movement and are used to collect surface runoff resulting from excess irrigation or rainfall. Within the field is a system of drainage-ways which are strips of land, approximately 1 m wide, left uncultivated down the middle of each fascia to drain surface water from the field into the collector drain. Field observations, however, have shown that the drainage-ways are generally ineffective because of blockages caused by the construction of aquiole channels across the line of the drain. The main purpose of drainage-ways is evident in the harvesting period when they are used as a means of access into the fascias.

4.3.2 Irrigation Practices

The individual farms are responsible for the control and distribution of water once it leaves the secondary canal - that is, from downstream of the tertiary canal head regulator.

Figure I.4.3

Existing Field Layout



Irrigation is entirely by surface methods and is based on a furrowed-basin system. The basin size is 50 m wide by 25 m long and is bounded by the quaternary and aquiole channels. Within the basin are furrows at 1.7 m spacing running down the 25 m length in the direction of the major slope.

The furrows are formed in the early stages of plant growth but are gradually broken down through periodic inter-row cultivation and the system essentially reduces to basin irrigation.

The basin size appears to be dictated by the micro-topography of the field. Basins have been kept small to avoid the costs involved in accurately levelling and grading the field to produce larger basins or longer furrows.

Water is supplied to the basins from the aquiole by breaching the channel. Siphon pipes are not normally used on the Estate but have been used for experimental purposes in the Burey area.

Basin irrigation methods were developed at a time when the Estate was small and the availability of water for irrigation was not an important consideration. The method was obviously chosen for its simplicity, but experience has shown that much supervision is required to ensure that the field is watered properly and there is often prolonged flooding of the lower parts of the fascia. This leads to ineffective use of irrigation water. As the cultivated area has now increased considerably, water shortage often occurs and there is now a need to make the most effective use of irrigation water by adopting a more efficient and more easily controlled irrigation method.

4.4 Summary

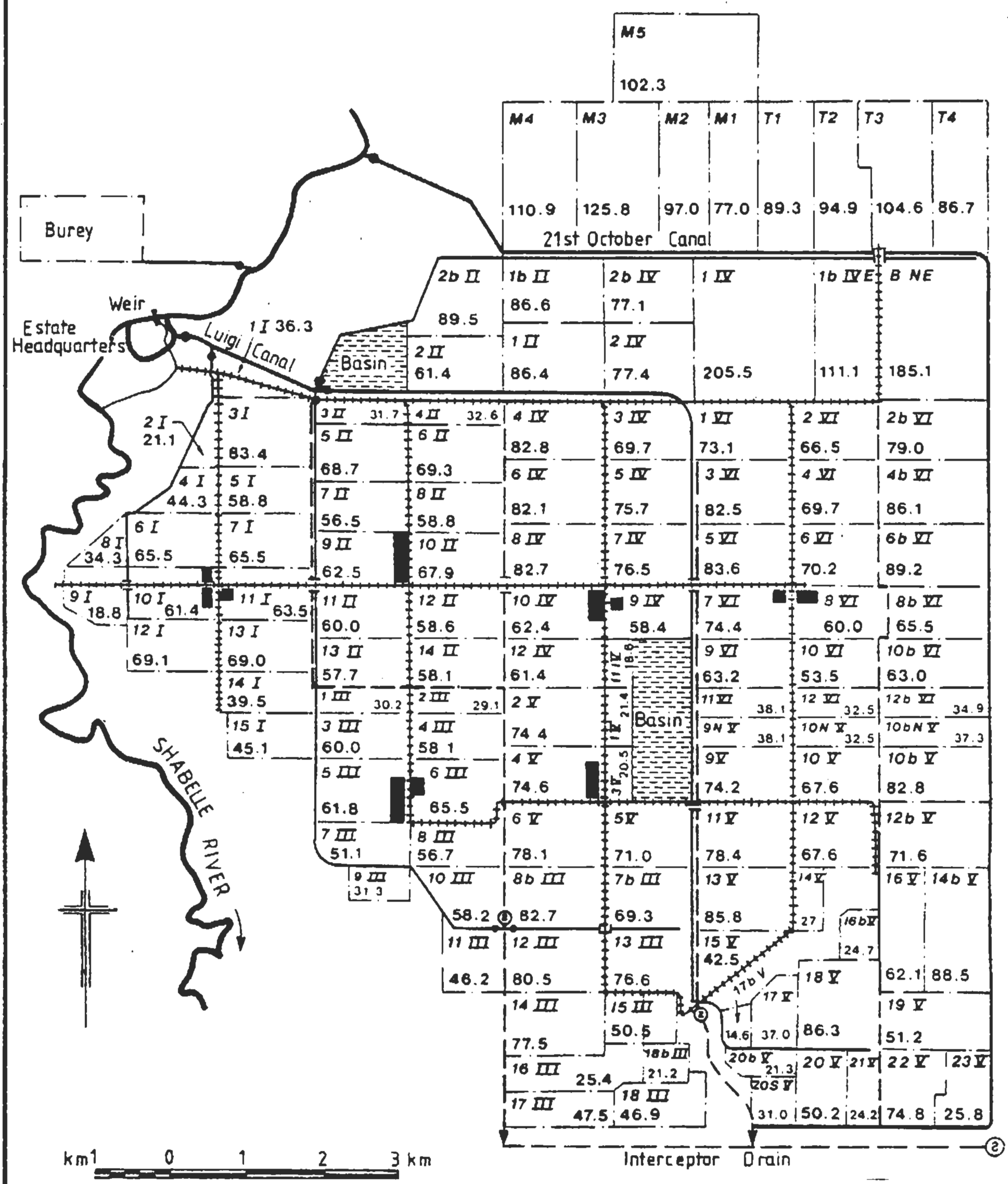
In its present condition the irrigation system at Jowhar places a considerable constraint on cane production. The system is not capable of providing an assured supply of water to all parts of the Estate at any time. The major problems are summarised below:

- sediment in canals reducing their capacity,
- extensive weed and reed growth in channels, restricting flow and promoting further sediment deposition,
- absence of flow measurement structures,
- poor condition of many control gates, making flow regulation difficult,
- parts of the Estate are too high to be commanded by the system at low river levels,
- shortage of adequate mechanical plant for maintenance,
- shortage of trained staff,
- poor control of in-field irrigation, especially at night.

Ways in which these problems can be reduced or overcome are discussed in subsequent chapters.

Figure I.4.4

Jowhar Sugar Estate
Existing Net Field Areas



LEGEND

15 I Field Reference Number

61.8 Net Field Area (ha)

NOTE

1. NET FIELD AREA ESTIMATED AS 0.93 x GROSS FIELD AREA (BASED ON EXISTING FIELD LAYOUT)

CHAPTER 5

IRRIGATION REHABILITATION REQUIREMENTS

5.1 Introduction

There is no unique solution to the problems experienced at Jowhar, rather there is a whole spectrum of rehabilitation options. Nor can the improvements required to the irrigation system be considered in isolation. Such improvements must be matched by equivalent works in the drainage system. Furthermore the extent and nature of the rehabilitation works is clearly dependent on the extent and location of the proposed cane-growing areas, and of areas of any other crops which might be introduced on a significant scale.

This chapter discusses the range of rehabilitation options considered, indicating the advantages and disadvantages of each. It has generally been assumed that canals would be rehabilitated to have sufficient capacity to provide irrigation for sugar cane on all of the Estate's 7 950 ha net. This policy ensures that even if initially the Estate is only partially rehabilitated, ultimately it would be possible to irrigate the full net area.

The question of water availability and possible ways of improving this are discussed separately in Chapters 2 and 3 of this annex. Irrigation water requirements are calculated in Annex II.

5.2 The Sediment Problem

5.2.1 Introduction

The problem of sediment deposition is probably the single most important cause of poor performance in the past few years. Certainly it is the easiest to identify as a problem. It is only over the last 7 years or so that sediment deposition has become such a serious constraint - the combined effect of a significant increase in river sediment load and a general run-down in the Estate's maintenance plant.

Sediment load in the river is generally at its highest at the beginning of the gu season floods in April and May. There is considerable scope during this period for closing the main river intakes, because irrigation requirements are relatively low. Continuous monitoring of sediment load in the river could readily be achieved by taking daily samples from which a rough idea of the sediment content can be gained by allowing the material to settle out. A policy of intake closure would clearly reduce the sediment problem.

5.2.2 General Design Considerations

(a) Introduction

In considering sediment control works it is important to understand the nature of sediment and its transport. For engineering purposes three categories of sediment can be distinguished. These are 'bed load', which moves along or very near the river bed; the suspended bed material load which consists of particles larger than 0.06 mm diameter kept in suspension by turbulent action of the flow; and 'wash load' which consists of suspended material finer than 0.06 mm diameter.

While the quantities of bed load and suspended bed material load are limited by channel flow conditions, no similar limits are known for wash load. However, wash load will partly settle out in canals when conditions of sluggish flow or choking weed growth exist.

The standard approach to design of canal systems is to exclude as much river bed load as possible from the intake and to remove sufficient of the suspended bed material load downstream of the intake to enable the canal system to transport the remainder through to the fields, with minimal deposits accumulating in the canals. Wash load is normally assumed to be carried through the system without need for control.

(b) Sediment Data and Criteria

During the fieldwork for the present study a number of bed samples taken from the canal system and near the headworks were analysed for particle size distribution. The results are shown in Figure I.1.1. Using this information, a design grading curve for the river suspended load has been prepared. It is based on the average of the curves for the separate samples. This curve has been used for the preliminary design of sediment removal works. It has been assumed that the grading of suspended sediment entering the remodelled canal system should be finer than the finest sample taken from the existing canal system.

As has been explained in Chapter 1, data on suspended sediment concentrations in the river Shabelle are relatively sparse. It is impossible to predict with confidence average suspended sediment loads through the year. Nevertheless, since the design of sediment exclusion works cannot proceed without this information, we have estimated monthly averages and these are presented in Table I.5.1. The 'more realistic' values have been used for preliminary design, but even these levels of sediment are very high and are unlikely to persist over a long period.

Information gathered in the field regarding the removal of sediment deposits at the canal headworks indicates that while frequent clearance of the Luigi intake channel is required, much less deposition occurs at the 21st October canal intake. This is partly explainable by difference in entry conditions between the two sites. The length of the Jowhar weir in excess of the natural river channel width leads to unfavourable curvature of flow in the approach to the Luigi intake channel at the side of the weir. In contrast, the 21st October canal intake is located favourably on the outside of a bend. Bearing in mind the fine grading of the bed samples from the Luigi intake, with particle sizes largely in the wash load range, it is considered that bed load is not a significant problem. The major problem is therefore one of suspended bed material load and wash load, and the main difficulty in designing appropriate control works is the uncertainty in the future variations in monthly yield of wash load. The approach adopted is therefore to design works which are adequate for realistic levels of sediment concentration even though the present concentrations are substantially less.

5.2.3 Alternative Solutions

In order to reduce the concentrations of suspended sediment to a level where the amount and grading of material escaping can be transported by the canal system, a settling basin is required at immediately downstream of the river intake. Apart from discharge and sediment criteria the proportions of a settling basin are influenced by the method adopted for removal and disposal of deposits from the basin.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Suspended Sediment Concentrations

Month	Abstraction (m^3/s)(1)	River flow (m^3/s)(2) (50% exceedance)	Suspended sediment load (ppm)(3) Pessimistic values	More realistic values	Recent records (1983)(4)
January	5.09	11.6	2 500	1 500	
February	5.51	8.7	2 000	1 500	
March	6.04	9.7	2 000	1 500	
April	3.29	33.9	1 500	1 250	
May	3.02	91.4	8 000	6 000	
June	3.39	48.2	6 000	4 500	300
July	3.23	39.9	4 500	4 000	160
August	3.98	97.4	4 000	4 000	840
September	5.57	128.0	4 000	3 500	$<$100
October	2.81	116.1	2 500	2 250	$<$100
November	3.18	81.8	2 500	2 000	
December	3.29	32.1	2 500	1 500	
Average			3 500	2 800	

Notes :
(1) Abstraction rate for 5 300 ha cane (see Annex II).
(2) River flow data presented in Chapter 2.
(3) The 'more realistic' values have been used in the preliminary design.
(4) Very low values of sediment concentration observed during 1983. See Section 1.3.1.

Since there is insufficient head available for gravity flushing due to high flood levels in the river associated with high sediment loads, the two main alternatives considered are:

(i) removal by land-based mechanical excavators and disposal by spreading with bulldozers;

(ii) removal by floating dredger and pumped disposal by pipeline to a settling lagoon.

The operation and maintenance of a dredger requires considerably more skill than an excavator. It does have the advantage however that it can reach all parts of a settling basin regardless of width. The most appropriate type of excavator is a dragline which has the largest reach compared with other excavators. The reach does however limit the width of the settling basin even when two draglines are operating, one on each bank. An advantage of draglines is that they are more flexible and are multi-purpose compared with a dredger. They can be moved and used for other duties during periods of low sediment inflow. Alternatively, in the initial phase after remodelling only one dragline (half capacity) could be provided on the basis that a second may be required when experience is gained of actual quantities of deposit requiring removal and disposal.

Depending on annual quantities, the disposal of sediment removed from the basin will require space for spreading or for settling lagoons; in either case land should be acquired for this purpose with allowance for possible extension in the future. With dredging and pipeline disposal to lagoons containment bunds would be required with provision of return flow of water to the canal downstream. With excavation by dragline and dumping of wet material beside the settling basin some delay may be required before it can be rehandled by a bulldozer. In both cases the disposal operation must be considered an integral part of the overall operation of a settling basin.

Due to generally greater flexibility and lower skill requirements draglines have been adopted as the preferred method. However, if pessimistic levels of sediment concentration prevail in the future, it may be necessary to review the method of sediment removal and disposal.

5.2.4 Location of Basin

In view of the fundamental importance of the sediment removal works in reducing the overwhelming maintenance load in the canal system, it is clearly preferable if sediment removal operations are concentrated in one location.

Of the two existing river intakes, the 21st October offers the best potential for sediment removal. The Luigi intake has an inherent problem in its geometry (Figure I.4.1) for which there is no simple solution. The 21st October intake is much more recent, is well sited and, in contrast to the Luigi intake, there is plenty of space in the head reach for a sediment basin of up to 1.5 km long. It is therefore recommended that, as far as possible, the 21st October canal is used to supply the Estate and the Luigi canal is abandoned. The implications of this are discussed in Section 5.4.

5.2.5 Settling Basin Design

Three main criteria must be satisfied in selecting the dimensions of a settling basin and the required rate for removal of deposits. They are:

(i) the loading (discharge/effective basin area) required to achieve a given settling efficiency for a critical particle size;

(ii) the flow-through velocity required to avoid resuspension of settled particles;

(iii) the volume below the settling zone required to provide adequate storage of deposits to allow for seasonal fluctuation in sediment and discharge inflow.

Having satisfied these criteria other requirements must be satisfied mainly to ensure minimum losses of settling efficiency due to instability and short circuiting of flow at the inlet and outlet. For this reason particular care has to be taken with the design of the inlet and outlet zones.

Concerning the main criteria which affect the design of the settling zone, preliminary calculations have been carried out to illustrate the relationship between basin loading and the particle size distribution of sediment expected to be deposited and to escape into the canal system downstream. These calculations were based on the method of Vetter (1940), selected for its ease of use for numerical calculation. Values of settling efficiency predicted by the Vetter method are only slightly lower than by the better known Camp removal function (Camp 1946).

The results of these calculations are plotted in Figure I.5.1 for a range of basin loading from 1.6×10^{-4} m/s to 1.3×10^{-3} m/s. From these results it is concluded that to achieve a satisfactory grading of escaping sediment the basin loading should not be greater than 3.3×10^{-4} m/s.

Regarding the flow-through velocity, an upper limit of 0.2 m/s has been adopted with monthly average velocities generally well below this value.

In order to match deposit removal rate with storage volume below the settling zone, the capacities of draglines readily available on the market have been used to provide indicative work rates and corresponding values of bucket reach and settling basin bed width. The range of values considered is shown in Table I.5.2.

TABLE I.5.2

Dragline Output, Maximum Reach and Settling Basin Bed Width

Number of machines	Bucket size (m^3)	Hourly output (m^3/hour)	Annual output (m^3)	Maximum reach (m)	Settling basin bed width (m)
2	0.80	64	221×10^3	23.4	25.8
2	0.96	76	263×10^3	22.1	23.2
2	1.15	92	318×10^3	19.5	18.0

To facilitate the selection of the settling basin dimensions with matching dragline capacity a computer program BUNKER has been used. This program assesses the performance of a basin of given dimensions with monthly average input values of discharge and sediment concentration. The particle size grading of the inflowing sediment and average daily removal rates are also input. The program calculates in 10 day periods:

- the bed level of the deposit (assuming uniform distribution over the basin length);
- the period and accumulation volumes settled, removed and stored;
- the settling efficiency for 0.06 mm particle size and for the overall particle size grading;
- the flow-through velocity at the beginning of each period.

Afer a series of trials the minimum lengths of settling basin assuming a working maximum water depth of 4.0 m were determined for the adopted sediment criteria and alternative dragline criteria. The results are presented in Table I.5.3 together with maximum values of basin loading and annual deposit volume.

Of the three options considered, the first is not satisfactory due to inadequate dragline capacity and the third is not satisfactory due to the basin loading exceeding the adopted design limit. The second option however does satisfy the criteria adopted and is therefore the preferred solution.

TABLE I.5.3

Summary of Settling Basin Dimensions and Performance

Option (Nr)	Bed width (m)	Length (m)	Maximum loading (m/s)	Annual deposit volume (m^3)	Annual dragline capacity (m^3)
1	25.8	750	2.6×10^{-4}	241×10^3	221×10^3
2	23.2	700	3.0×10^{-4}	237×10^3	263×10^3
3	18.0	600	4.3×10^{-4}	225×10^3	318×10^3

Maximum depth = 4.0 m

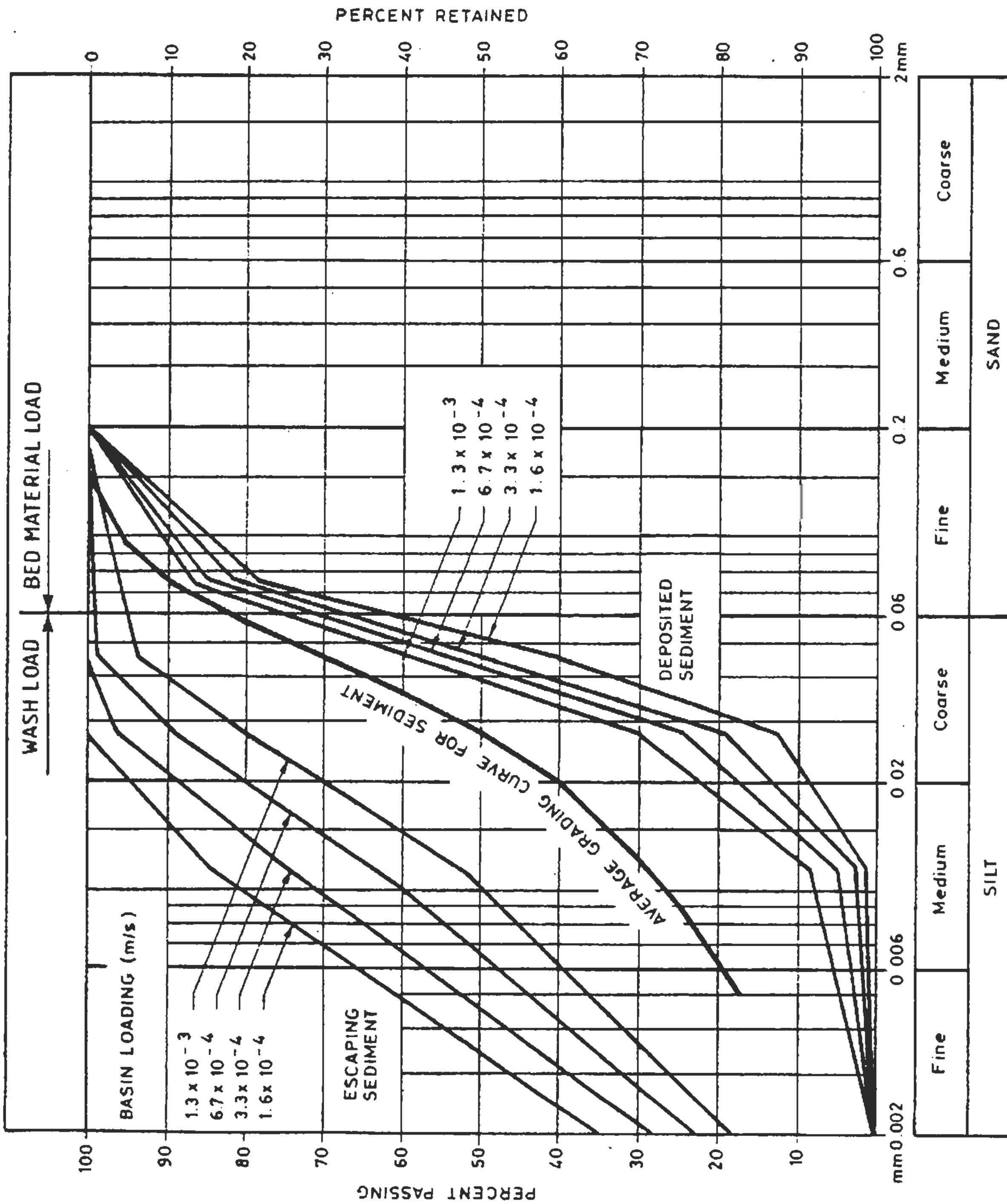
Side slopes = 1 : 1.5

Freeboard = 1.0 m

Dimensions exclude inlet and outlet zones

Figure I. 5. 1

Particle Sizes of Deposited and Escaping Sediment



5.2.6 Recommendations

In view of the uncertain accuracy of the sediment data available, and the short period of record, it is recommended that the sediment basin construction is phased. The first phase would comprise a basin of about half the calculated capacity, having the full design length (700 m) but only half the width. One dragline would be purchased to remove the sediment.

After implementation of the rehabilitation works, in say 1988, the need for increasing the basin size (by widening on one side) could be reviewed and, if necessary, another dragline purchased.

5.3 Flow Regulation and Control

Two fundamental requirements of any supply-based irrigation system (as distinct from a demand system) are some means of measuring flows and the ability to regulate the flow to achieve the desired objective. There is no simple means of measuring flows at any point in the supply and distribution system at Jowhar - there are no weirs, flumes or even water level gauges. Control of canal flows is also somewhat restricted by the shortage of gates, particularly on cross regulating structures. Any improvements to the canal system should therefore include the provision of some flow measuring devices, if only at the head of the system, and the improvement of flow control.

Reference to Appendix A herein reveals the full extent to which new control gates are required. More than half of the tertiary canal head regulators need replacement gates to ensure effective control. Of the 35 or so main and secondary canal regulating structures examined, only 5 had gates in operating condition, 15 needed new gates, and 15 had no gates at all.

It should be mentioned that it is not essential that all these structures are provided with the means of flow regulation. The replacement of gates on key control structures will be sufficient to provide the necessary degree of flexibility to improve system operation.

The new gates should be of a more robust construction than the existing ones. This will also necessitate the replacement of lifting frames and gear in many cases because these too are relatively flimsy.

One of the major factors which has influenced the design of the rehabilitation works is the requirement to avoid night-time irrigation. This is discussed in detail in the following section.

5.4 Night Storage

5.4.1 General

Previous experience on the Estate, and on irrigation schemes elsewhere in Africa, indicates that night-time irrigation is rarely carried out effectively. This is especially true of irrigation using siphon pipes because these need constant supervision to ensure that they remain primed. It is therefore desirable that tertiary canals operate only during the daytime (say 12 hours). There are a number of alternative ways of achieving this:

(a) Operate the entire canal system for 12 hours/day only. This option works well in systems where it is possible to maintain canals full or near full throughout the night. Otherwise there is a lengthy 'filling-up' time in the morning before irrigation can commence. At the end of the day it is necessary to close all regulators on main and secondary canals to prevent loss of water from one reach to another. This option is not considered to be ideal for Jowhar because it will be very difficult to prevent the canal system from emptying overnight. The following morning it would take up to 8 hours for the system to refill completely.

(b) An alternative is partial shut-down at night. With this option the main and secondary canals are throttled back at night to, say, 20% of design flow. In this way the filling up time the following morning is reduced. The water in the canals at night would be run to waste or could be used to provide water for non-cane areas at the southern end of the Estate.

(c) The preferred option is to introduce storage into the system so that no water is wasted and the intake can operate for 24 hours/day.

5.4.2 Alternative Systems

Water can be stored in reservoirs or in oversized canals. The in-canal storage is the most efficient system because the stored water is spread through the whole irrigated area and there is no 'filling-up' time. Unfortunately in-canal storage requires oversized canals and specially designed canal structures, and therefore its introduction at Jowhar would require complete redesign of the canal system.

The alternative of using reservoirs may be more appropriate. Reservoirs can be located at various positions within the system. The further down the system the reservoirs are, the less time is wasted filling canals in the morning. A reservoir at the head of the system would have the disadvantages of option (a) since the canal system downstream would have to be re-filled every morning. Reservoirs at the heads of the secondary canals is a better option since the main canal can then operate 24 hours/day and only the secondaries need filling every morning. A further alternative would be reservoirs at the heads of tertiary canals or groups of tertiaries. This would require a large number of relatively small reservoirs, but would enable 24-hour operation of all main and secondary canals.

The choice of where to have the storage reservoirs is determined to some extent by the design of the existing system. Generally speaking the existing secondary canals are higher than they need to be, as are the tertiary canals. In some parts of the Estate this excessive command in secondaries is sufficient to enable the construction of storage reservoirs at the heads of tertiaries, without requiring re-design of the secondaries. Unfortunately this is not true of all secondary canals. In some areas the levels of canals are more suited to reservoirs at the head of secondaries. In addition the introduction of reservoirs at the heads of tertiaries will be more expensive to construct.

5.4.3 Command Problems

Unfortunately, any system of night storage requires a greater command level at the head of the system - generally at least 0.50 m more than that required for a continuous flow system. With a reservoir system 0.50 m is a practicable minimum for the usable storage depth, and there are necessarily head losses into and out of the reservoir, so an increase in command of at least 0.70 m is required. At present the minimum water level in the river is about 103.8 (the main weir crest level). River levels at the weir are no more than 0.5 m higher than this for 6 months of the year. In order to command the whole irrigable area and to incorporate night storage, a water level in the river of 105.3 m would be required, which is only achived during high floods. Even if the highest fields are eliminated from the system a river level of 105.0 m would be required to permit night storage - river levels are lower than this for most of the year. There are a number of possible ways of achieving the increased level.

(a) Pumping

A new pumping station could be constructed on the river at the 21st October intake. This would pump the Estate's requirements whenever river levels were too low for gravity flow. Even though the pumped static head would be less than 2.0 m, recurrent costs would amount to some SoSh 3.0 million/year and a high capital investment would be required. This option has not therefore been considered further.

(b) Raising the Existing Weir

No information on the structural design of the weir is available but, in view of the fact that it is some 60 years old, this is unlikely to be a viable option except at high cost. Improved flood embankments would also be required upstream.

(c) Constructing a New Gated Weir (barrage) near the 21st October Intake

The present day cost of building Sabuun barrage and head regulator would be about SoSh 80 million. A new barrage for the Estate would be less expensive, perhaps SoSh 60 million, and this would ensure a gravity supply to the Estate at all river stages. However, it is unlikely that this cost can be economically justified.

(d) Make Use of the Existing Gated Barrage at Sabuun

This is the most realistic option but it does involve, in addition to a new head regulator on the river, an underpass carrying the irrigation water under the JOSR Supply Canal, and about 10 km of new main canal. With this option it should be possible to maintain a water level at the head of the canal system of about 105.0 m. This would require the Sabuun barrage pond level to be maintained at about 107.0 m. This will undoubtedly lead to increased sediment deposition upstream of the barrage and hence increased risk of flooding. The operation of the barrage at Sabuun is, of course, the responsibility of the Ministry of Agriculture and there may therefore be some conflict between the operational needs of the Estate and the Offstream Storage Reservoir. This would have to be resolved before any works are constructed. The cost of this option would be about SoSh 34 million.

5.4.4 The Preferred Option

In view of the high costs of options which require a river level higher than the present minimum level, the preferred option is based on existing levels. This inevitably means that the higher parts of the Estate cannot be commanded, as indeed they cannot with the present system when river levels are low. Such areas will require pumping throughout the year at a fuel cost of some SoSh 500/ha/year. Details of the proposals are given in Section 5.5. Of the proposed 5 300 ha net cane area, one third would require pumped irrigation supply on a perennial basis. In addition, an area of 450 ha would require pumping during months of low river level.

5.5 Improved Canal Layout

5.5.1 Introduction

The existing canal network has been constructed progressively in response to the increase in cane area throughout the scheme life. As a result the layout of canals is not ideal. The following poor features can be observed:

- tertiary canals feeding uphill (leading to excessive command at the upstream end);
- tertiary canals crossing the main drainage channels (this necessitates frequent culverts on the main drains; it thereby increases the problems of maintenance of these);
- tertiary canals served from both ends (i.e. connected to two secondary canals);
- secondary canals higher than necessary (because they serve high areas which would be better served from another canal).

Figure I.5.2 shows the proposed improved canal layout to eliminate the above problems. It is based on a single river intake (the 21st October canal intake - as discussed in the previous section) and incorporates night storage reservoirs as discussed in Section 5.4. The proposals are described in detail below.

5.5.2 Rehabilitation Proposals

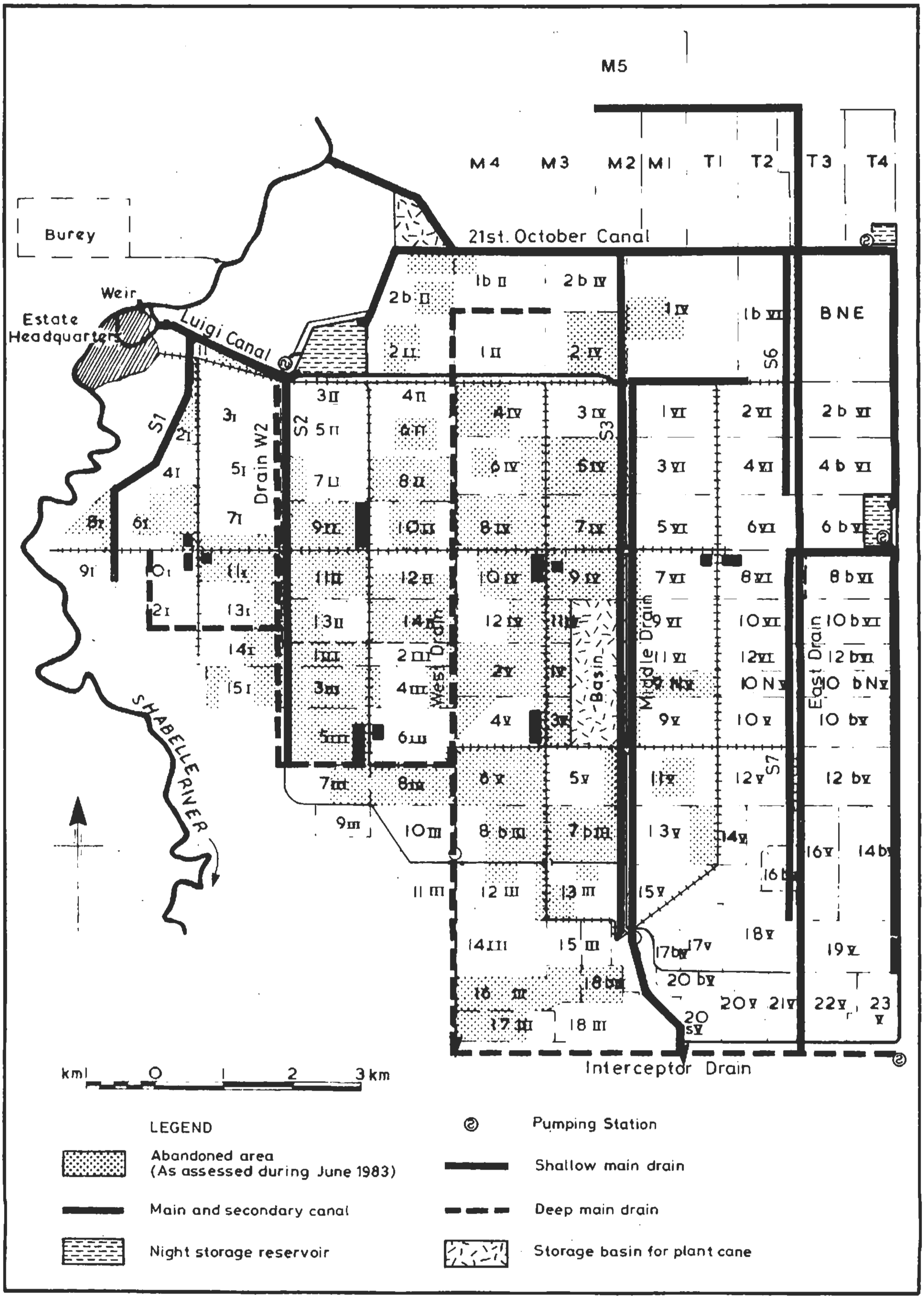
The rehabilitation proposals cover the entire scheme, even though it is likely in the future that part of the area will not be under cane. Selection of the future cane-growing area has been influenced by many factors, but the principal of these are drainage and land suitability, as discussed in Chapter 11 herein. The proposed rehabilitation measures for the irrigation system are described below. The recommended changes to field layout and in-field irrigation and drainage are described in detail in Chapter 6.

(a) Existing Canals

Wherever possible the existing canals have been retained - this is essential if costs are to be kept to an acceptable level. The rehabilitation measures from the existing canals include:

Figure I.5.2

Jowhar Sugar Estate
Proposed Layout of Canals and Drains



- reforming the channel section to the required capacity by removing silt and weeds and reshaping as necessary;

- removal of obstructive trees, shrubs and other vegetative growth from canal banks to improve access and visibility;

- grading the surfaces of canal banks to a cross fall outwards;

- repairs to brick and concrete structures as necessary (generally relatively minor works to ensure the continued life of the structure, but also some rebuilding work where the present condition is very poor);

- demolish a few redundant or dilapidated structues;

- provide selected control structures with new gates and lifting gear to improve flow distribution and control (some cross regulators can be left ungated), particularly these in canals serving non-cane areas);

- provide new gates for about half the tertiary head regulator structures.

In order to assess the cost of these proposals an estimate has been made of the numbers and sizes of structures involved. Preliminary proposals for main and secondary canals are presented in Appendix C; a more detailed assessment will be necessary before rehabilitation can commence.

Generally speaking the existing canal sections and structures are of sufficient capacity to serve their command areas even though in the future flow will be confined to 12 hours/day. The spacing between canal and parallel drain is also sufficient to allow for the provision of access and a reservation for silt dumping.

(b) New Canals

The proposals include two entirely new secondary canals (S6 and S7) having a total length of about 10.3 km, and two new reaches of canal connecting S1/S2 and S3 to the 21st October canal. In fact the connecting canal from the 21st October to S2 makes use of existing canal S5 and hence is not really entirely new.

The two new canals S6 and S7 are designed to serve the area between the Middle and East drains. This area is currently served by long tertiary canals supplied at one end by canal S3 and at the other by the 21st October canal. These long tertiaries thus cross both the Middle and East drains. The new secondary canals will ensure that tertiary canals are reduced to about 2 km in length and that there is no need for the tertiaries to cross the main drainage lines (see Figure I.5.2). Details of the new canals are given in Appendix C.

(c) Night Storage Reservoirs

The design philosophy for siting night storage reservoirs is as follows:

- for those canals serving relatively low areas (principally canal S2) provide gravity-fed reservoirs at the head of the canal;

- for higher areas which can currently be served by minimum river level do not provide night storage;

- for the highest areas which are difficult or impossible to serve from minimum river level provide reservoirs from which water has to be pumped in secondary canals;

- locate the reservoirs as far down the system as practicable.

Locations of the three proposed night storage reservoirs are shown on Figure I.5.2. Use has been made of the existing storage basin at the end of the Luigi canal. This will provide water for canal S2, and also for canal S1 at times when the latter cannot be served from the Luigi canal. Fields T3 and T4, in the north-eastern corner of the Estate, will have a separate small reservoir off the 21st October canal, from which water will be pumped (field T4 receives a pumped supply at present). The third reservoir will be located about two-thirds of the way down the 21st October canal. Water will be pumped from this into new canal S7 and into the tail reach of the 21st October.

The system will operate with the reservoirs being filled at night and no irrigation taking place during night-time. During the day, those areas which are served by reservoirs will receive their irrigation water solely from the reservoirs; and the day-time flow in the 21st October canal will be diverted to canals which do not have storage.

The principles of the system are illustrated diagrammatically in Figure I.5.3. It can be seen that there is an imbalance between night and day flow in the main canal. This is because the area served by storage reservoirs is less than the area served direct from the main canal. It will be necessary therefore to adjust the intake regulator twice daily to accommodate this pattern. It can also be seen from Figure I.5.3 that there are large differences between day and night flow for the lower reaches of the 21st October canal. This will lead to sluggish flow conditions during the day. However, sediment deposition rates should be acceptably low because most material will have already been removed in the settling basin. The required water levels for the offtaking tertiary canals in the affected reaches will be maintained by adjustments to the cross regulator gates.

As was mentioned earlier, it is assumed that canal S1 will continue to be supplied from the Luigi canal. Canal S1 serves some of the highest land in the Estate and cannot command its area when river levels are low; pumping will therefore be necessary for some of the year. It is proposed that when pumping is required this is carried out from the night storage reservoir which supplies canal S2, and which is filled from the 21st October canal. In this way use of the reservoir is maximised and the imbalance between night and day flows in the main canal is reduced. Pumping will be required for between 3 and 6 months a year, depending on river flows. Maintenance of the Luigi intake will continue to be a problem but the idea is to make full use of the high river levels whenever possible. It will not therefore be necessary to keep the intake channel open when river levels fall, and this should reduce the very high desilting requirements which are a feature of the existing operation. If however it proves uneconomic to maintain the Luigi intake, even during high river levels, then the system can be operated with the supply to canal S1 being pumped from the reservoir on a perennial basis.

Figure I.5.3
Operation of the
Night Storage System

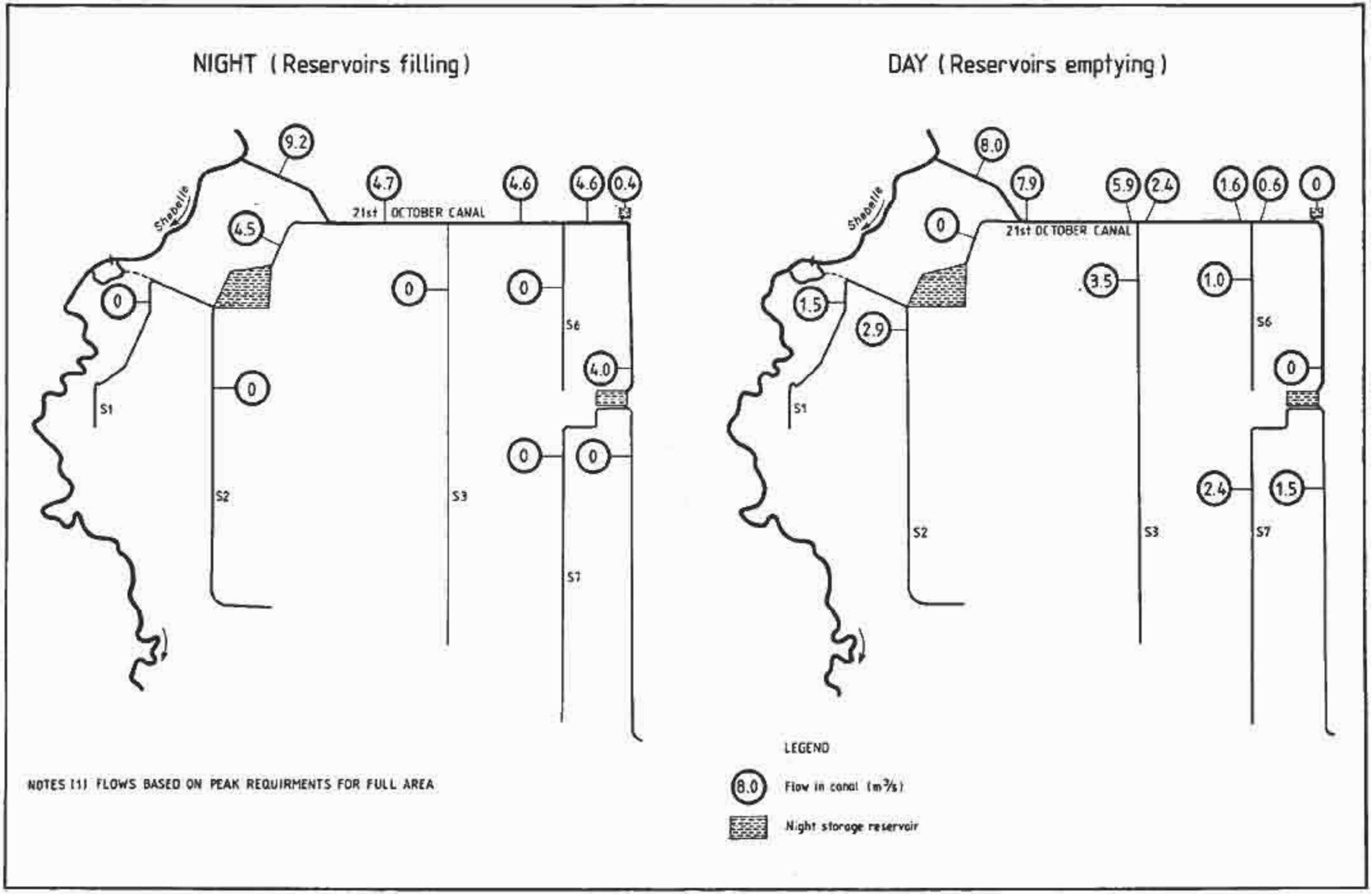


Figure I.5.4 shows the proposed cane area divided into canal commands and also indicates the areas which will always be supplied by pumping. As described above, the S1 command will require pumping for part of the year.

More details of the operation and maintenance of the canal system are given in Chapter 13 of this annex. Design details of the proposals are presented in Chapter 7.

Figure I.5.4

Jowhar Sugar Estate
Moderate Investment Strategy
Cane Areas

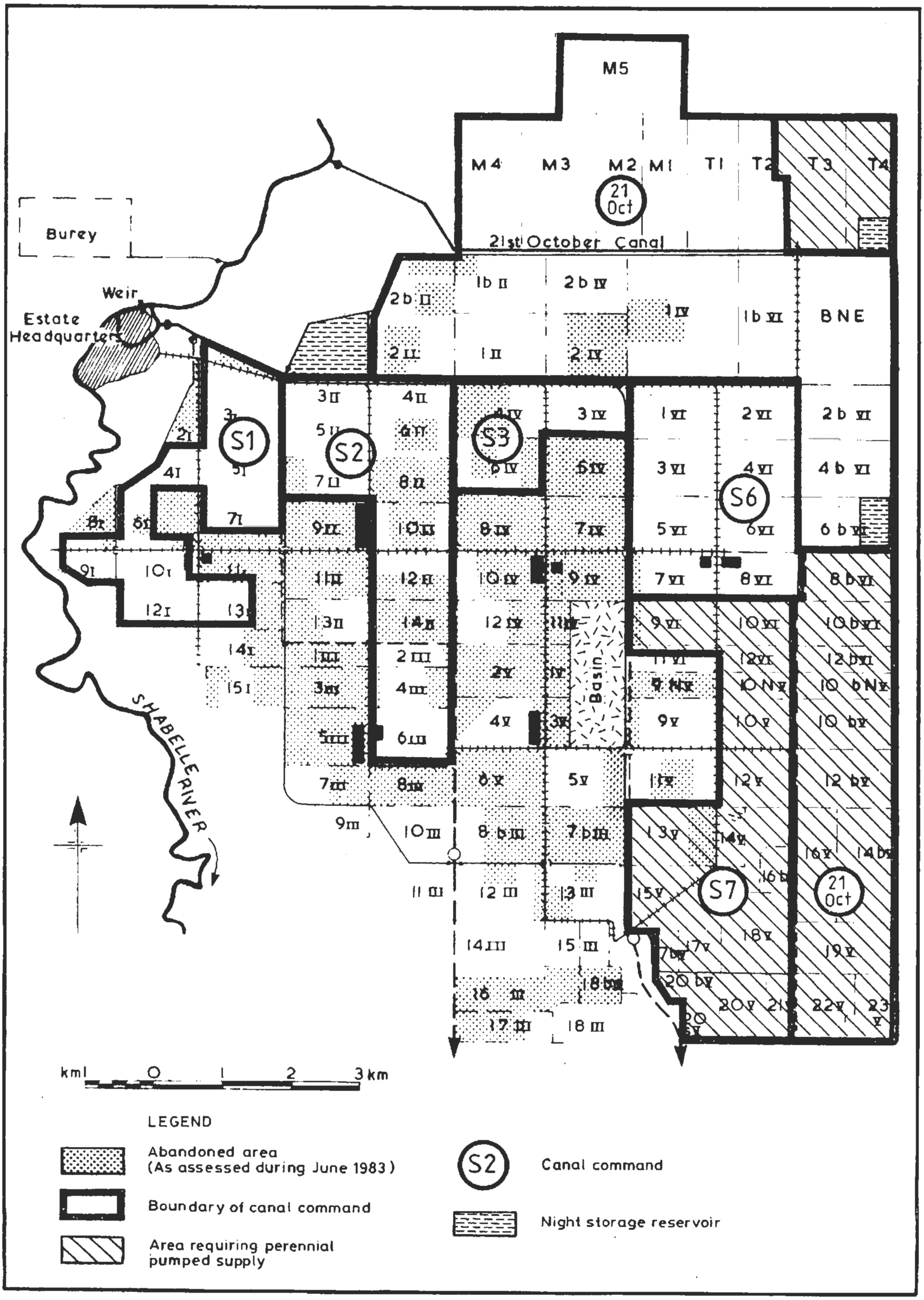
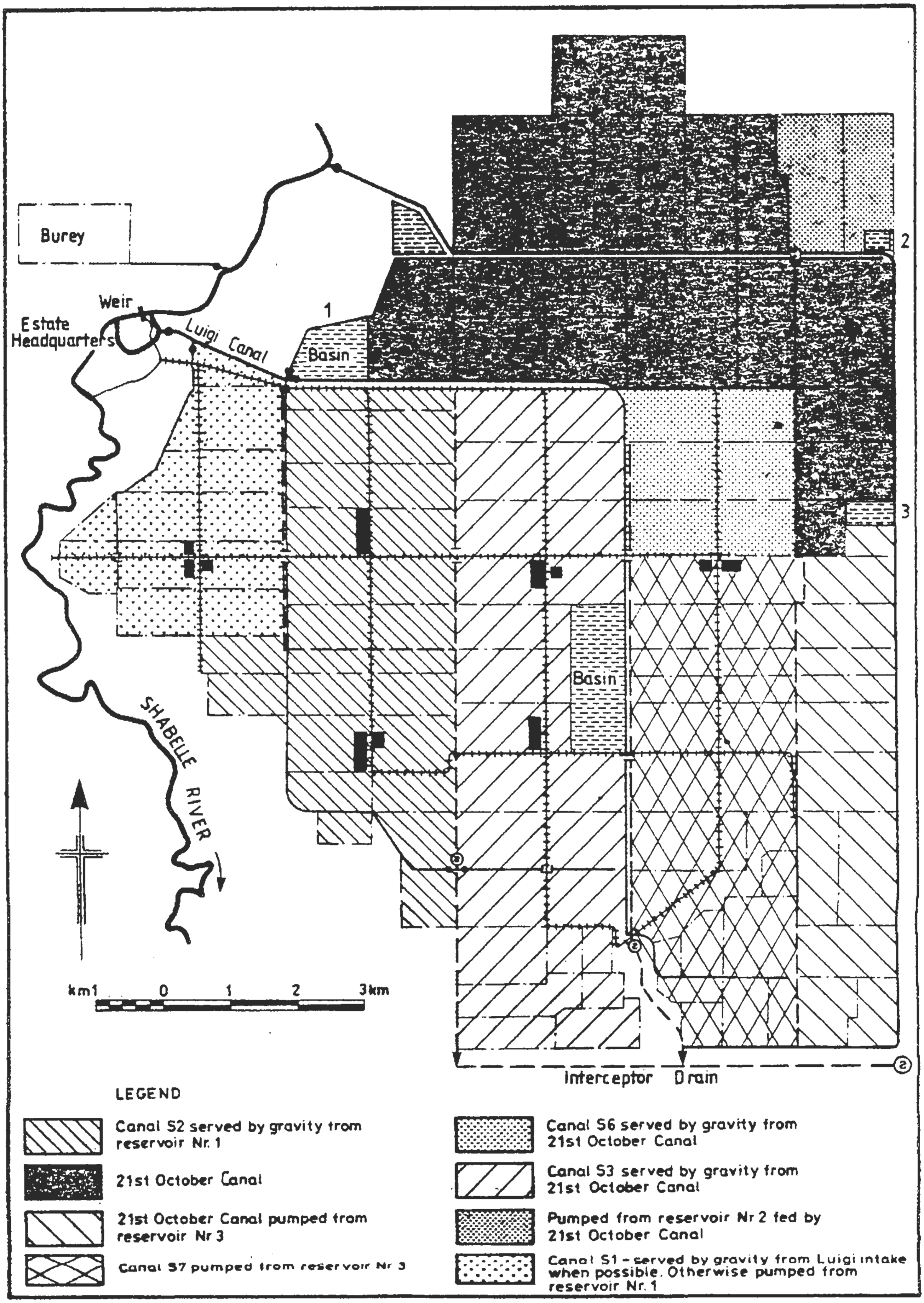


Figure I.5.5

Jowhar Sugar Estate
Proposed Command Areas



CHAPTER 6

PROPOSED FIELD LAYOUT

6.1 General

The operations carried out from tertiary outlet to collector drain are crucial to the success of an irrigation scheme (the 'workface' of irrigation). Good control with an even application of water will result in even stands of cane, no waterlogging and conservation of the limited water resource. Poor control will result in poor cane yields and excessive demands on the water supply, drainage and the soil/water resource.

A substantial emphasis should thus be placed on operations at this level. However, it should be realised that with some 81 fields to be cultivated small additional costs at the field level multiply up to a substantial investment.

Conversely, small increased benefits at the field layout will multiply up to substantial improvements to the returns of the scheme.

6.1.1 Field Layout

It is recommended that the present furrow/basin method of irrigation used on the Estate is changed to that of furrow irrigation, both to obtain better control over the application of irrigation water and to facilitate mechanical operations on the fields.

Furrow irrigation has become the most extensively used method of surface irrigation on sugar estates in the world, largely because of the above-mentioned advantages. The length of furrow is determined by a balance of two factors:

(i) labour requirements, mechanisation and irrigation costs which are all reduced with longer furrows;

(ii) irrigation application efficiency which is usually reduced with increase in furrow length.

It was previously proposed (MMP, 1976) that a reasonable balance of these factors was obtained with furrows of 300 m length.

This assumption has been reviewed as discussed in Section 6.1.4 below and, by allowing some flexibility to suit actual field dimensions, furrow lengths in the range 200 m to 400 m have been used with an average length of 310 m. A typical field layout is given in Figure I.6.1. Note that no deep collector drain is shown, the layout would be modified when subsurface field drainage is installed with deep collector drains being installed parallel to the surface drains. See Figures I.6.2 and I.6.3.

6.1.2 Net Areas

The areas to be kept in sugar cane are discussed in Chapter 11. The areas given as gross field areas include the tertiary reservation, field roads, quaternaries, and surface drain reservations, but exclude the area of secondary canals, collectors and main drains, farm buildings, stores and villages.

The net area is calculated as that proportion of the gross area actually under cane. Sample calculations on typical Estate fields give this conversion ratio of net : gross area at 90% for the proposed field layout.

6.1.3 Water Application into Furrows

A new header channel is to be constructed along the head of each furrow. Irrigation is to be applied by means of siphons into the furrows. This operation, although apparently straightforward, is the most common cause of inefficient irrigation. The following observations are given on factors that determine efficiency of in-field irrigation.

(a) Siphon Size and Type

The recommended delivery method into the furrow is that of plastic siphon pipes. These are relatively cheap, easy to install, can be used without disturbing the channel bank, and their portability reduces the number required. The flow can be regulated by changing the pressure head, varying the size or number of siphons used in each furrow or by adjusting a slide gate attached to the inlet end of each siphon. The method recommended is to use several smaller diameter siphons to deliver the maximum non-erosive stream into each furrow at the start of irrigation and to remove siphons as necessary when the water reaches the end of the furrows; thereby adjusting the flow until the desired irrigation amount is applied. The recommended irrigation is for 154 mm field application (which at 65% application efficiency amounts to a net field application of 100 mm).

For the average furrow length of 310 m with furrow spacings of 1.5 m and a furrow discharge of 5 l/s, 30 furrows will have 154 mm gross applied in 4 hours. This is on a net area of 1.4 ha, thus a typical cane field of 70 ha net will take 200 hours (17 days at 12 hours/day). It is recommended to use siphons of 50 mm internal diameter (as were used on the drainage trial). These will give a flow of 1.5 to 2.0 l/s depending on the pressure head (7.5 to 15 cm respectively). Ninety siphons will be required per 70 ha field.

(b) Turning Area for Tractors

The area immediately downstream of the header channel is used to turn the tractors before returning for their next run down the field. This length of some 5 m should be repaired after tillage and prior to irrigation. A less preferable alternative is to treat this top 5 m as a forebay, allowing water to pond in it from which water is siphoned into the furrows.

(c) Furrow Control and Release of Excess Water

The speed of advance down the furrow is dependent on the flow discharge, the slope and roughness. Each furrow will therefore be different and the application efficiency will be dependent on the skill of the irrigator. Actual application practices will be set out under the guidance of the Irrigation and Drainage Manager and his supervisory staff. Excess water should obviously be avoided but any standing water left on the irrigated field should be removed after 24 hours. Surface drains are discussed in Section 12.4 and land levelling is discussed below.

Figure I.6.1

Alternative Field Layouts

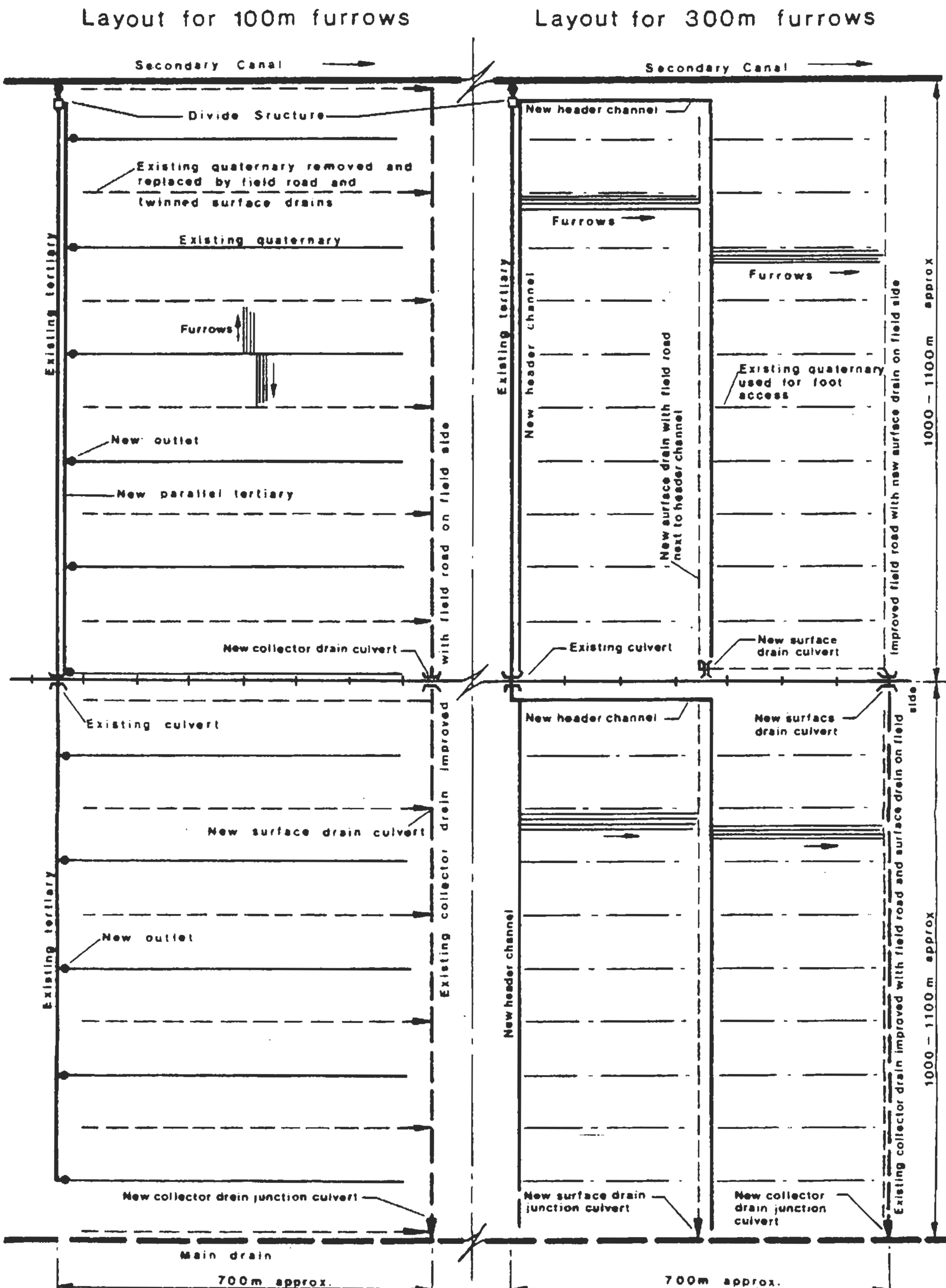
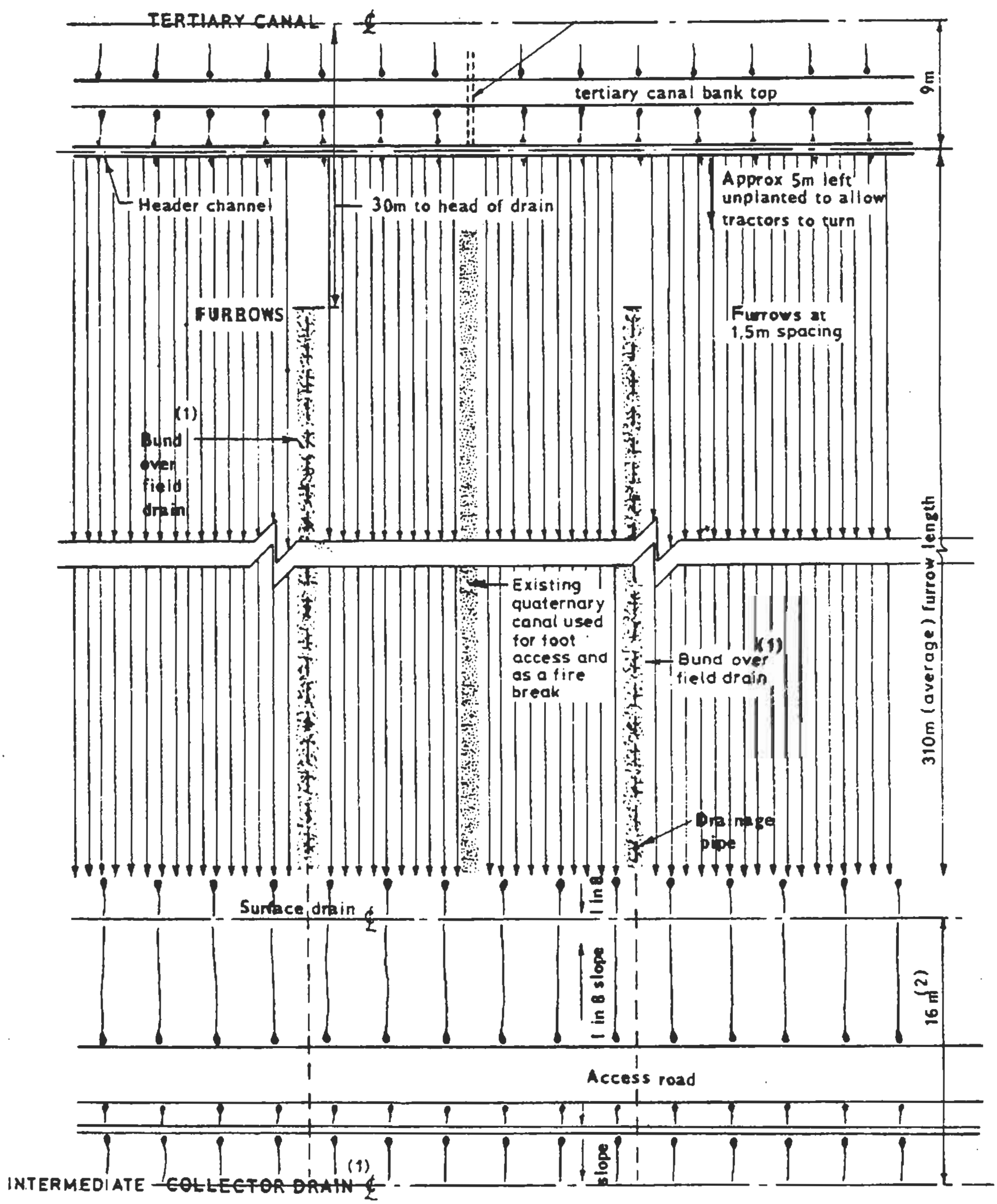


Figure 1.6.2

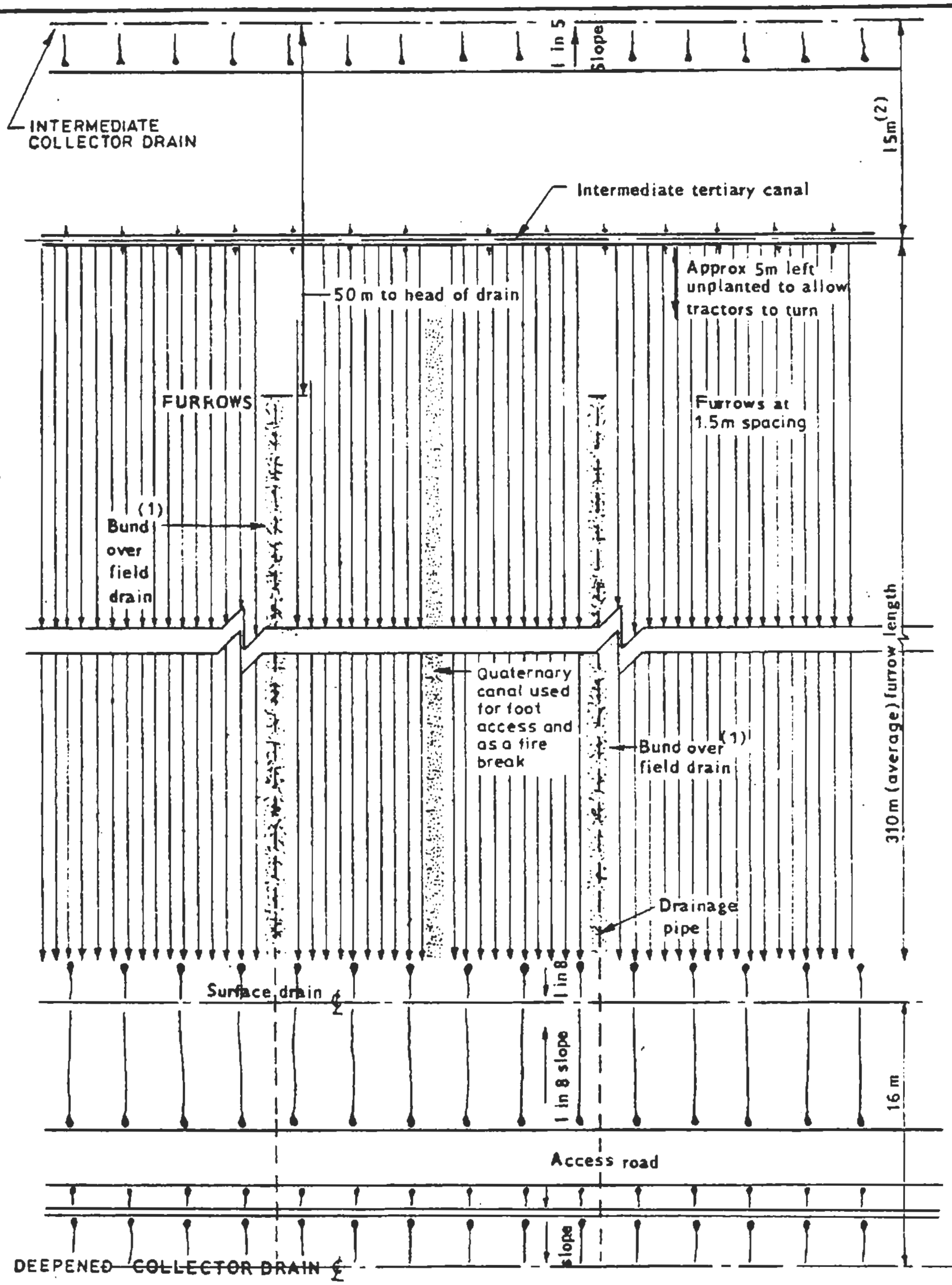
Proposed Field Layout
Where Field Drains are Installed
Top Half of Field



NOTE
1. THE INTERMEDIATE COLLECTOR DRAIN AND FIELD DRAINS ARE ONLY REQUIRED ON CERTAIN FIELDS. SEE SECTION
2. THE RESERVATIONS SHOWN IS ONLY REQUIRED WHEN FIELD DRAINS ARE INSTALLED

Figure I.6.3

Proposed Field Layout Where Field Drains are Installed Bottom Half of Field



NOTE

1. THE INTERMEDIATE COLLECTOR DRAINS AND FIELD DRAINS ARE ONLY REQUIRED ON CERTAIN FIELDS, SEE SECTIONS
2. THE RESERVATIONS SHOWN APPLIES WHEN FIELD DRAINS ARE REQUIRED; IN THE CASE WHERE FIELD DRAINS ARE NOT INSTALLED THEN THE RESERVATION FROM SURFACE DRAIN TO TERTIARY CANAL IS 13m.

6.1.4 Furrow Length

Consideration of the factors determining furrow length mentioned in Section 6.1.1 for clay soils generally gives a reasonable balance between 300 to 400 m long (FAO 1967). The field trial carried out in 1977/78 (MMP 1978) used furrow lengths of 300 m and irrigation control was good. Furrows of these lengths would be accommodated satisfactorily within the present field system and it is recommended that a maximum length of 400 m should be adopted. An analysis dividing up the 5 300 ha of the estate recommended for continued cultivation has been carried out. See Tables I.6.1 and I.6.2. Using a maximum furrow length criteria of 400 m and dividing the fields suitably as shown gives an average furrow length of 310 m.

An alternative proposal using shorter furrow lengths of some 100 m has been considered as it has been suggested that this would result in a more even irrigation application. However, consideration of the alternatives as given in Table I.6.3 leads to the conclusion that the longer (average 310 m) length furrow alternative is preferable; the significant advantages being that of a cheaper remodelling cost and a better suitability for mechanisation (longer uninterrupted runs). A major disadvantage of the 100 m alternative is that it is ill-suited to subsurface field drainage installation, in that it would entail either subsurface drainage running at right angles to the furrows (not to be recommended) or excessively long field drains (with a resultant increased depth and cost of both installation and pumping).

The comparison as given in Table I.6.1 applies in the 100 m furrow length case to furrows running parallel to the tertiary canal taking off from the existing quaternaries. The Estate management, although generally agreeing to the longer furrow alternative as set out in the Interim Report (MMP 1983), have asked if the 100 m furrow option can be maintained as an option in the event that 300 m long furrows prove unsuitable.

The proposed field layout (see Figure I.6.1) with the longer 300 m furrows would indeed allow for 100 m furrows (or 150 m long furrows) to be used if so required at a later date. This would require an additional surface drain and header channel to run across the head of each group of 100 m furrows which for a typical 70 ha field would result in four more 1 km long header channels, surface drains and field roads plus four surface drain culverts under field roads. Some 6 ha of cropped field area would be lost (per 70 ha field) due to the area of additional canal/drains required, but is otherwise a workable solution.

6.2 Land Levelling Requirements

6.2.1 General

For uniform application of water it is important that an even furrow slope is obtained. Major advances have been made in land levelling techniques during the last decade and it is recommended that these methods using laser control are adopted on the Estate.

6.2.2 Furrow Slopes

Land levelling quantities can be minimised by levelling to a slope close to that existing on the Estate fields. However for successful furrow irrigation there are optimum slope limits which will allow for control as described in Section 6.1.3(c).

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Field Dimensions

Field Nr	Dimension(1) East-west (m)	North-south (m)	Existing collector drains (Nr)	Required surface drains (Nr)	Furrow length (m)	Direction furrows
Farm I						
1	1 200	350	1	1	320	N-S
3	1 200	850	1	2	200	N-S
4	600	580	1	2	265	N-S
5	1 200	580	1	2	265	N-S
6	400	380+310	2	2	355+285	N-S
7	1 200	380+310	2	2	355+285	N-S
8 s	660	260	1	1	210	N-S
9	630	400	1	1	370	N-S
10	800+150	220+430	2	2	190+400	N-S
12	300+800	700	1	2	325	N-S
13	750	700	1	2	325	N-S
Farm II						
2 b	470+500	1 000	2	4	2 x 235 2 x 220	E-W
1 b	380+790	800	2	3	350, 2x380	E-W
2	1 180	610	1	2	280	N-S
1	1 170	820	1	2	385	N-S
3	1 160	330	1	1	300	N-S
4	1 230	330	1	1	300	N-S
5	1 160	690	1	2	320	N-S
6	1 230	690	1	2	320	N-S
7	1 160	590	1	2	270	N-S
8	1 230	590	1	2	270	N-S
10	1 220	670	1	2	310	N-S
12	1 220	580	1	2	265	N-S
14	1 220	600	1	2	225	N-S
Farm III						
2	1 210	350	1	1	320	N-S
4	1 200	630	1	2	290	N-S
6	1 190	650	1	2	300	N-S
Farm IV						
M5	1 530	800	1	2	375	N-S
M4	850	540+1 160	1	2	400	E-W
M3	900	1 700	1	3	273	E-W
M2	690	1 700	1	2	320	E-W
M1	550	1 700	1	2	250	E-W
T1	620	1 700	1	2	285	E-W
T2	700	1 700	1	2	325	E-W
T3	690	1 660	1	2	320	E-W
T4	330+300	1 700	2	2	300+270	E-W
2 b	800	830	1	2	375	E-W
2	800	810	1	2	375	E-W
1	770	300+1 400	1	2	360	E-W
4	1 190	720	1	2	335	N-S
3	1 140	730	1	2	340	N-S
6	1 190	770	1	2	360	N-S

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Field Dimensions

Field Nr	Dimension(1) East-west (m)	Dimension(1) North-south (m)	Existing collector drains (Nr)	Required surface drains (Nr)	Furrow length (m)	Direction furrows
Farm V						
10 N	1 030	350	1	1	320	N-S
10 b N	1 280	380	1	1	350	N-S
10	1 050	710	1	2	330	N-S
10 b	1 290	710	1	2	330	N-S
12	1 060	770	1	2	360	N-S
12 b	1 260	840	1	2	395	N-S
13	1 150	710	1	2	330	N-S
14	420	540	1	1	390	E-W
16 b	270	800	1	1	240	E-W
16	470	1 350	1	2	210	E-W
14 b	730	1 400	1	2	340	E-W
15	800	400	1	1	370	N-S
17 b	400	450	1	1	370	E-W
17	530	850	1	2	240	E-W
18	890	1 090	1	3	337	N-S
19	1 290	580	1	2	265	N-S
20 b	580+310	270	1	1	240	N-S
20 s	620	600	1	2	225	N-S
20	600	880	1	2	225	E-W
21	330	880	1	1	300	E-W
22	860	1 020	1	3	317	N-S
23	380	960	1	1	350	E-W
Farm VI						
1 b	740	1 710	1	2	345	E-W
BNE	410+300+550	1 010+590	3	4	380+270 +2x250	E-W
1	1 210	710	1	2	330	N-S
2	1 010	710	1	2	330	N-S
2 b	210+600+470	800	1	2	375	N-S
3	1 210	750	1	2	350	N-S
4	1 010	750	1	2	350	N-S
4 b	620+700	780	1	2	365	N-S
5	1 210	820	1	2	335	N-S
6	1 010	800	1	1	375	N-S
6 b	620+710	820	1	2	385	N-S
7	1 180	590	1	2	270	N-S
8	1 040	640	1	2	295	N-S
8 b	110+1 190	650	1	2	300	N-S
10	1 040	560	1	2	255	N-S
10 b	1 300	580	1	2	265	N-S
12	1 050	350	1	1	320	N-S
12 b	1 320	360	1	1	330	N-S

Note : (1) Dimensions shown are from centre line of tertiary canal to collector drain or centreline of road to main drain.

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Length of Drains[1] and Average Furrow Length

Farm Nr	Surface drain at edge of field (m)	Total surface drain (m)	Average furrow length (m)
I	12 440	17 290	290
II	16 780	30 370	291
III	3 600	5 990	296
IV	23 650	45 600	325
V	25 600	44 440	322
VI	21 580	39 120	309
TOTAL[2]	103 650	182 810	310

Notes : (1) The lengths given are only for the 5 300 ha to be included in the rehabilitation.

(2) The total length of drains in the fields to be rehabilitated that drain into the West drain are 36.2 km of collector and 59.4 km of surface drain.

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Comparison of Alternative Field Layouts(1)

	Item	Details for typical 70 ha field	
		300 m length furrows	100 m length furrows
a.	Tertiary canals	Present tertiary retained but requires two new header channels on each field and a division box on alternate fields	Tertiary retained but requires a new header channel on alternate field
b.	Outlets from tertiaries	Existing outlets to be removed	New outlets required on header channel (upper field). Outlets on tertiary serving lower field retained
c.	Quaternaries	Kept as a firebreak and foot access. Can be simply levelled off and used as additional field road access required	Alternate quaternaries retained
d.	Siphons	90 siphons required to be moved round the field	90 siphons required but more frequent movement to new furrows required
e.	Tractor turning area	5 m below header channel allowed for turning of tractors	Mechanical operations require either to cross the quaternaries and surface drains or use a short 100 m operating run
f.	Surface drains	2 km of new surface drain required with associated field road	3.5 km of twinned surface drain with associated field road, plus one 700 m single surface drain.
g.	Collector(2) drains	1 km of existing collector; cleared on alternate fields	Existing collector cleared
h.	Collector drain structures	1 new collector drain junction culverts on alternate fields	1 new collector drain culvert
i.	Surface drain structures	1 new culvert under field road plus 1 culvert on alternate fields	6 new culverts into collector

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REHABILITATION OF JOWHAR SUGAR ESTATE

Comparison of Alternative Field Layouts(1)

Item	Details for typical 70 ha field	
	300 m length furrows	100 m length furrows
j. Field drainage	300 m long field drains easily installed where necessary	Unsuited to subsurface field drainage
k. Mechanisation	Allows for 300 m long machine runs. Suitable for mechanised harvesting	Necessitates breaking through quaternaries and crossing surface drains to obtain machine runs of longer than 100 m
l. Land levelling	Earthmoving estimated at 218 m^3/ha on average	Earthmoving estimated at 204 m^3/ha on average
m. Costs(3)	Estimated at SoSh 14 600/ha	Estimated at SoSh 16 000/ha

Note : (1) See Figure I.6.1 for the field layouts.

(2) Collector drains are to be deepened in the West drain area.

(3) Costs include new field channels and in-field structures, collector drains and drain culverts, and land levelling.

Experience from the 1977/78 Drainage Trials (MMP 1978) and international guidelines (FAO 1967) lead to the recommendation that for 300 m long furrows minimum and maximum slopes of 0.0002 (0.02%) and 0.002 (0.2%) respectively should be adopted.

The minimum slope is to ensure that waterlogging does not occur and will enable excess water to be drawn off the end of the furrows into the surface drain.

The maximum slope is to ensure a reasonably even water distribution along the furrow. If the slope is too steep the water will quickly run to the bottom end and will not allow for adequate water to infiltrate the soil at the top end of the furrows. The maximum slope is also chosen to prevent soil erosion due to high water velocity in the furrows.

6.2.3 Land Levelling Requirements

Sample level surveys have been carried out on 6 fields covering an area of 100 ha gross. Land levelling requirements have been estimated for each sample area for both the proposed layout with 300 m furrows and the alternative layout with 100 m furrows. The land levelling criteria were as follows:

	300 m furrows	100 m furrows
Slope along furrows	0.02 to 0.2%	0.02 to 0.2%
Slope perpendicular to furrows	0 to 0.05%	0 to 0.05%
Land levelling unit	350 m x 50 m (1.75 ha)	100 m x 100 m (1 ha)

Quantities were calculated for the sample fields as shown in Table I.6.4. The average earthmoving requirements were 218 m^3/ha and 204 m^3/ha for the 300 m and 100 m long furrow alternatives respectively.

TABLE I.6.4

Land Levelling Requirements

Field Nr	Area surveyed (ha)	300 m furrows (m^3/ha)	100 m furrows (m^3/ha)
6 I	5	233	238
8 III	5	255	239(1)
2 IV	7	216	164
3 V	71	119	112(1)
4 IV	5.5	250	221
4 V	6.5	235	252
Average(2)	-	218	204

Note: (1) Quantity for 100 m furrows estimated from that calculated for 300 m furrows.

(2) The quantities given are taken as being representative of the field. Average quantities are taken as a mean of the representative values.

6.3 Land Levelling Methods

6.3.1 Laser System

The laser system of control for land levelling is recommended for the following reasons:

(i) Present land levelling practice and the drainage trials at the Estate (MMP 1978 b) have shown that the required accuracy of $\pm$ 50 mm in land levelling is difficult to achieve with the manual systems. The laser system has been shown elsewhere (MMP 1976) to work to much closer tolerances (as low as $\pm$ 25 mm).

(ii) The laser system requires fewer machines and thus lower maintenance, operation and capital costs.

(iii) Less survey work is required with the laser system. This reduces demand for training of staff.

6.3.2 Equipment Requirements

The rehabilitation programme allows for 5 300 ha to be land levelled in 5 years, i.e. 1 060 ha per year. Assuming land levelling can be carried out for 265 days per year this will require a work rate of 4 ha per day. Experience in other countries shows that land levelling is most efficiently performed by a combination of box scrapers and drag scrapers or landplanes, together with a laser system. It would be possible to make use of the Estate's Cameco box scrapers in this work. If a manual control system were to be used approximately double the number of work machines would be required.

CHAPTER 7

DESIGN CRITERIA

7.1 Introduction

For this Feasibility Study, design criteria have been adopted for the outline design work undertaken. These criteria are considered to be appropriate for this level of design, but should be reviewed before final design work commences.

Design flows for canals have been based on the irrigation requirements calculations presented in Annex II. Channel capacities have been designed to meet the peak requirement which occurs in March.

7.2 Tertiary Canals

The introduction of the new field layout, discussed in the previous chapter, will require new header channels as indicated on Figure I.6.1 with the existing header channels retained but reformed to feed the new header channel on alternate fields. Water will be abstracted from the header channels using siphon pipes, and a minimum command (water level above field level) of 0.15 m has been assumed to facilitate this.

The proposed tertiary canal/header channel cross section is illustrated in Figure I.7.1. The channel is designed to carry a peak discharge of 150 l/s, calculated as follows:

Peak net irrigation requirement = 181 mm in March

Field irrigation requirement = $\frac{181}{0.65} = 278$ mm

This is equivalent to a continuous flow at the head of the tertiary canal of 1.04 l/s/ha. However, irrigation will only take place for a maximum of 12 hours/day, and the design flow rate has to be 2.08 l/s/ha.

The average field unit size is about 70 ha net:

$70 \times 2.08 = 150$ l/s, say.

For larger fields presently served by one tertiary, it may be necessary to subdivide the field to avoid excessive discharges in the tertiary canal/header channels. However, even on minimum slope, the standard tertiary section can take up to 190 l/s whilst still retaining 0.15 m freeboard. This would be sufficient for a net field area of up to 90 ha.

The tertiary canal/header channel section is designed to be machine-formed in two stages. Firstly an embankment is formed with a top width of 3.5 m and a top level corresponding to the design water level in the tertiary. This embankment is compacted during construction by the passage of earthmoving machinery. Subsequently the V-channel is formed using an appropriate ditching machine, with the excavated material dumped to form embankments.

7.3 Secondary Canals

7.3.1 Canal Flows

Secondary canal design flows have been based on the offtaking tertiary requirements, plus an allowance for seepage and evaporation losses. During the fieldwork for this study, canals were examined for evidence of seepage losses and, in general, seepage appeared not to be a severe problem, even on the canals in high command. Previous investigations (MMP, 1976) suggested a seepage loss of 1.4 m^3/s per million m^2 of wetted perimeter, which is very low. Seepage is likely to be most severe when canals have been dry for some time and the soil has been allowed to crack. This will not be the case for secondary canals because water will be retained in the canals overnight by a strict programme of gate closure.

Evaporation losses from the canals in their present condition may be quite significant since extensive weed and reed growth in the channels, and bushes and trees in the banks, will transpire freely. However, with improved maintenance, this problem will be reduced and evaporation losses will be very small.

For design purposes secondary canal losses have been determined for each reach using the expression:

$$\text{Loss in reach} = 0.015\, Q^{\frac{1}{2}} L \quad (m^3/s)$$

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

This is equivalent to something like 3.0 m^3/s per million m^2 of wetted perimeter, and typically amounts to some 5% of the canal head discharge. Further details of canal losses are presented in Section 7.6 herein.

7.3.2 Water Surface Slope

Because of the severe command problems affecting parts of most canals, canal slopes have been kept as flat as possible, similar in fact to the existing slopes. Some siltation will inevitably occur as a result of this, but most of the suspended sediment will be removed from the water in the settling basin at the head of the system.

Secondary canal slopes have been determined using the Lacey formulae with a minimum silt factor of 0.45. This is considered appropriate in view of the very fine sediment sizes found in the existing canal beds.

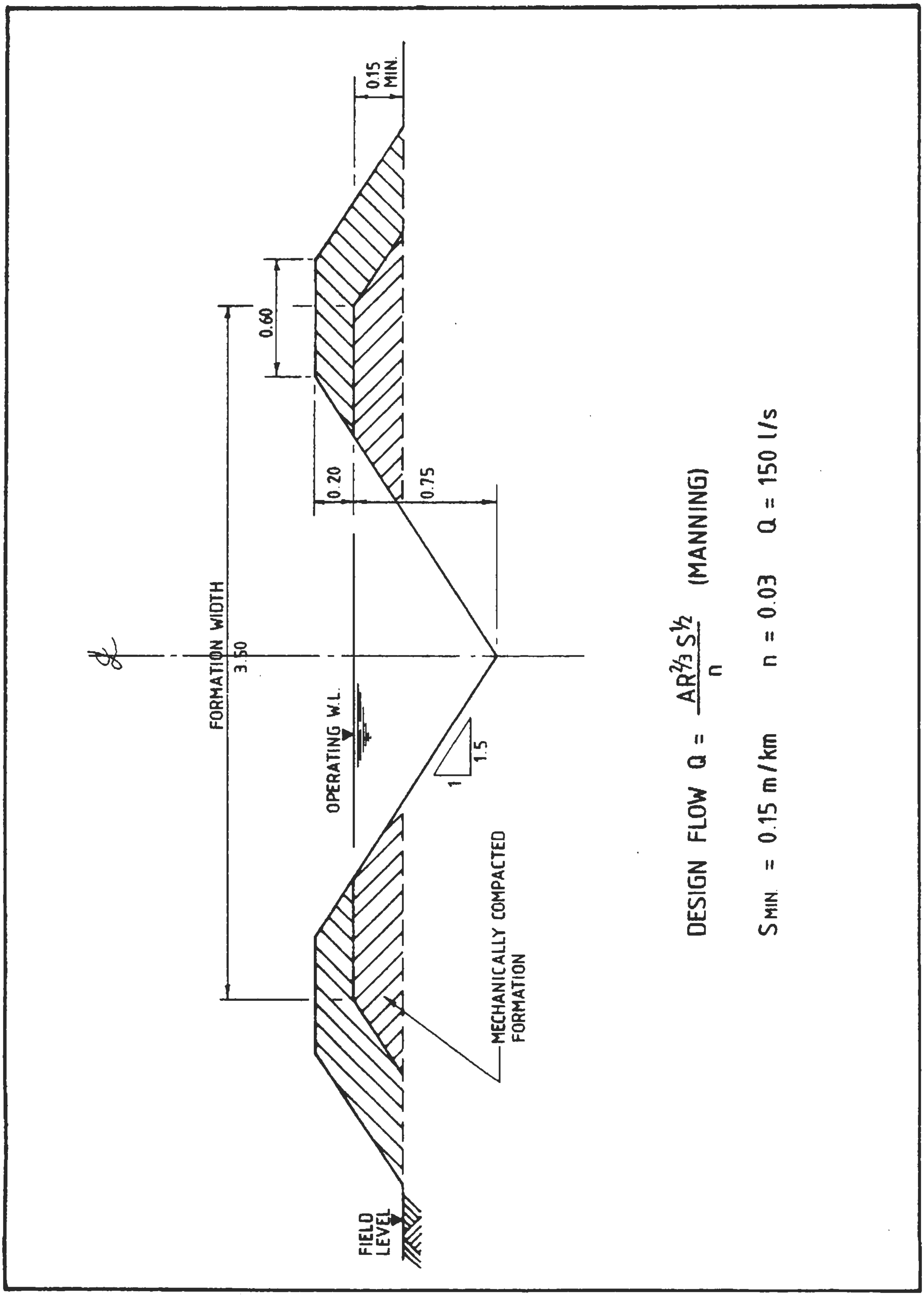
The appropriate Lacey equations are:

$$\left.\begin{aligned} Ws &= 4.83\, e\, Q^{\frac{1}{2}} \\ f &= \frac{2.46\, V^2}{Dm} \\ Dm &= \frac{0.4725\, Q^{\frac{1}{3}}}{e^{\frac{2}{3}}\, f^{\frac{1}{3}}} \end{aligned}\right) \quad \text{valid for all values of median grain size of sediment}$$

$$S = 0.000206\, e^{\frac{1}{3}}\, f^{\frac{2}{3}}\, \frac{E}{Q^{\frac{1}{6}}} \qquad \text{(for median grain size less than 0.2 mm)}$$

Figure I. 7.1

Tertiary and Header Canal



where Dm = mean depth, $\frac{area}{Ws}$ (m)

Ws = water surface width (m)

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

V = mean velocity (m/s)

Q = discharge (m^3/s)

S = water surface slope (m/m)

e = width factor

f = Lacey silt factor

7.3.3 Channel Section

Having determined the water surface slope using Lacey regime theory, the channel sectional properties have been chosen using the Manning equation. Lacey channels tend to be wide and shallow and assume a fairly smooth section (equivalent Manning's roughness of 0.025). To allow for the inevitable silting and weed growth a Manning's n value of 0.03 is recommended. The proposed secondary canal design charge based on this is given in Figure I.7.2.

Where existing canals are being remodelled the new section design will be selected to suit the existing section as far as possible. If necessary a wider, shallower channel will be used.

7.4 Main Canal

No new reaches of main canal are required for the rehabilitated scheme. The remodelled section will be designed using the same equations as for secondary canals.

7.5 Regulating Structures

7.5.1 Introduction

All the existing regulating structures on the Estate have been designed for vertical lifting gates. These structures can be divided to four basic groups:

(i) the two intake structures (Luigi canal and 21st October canal). These are basically main canal head regulators;

(ii) main and secondary canal cross regulators;

(iii) secondary canal head regulators;

(iv) tertiary canal head regulators.

There are only two structures in group (i) and no new structures of this type are planned. Group (ii) covers a wide range of structures which are similar in format and different only in detail. Cross regulators on the 21st October canal are relatively recent concrete structures in sound condition, whereas those on other canals are up to 60 years old and are generally of brick. All suffer from problems associated with the control gates - very few of these structures have steel gates in an operable condition. Group (iii) structures are very similar to secondary canal cross regulators and suffer from the same problems. The last group, tertiary canal head regulators, is the most numerous. Again these structures follow a standard format generally comprising an upstream headwall with gated orifice, a pipe or culvert through the secondary canal bank, and some form of outlet into the tertiary canal. Something like half of the existing tertiary canal head regulators need new gates.

Where existing canal cross regulators are to be retained it will be necessary to provide new control gates, since nearly all of the existing gates are in poor condition. These will be similar to those proposed for the new cross regulators (Section 7.5.2). A typical gate is illustrated in Figure I.7.7. Not all regulators will be provided with new gates - some will be left ungated since, with the flat canal slopes, gated regulators are only required every 2 km or so (typically at every other offtake). Drawing Nr 12700/12 shows the locations of all main and secondary canal structures and indicates those which are existing and those which are new.

7.5.2 New Structures

One of the main aims of the rehabilitation works is to provide the means for better regulation and control of canal flows. This can only be achieved by incorporating some means of flow measurement. Flow measurement cannot readily be achieved using the sort of lifting gate structure found in the existing scheme. Flow measurement is much more easily and accurately achieved with a weir-type regulator. Unfortunately, these are more expensive than the lifting gate alternative and, in any case, it is not practicable to convert the existing structures to this type. The proposal is, therefore, to introduce five new movable weir structures (Figure I.7.3) into the canal system, three cross regulators on the 21st October canal and two new secondary canal head regulators for S6 and the canal serving S1/S2 from 21st October canal. For the remaining secondary canal head regulators (for canals S1, S3, S7) new lifting gate structures (Figure I.7.4) will be used and approximate flow measurement will be facilitated by the inclusion of a downstream water level gauge. Lifting gate structures will also be used for new secondary canal cross regulators.

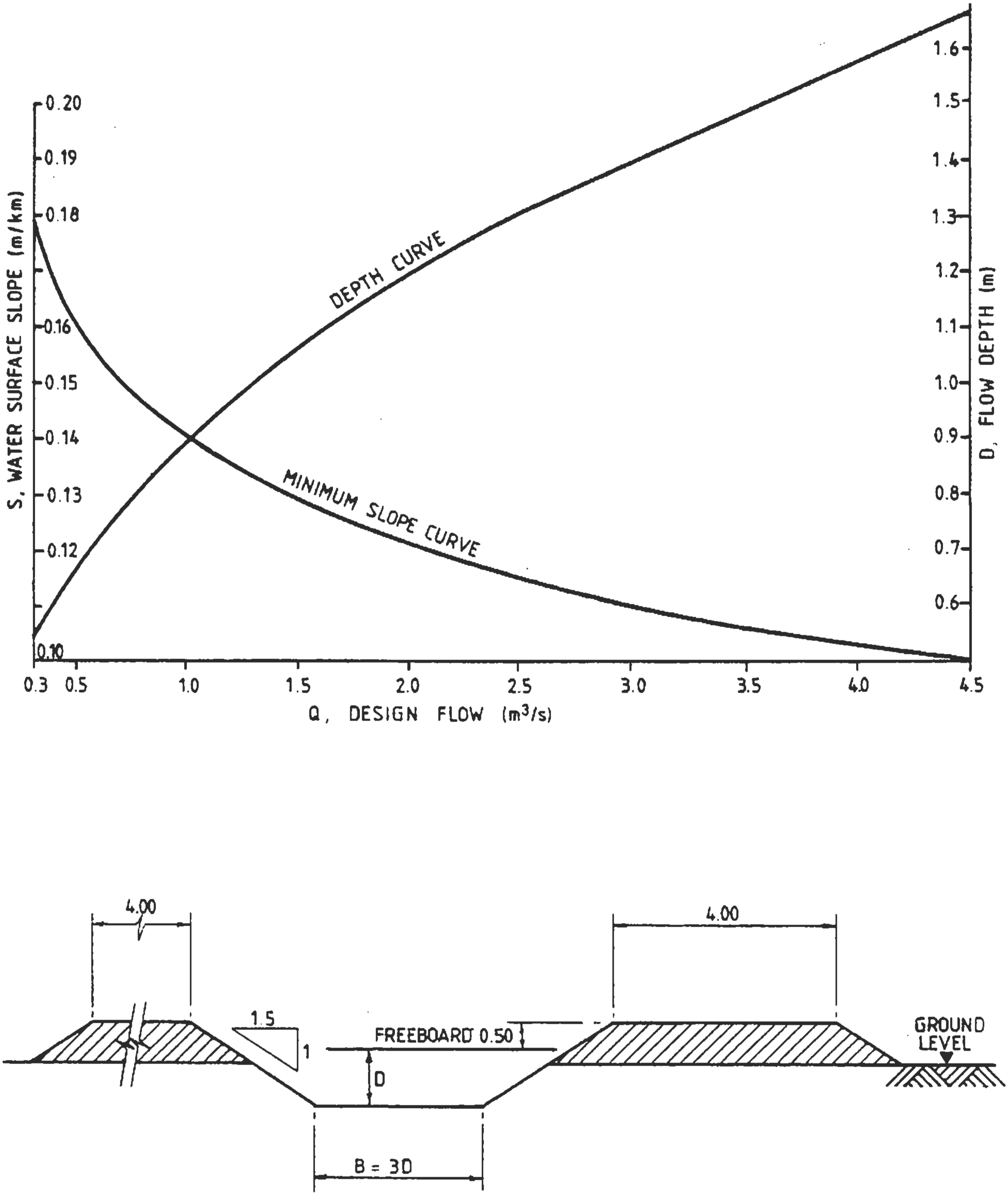
For the tertiary canal head regulators a similar structure to the existing regulator is proposed (Figure I.7.5). For standard sized field units a 0.60 m diameter pipe will be required. For larger fields 0.75 m diameter pipe will be appropriate.

For preliminary design purposes, the following sizes of structure gave been adopted:

Structure	Design flow range (m^3/s)	Size
Movable weir	<2.2	Single 2.0 m span
	2.2 to 5.5	Double 2.5 m span
	5.6 to 9.9	Triple 3.0 m span
Lifting gate	<0.8	Single 1.05 m dia.
	0.8 to 1.5	Double 1.05 m dia.
	1.6 to 2.5[(1)]	Triple 1.20 m dia.

Note: (1) Upper limit can be increased for head loss greater than 0.10 m.

Secondary Canal Design



TYPICAL CROSS SECTION

Figure I.7.3

Typical Movable Weir Head Regulator

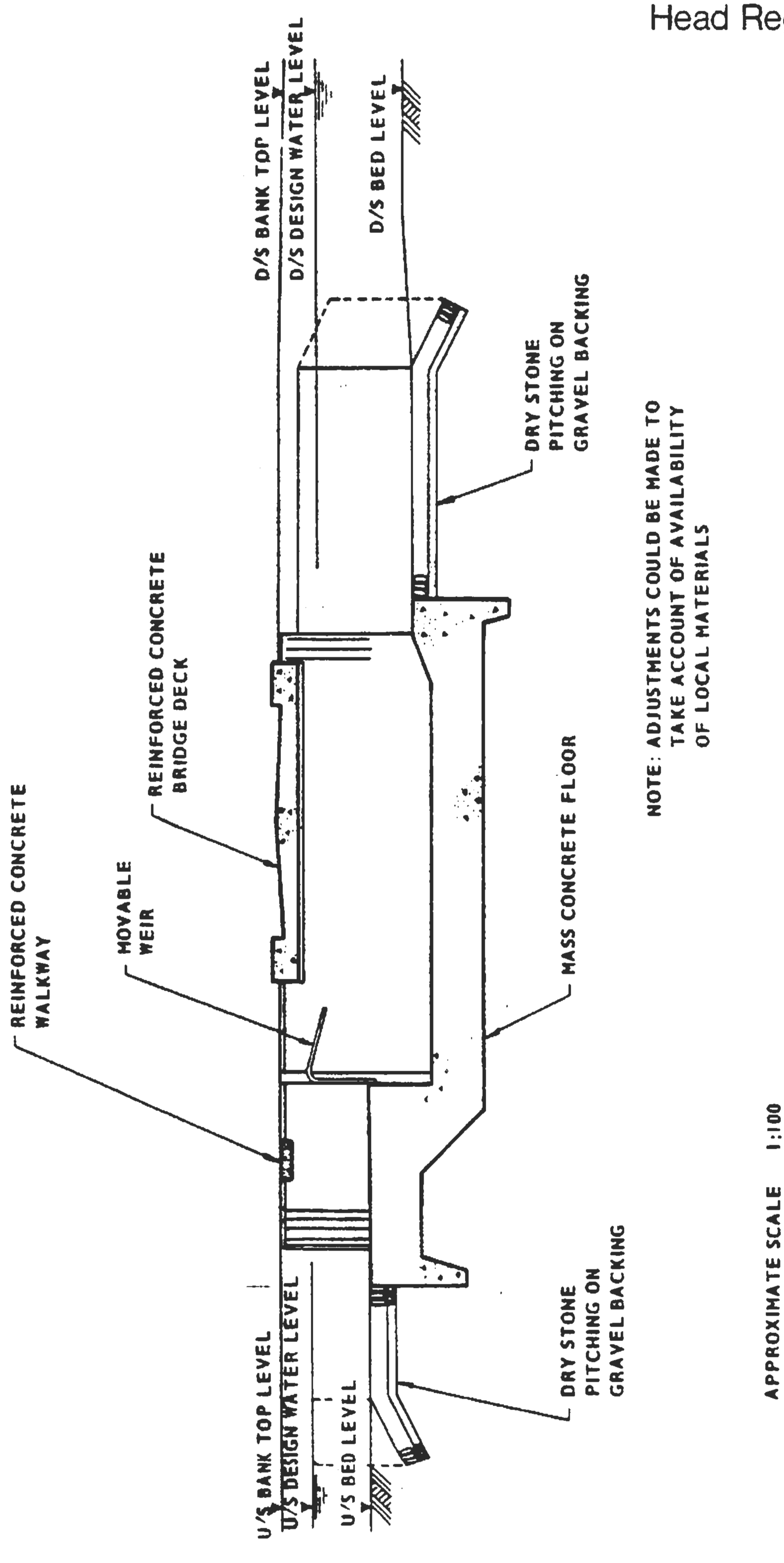


Figure I. 7. 4

Typical Canal Cross Regulator

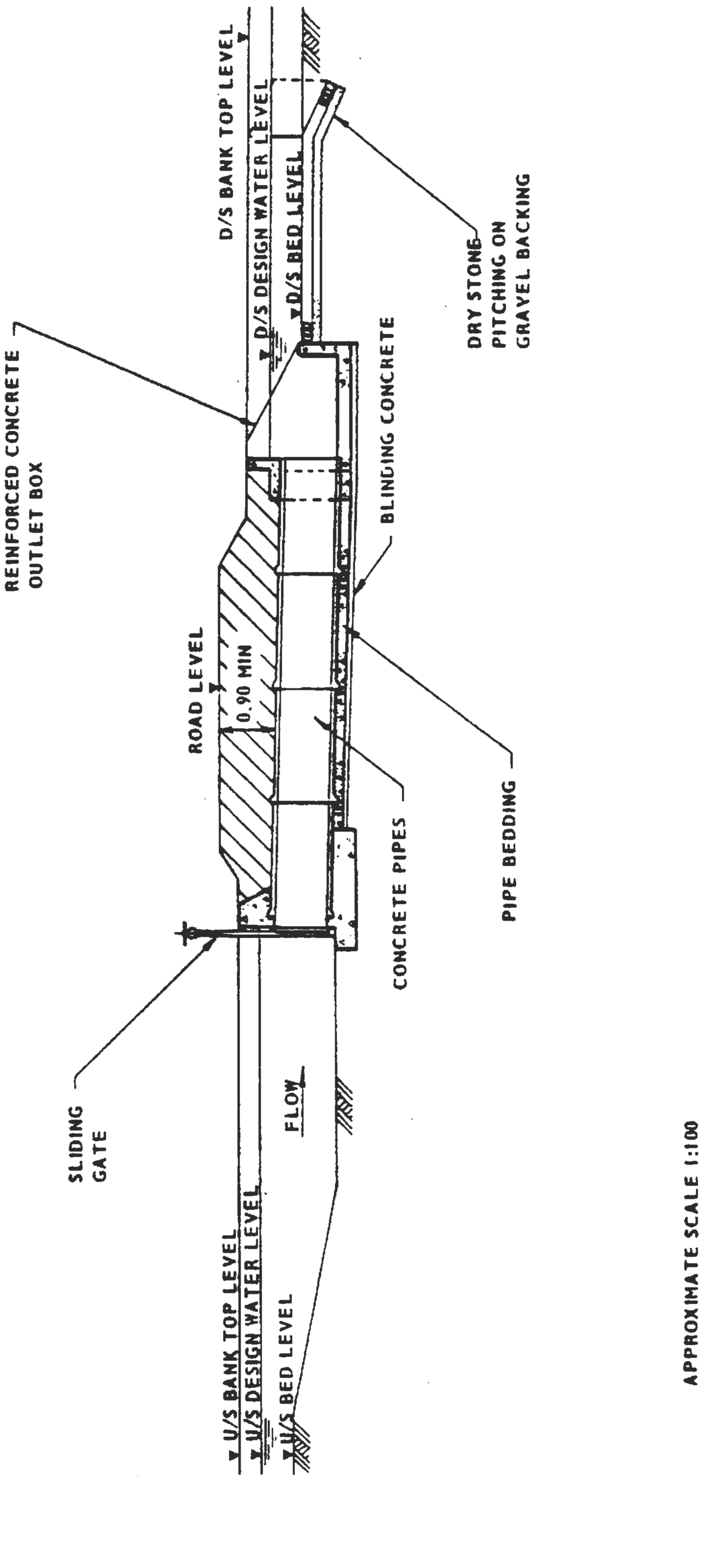
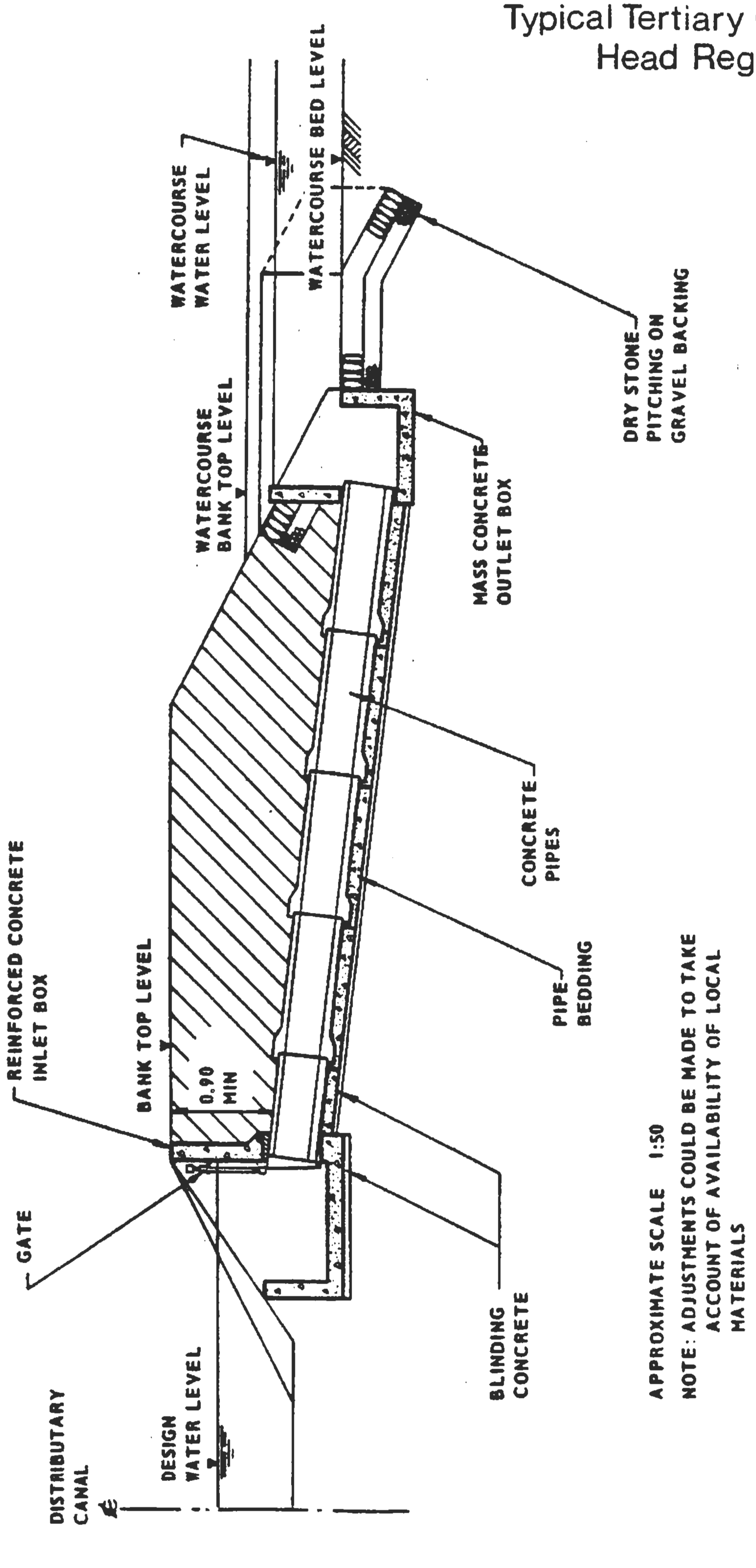


Figure I.7.5

Typical Tertiary Canal Head Regulator



Movable weirs have been used wherever flow measurement is essential. A minimum head loss of 0.15 m at design flow has been allowed for.

Lifting gates have been used on all other regulating structures, with single, double or triple gates depending on the flow. A minimum head loss of 0.10 m has been allowed for, but larger head losses have been adopted where necessary.

For the final design a greater range of structure sizes can be included so that the particular requirements of any one regulator can be met more precisely. However, this should be tempered by the need for standardisation of gate sizes and pipe diameters.

Table I.7.1 presents a list of the new canal structures with location, design flow, size and type.

In assessing the need for tertiary canal head regulators it has been assumed that one regulator serves two field units and thus has a design discharge of 300 l/s. Fields served by new canals will of course have new tertiary canal head regulators, but new regulators have also been allowed for where the existing structure is considered beyond repair or where the field is to be served from a different location. A total of 22 new head regulators has been allowed for - 6 on the 21st October canal, 2 on canal S1, 3 each on canals S3 and S6, and 8 on canal S7.

7.5.3 Pumping Stations

The estate already has a number of operating pumps delivering irrigation water to parts which cannot readily be served by gravity. There are also two unused units, complete with engines, at Farm II. Many of the pump units and diesel engines in use at present are relatively new and in sound condition. Most of the Estate's pumps were obtained from the GDR and are as follows:

Pumps	-	centrifugal pumps rated at 1 240 m^3/hour at 12 m head
Engines	-	6 cylinder diesel units rated at 70 kW

At the middle drain pump station there is an electric powered pump which receives its power from Jowhar via a transmission line. This pump was brought back into service in July 1983. The Estate also has a number of mobile pump units in operating condition.

It is assumed that these existing pumps will be used in the rehabilitated scheme for the following:

- pumping into canal S1 when necessary
- pumping for fields T3 and T4
- pumping out of storage reservoirs to irrigate plant cane during a severe drought

There are more than sufficient units for this purpose, including standby capacity.

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List of New Canal Structures[1]

Canal	Chainage (km)	Design flow (m^3/s)	Structure size and type[2]
S1	0.000	1.4	2 x 1.05 m dia. lifting gate regulator
	0.600	1.1	2 x 1.05 m dia. lifting gate regulator
	2.500	0.8	1 x 1.05 m dia. lifting gate regulator
S2 (headreach upstream of reservoir)			
	0.000	4.4	2 x 2.5 m movable weir head regulator
S2 (downstream of reservoir)			
	0.000	2.8	3 x 1.2 m dia. lifting gate regulator
	5.350	1.3	drain underpass (see drain structures)
S3	0.000	3.5	3 x 1.2 m dia. lifting gate regulator
S6	0.000	1.0	1 x 2.0 m movable weir regulator
	1.810	0.7	1 x 1.05 m dia. culvert
	2.580	0.3	1 x 1.05 m dia. lifting gate regulator
S7	0.000	2.4	3 x 1.2 m dia. lifting gate regulator
	1.750	2.4	drain underpass (see drain structures)
	2.400	1.8	3 x 1.2 m dia. lifting gate regulator
	3.800	1.2	2 x 1.05 m dia. lifting gate regulator
	4.500	0.9	2 x 1.05 m dia. lifting gate regulator
	5.400	0.6	1 x 1.05 m dia. lifting gate regulator
21st October canal			
	2.600	6.7	3 x 3.0 m movable weir regulator
	4.850	4.7	2 x 2.5 m movable weir regulator
	7.200	4.2	2 x 2.5 m movable weir regulator
	7.200	4.2	drain underpass (see drain structures)
21st October canal (pumped tail reach)			
	0.000	1.4	2 x 1.05 m dia. lifting gate regulator

Notes: (1) Existing canal structures are listed in Appendix C1 to this Annex, and their condition is discussed in Appendix A.

(2) For simplicity structure sizes have been standardised. Details are given in the text of Section 7.5.2.

However, for the new pumping station serving canal S7 and the tail reach of the 21st October canal, new units will be required. For this station inclined axial flow pumps are proposed. These are more suitable than centrifugal pumps for the low pumping lift required, and they do not require expensive civil engineering works. A typical arrangement is shown in Figure I.7.6. The units are similar to those which will be installed in the main drainage pumping station (these pumps have been ready for shipment for some years and await the establishment of the letter of credit).

7.6 Canal Losses

Following submission of the Draft Final Report it was suggested that the 10% allowance for canal losses (main and secondary canals) might be an underestimate. Particular concern was expressed with regard to losses from secondary canals overnight. We have therefore examined this aspect in more detail, as descibed below.

It has been emphasised (Section 13.4 in this annex) that a strict closure sequence will be necessary at the end of each day for all the secondary canal regulator structures. This would proceed downstream from the head regulator, with each cross regulator and the tertiary canal offtake regulators being closed in sequence. This will help to keep losses to a minimum.

Canal design capacities have been based on irrigation requirements for the full development of about 8 000 ha (both cane and non-cane areas), for the peak month.

(a) Secondary Canal Losses

A summary of secondary canal data is presented in Table I.7.2.

TABLE I.7.2

Secondary Canal Details

Canal	Length (km)	Flow at head (for full development) (m^3/s)	Number of X-regs (including tail reg)
S1	3.5	1.5	4
S2	7.0	2.9	8
S3	8.5	3.5	9
S6	4.2	1.0	2
S7	7.4	2.4	5
21st October tail	6.9	1.5	3
Average	6.2	2.1	5.2

It should be noted that the data refer to the full development. It can be seen that the 'average' secondary canal is 6.2 km long, with a head discharge of 2.1 m^3/s and has 5 cross regulators (including the rail regulator). In fact not all cross regulators will be gated; on average there will be a gated cross regulator every 2 km. So the average secondary has a gated head regulator, 2 gated cross regulators, and a gated tail regulator.

It is estimated that it will take the gate operator plus a labourer about 10 minutes to close the gates at each regulator. The water 'losses' occurring during the closure period can thus be estimates for the typical canal, as shown in Table I.7.3.

TABLE I.7.3

Estimation of Secondary Canal Operational Losses (for the 'average' canal)

Regulator	Time for closure plus travel (minutes)	Flow at head of reach (m^3/s)	Volume lost (m^3)
H-reg	15	2.1	1 890
X-reg	15	1.5	1 350
X-reg	15	0.9	810
Tail	10	0.3	180
Total			4 230

So a total of 4 230 m^3 is lost. This is equivalent to less than 5% of the total 12 hour volume flow in the canal, and this volume has to be replaced the following morning. It is not, however, 'lost' in terms of irrigation since the water will continue to flow into the tertiary canals and on to the fields until the siphons de-prime.

Additional losses will occur during the day as a result of seepage and gate leakage. Evaporation losses will be negligible for a well maintained (weed free) canal. In the typical canal evaporation at 6 mm/d would account for only some 220 m^3/d (0.2%). Gate leakage losses will be low for well maintained structures. If one assumes, for example, a loss of 1.0 l/s/gate for say 10 tertiary head regulator gates on a secondary canal, the total loss over 12 hours (at night) would be 430 m^3 (0.5%).

Seepage losses from canals were measured during the 1976 study (see Section 7.3.2). A seepage rate of 1.4 m^3/s per million m^2 of wetted surface was determined. In the typical secondary canal, this would result in a total loss over 24 hours of some 4 500 m^3, equivalent to some 5% of the 23 hour flow.

Figure I.7.6

Typical Irrigation Pumping Station

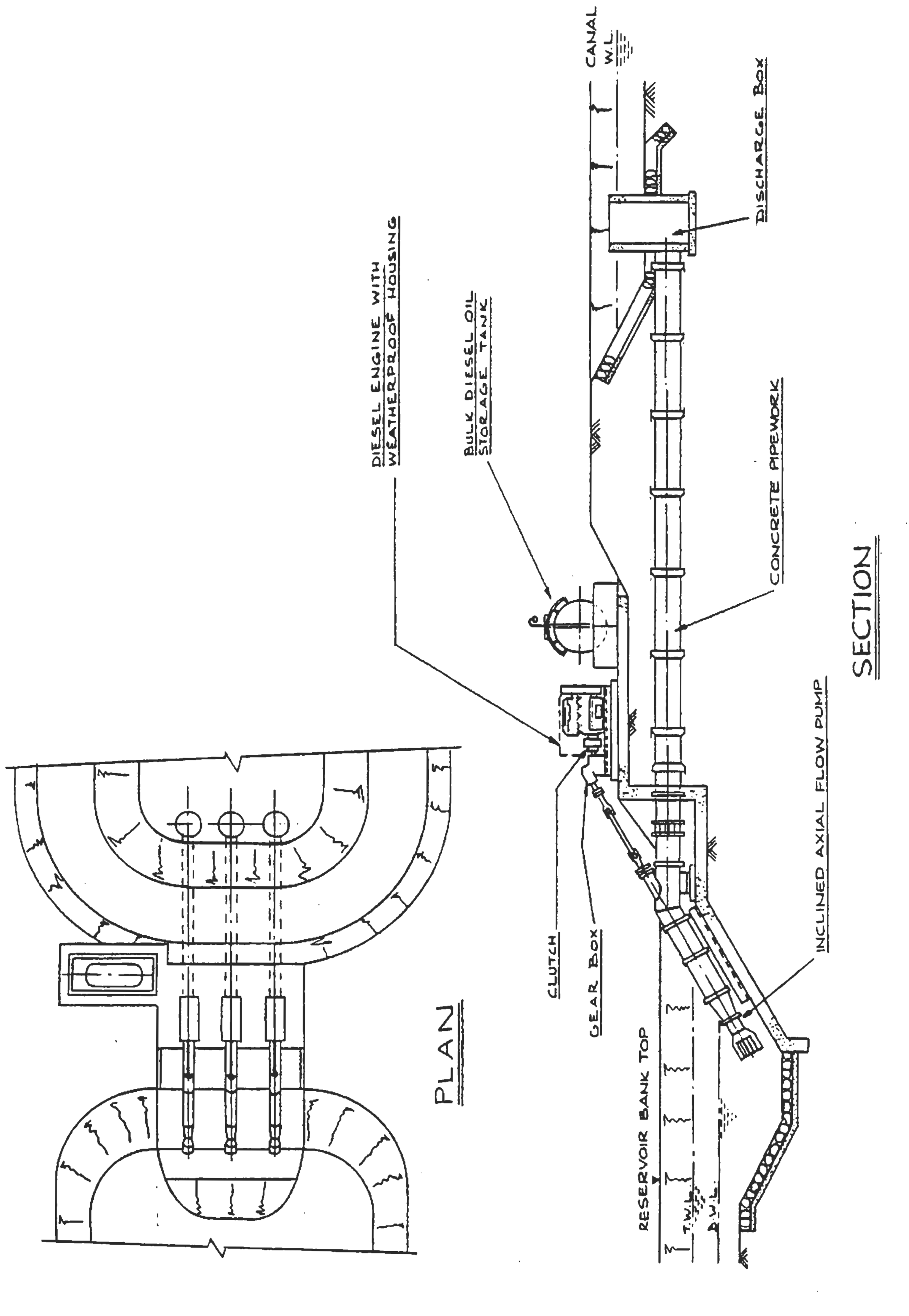
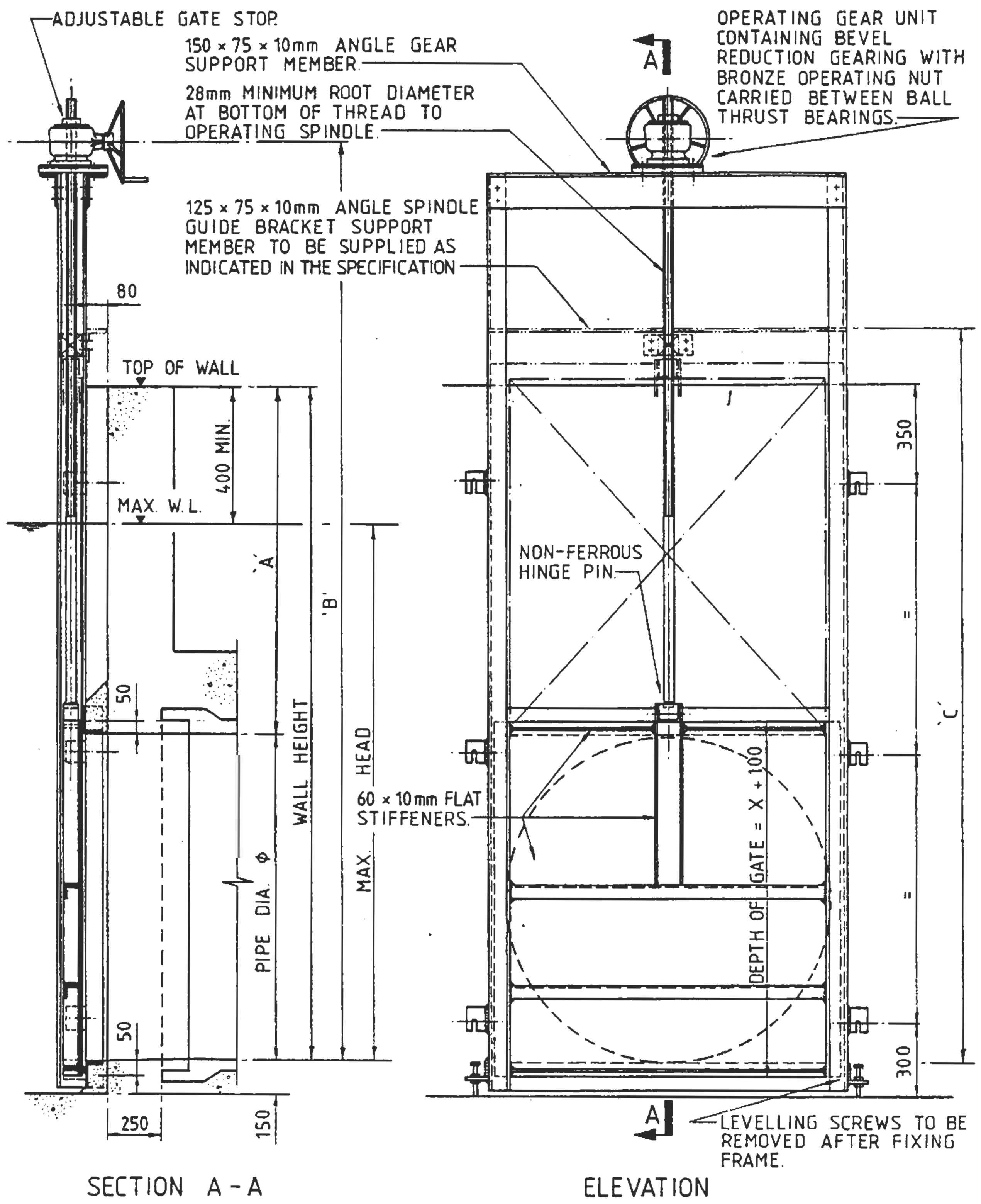


FIGURE 1.7.7

Detail of Sliding Gate for Cross Regulator



NOTE.

ALL DIMENSIONS IN MILLIMETRES

(b) Summary

The total estimated losses from secondary canals, thus amount to a little over 10%. However, only some 6% of this is a true loss, since the balance flows into the tertiary canals and is available for irrigation. To be added to this are the losses from night storage reservoirs and losses in the upper part of the 21st October canal (the main canal).

Seepage rates from reservoirs are expected to be very low. Losses from the nearby Jowhar Offstream Reservoir have proven to be less than 5 mm/d - similarly low rates should apply to the much smaller reservoirs in the Estate. With a total evaporation and seepage loss of 10 mm/d, the volume of water wasted per day would be some 8 000 m^3 (80 ha of reservoir area), which is 1.5% of the total 24 hour flow in the main canal for the peak month.

Seepage and leakage from the 21st October canal (12 km) would add another 5% to the total losses. Thus real losses would amount to some 11%, which is a little higher than the 10% adopted in the Draft Final Report.

It is concluded that the 10% system loss adopted in the Draft Final Report is adequate for assessing canal capacities and overall irrigation needs (for comparison with water availability).

SECTION C

DRAINAGE

CHAPTER 8

PRESENT DRAINAGE SYSTEM

8.1 General

The existing drainage system has evolved over many years with the development of the Estate and at present apparently fulfils the following functions which are partially conflicting:

- to remove from the fields surplus surface runoff from field irrigation and rainfall and to convey this to outfalls at the southern boundary of the Estate;

- to act as a supply system to provide irrigation to fields which are not adequately served by the canal systems;

- to provide temporary storage which can be used for escaping surplus canal water (particularly at night) and from which water can be pumped into canals as and when required.

In addition to problems which arise from the conflicts between these objectives, there has been a serious deterioration in the condition of the drainage system, and together these result in very poor performance.

8.2 In-field Drainage

The existing field layout is shown in Figure I.4.3. Each field within the Estate has provision for surface drainage of excess rainfall or irrigation. This takes the form of a 1 m wide drainageway running down the major field slope in the middle of each fascia. Generally every second drainageway is connected to a collector drain at the bottom of the field by a pipe culvert across the access road, but these drainageways are not always effective in removing excess surface water, as they are bunded at regular intervals by aquiole channels. One important effect of these aquiole bunds is to provide almost complete retention of storm runoff. Rainwater is ponded by each aquiole bund down the field so that very little of it discharges into the collectors and thence into the main drains. This reduces the required capacity of the drainage pumps, but has the disadvantage that it can cause waterlogging on the fields, although sugar cane can tolerate short periods of waterlogging (see Section 10.3.1).

Many of the fields are provided with a collector drain which runs along the bottom of the field and connects in some way with the main drain system, whether this be by gravity or by pumping. The shape and size of the collector drains vary considerably from a very wide shallow channel to one which can be up to 2.0 m deep.

Several collector drains have been constructed very close to tertiary canals, and it is thought these drains may have been constructed more with a view to intercepting seepage from the tertiary rather than draining the field. This is borne out by some collector drains having been constructed between the tertiary canal and the field which it is irrigating, where the only reason can be one of restricting canal seepage from waterlogging the top of the field. In such cases small culverts have been provided to take the quaternary channels across the drain. This is a good example of the problems associated with excessive commands in canals (Section 4.1.3).

The junctions between collector drains and main drains are generally through 0.30 m or 0.50 m diameter pipes in the main drain bank although the collector drains are often heavily infested with weed growth and it is not possible in some cases to see whether the pipe is clear. Some of the collector drains are not connected with the main drainage system because the water level in the outfall drain is higher than the surrounding land; mobile pumps are used to empty these drains when required.

The relatively new fields M1 to M5 and T1 to T4 have been recently provided with collector drains. These are adjacent to the 21st October canal with the result that these fields receive their irrigation supplies across the drains. The drains are not connected to the main drain system.

8.3 Main Drainage System

8.3.1 Channels and Structures

The existing main drainage system comprises the open channels shown in Figure I.8.1. There are three main drains called the West, the Middle and the East drains, which run parallel to one another in a north-south direction through the Estate towards the southern boundary. For most of their length the drains flow by gravity, but there are pump stations near the tail of each drain to discharge the water at a higher level. In the past these allowed disposal of the water into the area south of the Estate, but construction of the Jowhar Offstream Reservoir in this area has changed this situation. The pump stations and the outfall arrangements are described below in Sections 8.3.2 and 8.4.

The three main drains were inspected in May to July 1983 and an inventory of channel and structure condition is presented in Appendix B.

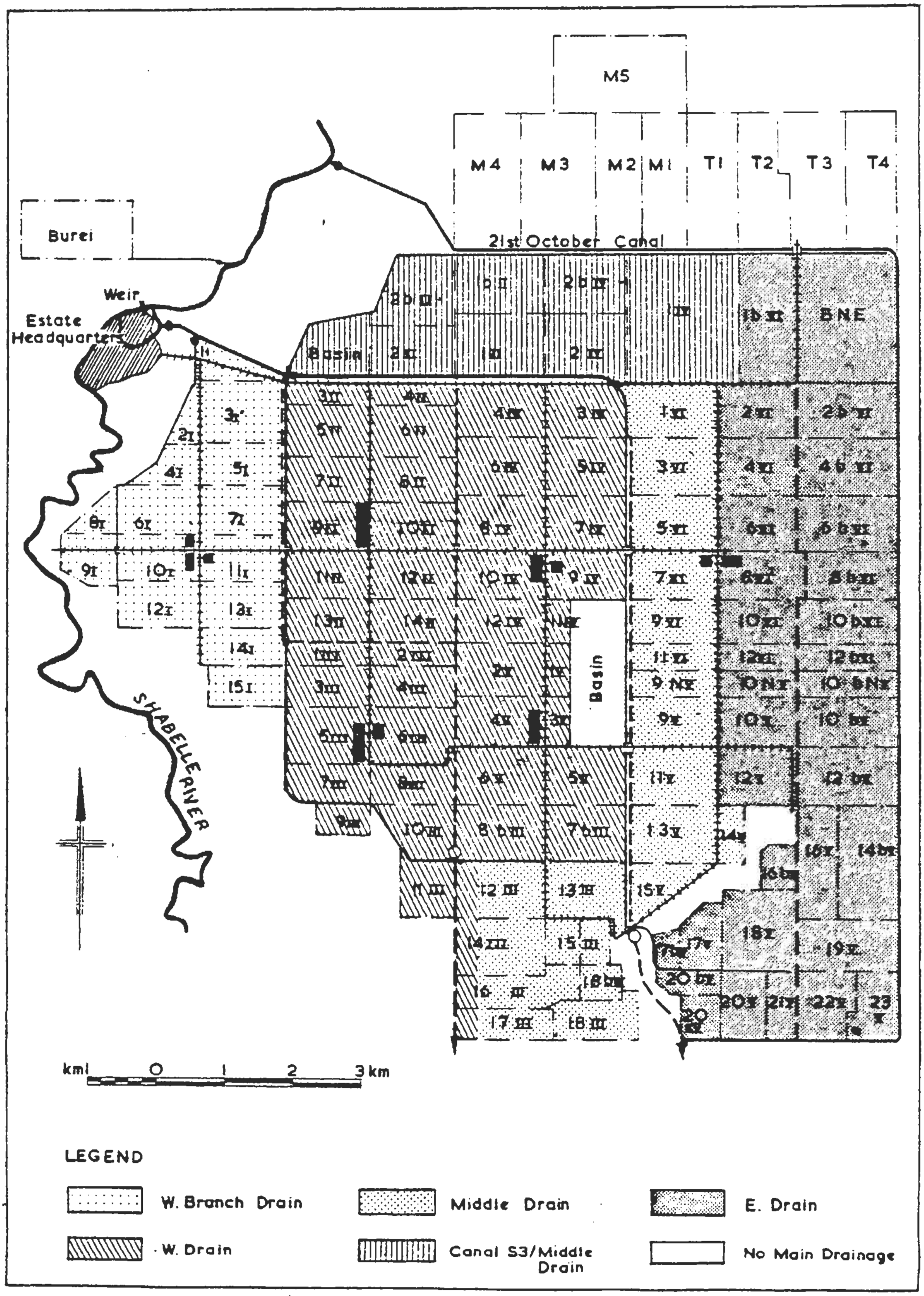
It is evident from the general growth pattern of the Estate that the West and Middle drains were the first to be constructed. They are generally wide channels with their cross sections reduced by heavy weed growth and siltation. Water levels in both drains were observed to be above the surrounding ground level at the southern end of the Estate and this meant that any field drainage in this area had to be pumped into the outfall drains. As the water level was above ground level this provided a gravity supply which was being used for irrigation. A branch drain was provided on the West drain to increase its effectiveness in draining Farm I and most of Farm II. Drainage from the land between the 21st October and the Duchessa canals has recently been connected to the head of the middle drain by construction of a culvert at the north-west of field 1 (VI) but downstream constrictions have prevented this from flowing properly. Drainage water from this area is also pumped into the head of canal S3 by a long established pump station at the Duchessa tail group.

The East drain is the most recently constructed main drain which drains the new parts of the Estate which are irrigated from the 21st October canal. One peculiarity of the East drain is that it is not a continuous channel and the section north of field 16 V is not connected to the drain channel south of 16 V.

Calculations made in 1976 indicated that both West and Middle drains have a free flow discharge of about 8 cumecs though the pumping capacity on each drain was much less than this. The free flow discharge of the East drain was estimated to

FIGURE 1.8.1

Jowhar Sugar Estate
Existing Drainage Catchment Areas



be less than half of that of the other two drains (MMP 1976). Catchment area of each drain is shown in Figure I.8.1, and Table I.8.1 shows the length of drains presently in use on the Estate.

TABLE I.8.1

Length of Existing Drains

Type	Length (km)
Collector	110.7
Branch	13.0
Main	26.4

Source : MMP 1976.

8.3.2 Pump Stations and Recycling of Drainage Water

Pump stations were located towards the tail of each main drain to raise the levels sufficiently for the drains to discharge from the Estate by gravity. Presumably these were sited on the original southern boundary of the Estate, and with the expansion of the Estate, the boundary has been shifted further south. The result is a confused irrigation and drainage system for these areas, with the water level in the drains being above ground level after pumping, and hence being used as the irrigation supply. Details of the situation on each drain are given below.

On the West drain there is a pump station to the north of field 10 III. This comprises a single pump rated at 1 240 m^3/h at 12 m head, driven by a 70 kW diesel engine. This lifts the drainage water to the higher level of the West drain downstream. Canal S2 joins the drain downstream of the pump station, and some 284 ha of land downstream is then irrigated solely from this section of the West drain.

To facilitate this, there is a cross regulator on the drain between fields 12 III and 14 III, which diverts water to irrigate fields 14 III, 15 III, 16 IIIW, 16 IIIE, 17 III, 18 III and 18bIII (though some of these may require additional pumping).

On the Middle drain there is a complex pump station which allows pumping of drainage water out of the Estate or from the drain into the tail of canal S3, whence the water can be used for irrigation of the south-east corner of the Estate. At this station there are two pump units similar to that on the West drain but these are powered by electric motors with standby diesel engines. As described in Section 8.3.1, there is also a pump station at the Duchessa tail group which pumps water which might otherwise flow into the new extension at the head of the Middle drain. This pump is diesel powered. The East drain, being much smaller than the other two, is not provided with permanent pumping equipment. Pumping is resorted to occasionally by using mobile plant. Drainage from the northern section of the East drain is used to irrigate field 16 V.

8.3.3 Salinity

During earlier studies observations were made of the electrical conductivity (EC) of the drain water at each of the three pumping stations. The readings are given below in Table I.8.2.

TABLE I.8.2

Electrical Conductivity of Drainage Water (mmhos/cm)

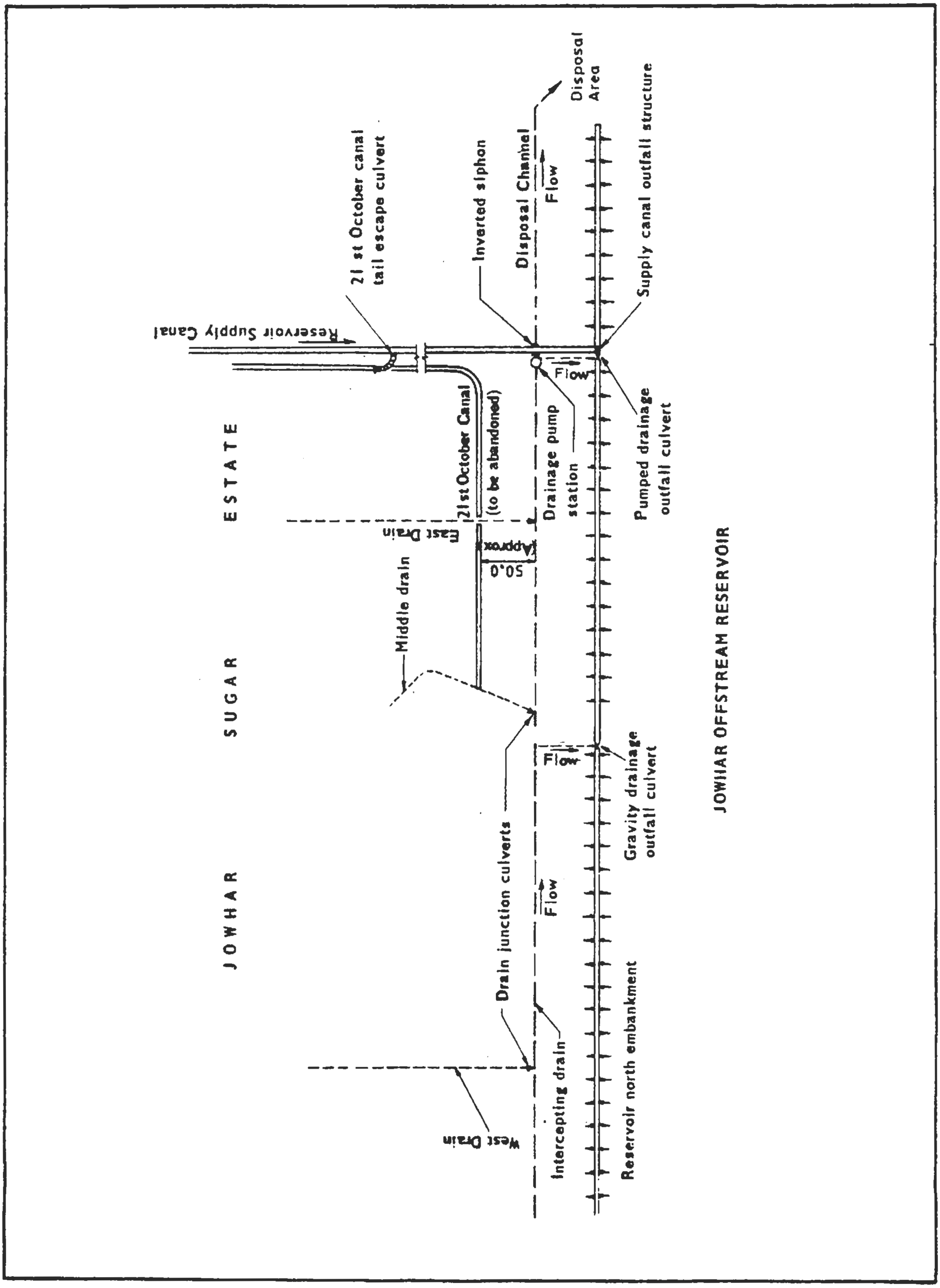
Date	West drain	Middle drain	East drain
27.07.75	0.45	0.90	-
2.08.75	0.60	1.20	-
16.08.75	0.40	0.83	0.85
27.08.75	0.50	0.85	0.75
4.09.75	0.50	0.75	0.80
9.09.75	0.65	0.95	0.75
5.11.75	1.20	1.05	1.30
1.12.75	1.25	1.70	1.36
31.12.75	1.10	1.60	1.10
1.02.77	1.50	1.70	1.40
1.03.77	1.10	1.30	1.30
4.04.77	1.70	2.00	2.30
7.05.77	1.70	1.10	-
31.05.77	1.80	1.70	1.40
8.02.78	2.00	1.60	2.10
6.03.78	2.30	2.00	2.40
9.04.78	1.40	1.40	0.70
18.03.78	2.20	1.40	0.80
1.06.78	1.50	1.60	2.05
2.07.78	1.90	1.40	-
8.07.83	1.00	1.00	0.80

These data show that the existing drainage water is generally little more saline than the irrigation water. This implies that most of the water in the drains is excess irrigation water and rainfall, and that there is little subsurface flow to these drains.

8.4 Main Drainage Outlets

The original outlets of the main drains into the depression south of the Estate have been interrupted by the construction of the Jowhar Offstream Storage Project in this area. Accordingly that project's works include an Intercepting drain along the southern boundary of the Estate and new disposal arrangements (described in Section 8.5), and also three junction culverts to provide outlets from the main drains into the Intercepting drain. These junction culverts were designed for the deep drainage system proposed earlier for the whole Estate (MMP 1976), to permit gravity flow from field drains at 2 m depth along deepened collectors and main drains to discharge into the Intercepting drain. In addition, to enable the drains to operate satisfactorily at their current levels, drain falls were designed to be sited in the main drains upstream of the junction culverts.

Schematic Plan of Intercepting Drain and Drainage Disposal System



These works are still incomplete. Details of the designs are given in the following Jowhar Offstream Storage Project drawings and the present situation at the outlet of each main drain is described below.

SOM 75 - Drainage Junction Culverts and Tail Escapes

SOM 78 - Drain Fall

The West drain currently passes over the Intercepting drain on a bund and discharges in a pipe culvert through the northern embankment into the reservoir. Some work has been done on the West drain junction culvert, but much remains to be done. No falls have been constructed.

On the Middle drain both the existing outlet and the situation with new works are similar to those on the West drain.

The East drain junction culvert has been completed and is discharging water from the southern section of the East drain into the Interceptor drain but the temporary drain falls have not been constructed.

8.5 Drainage Disposal System

A schematic diagram of the drainage disposal works is shown in Figure I.8.2. The main works are the Interceptor drain, the drainage pump station, the pumped drainage outfall culvert, the gravity drainage outfall culvert and the disposal channel. These were included in the Jowhar Offstream Storage Project because that project took over the Estate's traditional drainage disposal area and would otherwise have prevented satisfactory disposal of drainage water from the Estate.

The Interceptor drain is a deep open drain which runs between the northern embankment of the reservoir and the southern boundary of the sugar estate, and into which the Estate drains flow by gravity. It also acts as a cut-off drain to prevent local groundwater seepage from the reservoir affecting the Estate. The drain has not yet been excavated to its full design depth and a survey undertaken as part of this study revealed that an additional 120 000 m^3 of excavation is required. The channels were surveyed during the fieldwork for this study. The results are shown on Drawings Nr 12700/8 and 9.

The drainage pump station is to be equipped with five diesel-driven inclined axial flow pumps each with a nominal capacity of 900 l/s. One pump is provided as a standby, so the design maximum capacity of the pump station is 3.6 m^3/s, although the standby pump could be used to deal with short-term peaks. The pumping capacity anticipates full development of the buried field drainage system in the Estate. The pumps are to lift the drainage water from the Intercepting drain into a delivery pond from which it can flow by gravity either to the disposal area or into the reservoir at any reservoir water level. The pumps are not yet installed and it is hoped that they will be shipped to Somalia in late 1983. A temporary pump station on an adjacent site currently disposes of the drainage water by pumping into the reservoir.

A twin-barrelled reinforced concrete inverted siphon takes water from the pump station delivery pond under the supply canal and into a 4.5 km long unlined disposal channel. The inlet of the inverted siphon is fitted with hand-operated sliding gates so that the inverted siphon can be closed or operated with only one barrel open if required. Construction of the inverted siphon is complete but it has not been used pending completion of the pump station.

The pumped drainage outfall culvert allows the drainage water to be discharged from the pump station delivery pond to the reservoir. It comprises two 1.20 m diameter pipes with outlets fitted with flap gates and inlets fitted with hand-operated sliding gates. The inlet gates enable the culvert to be closed or operated with only one pipe open. The structure is complete, but has not been operated and its outlets have become buried with sediment during operation of the reservoir. These will require cleaning out and an outlet channel should be constructed downstream with a bund on its left (east) bank to prevent water from the reservoir supply channel flowing directly beside the bank to the culvert site.

The gravity drainage outfall culvert is similar to the pumped drainage outfall culvert except that the inlets have no gates. It is intended to operate as a gravity overflow in the event of the Intercepting drain becoming full following an exceptional storm or a prolonged breakdown at the pump station. The culvert will pass the design storm disposal discharge of 2.5 m^3/s with a minimum head loss of about 0.15 m. However, the water level in the drain at which the culvert begins to discharge will not be less than the inlet sill level of 97.39 (Jowhar Sugar Estate datum) and will also depend on the reservoir water level. When the reservoir level is higher than about 98.50, some small areas in the southern part of the Estate would be flooded before any water could pass through the culvert.

The disposal area (Figure I.3.1) comprises a series of depressions associated with the course of an old river channel which runs between the eastern boundary of the reservoir and the sand dunes. The topography in this area is very complex, and the depressions are not fully interconnected at the lower elevations, so that they will tend to fill in turn by overspill from one depression to the next. This means that maximum pond levels in the area will be successively lower from north to south, which has the advantage of avoiding the need for a high enclosing embankment at the southern end of the area. The actual required surface area cannot be accurately predicted at this stage as the disposal area is close to the sand dunes. However, it is likely that the seepage rate will be greater than the value of 5 mm/d which was assumed in the original design of the disposal area (MMP, 1973), and the drainable surplus from the Estate has also been reduced (see Section I.12.8) so that it is not anticipated that the intermediate embankments mentioned in the original design will be required.

The disposal channel has been designed to operate with a water level in the disposal area of up to 100.5 m.

For further details reference can be made to the following Jowhar Offstream Storage Project drawings:

SOM 70 - Inverted Siphon - General Arrangement
SOM 74 - Drainage Outfall Culverts - General Arrangement
SOM 79 - Drainage Outfall Culverts - General Details
SOM 80 - Drainage Pump Station - General Arrangement

Figure I.5.4

Jowhar Sugar Estate
Moderate Investment Strategy
Cane Areas

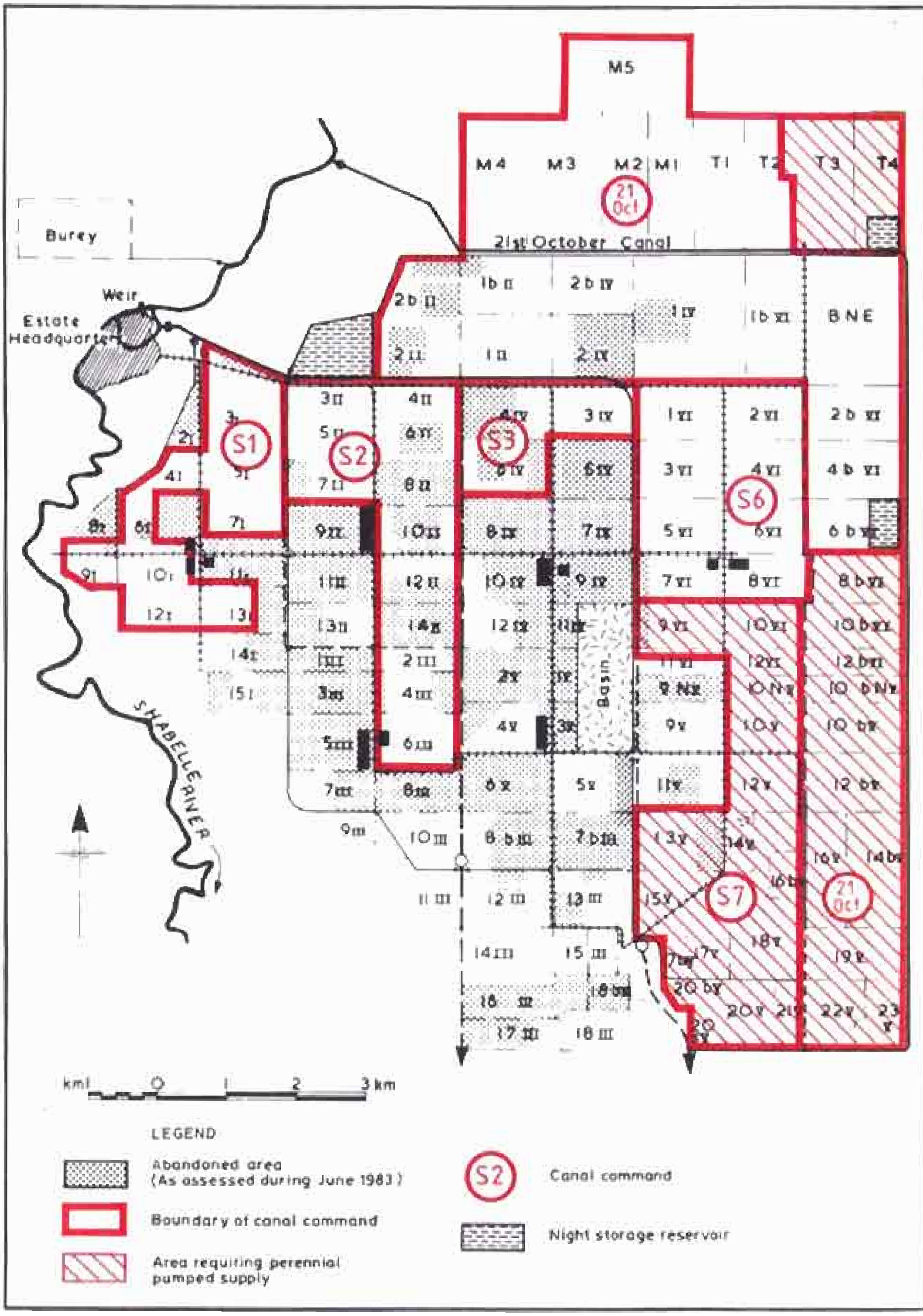
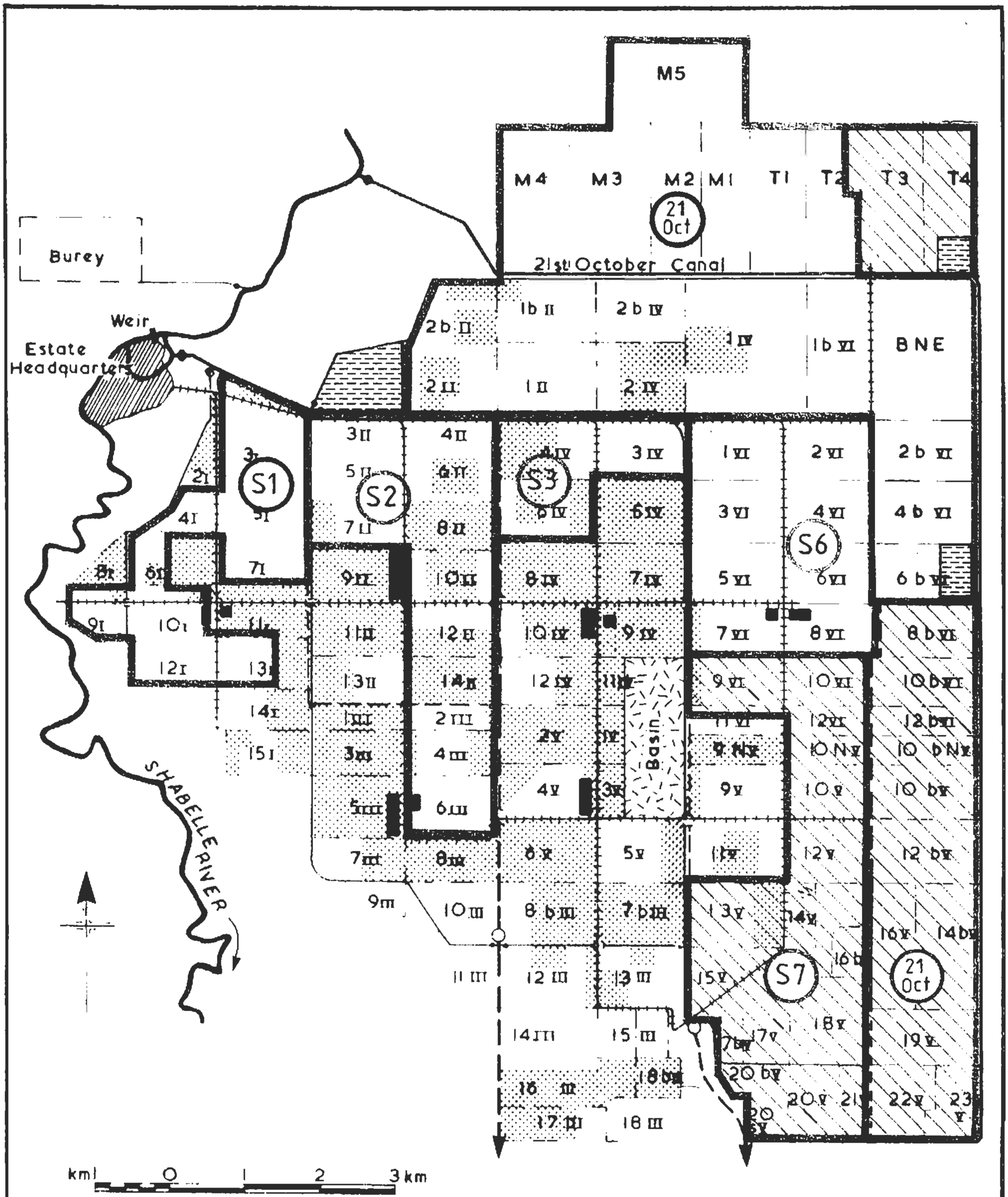


Figure I.5.4

Jowhar Sugar Estate
Moderate Investment Strategy
Cane Areas



LEGEND

Abandoned area (As assessed during June 1983)

Boundary of canal command

Area requiring perennial pumped supply

S2 Canal command

Night storage reservoir

CHAPTER 9

DRAINAGE INVESTIGATIONS

9.1 Trial Fields

The Drainage and Reclamation Study in 1976 (MMP, 1976) recommended that drainage trials be established to enable the most efficient designs for drainage and reclamation requirements to be determined. This was carried out in 1977 and 1978 (MMP, 1977; MMP, 1978) and recommendations were made in the 1978 report.

During this present study, the drainage trial fields 12 II and 8 III have been inspected but, since their collector drains have not been pumped for several years, only limited useful observations could be made. During the fieldwork a pump was reinstalled on field 12 II and the watertable lowered, enabling pits to be dug and three field drains to be inspected. The findings are summarised in Table I.9.1 and photographs of the exposed drains are shown on the following page.

The inspection indicated that, as expected, the pipe with the small sized gravel surround was in the best condition with little siltation and with the envelope still operating as a drain flow improver. The pipe with only backfilled soil surround had more siltation but more significantly the backfill had compacted with time and the entry flow into the pipe slots appeared restricted. It is therefore recommended that a small sized gravel surround is provided in any future field drain installation.

TABLE I.9.1

Inspection of Subsurface Field Drains on Field 12 II

Drain Nr	Type of filter surround	Depth of siltation (% of pipe diameter)	Evidence of iron ochre[1]	Remarks
4	No surround other than loosely back-filled soil	6-8 mm (8%)	Precipitation on top of silt in pipe	Soil surround appears to be blocking slots
8	Small sized graded gravel with a mean size of 3.5 m	4-5 mm	Slight colouration of pipe only	Good preservation of surround, acting as both filter and improver envelope
16	Coarse gravel, 5-30 mm in size	9-10 mm (12%)	Little evidence	Appears to be letting silt through the surround

Note : (1) Observations on iron ochre precipitations are of limited value as the field has not been pumped, hence the pipes have been kept under water where no oxidation of iron will occur.

9.2 Soil Characteristics

The soils of the Jowhar Sugar Estate have previously been divided according to parent material into Levee soils (L) and Floodplain soils (F). In terms of drainability the Levee soils can be considered as being of coarser texture than the Floodplain soils and are as a consequence generally well drained soils. Their significance to the sugar estate is, however, limited because they form a small percentage (10%) of the area and are generally under citrus cropping.

The Floodplain soils were previously subdivided into minor soil units according to differences in soil profile features. This subdivision was tentatively made in the 1976 Study Report on the basis of profile characteristics then available. The main characteristics used in the subdivision were that of the presence of a reddish brown horizon in the case of the F2 soil and that the F3 soil showed grey brown mottling from a shallow depth. These features distinguish the F2 and F3 soils from the basic Floodplain soil, F1, which was defined as a deep, imperfectly to moderately well drained dark brown calcareous clay soil (Chromic Vertisol).

The distribution of these subdivisions across the Estate is not easily identified due to the heterogeneous nature of the alluvial soils, and for the purposes of drainability and suitability of soils for crop use it is more appropriate to use the broader classification of Levee and Floodplain soils.

9.3 Permeabilities

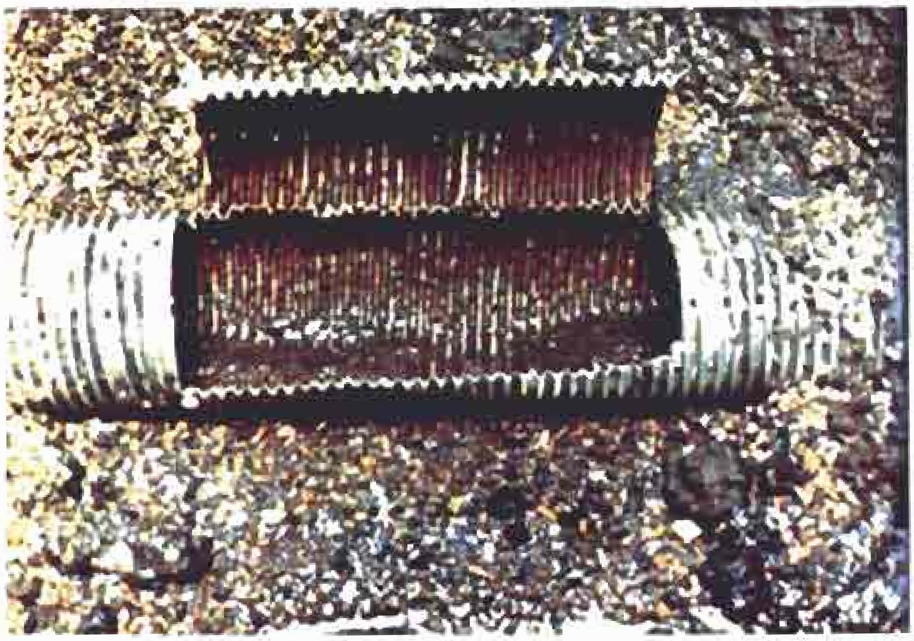
In the 1976 study and the subsequent drainage trials several measurements of hydraulic conductivity were made. One of the more significant conclusions of the 1977/78 drainage trials was that the single auger hole tests gave a good representation of hydraulic conductivity. During the present study a further 25 single auger hole tests have been carried out (see Section 10.2). A map showing these results (Figure I.9.1) shows that although there are areas of slower permeability on the Estate, there is no general pattern of good, moderate or poorly permeable soils; rather the presence of poorly permeable soil appears to be associated to that of water management of the particular field involved.

Consideration of the results leads to the conclusion that there is a correlation between permeability and soil topography (poorly permeable soils are generally on the lower lands where excess irrigation or drainage water congregates), also that fields which have been maltreated have lower permeabilities (e.g. the soils close to the central storage basin or those adjacent to inadequate drains). There is also the basic textural difference in that soils that are laid down with finer textures will have lower permeabilities than those of coarser textures. The tests also indicate that permeability decreases with depth.

Generally, however, the soils on the Estate are seen to be of reasonable permeability when they are satisfactorily managed and the major contributing factor affecting their present drainage characteristics is one of irrigation and drainage history. The areas of presently abandoned lands are concentrated around a central basin area where natural drainage is poor and where there is a long history of over-irrigation with inadequate drainage. There is little evidence to show, however, that these soils are fundamentally different from those elsewhere in the Estate presently under good cane. Furthermore, the drainage trials carried out on field 12 II in the middle of the abandoned lands have shown the soils to be readily reclaimable.

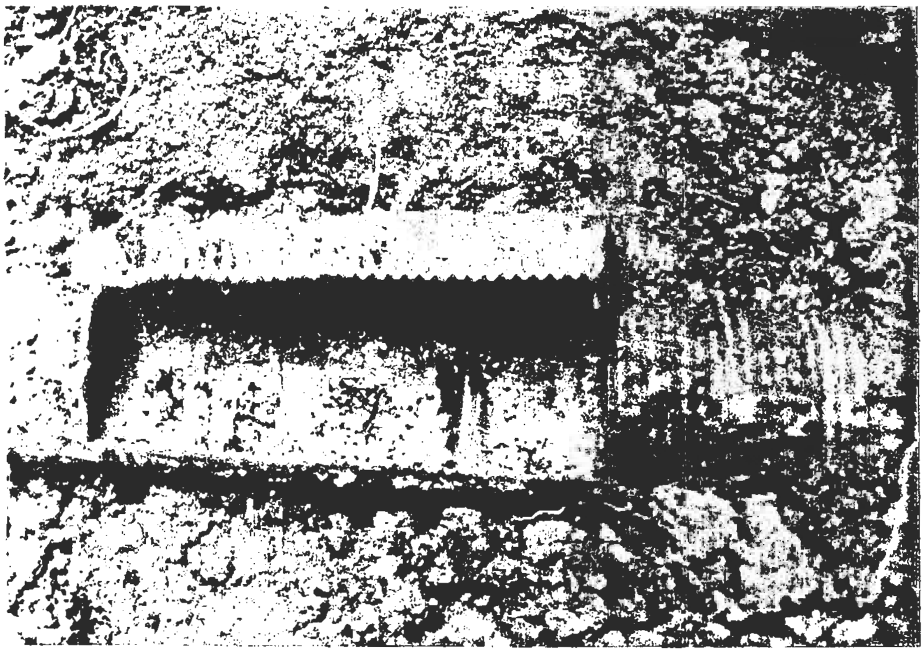
Examination of Field Drains

No surround other than loosely backfilled soil.
Field drain Nr 4,
Trial pit in field 12 II.
June 1983

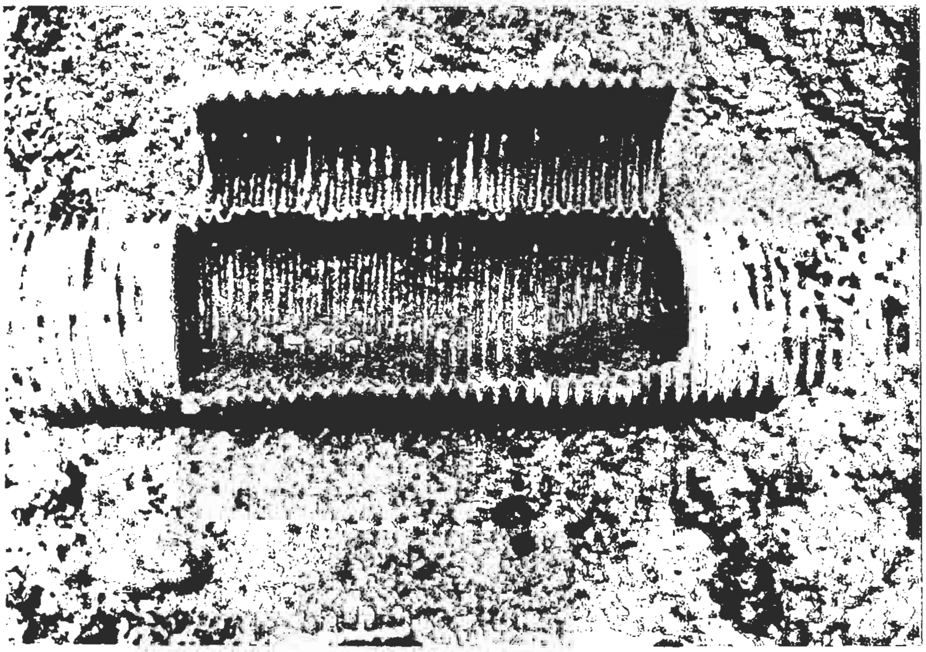


Small sized graded gravel with a mean size of 3.5mm.
Field drain Nr 8
Trial pit in field 12 II.
June 1983

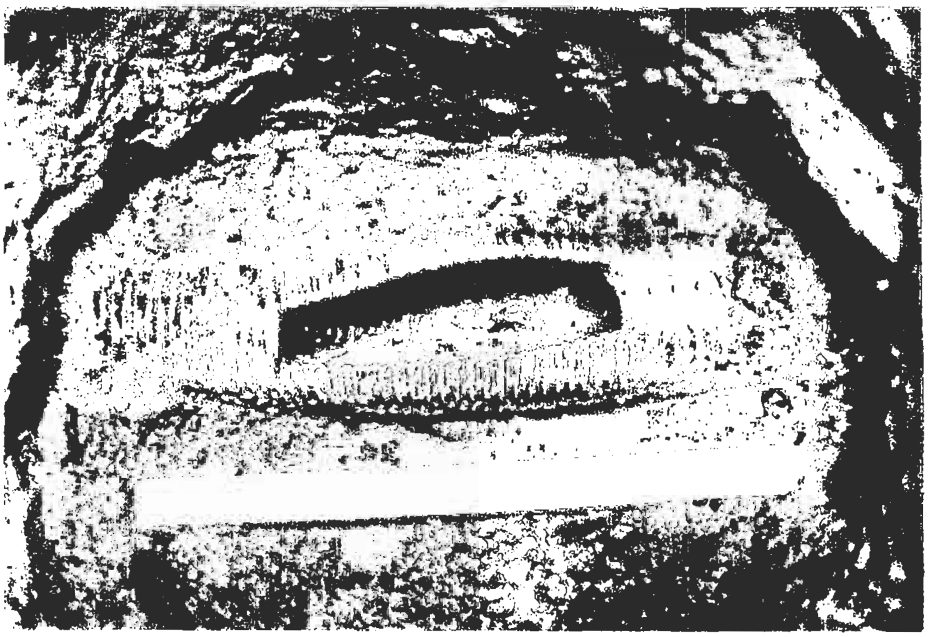
Coarse gravel,
5-30mm in size.
Field drain Nr 16
Trial pit in field 12 II.
June 1983



No surround other than loosely backfilled soil.
Field drain Nr 4,
Trial pit in field 12 II.
June 1983



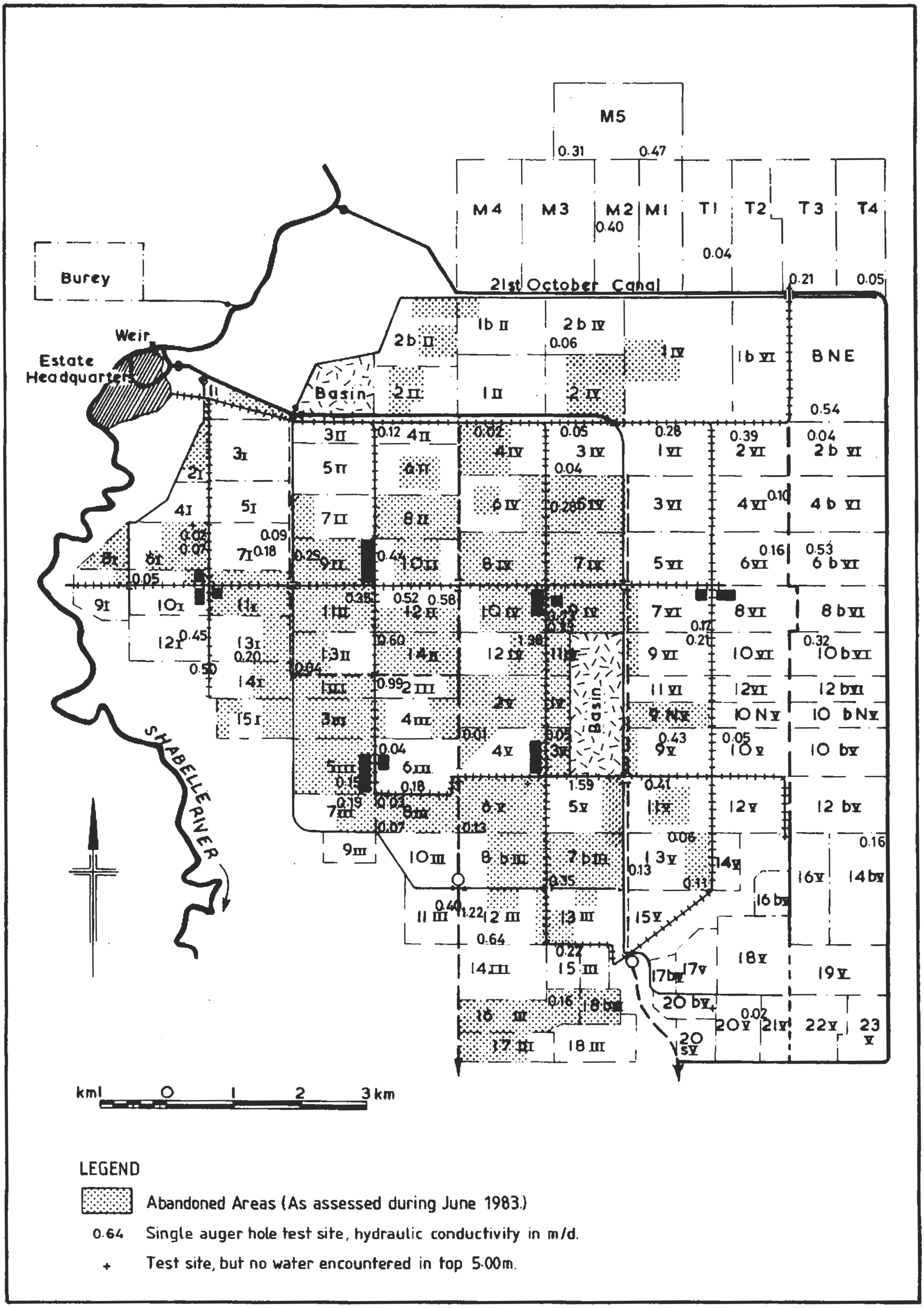
Small sized graded gravel with a mean size of 3.5mm.
Field drain Nr 8
Trial pit in field 12 II.
June 1983



Coarse gravel,
5-30mm in size.
Field drain Nr 16
Trial pit in field 12 II.
June 1983

Figure I.9.1

Jowhar Sugar Estate
Hydraulic Conductivity



9.4 Cultivable Land

Three classes of land suitability can be identified: (i) presently cultivable, (ii) marginally cultivable that can be easily rehabilitated, and (iii) fully abandoned land that will take some time and expense to be reclaimed before it can be brought back into good sugar production.

Figure I.9.2 shows this classification, which is justified as follows:

(i) Presently cultivable - Producing reasonable to good cane yields at present (even with an incomplete irrigation and drainage system) and thus can be expected to be improved with better water management control and basic modifications to the irrigation and drainage system.

(ii) Marginally cultivable - 'Abandoned' land but with reasonable permeability, and soil salinity status and either good natural drainage (near to the edge of the Estate) or easily connectable to an improved open drainage system.

(iii) Fully abandoned land - Presently out of cultivation and with a combination of poor drainage characteristics (hydraulic conductivities less than 0.1 m/day), high soil salinities (average EC_e's in the top 1 m of over 3.5 mmhos/cm) and with restraints on either connection to the main drainage system or difficulties in irrigation supply.

9.5 Hydropedological Measurements

Figures I.9.3, I.9.4 and I.9.5 show the measurements made in 1975, 1978 and 1983 on soil salinity and water levels. The figures are explained below:

9.5.1 Soil Salinity

In 1975 a soil survey was carried out on the Estate and comprehensive analyses made, as described in the 1976 report. Figure I.9.3 uses the results on EC_e given in Table III.1 of that report but averaged, taking the mean of the readings for 0 to 25, 25 to 50 and 75 to 100 cm depths. This average is assumed to give an indication of the soil salinity as it effects potential cane yields (the weighting towards the shallower depths being appropriate to that of root distribution). Check soil samples have been made on samples taken in 1983 and due to the unreliability of the Estate agricultural laboratory they were also taken for further analysis in England. Both sets of results are given in Table I.9.2.

The Jowhar laboratory equipment was found to be generally accurate but unfortunately rogue readings occasionally occurred. This is apparently due to a faulty connection in the electrical conductivity meter. Consequently Jowhar laboratory salinity results cannot be relied upon, until such time as their equipment is checked and calibrated.

Consolidation of the results, see Table I.9.2, shows that on the limited sample taken, average soil salinities have marginally decreased. This is probably due to the higher rainfall in recent years (1976 measurements followed 3 years of low rainfall and higher irrigation salinity levels). These results also indicate that recent declines in cane yield have not been caused by an increasing salinity problem.

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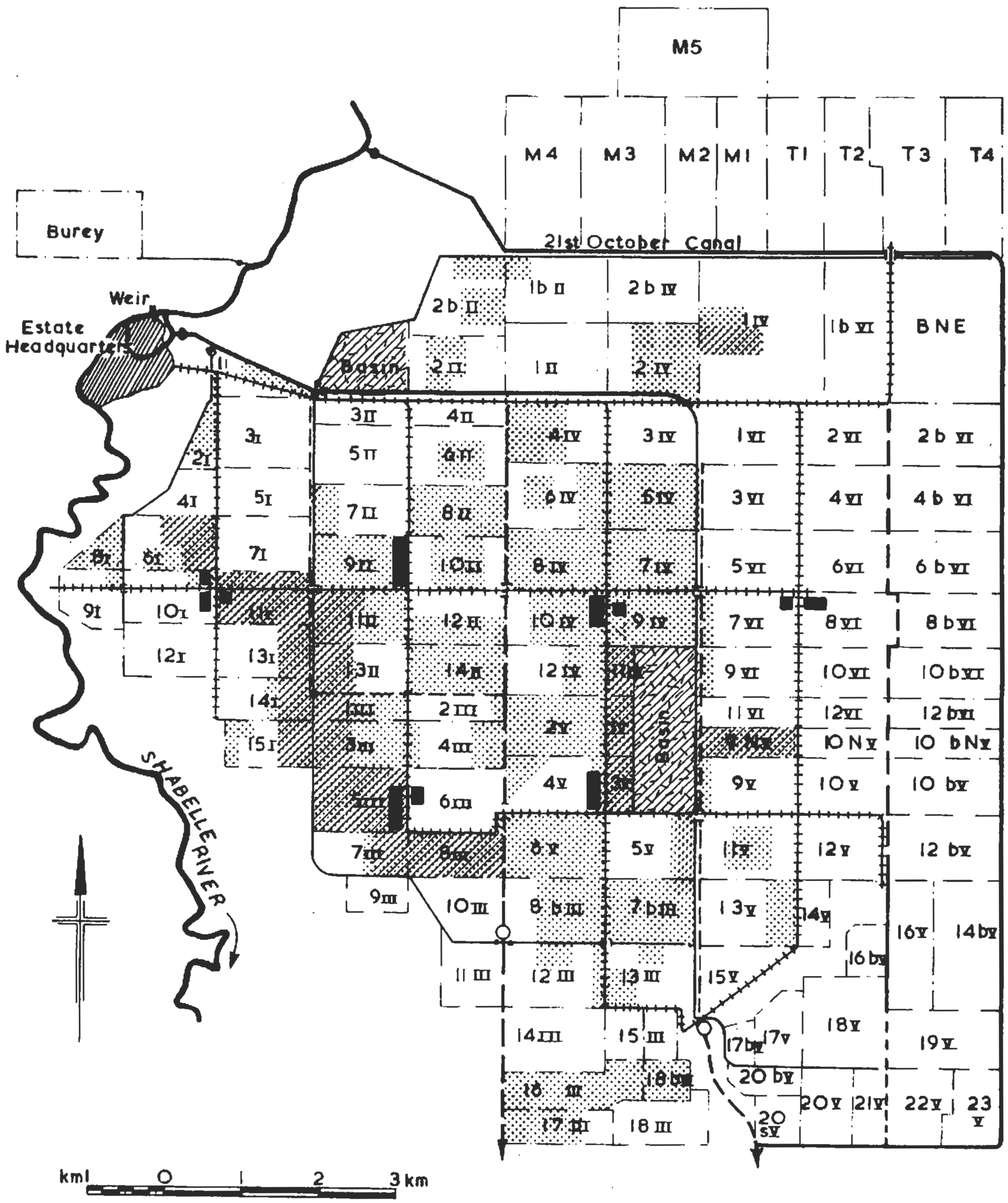
Soil Sample Anaysis[1]

Sample Nr	Depth (cm)	Field	Soil description	UK laboratory[2] EC_e (mmhos/cm)	UK laboratory[2] $EC_{1:2}$ (mmhos/cm)	Jowhar laboratory $EC_{1:2}$ (mmhos/cm)	1976[3] Site reference	1976[3] EC_e (mmhos/cm)
DAB 1	0 - 25	6 IV	(Dark brown	2.32	1.16	1.1	K180	7.4
DAB 1	25 - 50	6 IV	(clay	Sample lost	-	-	K180	7.7
DAB 1	75 - 100	6 IV	Red horizon	5.74	2.88	1.0	K180	4.4
DAB 2	0 - 25	9 VN	(Very dry	1.18	0.62	0.5	K145	6.5
DAB 2	25 - 50	9 VN	(grey brown	2.11	1.13	0.9	K145	6.3
DAB 2	75 - 100	9 VN	(clay	Not sampled; difficult to lift			K145	5.6
DAB 3	0 - 25	12 V	(Dark	0.96	0.57	0.6	K142	1.6
DAB 3	25 - 50	12 V	(brown	0.64	0.42	0.5	K142	1.3
DAB 3	75 - 100	12 V	(clay	0.67	0.47	0.5	K142	1.3
DAB 4	0 - 25	8 III	(Blackish	1.21	0.63	0.8	K150	3.0
DAB 4	25 - 50	8 III	(brown	3.29	2.91	2.2	K150	4.0
DAB 4	75 - 100	8 III	(clay	4.74	3.4	2.4	K150	3.3
DAB 5	0 - 25	12 II fascia 5	(Dark	1.47	0.57	0.7	K155	1.1
DAB 5	25 - 50	12 II	(brown clay	1.12	0.46	0.5	K155	1.2
DAB 5	50 - 75	12 II	(Reddish	1.52	0.6	0.7	K155	-
DAB 5	75 - 100	12 II	(horizon	1.84	0.67	0.8	K155	2.9
DAB 6	0 - 25	6 IN	(Silty clay	1.3	0.41	0.6	G10	2.4
DAB 6	25 - 50	6 IN	(Clay	3.12	1.24	0.8	G10	5.0
DAB 6	75 - 100	6 IN	(	7.17	3.48	0.9	G10	10.0

Notes :
1. Samples taken on 5th July 1983.
2. Tests carried out in July 1983 at Resource International Laboratories Ltd., London.
3. From Table III.1, MMP 1976.

Figure I.9.2
Jowhar Sugar Estate Land Suitability



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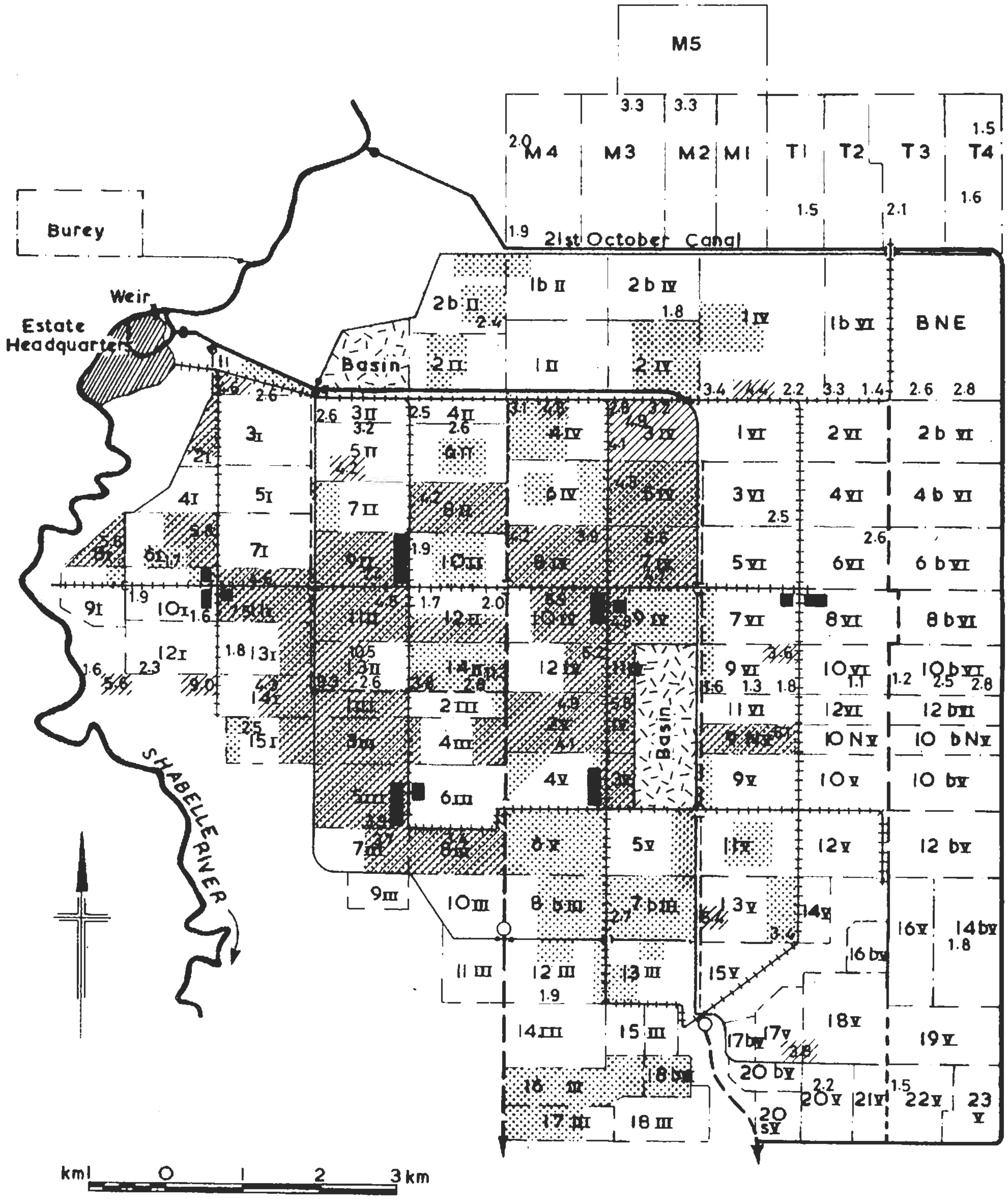
Marginally cultivable

Fully abandoned

Presently cultivable

Figure I.9.3

Jowhar Sugar Estate
Soil Salinity



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Abandoned Areas (As assessed during June 1983)

Areas of Salinity over 3.5 mmhos/cm, saturation extract (ECe)

1.9 ECe – Average of 0-25, 25-50 and 75-100 cm depths (mmhos/cm)

9.5.2 Water Levels

Groundwater levels have been observed in open wells on the Estate at various times since 1973. Water depths in these wells were recorded during the fieldwork, and further information has been collected from other related studies on groundwater in the area. Table I.9.3 and Figures I.9.4 and I.9.5 show the measured water levels across the Estate.

The difficulty with analysis of water levels in an irrigated area is that it is a dynamic situation with the watertable observation being dependent on how long it is made after irrigation. It is apparent, however, that average water levels in 1983 compared with 1975 and 1978 are higher by some 1.0 to 1.5 m near the East drain but lower near the West drain. This is presumed to reflect the recent irrigation history, considerable irrigation water having been applied via the 21st October canal (at the east and north of the Estate) but little irrigation applied near the West drain. A borehole drilled in 1983 on the south-east corner of the Estate near to Gerwane village see (Figure I.9.6) shows that the true groundwater level is some 34 m below ground level. This is consistent with previously recorded water levels in boreholes in the area (HTS, 1969), indicating that the shallow watertables on the Estate are caused by slowly permeable clays in the upper 10 m horizons. In the 1978 trials analysis the depth to impermeable layer was taken as 4 m; this assumption is presumed to be still valid.

9.6 Mole Drainage and Subsoiling

Alternative drainage improvement methods such as the use of mole drains (as proposed by Davy Agro) have been considered. The mole drains installed by the Estate in 1981 on a 1.5 ha trial plot were inspected and found to have collapsed at several places. The hydraulic stability of the soil was also tested both in Somalia and UK. It was found that the percentage of water-stable aggregates was low, with 9.5% of the fine soil particles (less than 0.5 mm diameter) remaining as water-stable aggregates. The method used for this test is described in Appendix D. A more qualitative test was also carried out as outlined in p.10 of Drainage Principles and Applications, Volume IV (IILRI 1980) whereby a sample of soil from 70 cm depth was rolled into a 20 cm diameter ball and placed in a container of water. It was found that the ball disintegrated after 12 hours, whereas a soil suitable for moleing should remain unaltered for several days.

The above tests indicate that effective working life of mole drains would be short (1 to 3 years). Mole drainage is not therefore recommended; however, the practice of subsoiling or chisel ploughing presently being carried out on the Estate does help to improve the soil drainage characteristics and this practice should continue.

9.7 Existing Water Balance

9.7.1 Steady-state Water Balance Equation

The general water balance equation for a field with a constant watertable level can be expressed as follows (from FAO 1980):

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Water Levels in Wells

Well Nr	1	2	3	4	5	6	7	8	9A	9B	10	11	12	A	B	C
Datum level (m) (1977 levelling)	101.10	102.47	101.38	101.23	101.16	100.86	100.55	100.69	100.16	99.78	101.25	(depth to water below datum)				
Ground level	100.82	101.48	101.26	100.07	101.43	100.04	100.74	100.33	99.70	100.41	100.35					
Date																
17/1/78	99.29	99.88	97.18	98.75	99.76	99.13	100.35	-	98.58	98.88	99.58	1.44	-	3.05	-	1.93
8/2/78	99.30	100.21	97.12	98.38	99.47	99.05	99.5	-	98.51	98.78	99.37	1.36	-	2.58	1.20	2.27
6/3/78	99.53	99.90	96.78	98.83	99.16	99.04	99.5	99.46	98.34	98.62	99.57	1.35	-	-	1.10	2.85
23/3/78	99.25	100.05	96.73	98.32	99.11	Flooded	100.30	99.19	98.46	98.78	98.78	1.21	-	2.45	1.20	3.00
6/4/78	99.1	99.95	96.63	98.13	99.06	-	99.67	99.69	98.84	98.83	98.95	1.33	-	2.30	1.05	3.00
3/5/78	99.07	99.91	96.46	98.10	-	-	-	-	98.88	99.10	98.75	1.37	2.33	0.90	1.79	2.34
18/5/78	99.65	-	96.38	97.81	99.51	-	-	-	99.16	99.36	99.32	-	1.90	0.85	2.10	2.30
1/6/78	99.38	99.87	96.19	99.04	99.54	-	-	-	98.92	99.08	99.08	1.19	2.11	-	1.12	2.10
15/6/78	99.1	99.97	96.03	98.48	99.46	-	99.35	99.39	98.66	98.83	98.75	1.45	2.33	0.90	-	2.25
29/6/78	99.0	100.38	96.36	97.43	100.08	-	99.35	99.61	99.08	98.73	99.15	1.80	2.45	1.15	-	3.00
18/7/78	99.4	99.71	96.03	99.03	99.36	-	-	98.48	98.11	98.23	99.09	1.80	2.50	1.20	-	3.50
2/8/78	99.16	100.10	96.00	98.33	99.33	-	99.94	99.87	98.65	98.85	98.91	0.89	2.45	1.20	-	dry
16/8/78	99.4	101.07	95.88	99.25	-	Flooded	99.57	99.69	98.51	98.71	98.91	0.85	2.56	1.35	-	-
28/8/78	99.32	100.17	96.33	98.67	99.25	Flooded	Flooded	100.01	98.35	98.47	98.90	1.15	2.51	-	1.35	dry
14/9/78	99.85	100.31	96.28	98.46	99.55	98.83	99.73	99.89	77.91	98.47	99.01	1.01	2.55	-	1.25	2.46
29/9/78	99.30	100.43	96.24	98.03	99.36	98.49	-	-	98.56	98.76	99.14	0.84	2.60	-	1.02	2.56
28/12/78	99.10	100.44	(99.31)	97.28	99.76	98.85	99.53	98.99	98.56	98.58	99.55	1.00	2.01	-	1.30	-
18/1/79	99.20	99.99	97.54	98.49	99.81	98.81	99.45	99.14	98.61	98.73	99.45	0.80	2.40	-	1.35	-
7/2/79	99.40	99.78	97.43	98.16	99.54	98.91	99.30	98.89	98.51	98.69	99.39	1.37	2.29	-	1.36	-
20/2/79	99.31	99.92	(96.73)	99.08	-	98.96	-	99.19	98.37	98.60	99.58	1.35	2.25	-	-	-
16/6/83	99.15	100.05	96.93	99.48	99.367	98.96	97.80	-	97.06	96.88	98.90	1.45	4.00	-	-	silted up
8/7/83	98.63	99.50	97.41	99.20	-	98.61	97.58	-	97.94	97.41	98.35	2.06	4.17			
July 1983(2)	2.19	1.98	3.85	0.87	-	1.43	3.16	-	1.76	3.0	2.00					
July 1975(2)	1.34	1.08	5.08	2.04	-	2.39	0.63	0.58	0.73	1.83	-					

Notes:
1. Table I.1 of MMP 1978 report has been extended; further readings are to be found in the 1976 and 1977 reports, as is the location of the well numbers.
2. Depth to water below ground level.
3. The location of wells is shown on Figure I.8.3.

Figure I.9.4

Jowhar Sugar Estate Water Levels, 1975

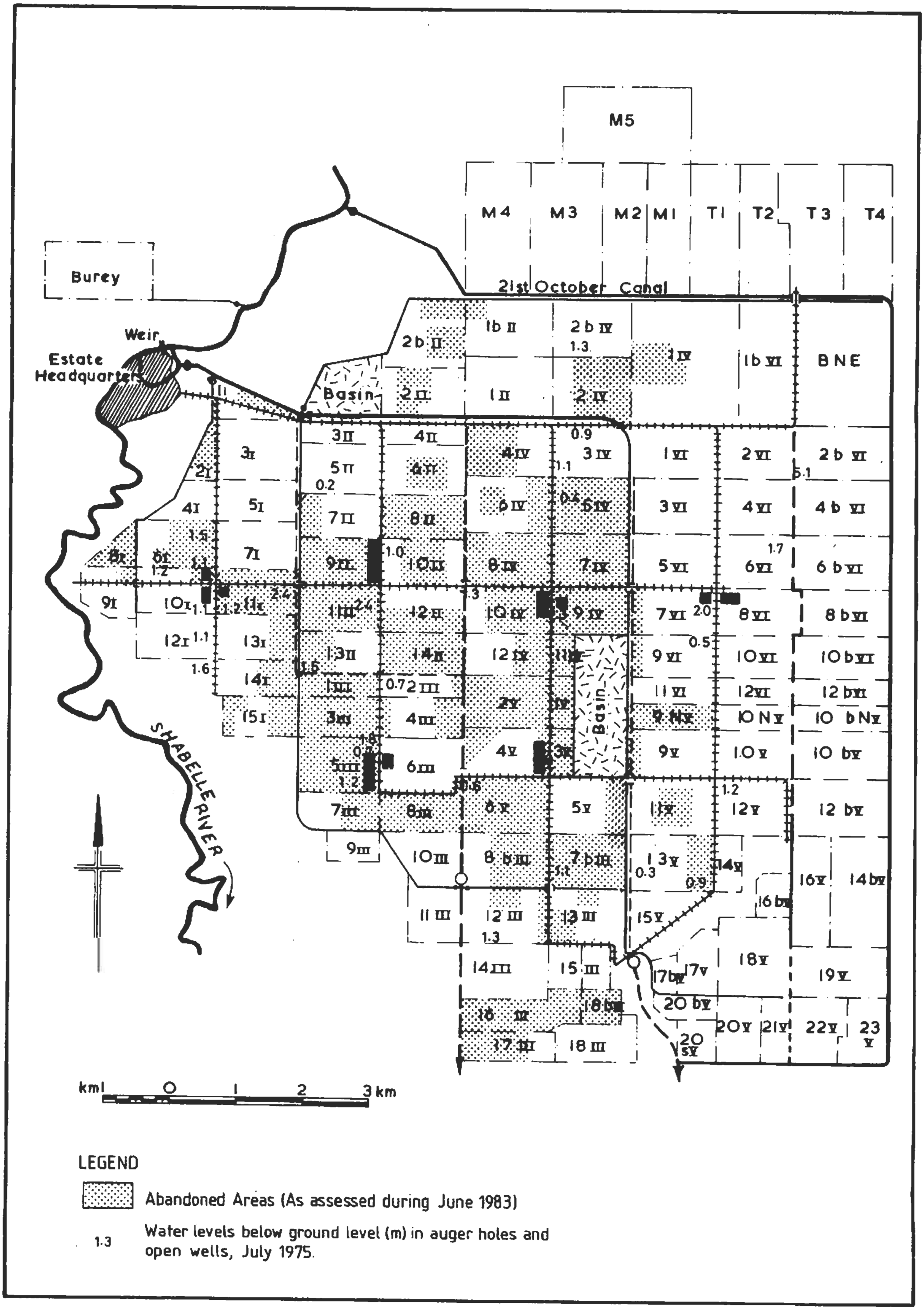
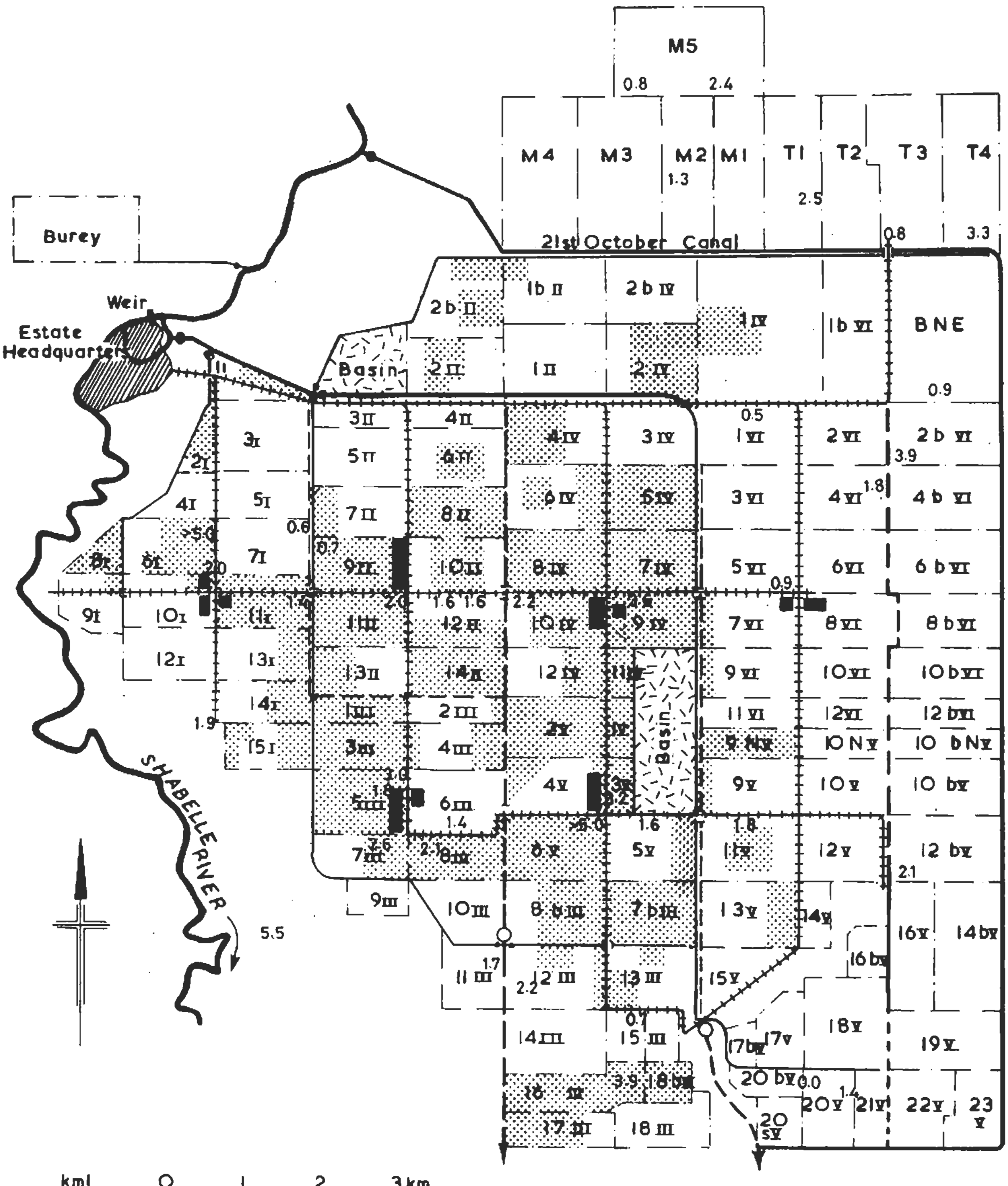


Figure I.9.5

Jowhar Sugar Estate Water Levels, 1983



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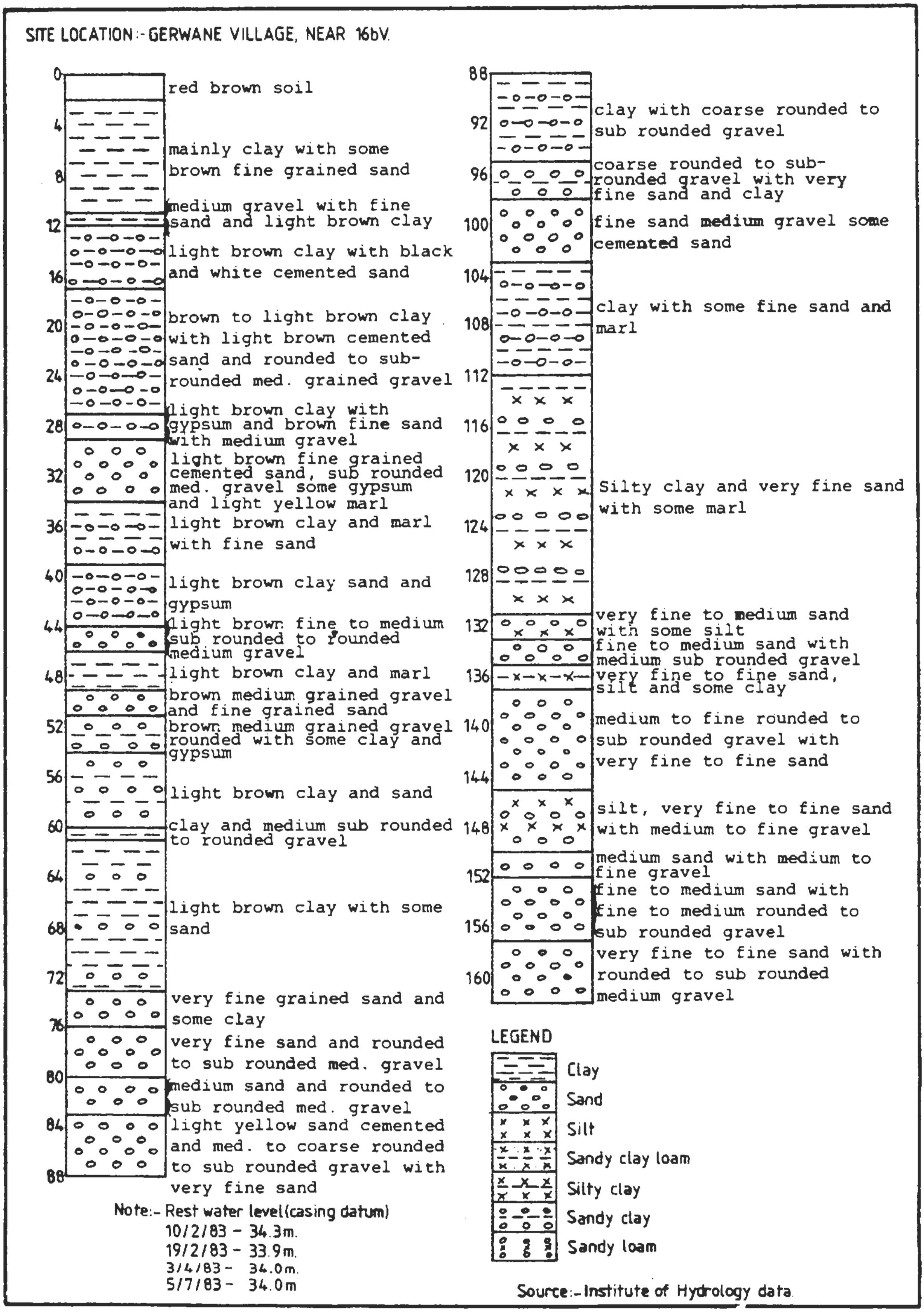
Abandoned Areas (As assessed during June 1983)

5.5 Water level below ground level (m) in auger holes and open wells, June/July 1983.

Figure 1.9.6
Jowhar Sugar Estate
Borehole Lithology



$$Q_s = R_{dp} + S_c - C - D \qquad \text{Equation 1}$$

where, in mm/d,

Q_s = design drainage rate for field drainage;

R_{dp} = recharge to groundwater by deep percolation from irrigation or rainfall;

S_c = seepage from canals to groundwater;

C = capillary rise from groundwater;

D = natural deep drainage (groundwater flow out of the field area net of any inflow or artesian flow).

This equation is used in Section 12.3 for estimation of the design drainage rate Q_s. This requires estimates of the natural deep drainage D, which are best derived by analysing the existing situation.

Equation 1 also applies to the existing situation, with $Q_s = 0$, provided the watertable level can be regarded as constant.

The data on average watertable levels (Table I.9.4) show variations between different parts of the Estate, particularly between the West drain and the East drain (northern sector). Between 1975 and 1983 the watertable in fields served by the West drain fell by approximately 1 m, from about 1.5 m to 2.5 m below ground level in abandoned fields, and from 1.0 m to 1.8 m in cultivated fields; while in fields drained by the East drain (northern section) the watertable rose about 0.5 m to a present level of approximately 2 m below ground level. However in terms of a daily rate of gain or loss of drainage water these changes in watertable level represent very low drainage rates indeed - the corresponding average rates of loss of water in the west drain area and of gain of water in the East drain are both considerably less than 0.1 mm/d. This compares to average rates of 1.6 mm/d for deep percolation (Rd_p) and 0.7 mm/d for canal seepage (S_c) which are derived in Section 12.3 for fields with optimum irrigation.

Since there is negligible daily gain or loss of drainage water the following water balance equation may be applied:

$$R_{dp} + S_c - C - D = 0 \qquad \text{Equation 2}$$

This equation has been applied separately to the fields on the West drain and to the fields on the Middle and East drains, to provide estimates of the natural deep drainage rate D. Unfortunately the necessary data on irrigation volumes are not available, because of the lack of measurement facilities and records of opening hours and discharges at the Luigi di Savoia and 21st October intakes. The following analysis is therefore only indicative.

TABLE I.9.4

Changes in Watertable Levels 1975 to 1983

	Drain catchment	Mean watertable depths below ground level (m) July 1975	July 1983	Approximate change in water level (m)
1.	West and west branch drains (abandoned areas)	1.5	2.5	1.0 fall
2.	West and west branch drains (cultivated areas)	1.0	1.8	0.8 fall
3.	Middle drain	0.9	1.6	? slight fall
4.	New lands (M5-T4)	-	1.9	?
5.	East drain (northern section)	2.7	2.2	0.5 rise
6.	East drain (southern section)	-	0.7	?

Note : These are approximate figures based on few measurements. In some cases 1975 and 1983 data are for different sites.

Source : Figures I.9.4 and I.9.5.

9.7.2 West Drain

The Estate's irrigation records for 1983 for Farm II (old system) on the west drain have been analysed as set out in Table I.9.5 and indicate an average of only 2.2 irrigations for each fascia between January and June. An estimated average rate of 0.7 mm/d deep percolation to the watertable from irrigation and rainfall has been derived from these data. Thus R_{dp} = 0.7 mm/day. Existing seepage rates from main, secondary, tertiary and quaternary canals are estimated at some 0.2 l/s/ha (1.7 mm/d) with full water supplies, based on 1975 measurements (MMP 1976). In recent years irrigation supplies to the cultivated fields on the West drain have been reduced by siltation, so a reduced average canal seepage rate (S_c) of 0.7 mm/d has been assumed.

Estimated rates of capillary rise from the watertable in a silty clay soil are shown in Table I.9.6 for various soil moisture suctions and watertable depths. This shows that cultivated fields served by the West drain with a watertable about 1.5 m below ground level in recent years would have a capillary rise C of approximately 0.6 mm/d. This may have helped to support cane growth during the long interval between irrigations but it will also have contributed to a rise in soil salinity in the surface horizon.

The existing water balance for these fields is thus

$$R_{dp} + S_c - C - D = 0$$

$$0.7 + 0.7 - 0.6 - D = 0$$

$$D = 0.8 \text{ mm/d}$$

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Irrigation Records, Farm II

Field (fascias)	Plant/ ratoon	Last harvest	Nr	Irrigations January to June 1983: 1	2	3	4
1II (1-4)	Ratoon	Feb 1982	1	May	-	-	-
1II (9-12)	Planted gu 1982	-	2	Jan/Feb	May	-	-
1bEII (1,6-8)	Ratoon	Aug 1982	3	Jan/Feb	Apr/May	May/Jun	-
(2-3)	Ratoon	Aug 1982	2	Feb	May	-	-
(4)	Ratoon	Aug 1982	1	Feb	-	-	-
1bWII (1)	Ratoon	Feb 1982	3	Jan	Feb	Apr	-
(2-5)	Ratoon	Feb 1982	2	Jan/Feb	Mar/Apr	-	-
(6-8)	Ratoon	Feb 1982	1	Jan/Feb/ Apr	-	-	-
2II (7-11)	Ratoon	Feb 1982	1	Apr/May	-	-	-
2bEII (1-6,9-10)	Ratoon	Feb 1982	1	May	-	-	-
2bWII (1-8,10)	Ratoon	Feb 1982	1	Apr/May	-	-	-
3II (1-3)	Planted gu 1982	-	4	Jan	Feb	Apr	Jun
4II (1-9)	Planted gu 1982	-	4	Jan	Mar	Apr/May	Jun
(10)	Planted gu 1982	-	3	Jan	Feb	Apr	
(11)	Planted gu 1982	-	1	Feb	-	-	-
5II (1-2,4-6, 8-11)	Planted der 1982	-	3	Jan/Feb	Feb/Mar	Apr/May Jun	-
(3,7)	Planted der 1982	-	4	Jan/Feb	Feb/Mar	Mar/Apr	Apr/May/ Jun
6II (1-11)	Ratoon	Feb 1982	2	Jan	May	-	-
7II (1-10)	Ratoon	Sep 1982	3	Feb/Mar Apr	Apr/May	Jun	-
10II (1-3,8-10)	Ratoon	Feb 1982	1	Apr/May	-	-	-
(11)	Ratoon	Feb 1982	2	Jan	Apr/May		
	Weighted mean irrigations per fascia		2.2				

TABLE I.9.6

Rates of Capillary Rise to the Ground Surface in a Silty Clay Soil

Approximate rates of capillary rise (mm/d)

Soil moisture suction (bar)	Watertable depth below ground surface (m) 0.5	1.0	1.5	2.0	2.5
0.1	1.5	0.2	-	-	-
0.5	3.0	1.0	0.5	0.3	0.2
1.0	3.6	1.2	0.6	0.4	0.2
5.0	4.0	1.5	0.8	0.6	0.4
16.0	4.0	1.5	0.8	0.6	0.5

Source : Ritjema (1969)

Therefore the rate of natural deep drainage from the fields served by the West drain is estimated at 0.8 mm/d. This is less than the estimated future recharge to groundwater and therefore subsurface drainage is required. Nevertheless it is a significant rate which helps to reduce the net drainage requirements. See Section 12.3.

For the abandoned fields the values are reduced and the water balance becomes approximately

$$0.1 + 0.3 - 0.3 - D = 0$$

$$D = 0.1 \text{ mm/d}$$

These low values, coupled with relatively static watertables, indicate that there is very little natural deep drainage from the West drain fields either vertically downwards or horizontally to adjacent abandoned fields with lower watertables.

9.7.3 Middle and East Drains

Much of the area served by the Middle and East drains is irrigated from the 21st October canal and in 1975 the discharge of this canal was measured on five occasions at an average 80% of the estimated 12 hour/d requirements (MMP 1976, Table 9.6). It is understood that the discharge has remained adequate since, whereas siltation of the Luigi/Duchessa/S3 canal system has caused considerable difficulties in irrigating the other areas. Hence for the Middle and East drain areas it is estimated that the average rate of natural deep drainage is sufficient to balance the total of deep percolation plus seepage minus capillary rise. Under present conditions the watertable has been rising at a rate of 0.05 to 0.1 m/year to a level of about 2 m below ground level. As discussed above this represents a very small drainable surplus. However it is a significant rise in watertable. The proposed development, by improving irrigation control, eliminating night irrigation and introducing a new field layout, should reduce losses. In addition the elimination of quaternaries would reduce canal seepage losses. This would help to improve drainage conditions.

Thus a reduced proportion of the irrigation supplies would be lost to recharging the groundwater. It is estimated that this would more than compensate for the increases caused by a somewhat higher water supply to this area, and hence the total colume of water going to groundwater would be reduced. Monitoring of irrigations and watertable levels is essential - see Section 12.3.

CHAPTER 10

SOILS AND LAND USE

10.1 General

The main characteristics of the project soils have been outlined in previous studies (MMP 1976, MMP 1978). This section deals with the soil characteristics as they relate to drainage.

10.2 Permeability

The Jowhar soils have originated from river or irrigation sediments. They are thus alluvial soils and show some stratification as is typical of such soils; however, they have been under irrigation for up to 60 years and their characteristics have been modified in this time.

The drainage trials carried out in 1977/78 showed that single auger hole tests give a good estimate of soil permeability and therefore further tests have been carried out in this present study. The results were discussed in Section 9.3 and are summarised in Table I.10.1.

10.3 Waterlogging and Salinity Effects on Cane Yield

Previous studies (MMP, 1976, 1978) have shown that the principal soil factors responsible for reducing cane yields and causing abandonment of fields at Jowhar are waterlogging and salinity.

These studies concluded that soil salinity levels over the Estate are not generally excessive (Figure I.9.3) but that there has been a build-up of salinity in the soils of the abandoned lands. This increase in salinity was attributed to the upward capillary rise of moisture from the high watertables which, in the absence of any leaching effect from irrigation, has resulted in an accumulation of soluble salts in the upper soil profile.

In the following sections an examination is made, in general terms, of the nature of the adverse effects of waterlogging and salinity on plant growth and the principles of their amelioration. This is based on previous work (MMP 1978).

10.3.1 Waterlogging

(a) Nature of Adverse Effects

Soil waterlogging poses practical disadvantages for crop production such as reduced ease of access and difficulty of cultivation. However, in terms of crop growth, the most adverse effects of waterlogging derive from physiological damage to root growth caused by the development of anaerobic soil conditions. Without active root growth plants are unable to take up adequate amounts of either nutrients or water.

Under waterlogging, the rate of diffusion of oxygen from the soil surface is severely reduced and as plants cannot actively translocate oxygen from their aerial parts to their roots, anaerobiosis develops in the root zone. The onset

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Hydraulic Conductivity Values

Field Nr	Hydraulic[(1)] conductivity s = 0	Hydraulic[(1)] conductivity s 0.5 h	Class[(2)]
7 I	0.10	0.09	2
9 II	0.28	0.25	3
12 III, fascia 5	0.64	0.52	4
12 II, fascia 10	0.72	0.58	4
6 III	0.22	0.18	3
7 III	0.23	0.19	3
8 III	0.03	0.03	1
11 III	0.54	0.40	4
12 III	1.82	1.22	4
15 III	0.24	0.22	3
16 III E	0.20	0.16	3
T 1	0.05	0.04	2
T 3	0.27	0.21	3
T 4	0.06	0.05	3
M 2	0.55	0.40	4
M 5, east	0.61	0.47	4
M 5, west	0.37	0.31	3
S V	2.06	1.59	4
6 V	No water	-	-
II V	0.46	0.41	4
20 V	0.02	0.02	1
20b V	No water	-	-
BNE	0.71	0.54	4
1 VT	0.35	0.28	3
4 VT	0.11	0.10	2

Notes : (1) Hydraulic conductivity determined from single auger hole tests in the 2.0 to 4.5 m depth using formula for depth to impermeable layer of $s = 0$ or $s > 0.5$ h (h = mean head during the measuring period). In general it is thought that $s = 0$ is the more appropriate condition.

(2) Class ratings (for $s = 0$); 1 $<$ 0.03; 2 = 0.03 - 0.12; 3 = 0.12 - 0.48; 4 $>$ 0.48 m/d.

of this condition can occur rapidly, with all oxygen being removed from the soil atmosphere within 24 hours of inundation (Crawford, 1978).

Two principal effects can result from anaerobic soil conditions, each of which is deleterious to root growth:

(i) Toxicity of Soil Ions

As oxygen depletion occurs in the soil, processes of chemical reduction can develop, accelerated by the build-up of anaerobic micro-organisms, and there may be a progressive reduction of soil ions. Nitrate can be converted to nitrite ion, manganic to manganous, ferric to ferrous and sulphate to sulphide ions (Turner and Patrick, 1968). These ions are not only more soluble but are highly toxic to plant roots. Also, in the case of nitrate, this process can result in the loss of a major plant nutrient.

(ii) Metabolic Effects

With depletion of oxygen supply to the roots the metabolism of the roots themselves is affected. In flood-sensitive plants anaerobic conditions in the root cause glycolysis and the formation of ethanol in toxic quantities. This compound causes damage to cell membranes and can prevent the return to normal aerobic respiration in the root even when waterlogging ceases and oxygen supplies are reinstated. Thus, root damage can be severe even after relatively short periods of inundation (MacManmon and Crawford, 1971).

(b) Jowhar Estate Conditions

Sugar cane is relatively tolerant to waterlogging and high watertables, (FAO/UNESCO, 1973) but the combination of climatic, soil and irrigation conditions at Jowhar are such that the harmful effects of soil anaerobiosis could be expected to be magnified. These harmful effects are normally more pronounced under conditions of high temperatures, as prevail at Jowhar, and in crops during an active stage of growth, which applies to cane throughout the year.

(c) Present Practices

Present irrigation practice at the Estate involves leaving water on the surface of the fields due to lack of effective field drainage. Such temporary surface waterlogging by static water can usually be tolerated for a few days provided there is air trapped in the pores of the subsurface soil horizons. However, the Jowhar soils are fine textured with low volumes of stable aerated pore spaces, with the result that reserves of soil oxygen are low and tolerance to surface waterlogging is reduced. Where watertables are high, most of the air space in the root zone of these fine textured soils will already be filled by capillary water at the time of irrigation and there will be negligible trapped air below the surface. The cane roots therefore have to rely on oxygen dissolved in the irrigation water.

Examinations of soil profiles show that cane rooting depth and distribution are better in soils which are less frequently waterlogged, suggesting that yield reductions are partly caused by the effects of poor aeration of cane roots. In the absence of readily apparent concentrations of reduced ions in the soils, such as sulphides, the deleterious effects of anaerobiosis are probably mainly physiological.

(d) Recommendations

Irrigation and soil management techniques on the Estate should therefore aim to reduce the period of soil waterlogging, and to increase soil aeration. Although some temporary waterlogging is inevitable when clay soils are irrigated, the provision of surface drains could be expected to have the immediate beneficial effect of enabling standing water to be removed from fields within 24 hours of an irrigation or within four days of the 1 in 5 year storm, and allowing a resumption of oxygen diffusion into the upper root zone. The lowering of watertables by the provision of deep under-drainage in the area served by the West drain would, by increasing the volume of soil pores occupied by air, increase the tolerance of the soil to temporary waterlogging during irrigation.

10.3.2 Soil Salinity

Nature of Adverse Effects

High concentrations of soluble salts in the soil adversely affect plant growth. The precise mechanisms by which growth is reduced are not fully understood but it appears to result from a combination of osmotic and ionic effects of the salts in the soil solution.

(a) Osmotic Effects

Soluble salts have the effect of increasing the osmotic suction of the soil solution and thus reducing the availability of water to plants. Water uptake by plants is a passive process influenced by suction gradients in the conducting tissues of the plant. Uptake of moisture from the soil is dependent upon the suction with which water is held in the root being greater than that with which it is held in the surrounding soil. As the soil dries, due to moisture removal by the root, the suction with which the remaining soil water is held - the matric suction - increases. Other factors being equal, water uptake can proceed and active plant growth continue as long as there is a sufficient suction gradient between the soil solution and the root cell solution. However, the ability of plants to develop suctions is limited and beyond a soil matric suction of some 15 bar, the gradients are insufficient to maintain an adequate flow of water into the root and permanent wilting and eventual death of the plant ensue.

An increase of soluble salts in the soil, by increasing the osmotic suction of the soil water, reduces the suction gradient across the root membrane. Plants therefore have to compete for water uptake against not only the matric suction of the soil solution but also the osmotic suction. Consequently, for a given soil moisture content, water is less freely available to plants if the soil contains soluble salts; moisture stress and permanent wilting point occur at progressively higher soil moisture contents (lower matric suctions) as the salinity of the soil increases. The osmotic suction effect of salts (O_e) is given by:

$O_e = 0.36 \times EC$ where EC is the electrical conductivity in mmhos/cm

Although concentrations of soluble salts in the soil undoubtedly have an adverse effect on the ability of roots to absorb water, the relationship between soil salinity and water uptake is far from clear and is very complex. In particular, plants exposed to saline soil conditions are known to have the ability to increase the osmotic suction in their cells, through absorption of salts or synthesis of soluble organic compounds (Bernstein, 1975). This osmotic adjustment enables suction gradients to be re-established and, provided overall suction gradients across the plant permit, water absorption continues and turgor is maintained in spite of soil salinity.

(b) Ionic Effects

Individual soluble ions in the soil can adversely affect plant metabolism and growth. The ions most commonly cited as being toxic are sodium and chloride, although some plants are sensitive to other ions such as sulphate.

It is almost impossible to dissociate the osmotic and toxic ion effects on plant growth, although it is probable that a plant's tolerance to salinity is largely a reflection of its ability to withstand otherwise toxic levels of ions, rather than its ability to take up water against high soil suctions.

Plant species vary greatly in their ability to tolerate salinity, sugar cane being a sensitive crop (FAO/UNESCO, 1973) which requires a soil EC_e of less than 4.0 mmhos/cm for active productive growth.

10.3.3 Conclusion

A comparison of existing conditions on the Estate and the background information on the processes involved suggests that waterlogging is a more serious problem than soil salinity at present. However it may be noted that these conditions have not deteriorated since 1975, and the reasons for the recent serious yield deterioration are to be sought elsewhere.

Drainage is required both to control waterlogging and to allow adequate leaching to prevent accumulation of salts. The leaching requirements are described in the following section.

10.4 Leaching Considerations

10.4.1 Introduction

An understanding of the leaching of the soils is fundamental to successful soil and water management at Jowhar as inadequate provision for leaching will lead to excessive salt accumulation and yield reductions such as have occurred in the West drain area over a long period.

To leach the salts, adequate water must be applied to allow water to pass through the root zone, and for this to occur there must either be natural vertical movement out of the root zone or water must be taken out by drainage. As discussed in Section 9.7, natural drainage is inadequate in the area served by the West drain and subsurface drainage is to be installed, but conditions are better in the Middle and East drain areas and it is thought that natural vertical flows would be adequate in these areas.

10.4.2 Theory

The amount of leaching is referred to as the leaching fraction (LF) and is defined as the fraction of the water entering the soil that passes beyond the root zone.

The leaching fraction required to achieve an acceptable salt balance is referred to as the leaching requirement (LR).

The FAO approach to estimating leaching requirements has been followed (FAO 1976) as in previous reports (MMP 1978). This allows for short term accumulations of salts within the lower root zone. These accumulations are permissible because of the uneven nature of moisture abstraction within the root zone (provided that the crop is adequately supplied with water in the upper root zone where the major water use occurs).

The FAO approach to estimating the leaching requirements can be expressed as follows:

$$LR = \frac{ECw}{5\, EC_e - EC_w}$$

where EC_w is the electrical conductivity of the irrigation water and EC_e is usually taken as the electrical conductivity level of soil saturation extract at which a 10% yield reduction could be expected (taken as 3.4 mmhos/cm for sugar cane (Maas and Hoffman, 1977)).

The leaching requirement calculated by the above procedure is usually adjusted to allow for leaching inefficiencies. These occur by :

(i) water passing along soil cracks, root holes and other easy routes as opposed to evenly through the soil mass,

(ii) also where drains are used more leaching water is found to pass through the soil next to the drains as compared with the between-drain area.

For the soils at Jowhar case (i) is considered to be negligible because of the homogeneity of the soils, but case (ii) will occur where drains have been installed.

For the salt balance analyses, the leaching efficiency has been taken as 50% and additional inefficiencies have been allowed for in the calculations below.

10.4.3 Estimated Leaching Requirement

For any particular salinity level of irrigation water a leaching fraction exists whereby acceptable soil salinity levels will be maintained. However, in a real situation the salinity levels of irrigation water vary through a season and thus, for a given leaching fraction, soil salinity levels will vary accordingly.

Using the FAO approach to leaching requirement (FAO, 1976) and taking the EC_e level of 3.4 mmhos/cm suggested to give a 10% yield reduction (Maas and Hoffman, 1977), and the mean EC_w value of (0.05 mmho/cm) of irrigation water; gives a LR as follows:

$$LR = \frac{850}{5 \times 3\,400 - 850} = 5.3\%$$

If we then allow for leaching inefficiencies due to the clay soil (say 50%), shallow rooting depth (say 90%) and furrow irrigation method (say 80%) this modifies equation 3 to give

$$LR = \frac{1}{0.50} \times \frac{1}{0.90} \times \frac{1}{0.80} \times 5.3\%$$

$$= 15\%$$

As described in Chapter 1, the river salinity (EC_w) varies from the mean value of 0.85 mmho/cm during a year in the range of 0.4 to 2.0 mmhos/cm. Previous work (MMP 1978) shows that for a leaching fraction of 15% at Jowhar the long term soil salinity (EC_e) corresponding to an EC_w of 0.85 mmho/cm is approxiimately 2 mmhos/cm. The extreme EC_e range corresponding to the EC_w variation through the year would be EC_e values of 1 to 5 mmhos/cm. These higher EC_e values at certain times of the year correspond to the short term salt accumulations proposed as acceptable under the FAO approach.

10.4.4 Conclusions

The adoption of the proposed approach should ensure potential optimum yields where the irrigation intervals are short. However; at times of water shortage (January to April at Jowhar) irrigation may have to be withheld and longer irrigation intervals will occur. This will result in lower cane yields both because of direct water shortage and because longer irrigation intervals will increase soil salinity causing high osmotic suction and making it more difficult for the plant to extract water.

The yield estimates presented in Annex II take account of reductions caused by these effects.

10.5 Land Use

10.5.1 Current Land Use

The Estate has a total gross field area of 8 850 ha. According to Estate records this area was used as follows in 1983 :

cultivated	:	6 360 ha gross
storage basins	:	190 ha gross
Burey experimental farm	:	115 ha gross
abandoned	:	2 185 ha gross

The locations of these areas are shown in Figure I.4.2. Sugar cane is grown on virtually all the cultivated area, though there are small (and negligible) areas of grapefruit and coconuts. With the existing field layout the net area is approximately 93% of the gross area (MMP 1976) so the net cane area currently amounts to some 5 910 ha.

10.5.2 Proposed Land Use

The major objective for the Estate is to produce enough cane to satisfy the factory's requirements of 468 000 t/year.

A package of rehabilitation measures has been devised which would be expected to give an estimated average sugar cane yield of 100 t cane/ha if adequate water supplies were available. Water shortage however is a function of the area cultivated, with larger areas requiring larger irrigation volumes and therefore suffering greater stress and yield reductions when water is short.

River flow analyses have revealed the frequency and extent of water shortages for a given cane area (Chapter 2) and the resulting yield reductions have been estimated (Annex II). It has been shown that an area of 5 300 ha net would be appropriate, as the long term average yield for this area (after allowing for water shortages) is estimated at 91 t/ha, which would satisfy the factory requirement and provide the necessary planting stock.

The selection of particular fields to make up this area is described in Section 11.2. These fields are shown in Figure I.5.4 and cover a total net area of 5 360 ha net. It is assumed that small areas totalling approximately 60 ha within these fields would be excluded from cultivation because of particular problems such as soil salinity, and the remaining 5 300 ha net would be used to grow sugar cane. The non-cane fields would be used for other crops as required (see Annex III). Details of existing and proposed land used are given in Table I.10.2.

10.6 Reclamation of Abandoned Land

The 5 300 ha cane area recommended for development excludes the worse abandoned areas and the rehabilitation programme concentrates on areas presently under cane (see Section 11.2). However, there are approximately 600 ha of presently abandoned land included in the proposed cane area. Most of these are presently uncultivated due to local topographic problems, they are either out of command or suffer from regular flooding. These areas are assumed to be re-cultivable with the proposed introduction of the new field layout and the imposition of an improved irrigation regime.

Fields that are identified as having poor soil water characteristics will require full reclamation including the installation of subsurface field drains. Field 8 II has been identified for initial reclamation and is scheduled for reclamation in Year 3 of the rehabilitation programme. It is presently abandoned due to a combination of high soil salinity and waterlogging and will be reclaimed by installing the new field layout with subsurface field drains at 50 m spacing and with two deep collector drains. Two or three leaching irrigations should be carried out before sugar cane is planted but it is recommended that the cane is planted one or two months after remodelling of the field. Reclamation of the field will continue as the cane is growing, as once the subsurface drainage is installed the soil water regime of the field will improve under future cultivation. The success of this regime was demonstrated during the drainage trials when cane was planted after only two irrigations.

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Details of Existing and Proposed Land Use Areas

	Existing land use		Proposed land use	
	Gross field area	Net area(1)	Gross field area	Net area(2)
1. Selected Cane Area				
Cultivated	5 270	4 901	5 889	5 300
Not cultivated	686	637	67	60
Total	5 956	5 538	5 956	5 360
2. Non-Cane Area				
Cultivated	1 055	981	?	?
Not cultivated	1 532	1 425	?	?
Total	2 587	2 406	2 587	2 328
3. Whole Estate				
Cultivated	6 325	5 882	5 889(3)	5 300(3)
Not cultivated	2 218	2 062	2 654	2 388
Total(4)	8 543	7 944	8 543	7 688

Notes :
(1) Net area = 0.93 x gross for existing field layout
(2) Net area = 0.90 x gross for new field layout
(3) Assuming the non-cane area is not cultivated
(4) Excluding storage basins and Burey experimental farm

During and after the rehabilitation of the Estate, it will become apparent that some fields continue to give poor yields. These fields should be examined to establish their soil salinity and watertable levels and, if found to be defective, reclamation, as described for Field 8II above, should be carried out. A 15 year programme of field drain installation has therefore been allowed for in the cost estimates, eventually covering 1 000 ha of the Estate.

CHAPTER 11

DRAINAGE REHABILITATION REQUIREMENTS

11.1 Options

11.1.1 Objectives and Background

The objective of the rehabilitation works is to ensure that the Estate will have available for the next 30 years a cane growing area of 5 300 ha net which can produce an average yield of 91 tonnes cane/ha. The drainage requirements for this are to overcome current problems with the drainage system and to safeguard the cane fields from waterlogging and salinity.

The existing drainage system was described above in Chapter 8, and the existing conditions on the Estate in Chapters 9 and 10. These reveal that over the Estate as a whole there is poor drainage provision and a serious drainage problem. Key features are as follows :

- inadequate existing in-field surface drainage;
- a collector and main drain system designed for surface drainage of a smaller Estate area and inadequate in the newer areas to the south, east and north, used in part for irrigation supply, and in poor condition due to reed growth and siltation;
- a disposal system partly constructed but not completed, designed to dispose of non-saline or saline water from a deep drainage system;
- average watertables approximately 2 m below ground level;
- average hydraulic conductivities in the range 0.1 to 0.6 m/day;
- existing soil EC_e values generally above 1.5 mmhos/cm;
- irrigation water quality of C2S1 according to the USDA classification (USDA 1954) giving a leaching requirement of 15%;
- poor control of irrigation application.

The problems of high watertable, low hydraulic conductivity and high soil EC_e are worst in the west of the Estate, but potentially serious over the whole Estate. A similar conclusion may be reached concerning the natural deep drainage rates (see Section I.9.7).

11.1.2 A Deep Drainage System

With the existing serious drainage situation, the only secure way to protect the cane fields from yield reductions due to waterlogging and salinity is to construct a deep drainage system. This would comprise the following works, as included in the Heavy Investment Strategy.

- buried field drains 2 m deep, at 50 m spacing, maximum length 370 m;
- deep collector drains, 1 in mid-field, 1 at the foot of the field, to convey the water from field drains to main drains;

- deep main drains to convey the water by gravity from the deep collector drains to the disposal system; these main drains would be constructed to the layout shown in Figure I.11.1 by deepening the existing main drains, deepening and enlarging existing collectors, and constructing new sections of main drain.

The costs of constructing this deep drainage system for 5 300 ha net depend on the fields selected to make up the area and consequently whether any of the main drainage works may be omitted. Indicative costs are given in Table I.11.1 for providing a deep main drainage system to all the Estate (some 8 500 ha net) and deep collectors and field drains for 5 300 ha net. Minor reductions could be made to the main drain system by careful choice of cane area (e.g. by omitting the fields drained by the west branch drain, or the fields to the north of the Estate). In this way it may be possible to reduce the total costs of the deep drainage system for 5 300 ha to some SoSh 280 million.

TABLE I.11.1

Costs of a Deep Drainage System

	Description	Rate per ha (SoSh)	Amount (SoSh million) (mid '83 prices)
1.	Deepen West main drain system	-	30.9
2.	Deepen Middle drain and East drain	-	50.0
3.	Deepen existing collector drains (5 300 ha net)	8 000	42.4
4.	Construct new midfield collector drain (5 300 ha)	9 500	50.4
5.	Install field drains at 50 m spacing (5 300 ha)	22 000	116.6
		TOTAL	290.3

Because of the high cost of this option, alternatives have been considered as follows for the Moderate Investment Strategy:

- a partial deep drainage system (deep collector drains and/or deep main drains, without field drains);

- a shallow drainage system for surface water disposal only;

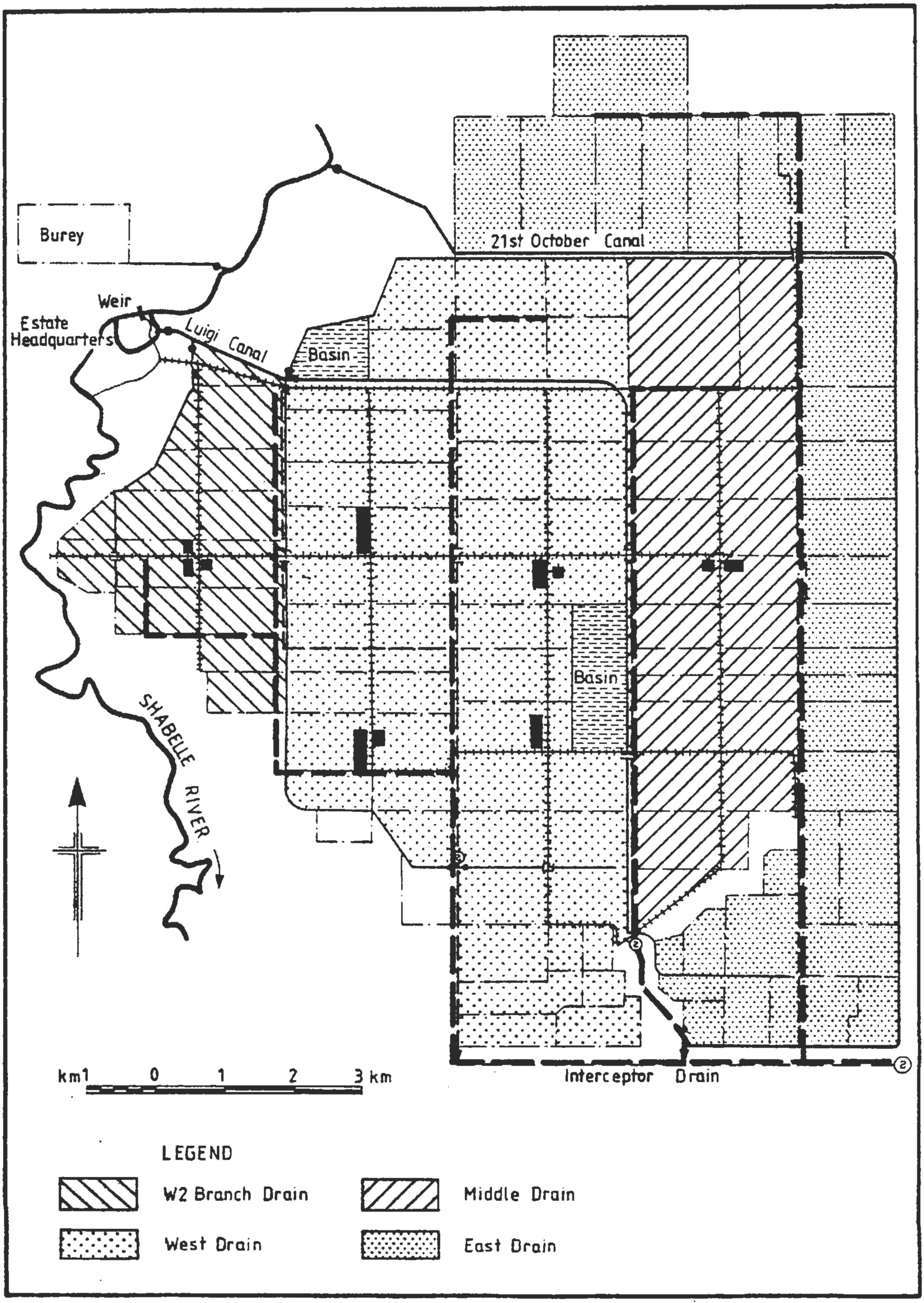
- a mixed system, with full deep drainage on one area, partial deep drainage on another area, and shallow drainage on the remainder;

- reducing the specification of the deep drainage system.

These options are discussed in turn below.

Figure I.11.1

Jowhar Sugar Estate Proposed Main Drainage System and Catchment Areas



11.1.3 A Partial Deep Drainage System

A partial deep drainage system would consist of main drains and collector drains deepened to the full design depth necessary to drain 2 m deep field drains, but without installing the field drains. Possibilities for the collector drains are:

- to deepen the existing collector drain only (equivalent to subsurface drains at about 700 m spacing);
- to deepen the existing collector and to install a new midfield deep collector, as in the field layout with field drains (equivalent to subsurface drains at about 350 m spacing);
- to retain the existing shallow collector drains (giving subsurface drainage by the main drains only, at about 2 km spacing).

Subsurface drains at these spacings give negligible control of the watertable. The standard analyses are not strictly applicable under these conditions, but nevertheless demonstrate that the drainage effect is minimal. Use of the Hooghoudt steady state equation for instance suggests that collector drains at 350 m spacing would remove only 0.2 mm/day with the watertable at 2 m above the drains (approximately 0.5 m below ground level). In practice in soils of these low hydraulic conductivities, such effects are not expected to extend beyond about 80 m from the drain and watertables would remain high for most of the distance between the drains.

Another minor and localised effect of the deep collectors and main drains would be to intercept groundwater which has already risen to drain level and above. Some of this appears to lie in sand lenses. When the collector drain was constructed in 1977 for the drainage trials on field 12 II it was observed to intercept this high groundwater before field drain trenches were opened. The subsurface drainage effects of intercepting this groundwater are likely to be negligible compared to the recharge from irrigation. However, some reclamation effects are anticipated in fields with high saline groundwater, particularly if collector drains are constructed and the field then left fallow.

The main benefits of a partial deep drainage system would be as follows:

- surface drainage disposal;
- the option of installing deep field drains at a later date;
- provision of suitable fill material for use in constructing new canals and roads;

The partial deep drainage system would dispose of surface drainage water without difficulty, and its depth provides temporary storage for any overload.

Once deep main drains have been constructed, field drains could be installed later on any field or part of a field which is found to have a drainage problem. If deep collectors are constructed at the same time as the main drains, then subsequent installation of field drainage as required would be relatively easy and non-disruptive. Thus a partial deep drainage system may be seen as providing the basic infrastructure which would enable the Estate (finance permitting) to maintain good drainage conditions in the future.

The subsoil excavated in deepening the drains is expected to be suitable material for canal embankments, and this would avoid the need for borrow areas to provide material for remodelling the irrigation canal network.

Indicative costs of a partially deepened drainage system can be derived from Table I.11.1. A deep main drain system for the whole Estate would cost SoSh 80 million. With deepened existing collector drains for 5 300 ha net the total cost would be SoSh 122 million, and with both deepened existing collectors and new midfield collectors for 5 300 ha net the total cost would be SoSh 173 million. As with the deep drainage system, it might be possible to reduce these figures by a maximum of SoSh 10 million by selecting the 5 300 ha from those areas which require least main drainage works.

11.1.4 A Shallow Drainage System

The minimum drainage provision for the Estate would be a system intended solely to remove excess rainfall or irrigation water from the surface of the fields, to convey this to the boundary of the Estate and dispose of it.

The existing drainage system is quite inadequate for this, and the following works would be required:

- new in-field surface drains as provided in the proposed new field layout;
- connections into the main drainage network;
- clearing reed growth and silt from existing main drains;
- remodelling existing main drains and constructing new main drains to provide gravity drainage from each field to the disposal system.

Indicative costs of a shallow drainage system for the whole Estate (some 8 000 ha net) are detailed in Table I.11.2.

TABLE I.11.2

Costs of a Shallow Drainage System

	Description	Rate per ha (SoSh)	Amount (SoSh million) (mid '83 prices)
1.	Clean and remodel West drain system		11.0
2.	Clean and remodel Middle drain and East drain (northern section)		18.3
3.	Rehabilitate collector drains and structures (5 300 ha net)	320	1.7
4.	In-field surface drains		Included in field layout costs
	TOTAL		31.0

11.1.5 A Mixed Drainage System

By a mixed drainage system is meant the division of the scheme into different areas, with full deep drainage on one area, partial deep drainage on another area, and shallow drainage on the remainder. This approach has the advantage that the intensity of drainage to be installed in a particular area can be decided from an assessment of its particular need for drainage, which should be more cost effective than a standard provision for all fields. In addition, partial deep drainage can readily be converted to full deep drainage later, on any particular fields which do not drain adequately with the partial deep drainage system. However, partial deep drainage and full deep drainage can only be installed on fields which are connected to a deep main drain. Thus a fundamental decision to be made for each main drain (and branch main drain) is whether to remodel it to a deep or a shallow level.

The indicative costs given in Tables I.11.1 and I.11.2 show that considerable savings can be made by adopting a shallow drainage system wherever possible. However surface drainage will tend to reduce future deterioration of field conditions rather than to improve the present situation, and deep drainage is essential in areas where watertable and soil salinity levels are to be reduced.

Based on these considerations a mixed drainage system is proposed for rehabilitation of the Estate. Details are given in Section 11.2.

The key cost saving measure is the adoption of surface drainage instead of deep drainage in the areas served by the Middle and East drains. The success of this is very much dependent on careful irrigation control to restrict the deep percolation to amounts within the vertical drainage capacity of the subsoil. If this is exceeded, watertables will rise, reducing yields. Recording of canal discharges and irrigation applications, monitoring of watertable levels and analysis of yields should enable problems to be identified and corrective action taken. If, however, serious problems do arise production could be maintained by bringing in additional land on the West drain, with subsurface drainage measures, to replace unproductive land on the Middle and East drain.

11.1.6 A Deep Drainage System with Reduced Specification

The key features of the deep drainage system which largely govern its cost can be divided as listed below, into those features which govern the depth of the system (particularly the depth of the main drains) and those which otherwise influence the unit costs.

Features which govern the depth of the system:

- field drain depth 2 m;
- open collector drain bed level 0.4 m below field drain outfall;
- minimum bed slope of open collector drain 0.25 m/km;
- minimum bed slope of main drain 0.10 m/km;
- minimum head loss of 0.05 m at all culverts;
- choice of open or buried collector drains.

Features which have a marked influence on unit cost :

- field drain spacing 50 m;
- field drains to have 75 mm thick gravel surround;
- open drain side slopes to be 1 : 1.2.

In general the listed depths, head losses and bed slopes are all regarded as the least required for the drainage system to operate satisfactorily. However, some reductions could be made for the particular critical cases which govern the depths of each main drain, to save capital cost at the marginal cost of slightly poorer drainage over a small area.

The choice of open or buried collectors is discussed in detail in Section 12.6. The costs given in Table I.11.1 assume that open collectors are used: changing to buried collectors would require a slightly deeper and more expensive main drain system.

The possibilities for reducing the cost of a full deep drainage system by changing those factors which have a marked influence on unit costs are discussed in turn below.

(a) Field Drain Spacing

Increasing the drain spacing would cause a rapid deterioration in the protection provided. For instance, analysis of Hooghoudt equation shows that a 10% increase in drain spacing would cause an increase of about 16% in the watertable height above the drains. Since the field drain costs are themselves only about 45% of the total costs of the deep drainage system there is a danger that increasing drain spacings would reduce the benefits of drainage considerably while making only a small reduction to the costs.

(b) Gravel Surround

It is difficult to assess the effects of changing the gravel surround to the drainage pipe. The 1983 inspections of the corrugated plastic pipe installed in field 12 II concluded that, where there was no filter, the soil backfill around the pipe had compacted which restricted the entry of water (Section 9.1). Therefore it is recommended that a gravel filter is used to prevent inflow being reduced by this formation of a layer of low permeability.

However, the cost of this gravel filter amounts to half the cost of the field drains. Thus the field drainage cost could be reduced by 50% (amounting to about 20% reduction in the total costs of a deep drainage system) by installing field drains without a filter. There is a danger however that the field drains might become ineffective in time. The need for a gravel filter is discussed further in Section 12.5.4 where it is proposed that four prototype field drains are installed without gravel early in the field drainage programme, and that filter requirements are kept under review.

Further compromise is possible by wrapping a filter cloth around the pipe instead of the gravel surround. However the fabrics tried on field 12 II (Terram 140 and Fibretex) showed evidence of severe blockage by iron ochre (MMP 1978) so it is recommended that fabric filters are not used in this way.

(c) Open Drain Side Slopes

A standard recommendation for side slopes on these soils in cut would be 1 vertical in 1.5 horizontal. However, the mean side slope of 10 samples of the existing drains (MMP 1976 survey) showed a mean side slope of 1 vertical in 1.2 horizontal, with some drains having steeper existing slopes. Therefore 1 : 1.2 side slopes are proposed.

A steeper side slope (e.g. 1 : 1) is likely to cause slumping of the drain banks and would therefore cause higher maintenance costs. The saving in capital cost by changing from a 1 : 1.2 side slope to 1 : 1 would be approximately 10% of the main drain cost and 12% of the collector drain cost, which amounts to a total reduction of only about 6% on the deep drainage system costs.

It is clear from the above that no substantial reduction could be made to the cost of a full deep drainage system by reducing the specification and therefore savings have been made by varying the intensity of the drainage provision according to need.

11.2 Proposed Rehabilitation Measures

It is considered that a mixed drainage system could provide, at reasonable cost, adequate drainage to sustain a yield of 91 tonnes cane/ha over 5 300 ha. The proposals for the works have been determined by assessing the appropriate drainage measures for each field (Figure I.11.2) and then deciding whether each main drain should be remodelled to a shallow or a deep level and selecting fields to make up 5 300 ha net at minimum drainage cost. The selection was made bearing in mind irrigation aspects and cane transport costs (approximately SoSh 4 per tonne /km) but it was found that the costs of the different proposed drainage measures governed the selection of cane area.

Field rehabilitation requirements for the Estate area of 7 700 ha net (with the new field layout), corresponding to the gross area of 8 500 ha, can be broadly grouped into five categories:

(i) New field layout and surface drainage. These fields have reasonable cane at present and few subsurface drainage problems. They are mainly concentrated in the northern and eastern sectors of the Estate and total some 3 800 ha net. The average cost of providing these areas with the new field layout and rehabilitated irrigation and drainage systems is estimated at some SoSh 32 000 per ha, excluding the cost of works for non-cane areas.

(ii) New field layout, surface drainage, deep collector drains, and a possible requirement for buried field drains in the medium term. There are some 2 400 ha net in this category, mostly located in the oldest part of the Estate near the factory, but including some land to the south which is currently irrigated by pumping drainage water. The fields have drainage problems and cane quality is generally not good. The estimated cost of rehabilitating these fields in about SoSh 45 000 per ha, including the new field layout on cane areas and primary works on non-cane areas.

(iii) New field layout, surface drainage, deep collector drains, and a reclamation programme, with an expected requirement for buried field drains in the medium term. These fields are marginally cultivable areas covering about 500 ha. They are mostly in the central low lying part of the Estate and abandoned at present but suffering less from salinity and groundwater problems than other abandoned areas. It is hoped that reclamation can be achieved by deepening the existing collector drain, constructing a new intermediate collector drains and conducting a leaching programme. It can be anticipated however that the majority of these fields would require installation of field drains in the short to medium term to maintain high cane yields. Capital costs of rehabilitation are estimated at SoSh 55 000 per ha, excluding field drains which would cost another SoSh 22 000 per ha.

(iv) New field layout, surface drainage, deep collector drains, buried field drains and reclamation. These fields cover an area of about 500 ha near the factory. They are presently abandoned and cover a range of conditions including some land with high salinity and/or high watertable. They would certainly require field drains to ensure reclamation over a two to three year period. The total capital costs of rehabilitating these fields would be approximately SoSh 77 000 per ha.

(v) Fully abandoned land which is not recommended for reclamation in the short term. This covers the worst of the abandoned land totalling about 500 ha.

Figure I.11.2 shows our assessment of the drainage measures required to enable the fields of the Estate to support the target yield in the short to medium term. In the medium to long term these measures probably would not prevent drainage problems arising on some fields (particularly if there is poor control of irrigation applications). Therefore future provision should be made for additional drainage measures to be installed on some fields as required.

Consideration of the drainage requirements set out in Figure I.11.2 led to the following proposals for rehabilitation of the main drains:

- shallow Middle drain and East drain;
- deep West drain system (West drain, west branch drain W2 and sub-branch W2/2);

A cane area was then selected to total about 5 300 ha as a convenient block with low cost in drainage requirements. This is shown on Figure I.11.3, and is made up as follows :

- surface drainage on 3 526 ha net (fields on Middle and East drains);
- partial deep drainage, with deepened existing collector drains on 1 722 ha net (fields on West drain, W2 and W2/2);
- partial deep drainage and reclamation with new deep collector drains in midfield and deepened existing collector drains on 56 ha net (field 14 II on West drain);
- full deep drainage, with deep collector drains and field drains on 57 ha net (field 8 II on West drain);

Thus the total area included in the rehabilitation programme is 5 360 ha net. This allows for possible exclusion of small areas (some 60 ha total) which are not currently cultivated in some fields (e.g. 15I, 2III, 4III), resulting in the required cultivable area of 5 300 ha net.

Figure I.11.2

Jowhar Sugar Estate
Drainage Measures Required

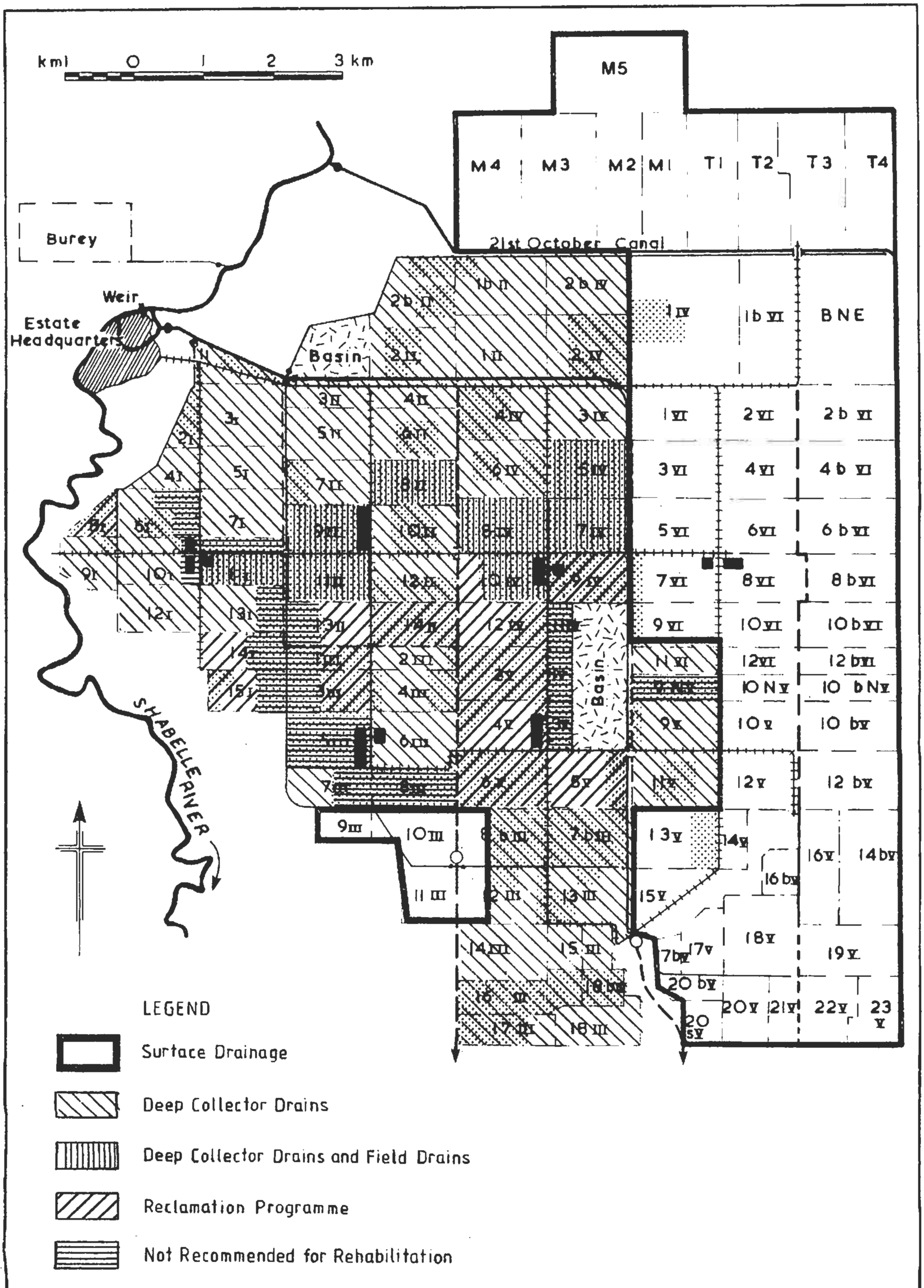
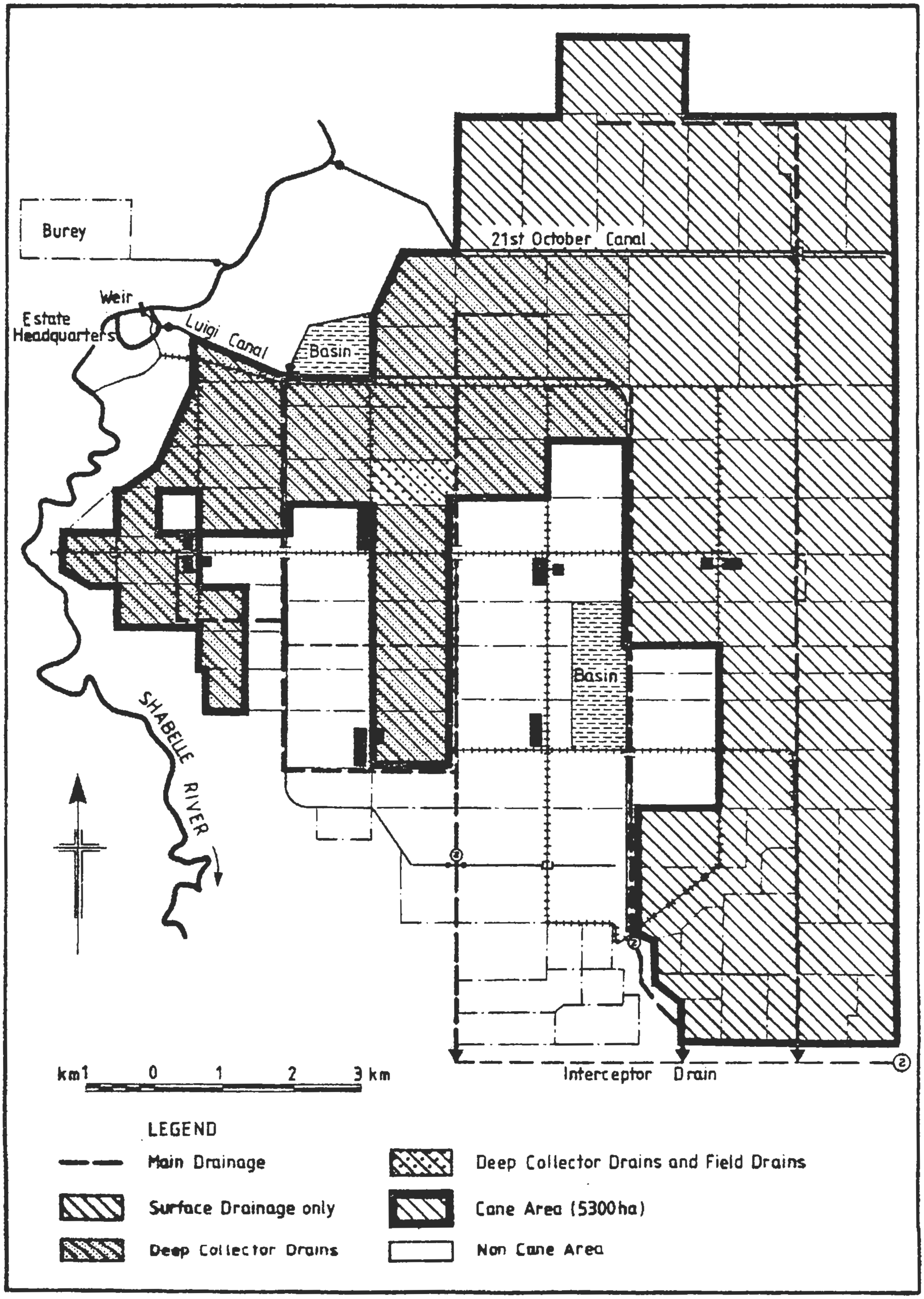


Figure I.11.3

Jowhar Sugar Estate
Proposed Cane Areas and Drainage Rehabilitation Measures



CHAPTER 12

DRAINAGE DESIGN

12.1 General

Both surface and subsurface drainage are to be considered. The drainage rates for these are derived in Sections 12.2 and 12.3.

Figures I.6.2 and I.6.3 show a typical field layout. There are four types of drain, as follows:

- surface drains (in-field);
- buried field drains;
- deep collector drains;
- main drains.

In Sections 12.4 to 12.7 the requirements for each type of drain are described and design criteria are prepared. In general all drainage is to be designed in accordance with the FAO recommendations on drainage design factors (FAO, 1980).

12.2 Surface Drainage Rate

12.2.1 Introduction

The drainage system is designed to satisfy two separate requirements for the drainage of surface water:

- to remove excess irrigation water, runoff from each irrigation;
- to remove excess rainfall, runoff from storms.

The irrigation runoff is a regular event. It is estimated at about 10% of the gross field irrigation requirement (another 25% is assumed lost to deep percolation, to give an overall field efficiency of 65%). The gross field irrigation requirements total 2 207 mm on average per year (see Annex II), which implies 221 mm per year irrigation runoff or 15 mm per irrigation. The runoff from peak storms (see below) is substantially more than this and is therefore used for design of the surface drainage system. However the regular flow of irrigation runoff to the drainage system has to be taken into account in the design of drains for the continuous disposal of subsurface water and surface water combined. This is described in Section 12.7.

The derivation of the design drainage rate for removing excess water from storms is described in the following sections.

12.2.2 Design Storm Rainfall

Annual maximum daily rainfalls have been abstracted from the 25 year records at Jowhar (Villabruzzi) (Fantoli, 1965). The records cover two periods of 1922 to 1939, and 1953 to 1960. The annual maximum series has been fitted by an extreme value type I (Gumbel) distribution (Figure I.12.1) and maxima estimated for selected return periods. These are detailed in Table I.12.1.

TABLE I.12.1

Annual Maximum Daily Rainfall at Jowhar (Villabruzzi)

Return period (years)	Rainfall (mm)
2	58.6
5	79.5
10	93.4
20	106.7

The daily total assumed for the design storm has been distributed according to US Soil Conservation Service Type II storm profile representing a convective storm (USDA 1968). The profile is symmetrical about the central hour in which 42% of the total falls.

12.2.3 Drainage Criteria

It has been assumed that the maximum duration of flooding caused by the 1 in 5 year 24 hour rainfall may not exceed 4 days without damage to the sugar cane. The minimum drainage rate to meet these criteria is taken as the design rate for the design of field outlet structures and collector drains which will run at about design capacity for the duration of flooding.

Evaporation losses have been estimated as 6.3 mm/d from the monthly Penman evaporation figures for Jowhar (MMP, 1976).

Rates of infiltration into the soil will depend on field conditions, particularly the existing moisture content. Therefore a range of infiltration rates has been used in the analysis from 75 mm/d to 5 mm/d to indicate the variation in rates depending on the length of time since the last irrigation.

Furrows on the Estate are to have bed slopes in the range 0.2 to 0.02%. Furrows are assumed for this analysis to be 1.5 m apart, 0.3 m deep and 350 m long.

12.2.4 Conclusions

A water balance analysis was undertaken to determine the drainage rate necessary to protect the fields for flooding for more than four days under the conditions described above. The results are as shown in Table I.12.2.

TABLE I.12.2

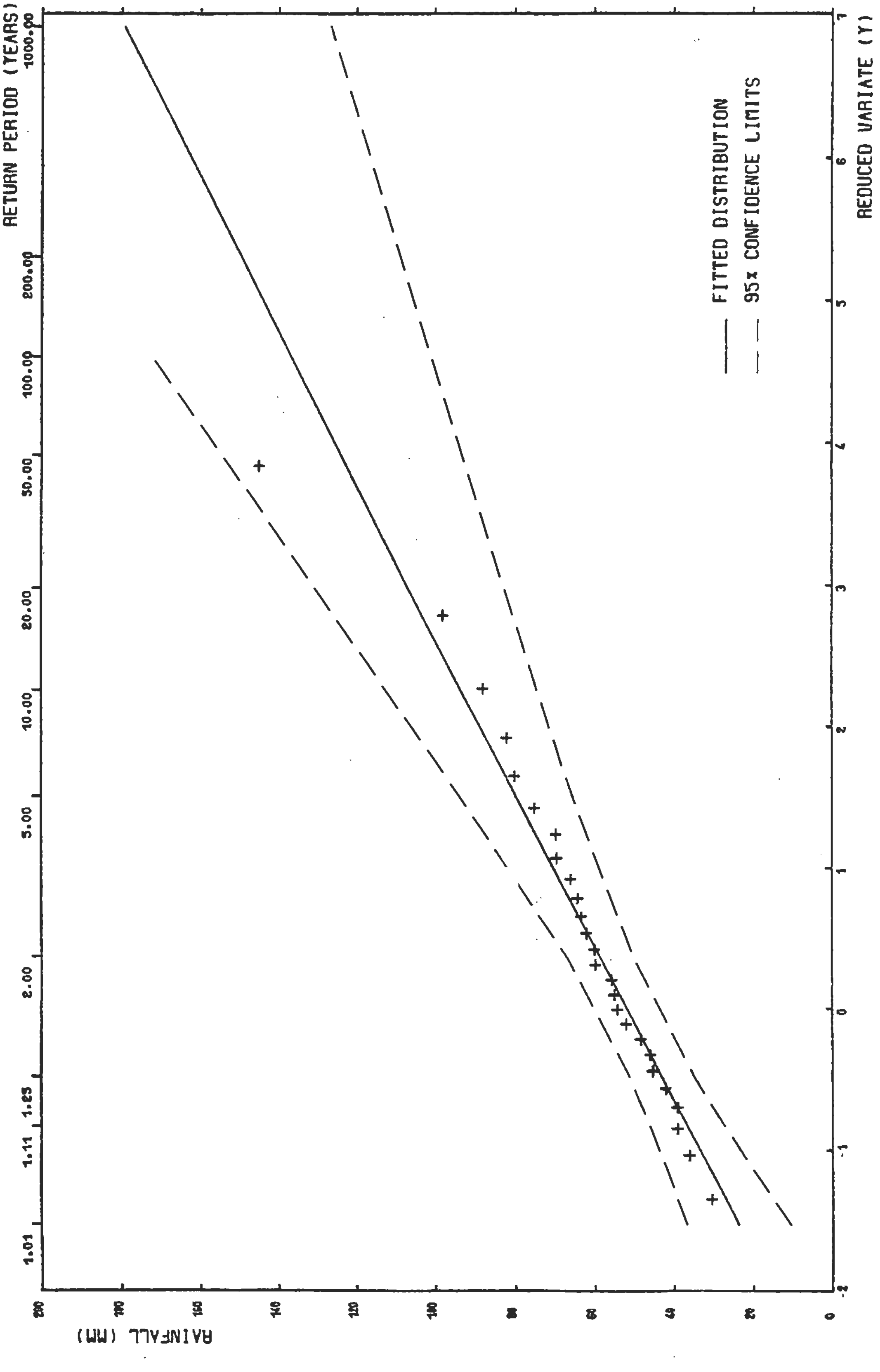
Required Surface Drainage Rates

Infiltration rate (mm/d)	Drainage rate (l/s/ha gross)		
	Furrow slope = 0.2%	Furrow slope = 0.05%	Furrow slope = 0.02%
75	0.2	0	0
25	1.3	0.3	0
10	1.7	1.0	0.3
5	1.9	1.2	0.6

ANNUAL MAXIMUM RAINFALL SERIES

VILLABRUZZI (JOWHAR)

TYPE 1 EXTREMAL (GUMBEL) DISTRIBUTION



For these conditions 10 mm/d is thought to be an appropriate design infiltration rate, to represent the condition that the fields are already wet (but not saturated) from previous rainfall or irrigation. The sample areas used for the land levelling quantity estimates showed approximately twice as many furrow slopes in the range 0.02% to 0.05% as in the range 0.05% to 0.2%. Therefore a design drainage rate of 1 l/s/ha gross has been adopted.

The peak flows in the main drains and the intercepting drains will be at a reduced rate because the peak storms would not normally fall uniformly over the whole area. Area reduction factors of 0.9 for main drains and another 0.9 for the intercepting drain have been assumed. The design drainage rates for disposal of excess water from the 1 in 5 year storm are therefore:

- for in-field surface drains and collector drains, 1.0 l/s/ha gross;
- for main drains, 0.9 l/s/ha gross;
- for the intercepting drain, 0.81 l/s/ha gross.

12.3 Subsurface Drainage Rate

The purpose of the subsurface drainage system is to remove from the subsoil the excess of inflow minus natural outflow which would otherwise cause a rise in the watertable. This is expressed by the water balance equation introduced in Section 9.7.1 (FAO 1980):

$$Q_s = R_{dp} + S_c - C - D \qquad \text{Equation 1}$$

where, in mm/d,

Q_s = design drainage rate for field drainage;

R_{dp} = recharge to groundwater by deep percolation from irrigation or rainfall;

S_c = seepage from canals;

C = capillary rise from groundwater;

D = natural deep drainage (groundwater flow out of the field area net of any inflow or artesian flow).

It was estimated in Section 9.7.1 that under existing conditions the approximate rates of natural deep drainage (D) were as follows:

- on the West drain, D = 0.8 mm/d;

The recharge to groundwater, R_{dp} is made up of two components:

- deep percolation from irrigation;
- deep percolation from non-effective rainfall.

The deep percolation from irrigation from the proposed well graded furrows is estimated as 25% of gross field water requirements (FAO 1980) which exceeds the leaching requirements. Thus the annual deep percolation from irrigation amounts to 0.25 x 2 207 = 552 mm/year or 1.5 mm/d.

The surface drainage system is designed to allow 1 l/s/ha to run off the furrows, amounting to 8.6 mm/d. This will remove most of the ineffective rainfall from the field, and it is estimated that only some 30% of the ineffective rainfall will infiltrate and contribute to deep percolation; this amounts to 0.3 x 93 = 28 mm/year or 0.1 mm/d.

The total rate of deep percolation to groundwater (R_{dp}) is thus

552 + 28 = 580 mm/year or 1.6 mm/d.

Seepage from tertiary canals is included in the deep percolation rates given above. Seepage from main and secondary canals (S_c) is estimated at 10% of the field irrigation requirements (see Annex II).

A monthly breakdown of these figures is given in Table I.12.3.

Capillary rise is estimated at an average of 0.2 mm/d for the low soil suctions with the improved irrigation regime and a steady state watertable 1 m below ground level.

These assumed values of the water balance components are shown in Table I.12.3 on a monthly basis. These have been used to give estimates of the required drainage rate for field drainage (Q_s), from Equation 1. For the cultivated fields served by the West drain Q_s varies from a minimum of 0.7 mm/d in July to a maximum of 2.2 mm/d in March, with a mean value of 1.3 mm/d.

The analysis above must be regarded as indicative only, and the accuracy of the various estimated values is low. However the general conclusions are not likely to be changed by more accurate data.

Deep drainage is clearly needed in the West drain area. A design drainage rate of 1.8 mm/d has been adopted to satisfy the drainage requirements most months of the year and give a contingency over the mean requirements. This has been derived for the fields currently growing sugar cane which have also been selected for future sugar cane production. It is assumed that this rate is also applicable to other fields (both cultivated and abandoned fields) which may be used in future for the production of other crops.

As discussed in Section 9.7.3 it is thought that deep drainage is not needed in the Middle and East drain areas provided that irrigation is controlled carefully and deep percolation and seepage losses are limited. It is thought that the new field layout, irrigation control system and training proposals would enable the Estate to achieve this and maintain this standard. Monitoring of irrigation quantities and watertable levels will be a necessary part of this improved water management. If watertables rise irrigation applications must be reduced and capillary rise will tend to contribute more to water requirements and help to stabilise the watertable.

In the above analyses it is assumed that water shortages have a negligible overall effect and the system is operated well, so that water delivered to the field equals field water requirements. Also lateral movement of water is assumed negligible.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Mean Monthly Groundwater Balance

	Mean monthly values (mm/d)													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean	Total mm/year
Gross field irrigation requirement	7.5	8.2	9.0	4.9	4.5	5.0	4.8	5.9	8.2	4.2	4.7	5.8	6.1	2 207
Deep percolation from irrigation	1.9	2.1	2.2	1.2	1.1	1.3	1.2	1.5	2.1	1.1	1.2	1.4	1.5	552
Deep percolation from rainfall	0.1	0	0	0.2	0.2	0	0	0	0	0.2	0.2	0	0.1	28
Total recharge to groundwater R_{dp}	2.0	2.1	2.2	1.4	1.3	1.3	1.2	1.5	2.1	1.3	1.4	1.4	1.6	580
Canal seepage S_c	0.8	0.9	1.0	0.5	0.5	0.6	0.5	0.6	0.9	0.5	0.5	0.6	0.7	243
Capillary rise C	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	73
West drain natural drainage D	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	292
$R_{dp} + S_c - C - D$	1.8	2.0	2.2	0.9	0.8	0.9	0.7	1.1	2.0	0.8	0.9	1.0	1.3	458

Table Nr I.12.3

12.4 Field Surface Drains

A surface drain is required at the foot of each set of furrows to collect runoff from excess irrigation or rainfall and convey it to a collector or main drain. Thus for each standard sized field (approximately 1 100 m x 700 m) two surface drains will be required, one in midfield parallel to the new header channel, and one at the foot of the field (see Figure I.6.1). For non-standard fields the surface drains will be located similarly according to the furrow layout). The surface drain will normally be separate from the collector drain, with a field access road in between. Where the existing collector is shallow, however, and no deep collector is required (i.e. on the Middle and East drain fields) it may be preferable to remodel this existing drain to the required slopes and depth. The surface drain will outlet into a main drain or into a cleared existing collector drain, depending on the location of the field, see Figure I.6.1.

The design cross section is shown in Figure I.12.6. The surface drain is to have 1 : 8 side slopes to make it easier for machinery to cross the drain and to turn here at the end of the cane row. Flow along this surface can be improved where necessary by running a furrow body along its centre line. The minimum bed slope is 0.15 m/km and the minimum depth 0.25 m which is adequate for the design runoff from 35 ha (one half of the typical field).

A 0.3 m diameter pipe culvert is required under the field road where the field is remote from the main drain or, where the field is adjacent to the main drain, a 0.3 m diameter pipe junction culvert into the main drain is required.

On the fields served by the West drain the surface drains would discharge into the deep collector and main drain system through a surface drain outlet. There is a 2 m head loss across this structure and a small diameter pipe is to be used to check the flow.

12.5 Buried Field Drains

12.5.1 Area Drained

The selected cane area includes one field, 8 II, for which installation of buried field drains is recommended to give adequate drainage (Figure I.11.3). Other fields which would require buried field drains have been excluded from the selected cane area. However it is anticipated that other fields in the selected cane area will show some drainage problems, and the costs include for phased installation of field drains over 15 years as required, on a total of 1 000 ha served by the West drain. The particular fields requiring drainage have not been identified and it is anticipated that the Estate management would select an average one field per year by monitoring drainage conditions and yields.

12.5.2 Drain Spacing Calculation

(a) Steady State Method

Although there are several methods for calculating drain spacings by steady state methods the quickest is by means of nomographs prepared by Boumans from data of Ernst. This is based on Hooghoudt's equation:

$$S^2 = \frac{8\,Kdm + 4\,Km^2}{q}$$

where d effective depth dependent on h and drain radius
K hydraulic conductivity
m hydraulic head
q drainage rate
S drain spacing

The effective depth can be calculated using the following relationship:

$$d = \frac{S}{\frac{8}{\pi} \ln \frac{h}{\sqrt{2}r} + \frac{(1 - \sqrt{2}\, h/S)^2}{h/S}}$$

h depth to the impermeable layer
r drain radius including any filter

The nomograph shown on Figure I.12.2, eliminates the necessity for calculating the effective depth and only values of K, q, h, m and U (wetted perimeter of the drain) are required.

(b) Drainage Rate

The field drains are all to be installed in the West drain area. The design subsurface drainage rate for this area is 1.8 mm/d as derived in Section 12.3 and this is used in the steady state drain spacing calculation. (Surface runoff flows direct to the collector drains and is therefore not included.)

$$q = 1.8 \times 10^{-3} \text{ m/d}$$

(c) Hydraulic Conductivity

The results of single auger hole test measurements (Table I.10.1) show that there is a wide range of hydraulic conductivities on the fields of the Estate. The trials on field 12II showed a close agreement between the values of hydraulic conductivity derived from monitoring the performance of field drains (and applying the drain spacing equations) and values obtained from single auger hole tests. This shows that effects such as anisotropy may safely be neglected, and hydraulic conductivity values derived from single auger hole tests are appropriate for design.

A mean value of 0.4 m/d has been adopted for the design of field drains in the selected cane area on the West drain.

$$K = 0.4 \text{ m/d}$$

(d) Watertable Level and Drain Depth

The subsurface drains are designed to keep the watertable below the normal root zone, so as not to restrict plant development. A watertable level of 1 m below ground level has been taken for the steady state drainage design.

A drain depth of 2 m is appropriate for controlling the watertable at 1 m below ground level. It is a practical depth for use of trenching machines and was assumed when the drainage disposal works were designed. Shallower drains would require a closer drain spacing. Because a considerable part of the cost is in the pipe and gravel, which are unchanged by depth, a reduction in drain depth would increase costs per unit area.

Deeper drains would have advantages of increasing the necessary drain spacing or improving the drainage with the chosen spacing. The increased depth however could lead to trenching difficulties and higher costs in downstream drainage channels and disposed costs.

Therefore a drain depth of 2 m has been used in the drain spacing calculations, and the collector drains and main drains would be designed for this. At the implementation stage individual field drains should be installed as deep as the trenching machine, permissible slopes and collector/main drain levels permit to get the maximum drainage benefits.

A watertable level 1 m below ground level would therefore give a hydraulic head (m) of 1 m on the drains.

(e) Effective Wetted Perimeter

The effective wetted perimeter U is obtained from the equation:

$$U = \text{trench width} + 2 \times \text{pipe diameter}$$

A value of U = 0.38 m has been used, for a 0.22 m trench width and 80 mm diameter pipe.

(f) Depth to Impermeable Layer

In general the hydraulic conductivity reduces with depth below ground level. An impermeable layer is assumed to lie at the depth below which the hydraulic conductivity is 10% of the mean value of 0.4 m/d. A depth of 4 m to the impermeable layer has been adopted, following previous studies (MMP 1978). For 2 m deep field drains, the depth to the impermeable layer is 4 - 2 = 2 m :

$$h = 2\text{ m}$$

(g) Drain Spacing Calculation

The required drain spacing has been calculated from Figure I.12.2 and the following data which represent typical conditions for the selected cane fields served by the West drain:

$$q = 1.8 \times 10^{-3}\text{ m/d}$$

$$K = 0.4\text{ m/d}$$

$$m = 1\text{ m}$$

$$U = 0.38\text{ m}$$

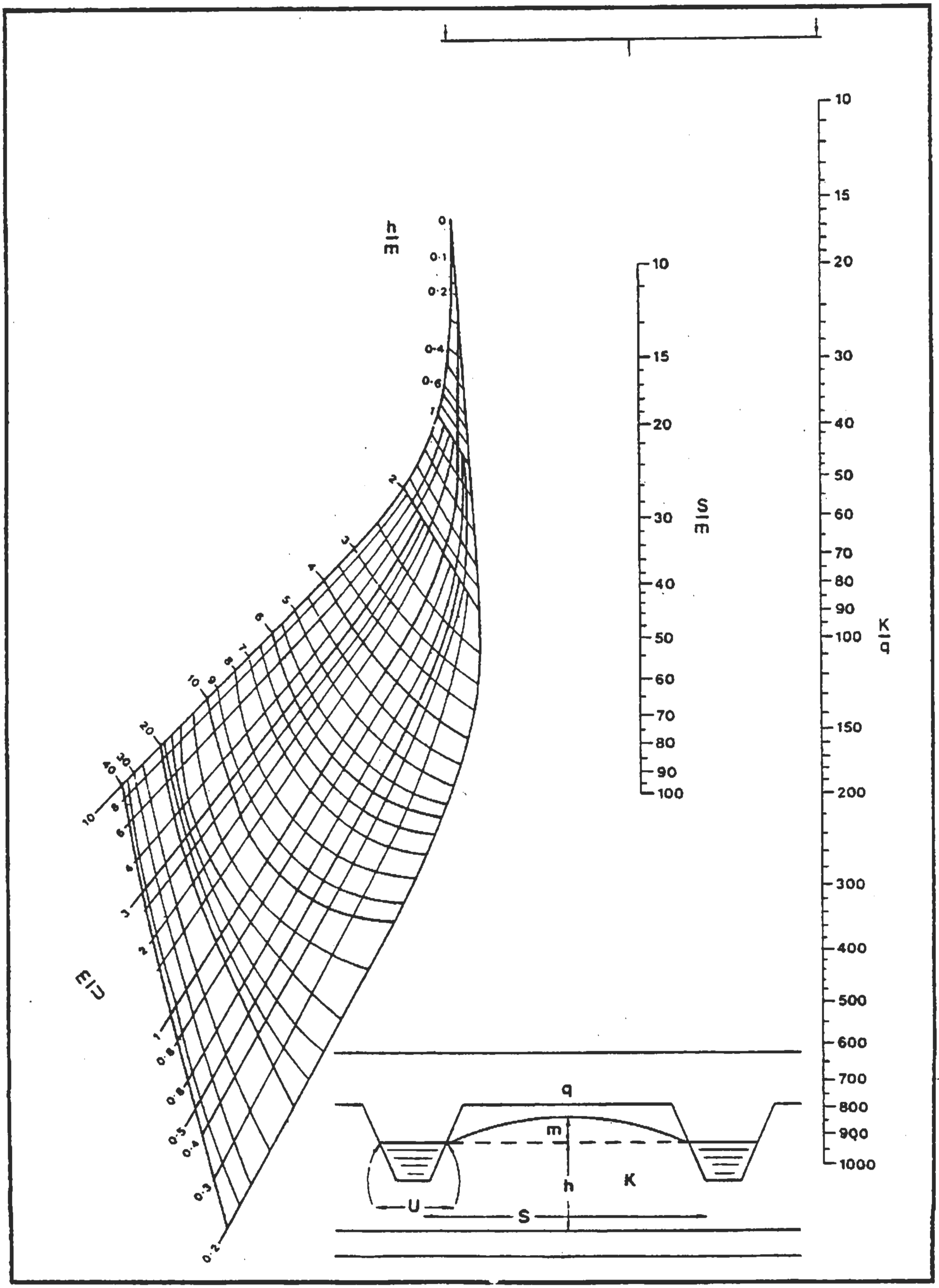
$$h = 2\text{ m}$$

This gives a required drain spacing of 62 m.

Figure I.12.2

Drain Spacing Diagram

Single Layered Soil ($\frac{S}{m}$ < 100)



The major uncertainties and inaccuracies in the above analysis are probably in the adopted values of hydraulic conductivity and drainage rate.

It is intended that fields selected for installation of buried field drains should be those which have shown non-typical drainage problems, (compared with the rest of the cane fields on the West drain, and as a result these fields are likely to have lower than average hydraulic conductivities and poorer than average natural deep drainage. Therefore the analysis has been repeated for a number of other cases as shown in Table I.12.4.

TABLE I.12.4

Theoretical Drain Spacings for Different Drainage Rates and Hydraulic Conductivities

q(m/d)	K(m/d)	S(m)
1.8×10^{-3}	0.4	62
1.8×10^{-3}	0.2	43
2.4×10^{-3}	0.4	52
2.4×10^{-3}	0.2	36
3.0×10^{-3}	0.4	46

In these circumstances a drain spacing of 50 m has been adopted, which should give reasonable watertable control over the wide range of conditions detailed above.

12.5.3 Pipe Size

As described in Section 12.5.4 corrugated uPVC pipe is recommended for the buried field drains. The pipe size is determined from the following equation which is based on work by Dekker (1978) with a constant depth of silt of 13 mm :

$$Q = 422\, D^{3.26}\, i^{0.667}$$

where D = pipe internal diameter

i = hydraulic gradient

This formula contains a correction factor to allow for spatially varied flow along the pipe. A comparison was also made with the Manning formula for non-uniform pipe flow in corrugated pipes (Smedema and Rycroft, 1983):

$$Q = 38\, s\, D^{2.67}\, i^{0.5}$$

where s = reduction factor for siltation

For continuous plastic pipe in stable soil :

$$s = 0.75$$

The Manning formula gave higher discharges for small diameters and lower discharges for large diameters than the formula proposed. This would agree with a concept of a fixed reduction factor for siltation using the Manning formula, whereas a variable one has now been used.

Field drains should have a minimum slope of 1 m/km but 1.5 m/km is preferred to give small pipe sizes and greater tolerance against reverse slopes. Table I.12.5 gives the discharges for recommended pipe sizes at a slope of 1.5 m/km.

TABLE I.12.5

Discharge for Corrugated Pipes with Recommended Slopes

Pipe diameter (mm)		Pipe discharge (l/s)
Nominal	Internal	
60	54	0.41
80	72	1.04
100	92	2.32
125	115	4.79

The discharge to the field drain varies with watertable height, and thus with time after irrigation. This can be represented by the equation :

$$q = \frac{2\pi K\, m\, (d + m/2)}{S^2}$$

(Luthin, 1966)

For $K = 0.4$ m/d and

$S = 50$ m

discharge varies with watertable height as follows :

m (m)	q (mm/d)	q (l/s/ha)
1.7	4.8	0.6
1.5	4.0	0.5
1.0	2.4	0.3

This shows that an 80 mm diameter pipe would be adequate to drain fields of 400 m maximum length (average 310 m) with drains at 50 m spacing. When the watertable is at least 0.3 m below ground level the field discharge to the drain would be carried by the pipe at the design hydraulic gradient. With reasonable irrigation control watertables should not arise above this level but, if local flooding causes this to occur, the pipe would be surcharged for a short time, which is considered acceptable. This pipe size of 80 mm is also compatible with the peak monthly steady stage drain discharge of 2.2 mm/d.

12.5.4 Pipe Type and Filter

The options for drainage pipe are as follows:

- perforated corrugated plastic pipe (uPVC)
- clay pipe
- concrete pipe

These have been compared in detail in previous reports (MMP 1976, MMP 1978).

Plastic pipe is recommended for Jowhar because the continuous pipe is more easily cleaned by jetting - the separate clay or concrete pipes tend to be pushed out of alignment on cleaning. Plastic pipe could be ordered in the required lengths from a supplier in Europe and now that only relatively small quantities are needed each year this would be more convenient than setting up pipe manufacturing facilities locally. There should also be no problems with quality control with plastic pipes supplied in this way.

Unlike clay and concrete pipe, plastic pipe does not require a gravel surround to give satisfactory hydraulic characteristics. In soils containing a large proportion of coarse silt and fine sand, however, a gravel or other filter is needed to prevent these particles moving into the pipe. The clay soils at Jowhar are not problem soils in this sense, having a clay to silt ratio of 4.3 compared with the critical value of 0.5 below which soils should be considered unstable (Olbertz, 1965). Figure I.12.3 shows the particle size gradings of the Jowhar soils compared to problem soils. However the trials revealed minor problems of iron ochre deposition (MMP, 1978) and the investigations reported in Section 9.1 found some compaction of soil around pipes which had been installed without a gravel filter. It is possible that this compaction was caused during the installation of drain pipe by hand in open trenches, especially as the height of the watertable above drain level caused continuous problems in keeping the trenches dry (MMP, 1978).

The decision on whether or not to install a gravel filter can be assessed in economic terms by comparing the cost of the gravel with the expected life of the drains. The cost of the gravel amounts to about 50% of the total cost of the field drains. With a gravel filter one may be confident that the pipe drains would last about 30 years with occasional cleaning by jetting (FAO 1979). The life of field drains installed without a gravel filter would probably be somewhat less, but no accurate predictions can be made at this stage.

A simple present value calculation with a discount rate of 12% has been used to compare the two options. This shows that, if the drains without a filter operate satisfactorily for seven years or more, and are then replaced by further drains without a filter and so on, this has a lower economic cost than installing drains with a gravel filter at double the cost which have a 30 year life.

Since the field drainage programme amounts to a small amount of work per year, and low usage of the trenching machine, it would not be a major problem to replace field drains every seven or more years. Therefore the use of field drains without a filter should be considered seriously. The recommended approach is to install a prototype area in field 8II comprising four adjacent field drains without gravel filter. A gravel filter would be used for the remaining drainage pipes (approximately 40 Nr), to safeguard against deteriorating performance due to restriction of inflow by a poorly permeable soil surround or to blockage by lenses of unstable soil moving into the pipe. Piezometers would be installed to measure the watertable profile under both conditions and drainage from the pipes with and without a filter would be monitored over a number of years, both on field 8II and the trial field 12II. This would enable a decision to be made for future field drain installation, on whether the life of drains without a filter is long enough.

The costs presented in Chapter 14 assume that a gravel filter is installed on all future field drain installations.

There are difficulties applying the gravel uniformly around the pipe, and a thickness of 75 mm should be specified to ensure that a minimum thickness is achieved. The recommended grading is shown in Figure I.12.4 (MMP, 1978). This is based on standard recommmendations for a soil with 60% of the material finer than 0.02 to 0.05 mm particle diameter (Winger and Ryan, 1970) which is appropriate for the Jowhar soils shown in Figure I.12.3. A typical field drain cross section is shown in Figure I.12.5.

As described in Section 9.1, replacing the gravel filter by a fabric filter wrapped around the cloth is not recommended because of the danger than iron ochre would block the filter. With gravel filters however it is recommended that a synthetic pervious membrane is laid on the top of the gravel to prevent the filter from damage by high flows down the trench after irrigation.

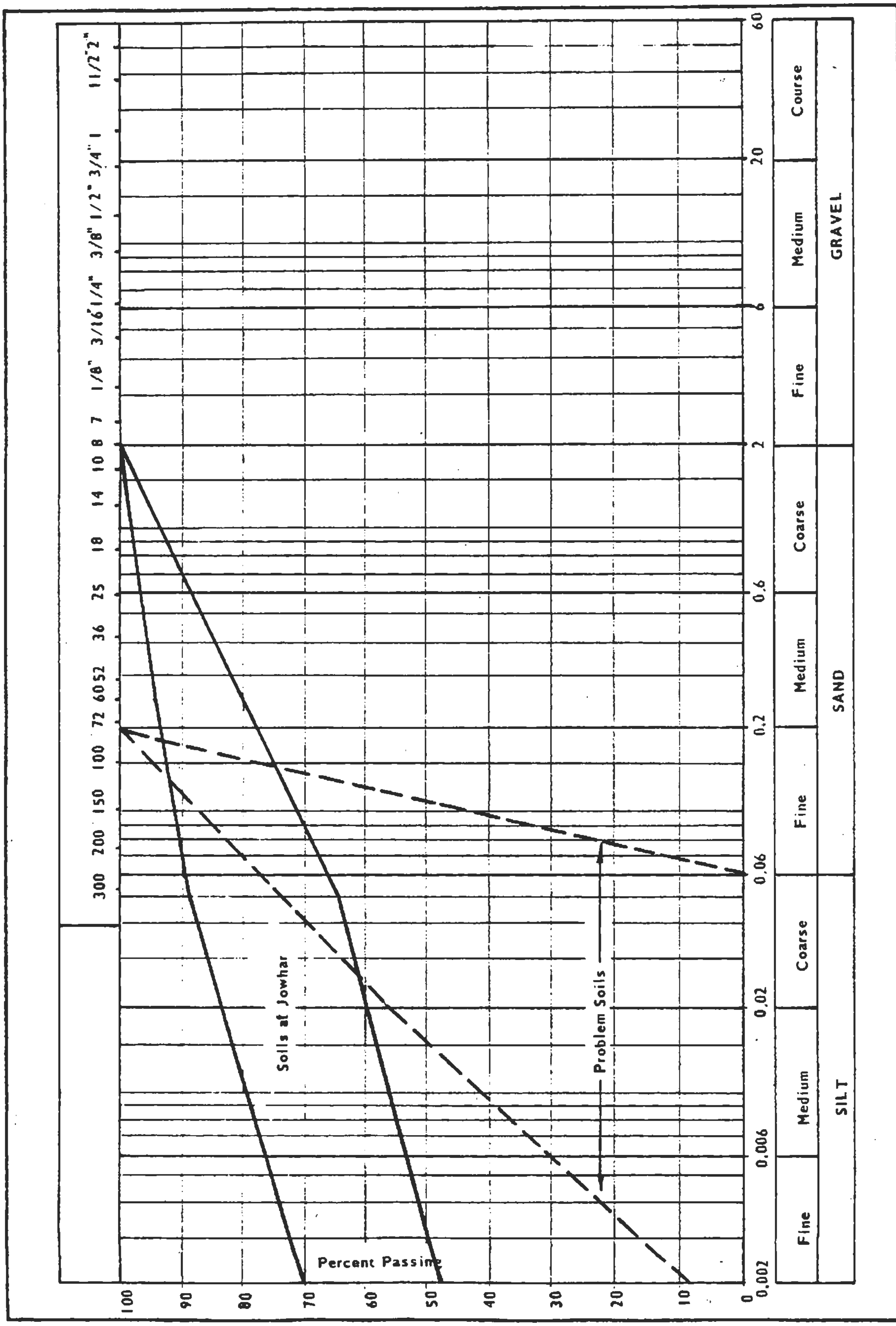
12.5.5 Construction Methods

Buried pipe drains should be laid by a specialist trenching machine. This will excavate a narrow trench and allow accurate placing of the pipe and gravel, even below the watertable. This will cause much less disruption of the field, use less gravel, and result in better laid drains than would the use of a general purpose hydraulic excavator. A trenching machine is preferred to a trenchless machine because the excavated trench itself assists drainage, and drains laid by a trenchless machine in sandy clay and clay soils have been found to have reduced discharges during the early years of operation (Naarding, 1978).

Level and grade control with a drainage machine may be achieved by use of a laser system as recommended for the land levelling works, or by lining up target level stakes (boning rods).

FIGURE 1.12.3

Particle Size Grading for Soils at Jowhar



Recommended Grading for Gravel Surround

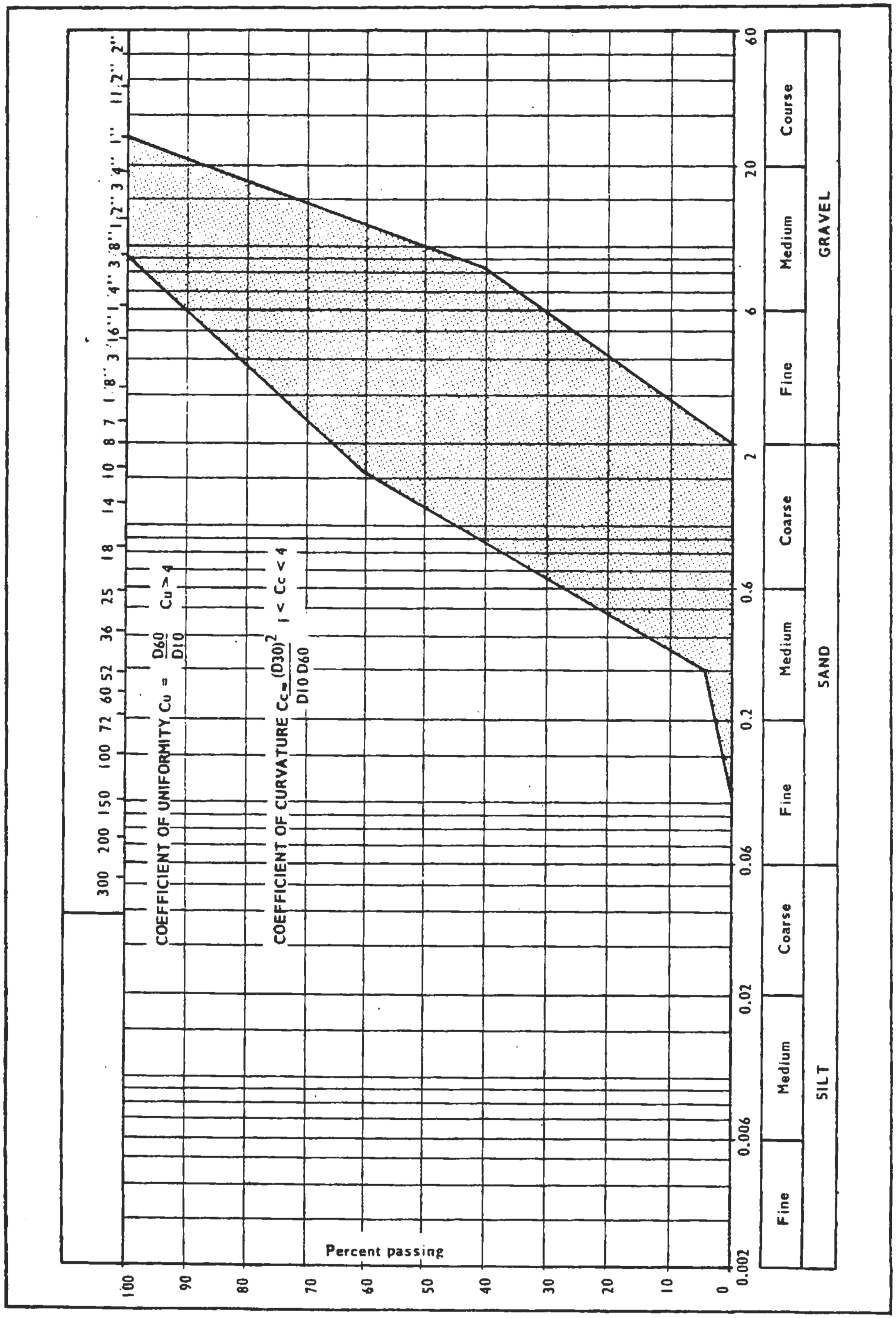
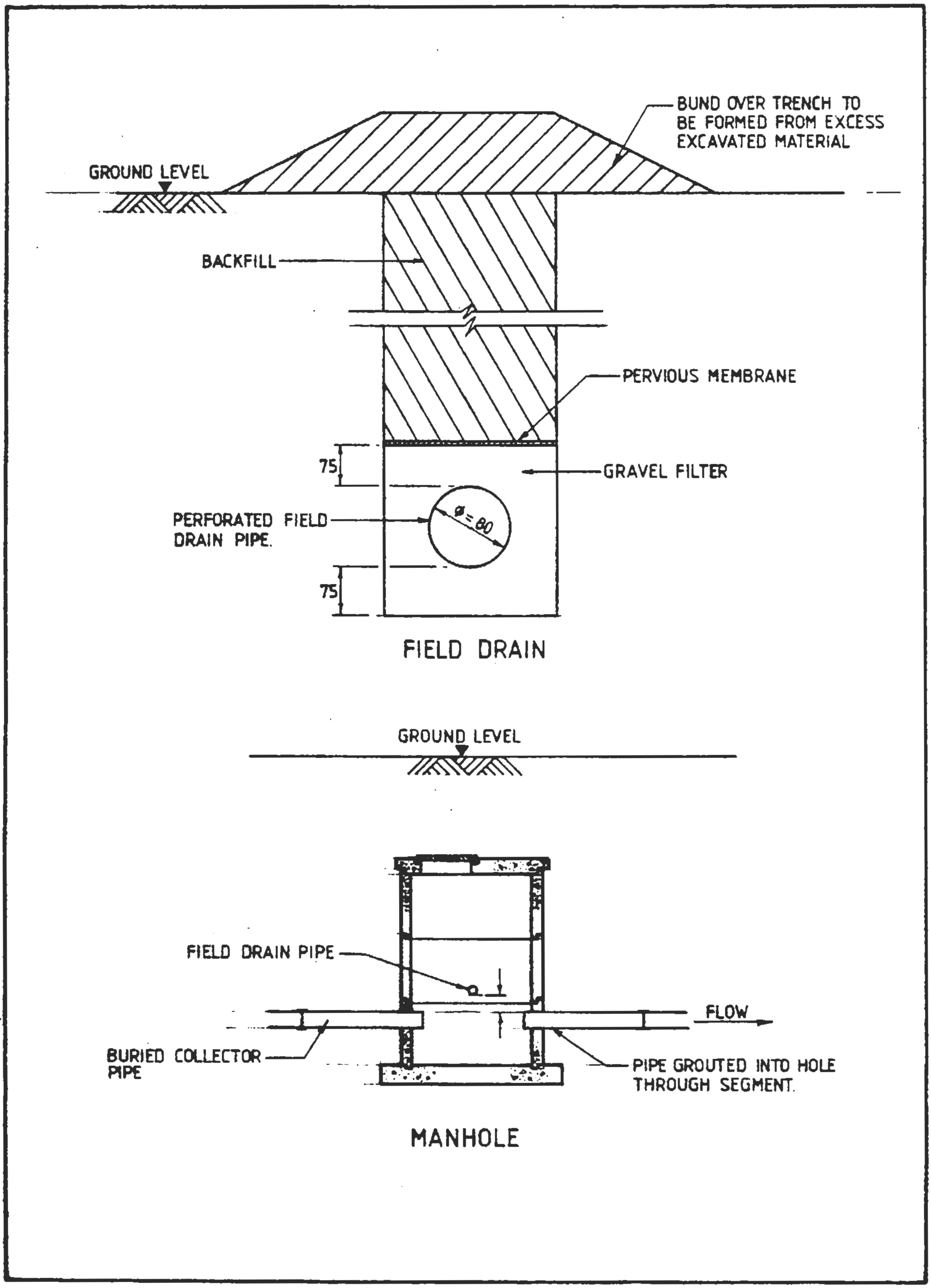


Figure I.12.5

Buried Field Drains and Buried Collector Drains



12.6 Deep Collector Drains

12.6.1 Objectives and Drainage Rate

The positions of the deep collector drains are shown in Figures I.6.2 and I.6.3. Deep collectors are to be constructed only on the selected cane fields served by the West drain, and are of two types:

- a deep collector across the foot of each field, to be constructed as part of the capital investment, without field drains;

- an intermediate deep collector across the middle of each field, to be constructed on those fields which are found to require buried field drains, at the same time as the field drains.

The deep collector across the foot of each field has the following purposes :

- to intercept groundwater and discharge it into the main drain system;

- to provide the basic infrastructure to conduct water from subsurface field drains (which may be installed at some future time) to discharge into the main drainage system.

The purpose of the intermediate deep collector is primarily to conduct water from subsurface field drains (installed at the same time) to the main drainage system. Groundwater interception is of minor importance for this drain because the buried field drains would perform this function.

For these purposes either open collectors or buried collectors could be used. These options are discussed in Sections 12.6.2 and 12.6.3 below, and recommendations are made in Setion 12.6.4.

In addition to the drainage objectives detailed above, the deep collectors have been included in the programme to provide suitable material for canal construction. This would be provided by open collectors, but not buried collectors.

Deep collector drains are to be installed in all the selected cane fields in the West drain area. The design subsurface drainage rate for this area is 1.8 mm/d as derived in Section 12.3, and this is used as the design discharge for the buried collectors. If buried collectors are used, surface drains would discharge into the main drains, and therefore surface runoff has not been included in the design discharge for the buried collectors. Open collectors however would be used to convey water from the surface drains to the main drains, and therefore are designed for the steady state condition of subsurface drainage flow and regular not peak surface runoff, at the design subsurface drainage levels. (Peak rates of surface runoff would cause higher water levels and poor subsurface drainage for several days, but this should not cause any problems.) The regular surface runoff derives from irrigation only, and is estimated at 10% of the gross field water requirements (following Section 12.3) which amounts to 0.1 x 2 207 = 221 mm/year or 0.6 mm/d.

Therefore the design drainage rates for deep collectors are as follows:

- for open collectors $q = 1.8 + 0.6 = 2.4$ mm/d
  $= 0.28$ l/s/ha

- for buried collectors $q = 1.8$ mm/d
  $= 0.21$ l/s/ha

12.6.2 Open Collector Drains

Open collectors would be designed with a trapezoidal cross section as shown in Figure I.12.6. The bed width would be 1 m and side slopes 1 vertical : 1.2 horizontal. The design bed level would normally be a minimum of 2.5 m below ground level with a minimum bed slope of 0.25 m/km. This will enable field drains to be installed at 2 m depth with an outfall level 0.4 m minimum above collector drain bed level. These design criteria for conducting the subsurface water from field drains to the main system would well satisfy the requirement for groundwater interception (wetted perimeter 1 to 2 m). The open collector drains would also be used to conduct field runoff (excess irrigation or rainfall) from the surface drains to the main drain system and would provide a buffer storage of about 2 days which might be useful in case of any breakdown of the main drainage system.

Open collectors would provide a valuable source of suitable material for rehabilitating existing canals and constructing new tertiaries.

A major problem of open collector drains is the high maintenance requirement to prevent the drains becoming blocked by reeds and weed growth. This is particularly important at Jowhar where the Estate already has problems maintaining the main drainage system. Mechanical methods (using tractor-mounted flail mowers and hydraulic excavators with rakes) are likely to cause damage to field drain outlets and therefore carefully controlled trials of chemical methods are recommended leading to the introduction of safe methods of weed control.

Open collectors are well suited to the requirements for the deep collector across the foot of each field. The existing open collector drain would be enlarged and deepened to the design section. The spoil from this work would be used for canal and road construction which is to be done at the same time. Field drains could be installed later and joined into the open collector by the same procedure as for drains installed at the same time as the open collector.

These advantages do not apply to the intermediate collector because there is no existing midfield drain, canal and road construction would be finished when most of the intermediate collectors are installed, and field drains would be installed at the same time.

12.6.3 Buried Collector Drains

Buried collector drains may be corrugated or straight walled uPVC pipe, depending on cost. For this study, imported corrugated pipe has been assumed, but local manufacture of straight walled pipe might also be considered. The pipe diameter depends on the area drained as shown in Table I.12.6.

As described above in Section 12.6.1, two situations are to be considered: the deep collector drain across the foot of each field, and the intermediate deep collector drain in midfield.

The deep collector drain across the foot of each field would be intended to intercept groundwater, and therefore slotted pipe would be required with a gravel filter and synthetic pervious membrane, like a field drain. The filter and membrane are not required for the intermediate deep collector, and the pipe need not be slotted because its function is not to intercept groundwater but to convey water from the buried field drains to the main drains. Figure I.12.5 shows typical cross sections.

Figure I.12.6

Typical Cross Section of Surface Drain and Deep Open Collector Drain

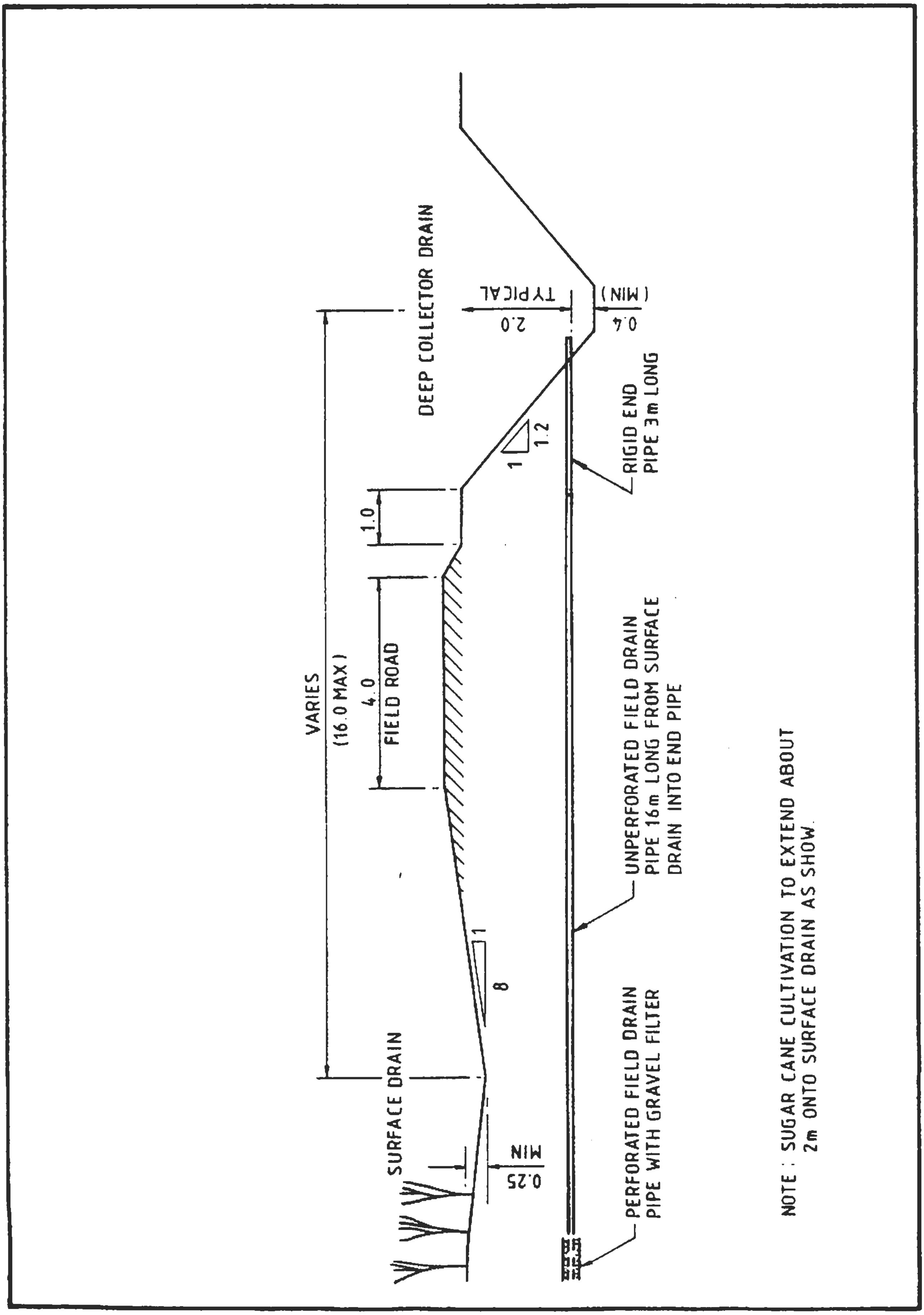


TABLE I.12.6

Discharge and Drained Area for Buried Collector Pipes

Pipe diameter Nominal (mm)	Internal (mm)	Pipe discharge (l/s)	Area drained (ha)
100	92	1.1	5
125	115	2.3	11
160	147	5.1	24
200	184	10.7	51

Note : Discharges calculated from the formula

$$Q = 422\, D^{3.26}\, i^{0.667}$$

for corrugated pipe, with allowances for spatially varied flow and 13 mm siltation.

The minimum slope on the pipe would be 0.5 m/km which is normally steeper than the ground slope. The collector drain pipe would be installed with an invert level of 2.1 m minimum below ground level. (Normally the minimum depth will apply at the upstream end of the drain.) This will enable field drain to be installed at 1.85 to 2.2 m depth with a minimum drop of 0.1 m from field drain invert level to collector drain soffit level. Manholes would be required at the junction between the field drain and the collector drain so that field drains (and the collector drains) could be cleaned if necessary to remove silt or iron ochre deposits. The manholes should be buried to prevent any interference, and metal manhole covers (or other 'tell tales') would be used to enable their position to be located easily by using a metal detector (See Figure I.12.5). At the outfall of the collector drain into the main drain a 0.2 m minimum drop is required from pipe invert to drain water level.

Buried collectors would be installed by using specialist trenching equipment. Modern machines have a variable width (eg. 0.22 m to 0.45 m) and changing from one width to another would involve changing the chains and cutting blades. This can be done on site, and it would be possible to use one machine for both buried collectors and buried field drains.

A buried collector system should require little maintenance. The silt traps at manholes should be cleaned periodically, and the pipe between manholes could be rodded if necessary. The outfall pipes into the main drains are vulnerable to damage by mechanical plant clearing the main drain, and the positions of the outfalls would need to be clearly marked.

The deep collector across the foot of the field is to be installed initially without an intermediate deep collector and without buried field drains. Therefore large and expensive pipe sizes would be needed and it would require a gravel filter. As a result the collector would be relatively expensive. In addition there would be practical problems making future connections of buried field drains to the buried collector. Therefore buried collectors are not particularly well suited to the requirements for the deep collector across the foot of the field.

Buried collectors are more suitable for the intermediate collector. This is to be installed at the same time as 300 m field drains and therefore would require smaller pipe sizes and no gravel. There is the additional advantage that the buried collector could be installed relatively easily in cane fields without loss of land.

12.6.4 Relative Costs of Open Collector Drains and Buried Collector Drains

(a) Direct Capital Cost

Earthworks requirements for deepening an existing open collector drain is estimated at 8 m^3/m cut, on average, which gives an indicative capital cost of SoSh 400/m (mid 1983 prices, construction by an international contractor). The requirements for a new intermediate collector are estimated at 10 m^3/m and SoSh 500/m or some SoSh 17 000 per ha gross.

Indicative capital costs of buried collector drains are shown in Tables I.12.7 and I.12.8. These show that, excluding the capital cost of the trenching machine, the cost of an intermediate buried collector drain would be some SoSh 16 000 per ha with the standard layout or SoSh 11 000 per ha with a revised layout which enables smaller diameter pipe to be used (Figure I.12.7).

In the circumstances at Jowhar it is reasonable to neglect the trenching machine cost because use would be made of the machine which has been purchased for laying field drains. (Strictly, a dual purpose trencher for field drains and collector drains might be up to 20% more expensive than one capable of laying field drains only. Thus SoSh 0.6 million could be allowed for the trenching machine cost. If the machine is used for 10 km of intermediate collector this would add SoSh 60 per metre to the costs given in Table I.12.7. The costs per ha would then amount to SoSh 18 000 for the standard layout or SoSh 14 000 for the revised layout.)

The direct capital costs of open drains and buried drains are thus very similar for the intermediate collector. A buried collector laid to the revised layout appears to be the cheaper solution on the assumptions made above but a contingency allowance should probably be added to reflect the greater uncertainty of the buried collector drain costs.

As discussed in Section 12.6.4, buried collectors are not very suitable for the deep collector at the foot of the field, and it can be concluded from the above that the costs for the larger pipe size and gravel filter required would be considerably higher than for an open collector.

(b) Indirect Capital Cost

A major problem with buried collectors is that their minimum slope is steeper than for open collectors and as a result the design water level in the main drain must be some 0.6 m lower at the collector drain junction (based on the

Figure I.12.7

Schematic Field Layout with Buried Collector Drains

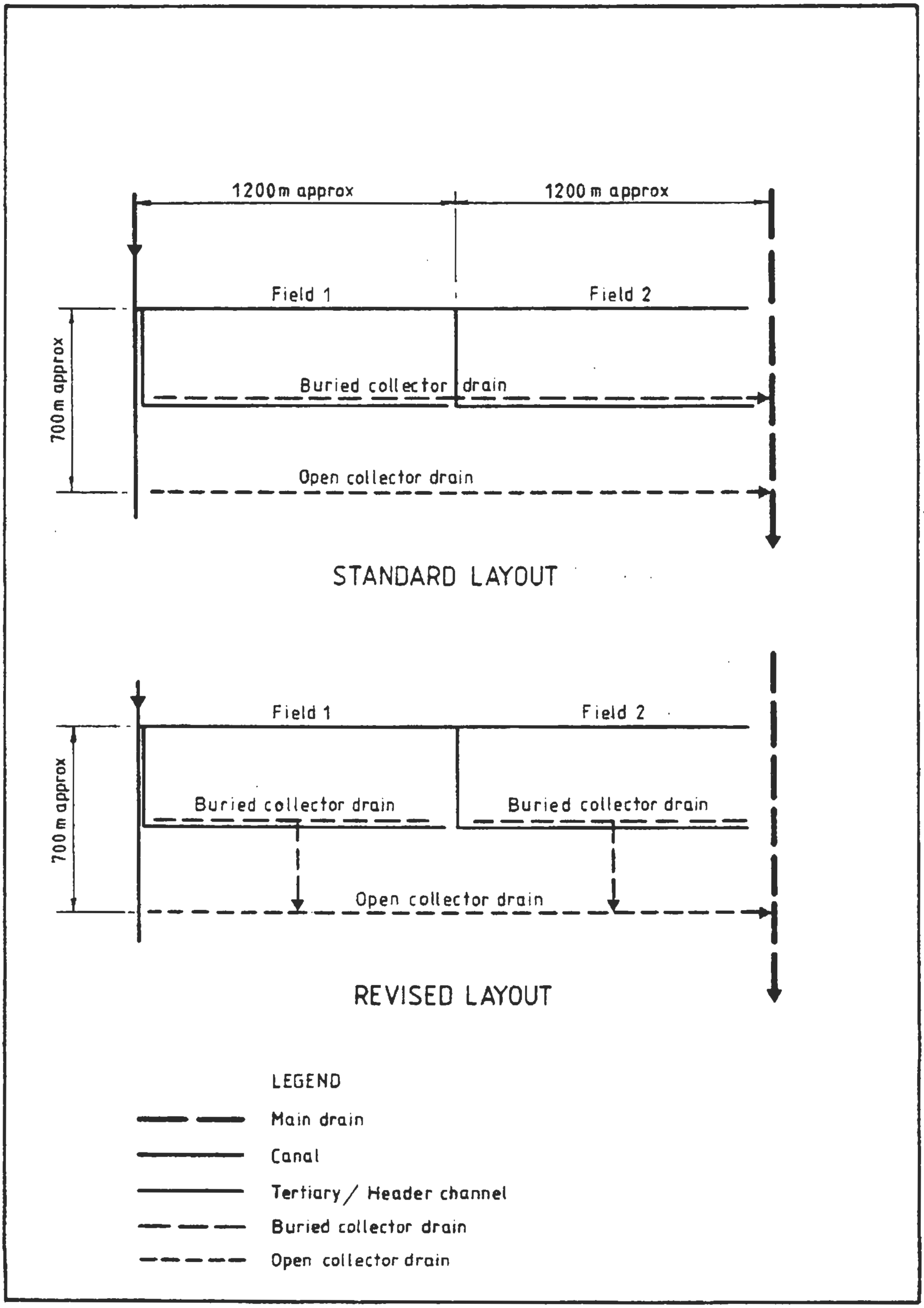


TABLE I.12.7

Indicative Costs per metre of Buried Collectors of Various Sizes

Pipe diameter	Costs per metre (SoSh) 100 mm	125 mm	160 mm	200 mm
Pipe	46	77	112	224
Gravel filter (75 mm)	39	44	53	64
Membrane	3	3	3	4
Fuel	3	3	3	3
Maintenance	1	1	1	1
Support machines	2	2	2	2
Labour	1	1	1	1
Supervision 10%(1)	10	13	18	30
Contractor's overheads 23%(1)	24	33	44	76
Contractor's profit 9%(1)	12	16	21	36
Total(2)	141	193	258	441
Total cost without filter and membrane(2)	99	146	202	373

Note: (1) Percentages as estimated for an international contractor on Homboy Irrigated Settlement Project, Somalia (MMP, 1980).

(2) Costs exclude capital cost of trenching machine.

standard layout). Increasing the depth of a deep outfall drain by 0.6 m would increase the earthworks' cost by approximately 40% or about SoSh 4 000 per ha of cane. This could be reduced by using minimum slopes on the main drain and relaxing the criteria on certain fields, but nevertheless to use buried collectors throughout would cause substantial increases in the main drain costs. It would also necessitate lower levels at the intercepting drain and drainage pump station, and increased costs as a result.

TABLE I.12.8

Indicative Costs of Buried Intermediate Collector

Pipe size	Cost per metre(1)	Standard layout(2) Pipe length (m)	Standard layout(2) Cost SoSh x 10^3	Revised layout(2) Pipe length (m)	Revised layout(2) Cost SoSh x 10^3
100 mm	99	nil	-	nil	-
125 mm	146	360	53	1 440	210
160 mm	202	440	89	860	174
200 mm	373	900	336	600	224
2 x 200 mm	746	600	448	nil	-
Manholes			161		161
Total		2 300	1 087	2 900	769
Cost per ha gross(1)			15.8		11.1

Note (1) Excluding the capital cost of the trenching machine.

(2) The standard and revised layouts are shown schematically in Figure I.12.7.

The depth of the main drains however is determined by collector drain outfalls from critical low lying fields, and some other collectors could be buried collectors rather than open collectors, without requiring an increase in main drain depth and cost.

(c) Maintenance Costs

Maintenance costs are considerably higher for open collectors than for buried collectors because of the high cost of cleaning weed growth.

For cleaning weed from an open collector drain by mechanised methods the costs of the necessary operations (eg. by tractor-mounted flail mower and hydraulic excavator with lifting bucket) would amount to about SoSh 2 800 per km. The weeds would be left on the drain bank. Assuming clearance is required four times per year this is equivalent to an annual cost of SoSh 11 200 per km or SoSh 400 per ha gross.

The present value of this maintenance cost is approximately SoSh 3 300 (discount rate 12%).

For buried collectors the annual maintenance costs are estimated at only SoSh 70 per ha or a present value of approximately SoSh 600.

The above costs are only indicative but demonstrate the scale of savings in maintenance costs which can be achieved by using buried collectors. If maintenance costs are shadow priced compared to capital costs to represent the relative difficulty the Estate authorities would have in financing recurrent costs, then the savings would be even higher. This difference in maintenance requirements supports the recommendation by FAO (1979):

'Where physical and economic conditions permit, pipe collector drains are always preferable to open ditch collectors for the collection of subsurface drainage effluent.'

(d) Conclusions

Buried collectors would be prohibitively expensive for the collector drain at the foot of the field. Deepening the existing open collector seems the best solution.

With a deepened open collector at the foot of the field, the intermediate collector could be either an open or a buried drain (to the revised layout) at about the same direct capital cost. The buried drain would save about SoSh 330 per ha in annual maintenance cost. In some cases however the buried drain would require a deeper main drain and possibly increased pumping costs.

12.6.5 Recommendations

The deep collector drain at the foot of the field should be an open drain constructed by deepening the existing drain.

The intermediate deep collector drain should be a buried drain to the revised layout wherever the main drain design permits this. Otherwise an open drain should be used.

The main drain design work should include preparation of a report recommending procedures and equipment for the phased construction of the intermediate collectors and buried field drains probably by the Estate. The buried collectors' economic advantages of reduced maintenance costs and reduced land take should also be taken into account in this and in the consequent design of the main drains.

For the cost estimate in Chapter 14 it has been assumed that open collector drains are used throughout. Any subsequent change will have minimal effect on the economic analysis.

12.7 Main Drains

12.7.1 Objectives and Drainage Rate

(a) Deep Main Drains

The West drain, west branch drain (W2), and west sub-branch drain (W2/2) are to be remodelled to remove the subsurface drainage flows from all the fields served and to convey these by gravity to the Intercepting drain. This includes both the proposed cane fields and the non-cane fields.

These deep main drains would also be used to conduct surface runoff from the surface drains and collector drains to the Intercepting drain.

There are thus two design conditions:

- steady state drainage at the subsurface design levels;
- peak surface drainage.

For steady state drainage, the design drainage rate is the sum of the subsurface drainage rate and the regular surface runoff, as for deep open collector drains (see Sections I.12.3 and I.12.6.1):

$$q = 1.8 + 0.6 = 2.4 \text{ mm/d}$$

$$= 0.28 \text{ l/s/ha}$$

The drains and structures must be capable of passing this discharge at the design water levels for subsurface drainage, which would enable deep collectors and deep field drains to discharge by gravity as designed.

For peak surface drainage the design rate is 0.9 l/s/ha gross as derived in Section 12.2. After storms causing such runoff water levels would rise and restrict subsurface drainage for several days. This is acceptable, and the main drain designed for subsurface drainage would be able to pass the peak surface flows satisfactorily at higher water levels. Structures however will have to be designed for the higher peak surface drainage rates.

(b) Shallow Main Drains

The Middle drain and East drain are to be remodelled to provide surface drainage only from all the fields served (Section 11.2). Design flows are to be conveyed by gravity to the Intercepting drain.

The design surface drainage rate for a 1 in 5 year storm with a 90% area reduction factor is 0.9 l/s/ha gross, as derived in Section 12.2.

12.7.2 Design Criteria

Both deep and shallow main drains would be designed by the Manning equation with the parameters detailed below:

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

- Manning's n = 0.033

- bedwidth = 3 x water depth (minimum 0.5 m)

- sideslope 1 vertical : 1.2 horizontal

- minimum slope 0.10 m/km

- maximum slope limited by permissable tractive force 4 N/m^2

Typical cross sections are shown in Figure I.12.8.

Structure head losses have been assumed as follows:

- culvert : 0.05 m

- drain underpass : 0.2 m

- drain junction : 0.2 m minimum

12.7.3 Preliminary Designs

Preliminary designs have been prepared for the main drains and used for costing purposes.

These designs allow for the drainage of both the selected cane fields and non-cane fields in the area served by each drain. The West, W2 and W2/2 drains were designed for subsurface drainage, and the Middle and East drains were designed for surface drainage.

Maximum design water levels were determined as follows. In each field the minimum field level (after land levelling) was identified from the 1983 survey map. A drop of 2.5 to 2.7 m was subtracted for deep main drains, or 0.4 to 0.6 m for shallow main drains, depending on distance from the low spot to the main drain. This gave maximum values for the main drain water level.

The preliminary designs are presented in Tables I.12.9 to I.12.13.

12.8 Main Drain Outlets, Interceptor Drain and Drainage Disposal System

12.8.1 Main Drain Outlets

These works have been designed under the Jowhar Offstream Storage project and partially constructed as described in Section 8.5. The designs would need to be checked for compatibility with the main drain and the works completed.

Although the Middle and East drains are now to be shallow drains, their design levels at the outlets will be too low to allow year-round drainage by gravity, as shown below :

drain design water level at outlet	98.0 - 98.1 m
reservoir maximum water level	99.8 m (Estate datum)

Construction of a special dual structure would allow drainage water to be discharged by gravity into the reservoir when reservoir levels were below 97.9 m (Estate datum) or otherwise to flow into the Intercepting drain. However,

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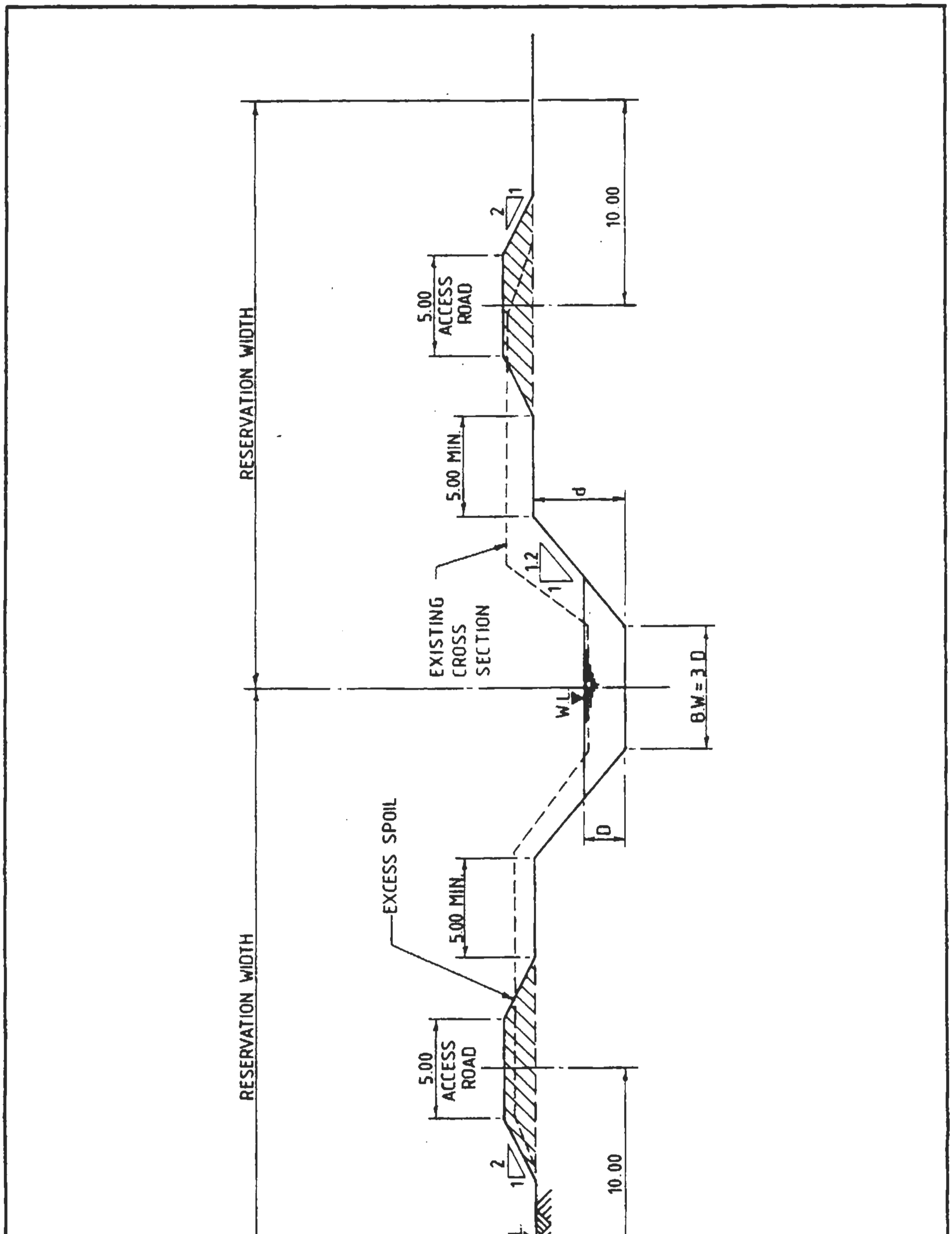
Preliminary Design of West Drain

Km	Structure	Design water level u/s and d/s of structure	Design discharge (m^3/s)	Slope (m/km)	Bed width (m)	Depth (m)	Bed level
11.600	-	99.13	0.05	0.44	0.75	0.25	98.88
11.300	Culvert	99.0	0.05	0.44	0.75	0.25	98.75
		98.95	0.10	0.44	0.96	0.32	98.63
10.100	Culvert	98.42	0.10	0.44	0.96	0.32	98.10
		98.37	0.14	0.44	1.09	0.36	98.01
9.200	Culvert	97.98	0.14	0.44	1.09	0.36	97.62
		97.93	0.42	0.31	1.77	0.59	97.34
6.750	Culvert	97.17	0.42	0.31	1.77	0.59	96.58
		97.12	0.66	0.33	2.07	0.69	96.43
4.000	Culvert	96.22	0.66	0.33	2.07	0.69	95.53
		96.17	0.70	0.33	2.10	0.70	95.47
3.800	W2 junction	96.10	0.70	0.33	2.10	0.70	95.40
			1.07	0.10	3.09	1.01	95.07
2.400	Culvert	95.96	1.07	0.10	3.09	1.03	94.93
		95.91	1.21	0.10	3.24	1.08	94.83
0.500	Footbridge	95.72	1.21	0.10	3.24	1.08	94.64
		95.67	1.23	0.10	3.24	1.08	94.59
0.000	Junction (intercepting drain)	95.62	1.23	0.10	3.24	1.08	94.54

Note : Drain designed for subsurface drainage at rate of 0.28 l/s/ha; drain structures are to be designed for surface drainage rate of 0.9 l/s/ha (see text).

figure 1.12.8

Typical Cross Section of Main Drain



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Preliminary Design of Drain W2

Km	Structure	Design water level u/s and d/s of structure	Design discharge (m^3/s)	Slope (m/km)	Bed width (m)	Depth (m)	Bed level
7.700	-	99.35	0.04	1.18	0.57	0.19	99.16
6.900	Culvert	98.40	0.04	1.18	0.57	0.19	98.21
		98.35	0.12	0.59	0.96	0.32	98.03
5.300	Culvert	97.40	0.12	0.59	0.96	0.32	97.08
		97.35	0.16	0.10	1.50	0.50	96.85
4.150	W2/2	97.23	0.16	0.10	1.50	0.50	96.73
		97.23	0.23	0.10	1.74	0.58	96.65
4.100	Culvert	97.22					
		97.17	0.23	0.10	1.74	0.58	96.59
3.700	Drain underpass	97.13	0.24	0.10	1.77	0.59	96.37
		96.93					
2.350	Drain underpass	96.79	0.24	0.10	1.77	0.59	96.20
		96.59	0.24	0.10	1.77	0.59	96.00
1.200	Culvert	96.47	0.24	0.10	1.77	0.59	95.88
		96.42	0.24	0.10	1.77	0.59	95.83
0.000	Junction (west drain)	96.30	0.24	0.10	1.77	0.59	95.71
		96.10	0.24	0.10	1.77	0.59	95.51

Note: Drain designed for subsurface drainage at rate of 0.28 l/s/ha; drain structures are to be designed for surface drainage rate of 0.9 l/s/ha (see text).

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Preliminary Design of Drain W2/2

Km	Structure	Design water level u/s and d/s of structure	Design discharge (m^3/s)	Slope (m/km)	Bed width (m)	Depth (m)	Bed level
2.600	-	98.70	0.03	0.75	0.50	0.16	98.54
2.200	Drain underpass	98.40 98.20	0.03 0.05	0.75 0.42	0.50 0.75	0.16 0.25	98.24 97.95
1.200	Culvert	97.78 97.73	0.05 0.06	0.42 0.25	0.75 0.87	0.25 0.29	97.53 97.44
0.000	Junction (W2)	97.43 97.23	0.06	0.25	0.87	0.29	97.14

Note: Drain designed for subsurface drainage at rate of 0.28 l/s/ha; drain structures are to be designed for surface drainage rate of 0.9 l/s/ha (see text).

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Preliminary Design of Middle Drain

Km	Structure	Design water level u/s and d/s of structure	Design discharge (m^3/s)	Slope (m/km)	Bed width (m)	Depth (m)	Bed level
10.900	Head of drain	100.50	0.31	0.44	1.47	0.49	100.01
9.400	Culvert	99.84	0.31	0.44	1.47	0.49	99.35
		99.79	0.74	0.44	2.04	0.68	99.11
8.100	-	99.22	0.74	0.44	2.04	0.68	98.54
		99.22	0.74	0.10	2.70	0.90	98.32
7.000	Culvert	99.11	0.74	0.10	2.70	0.90	98.21
		99.06	1.11	0.12	3.00	1.00	98.06
4.300	Culvert	98.74	1.11	0.12	3.00	1.00	97.74
		91.69	1.18	0.12	3.27	1.09	97.60
1.700	Culvert	98.37	1.18	0.12	3.27	1.09	97.28
		98.32	1.18	0.12	3.27	1.09	97.23
0.000	Junction intercepting drain	98.12 95.27	1.18	0.12	3.27	1.09	97.03

Note: Drain designed for surface drainage at rate of 0.9 l/s/ha (see text).

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Preliminary Design of East Drain

Km	Structure	Design water level u/s and d/s of structure	Design discharge (m^3/s)	Slope (m/km)	Bed width (m)	Depth (m)	Bed level
15.200	Drain underpass	101.40 101.20	0.78	0.24	2.34	0.78	100.42
10.800	Drain underpass	100.14 99.94	0.78 1.04	0.24 0.14	2.34 2.88	0.78 0.96	99.36 98.98
9.100	Culvert	99.70 99.65	1.04 1.29	0.14 0.10	2.88 3.30	0.96 1.10	98.74 98.55
7.500	Culvert	99.49 99.44	1.29 1.48	0.10 0.10	3.30 3.48	1.10 1.16	98.39 98.34
6.700	Drain underpass	99.36 99.16	1.48 1.63	0.10 0.10	3.48 3.63	1.16 1.21	98.20 97.95
3.900	Culvert	98.88 98.83	1.63 1.63	0.10 0.10	3.63 3.63	1.21 1.21	97.67 97.62
3.100	Culvert	98.75 98.70	1.63 1.98	0.10 0.21	3.63 3.39	1.21 1.13	97.54 97.57
0.900	Culvert	98.24 98.19	1.98 2.20	0.21 0.21	3.39 3.51	1.13 1.17	97.11 97.02
0.000	Junction (intercepting drain)	98.0 95.05	2.20	0.21	3.51	1.17	96.83

Note : Drain designed for surface drainage at rate of 0.9 l/s/ha.

hydrological analysis on the records from 1961 to 1973, which were used to plan the Jowhar Offstream Resevoir, show that, with the recommended operating procedures, the reservoir level would not fall below 97.9 m (Estate datum) in seven years out of twelve, and in the other years it would only occasionally fall below this level in March to May or July to August. Therefore special provision to enable gravity flow direct into the reservoir is not justified with the preliminary design drain levels.

12.8.2 Interceptor Drain, Drainage Pump Station and Disposal Channel

Like the main drain system the Interceptor drain, drainage pump stations and disposal channel are to serve two purposes :

- at subsurface drainage levels, to remove the daily flows resulting from subsurface drainage and regular surface runoff; these amount to 0.28 l/s/ha over 4 600 ha (West drain) plus 0.07 l/s/ha over 4 000 ha (Middle and East drains) totalling 1.6 m^3/s;

- at surface drainage levels, to remove the peak surface drainage flows from all the fields on the Estate (cane and non-cane fields); applying an area reduction factor of 0.9 to the main drain rate of 0.9 l/s/ha gives a requirement of 0.81 l/s/ha over 8 600 ha or a total of 7.0 m^3/s.

The drainage pump station is to comprise five pumps rated at 0.9 m^3/s at the subsurface drainage levels. This gives ample cover for the daily subsurface drainage requirement of 1.6 m^3/s, but is inadequate for the peak flows deriving from the design storm. In this situation the Intercepting drain would fill in less than 24 hours and the gravity drainage outfall culvert would then start to operate, discharging about 2.5 m^3/s into the Jowhar Offstream Reservoir. The pumped discharge will increase somewhat at higher water levels (lower heads). However, the rated capacity of the pump station with all five pumps operating (4.5 m^3/s) and the design capacity of the gravity drainage outfall culvert (2.5 m^3/s) together provide no safety margin over the design inflow (7.0 m^3/s). This clearly gives inadequate protection: one cannot, for instance, rely on all five pumps being operational at the time of the storm.

The disposal of peak surface drainage needs to be reviewed at the design stage, and it appears that additional provision will be required (e.g. a second gravity drainage outfall culvert) to provide drainage for the whole Estate. This is not expected to require any changes to the works previously designed under the Jowhar Offstream Storage project, and these should be completed as soon as possible. The project costs in Chapter 13 do not include for completion of these works, nor for any additional works required. The cost of additional works would be relatively small and would not affect the feasibility of the project;. they would also be primarily attributable to development of the non-cane areas.

12.8.3 Drainage Disposal System

(a) Previous Studies

The drainage disposal system, as described in Section I.8.3, was designed for the Jowhar Offstream Storage project assuming cane cultivation over the whole sugar estate with a drainable surplus of 2.8 m/d on the total area of 10 000 ha. This resulted in an annual drainage disposal volume of 102 Mm^3, and it was recommended that this water be pumped either into the offstream reservoir (when EC was less than 2 mmhos/cm) or into the drainage disposal area (when EC was above 2 mmhos/cm).

The various drainage disposal options were studied in some detail in the 1976 Study (Appendix XIV, MMP 1976), and it was concluded that acceptable salinity levels would be maintained in the reservoir if drainage water is only released into the reservoir from July to November when the drainage water should be less saline (due to the low irrigation water salinity and the higher rainfall runoff contribution). This practice would require a drain disposal area for the remaining more saline water with a volume of some 60 Mm^3.

(b) Proposed Design

The drainable surplus for the proposed rehabilitation is lower than previously assumed due to the anticipated improved irrigation efficiency, the smaller area under cane cultivation and the smaller area with subsurface drainage. The mean drainable surplus is taken as 2.7 mm/d on the 2 100 ha drained by the West drain (with deepened collector drains and some field drains); 0.9 mm/d on the remaining 3 800 ha of cane cultivation (allows for surface runoff from irrigation and ineffective rainfall), and 0.3 mm/d on the 2 400 ha of the Estate not under cane cultivation (where only runoff from ineffective rainfall is allowed for). This gives a total annual volume of drainage disposal water at 34 Mm^3 per year, see Section 14.8. Note that this assumes no irrigation runoff occurs from the non cane area and the annual volume then is one third of that originally envisaged in the design of the drainage disposal area.

It is recommended that the practice of releasing drainage water into the storage reservoir when salinity is less than 2 mmhos/cm is adopted. This will reduce the volume requiring separate storage in a disposal area to less than 30 Mm^3 per year; half of that originally envisaged. (Note that drainage salinity levels are now expected to be higher than those used in the 1976 computer model, due to the lower leaching percentage.)

It is now estimated that in order to dispose of these drainage volumes a disposal area is required which can provide an average water surface area of 10 km^2. This figure assumes that water will be lost both by evaporation (not of average rainfall) and by seepage (taking a seepage rate of 5 mm/d).

The disposal area identified for the Jowhar Offstream Storage project with embankments was estimated to provide storage with some 20 km^2 of surface area. With the smaller storage now required it is anticipated that no check embankments will be necessary.

The drainage disposal channel and pump station are designed to allow for a maximum pond level of 100.5 m, which is consistent with the embankment levels of the storage reservoir.

12.9 Drain Structures

The following are the principal structures required :

- collector drain culvert
- collector drain junction culvert
- main drain culvert
- main drain junction culvert
- main drain underpass

These are described in turn below.

12.9.1 Collector Drain Culvert

The collector drain culvert would have a design discharge less than 0.1 m^3/s and minimal head loss. A 0.45 m diameter pipe culvert would be used, with concrete pipe on a mass concrete bed and pitching at the inlet and outlet. Similar structures would be used on both existing shallow collector drains (where needed) and deep collector drains. The structure would be a similar type to the main drain culvert described in Section 12.9.3.

12.9.2 Collector Drain Junction Culvert

The collector drain junction culvert carries a design discharge normally less than 0.15 m^3/s (surface drainage from 150 ha gross) but must operate satisfactorily over a wide range of head loss. The pipe culvert shown in Figure I.12.9 is suitable for these conditions. It comprises a 0.45 m or 0.6 m diameter concrete pipe on a mass concrete bed, with mass concrete inlet and outlet boxes. Similar structures would be used on shallow collector drains (where a new culvert is needed) and on deep collector drains.

12.9.3 Main Drain Culvert

Pipe culverts would be used for design discharge up to about 1.5 m^3/s. These would be made of concrete pipe on a mass concrete bedding, with a pitched inlet and outlet. Pipe velocities at design discharge would be limited to 0.7 m/s. A typical structure is shown in Figure I.12.10. For larger discharges mass concrete slab culverts have been assumed for costing purposes. Reinforced concrete box culverts would be a possible alternative.

Similar structures would be used for both shallow and deep main drains.

12.9.4 Main Drain Junction Culvert

The main drain junction culvert must be able to accept a variable head loss across the structure. The proposed structure is a pipe culvert with a drop inlet and a USBR Type VI baffled outlet. Concrete pipe would be used, on a mass concrete bed, with a mass concrete inlet box and a reinforced concrete outlet.

The type structure is shown in Figure I.12.11. Similar structures would be used both on shallow main drains and on deep branch and main drains.

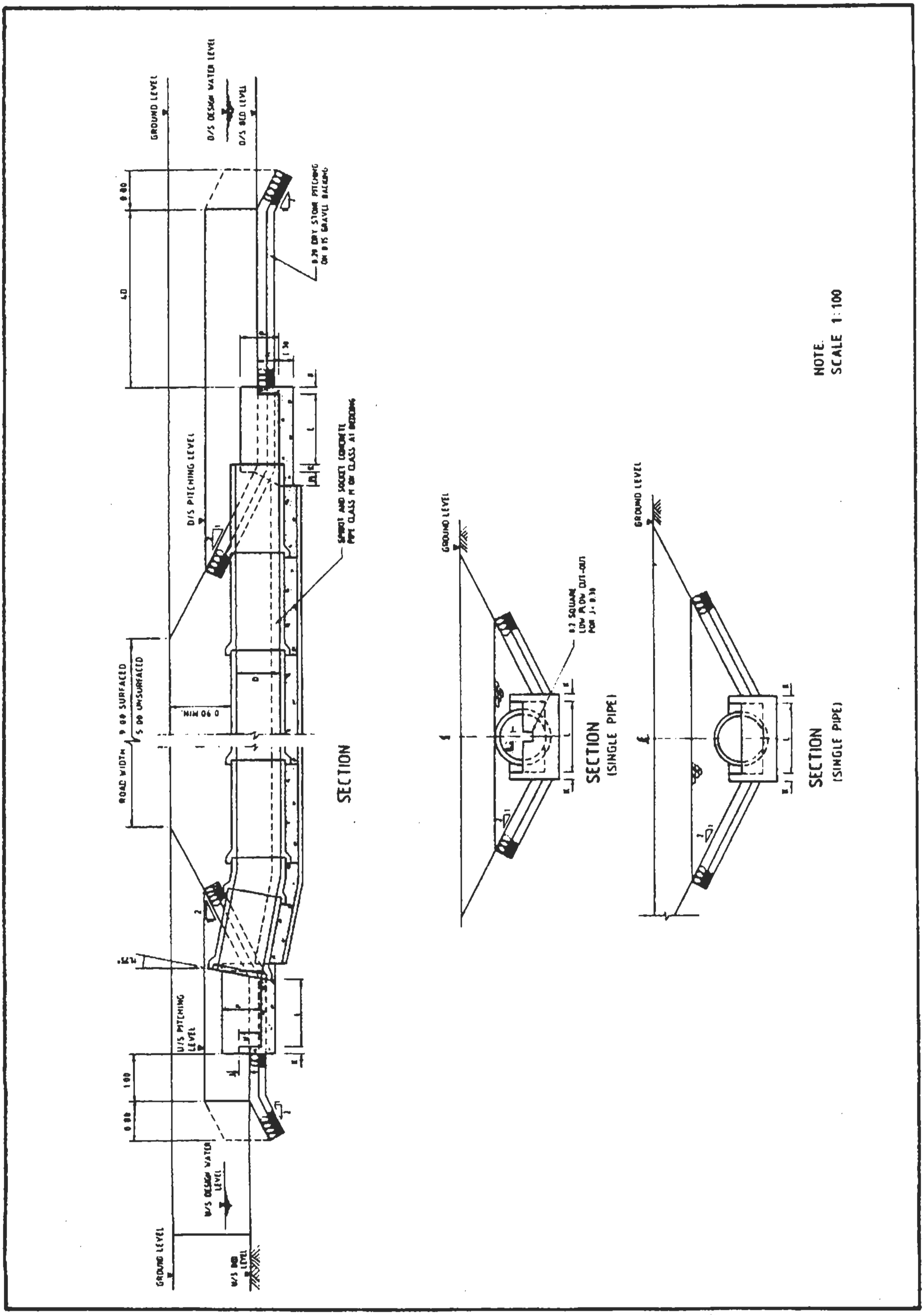
12.9.5 Main Drain Underpass

Drains would generally be put in underpasses under canals, rather than the other way round, because it should be possible without depressing the drain bed. There would be minimum head loss through the structure.

A pipe culvert would be used, comprising concrete pipe on a mass concrete bed and concrete inlet and outlet boxes. The pipe velocities at design discharge would be limited to 0.7 m/s.

FIGURE 1.12.9

Typical Collector Drain Junction Culvert



Typical Main Drain Culvert

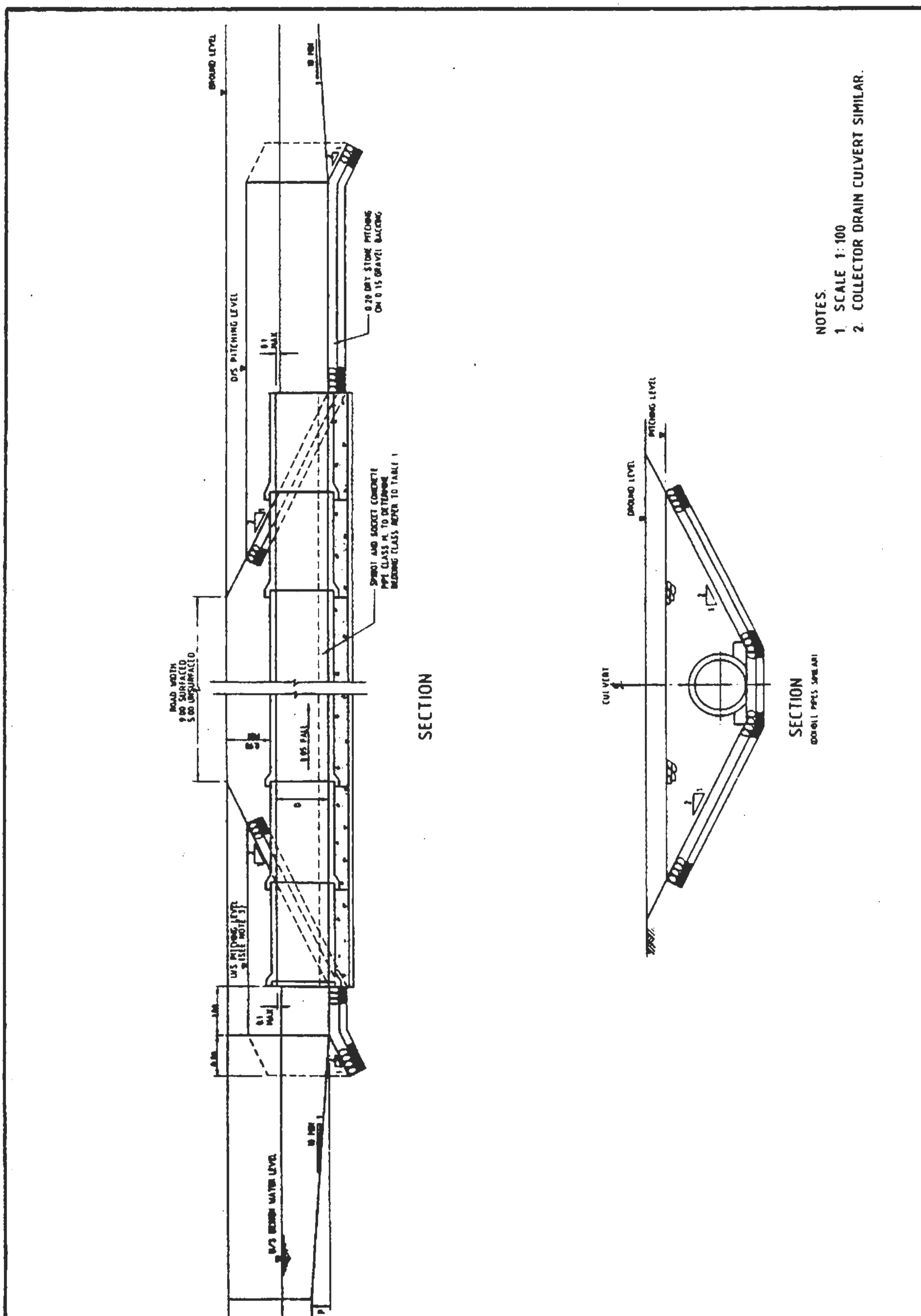
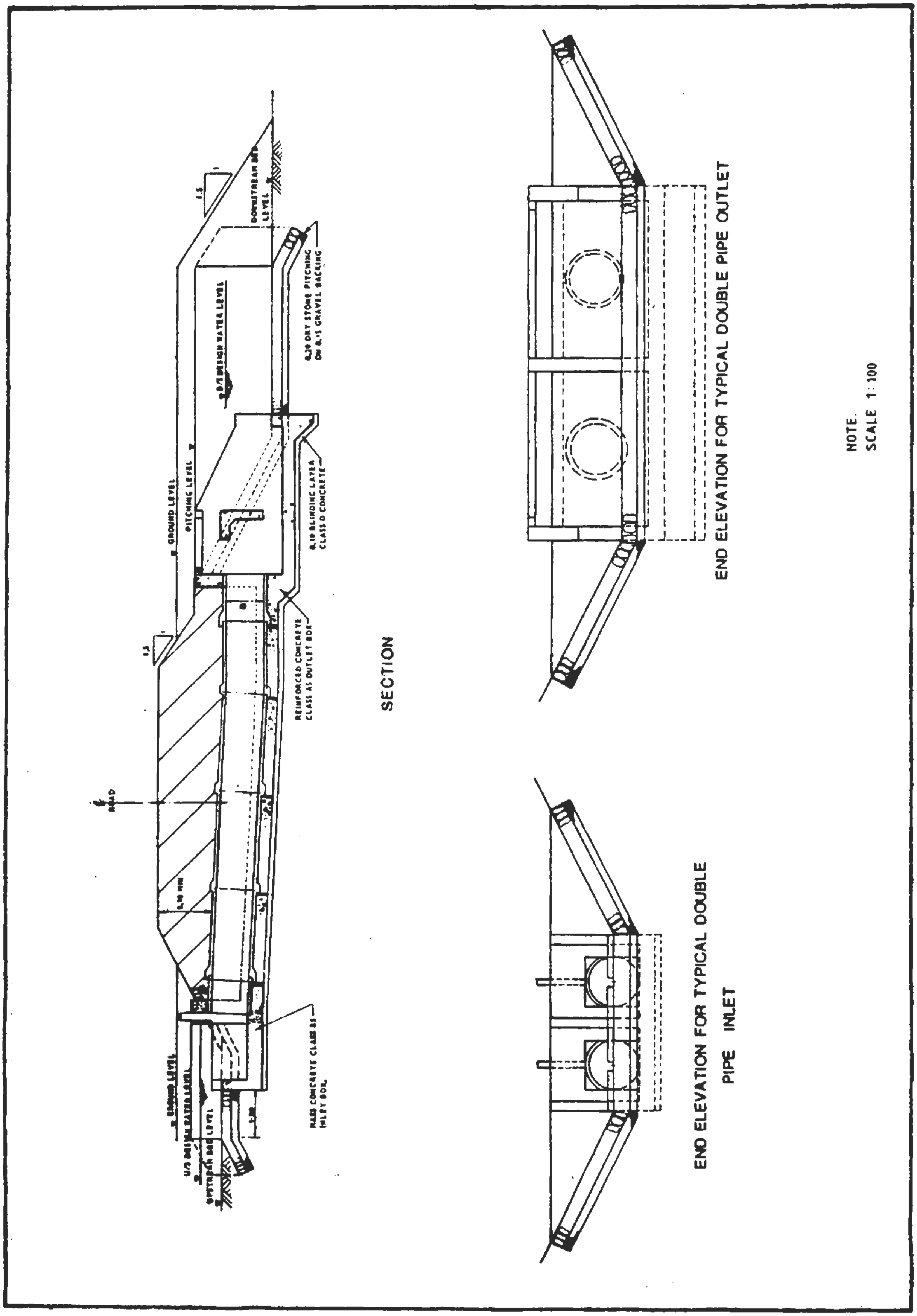


Figure 1.12.11

Typical Main Drain Junction Culvert



SECTION D

OPERATION AND MAINTENANCE

CHAPTER 13

OPERATION AND MAINTENANCE

13.1 Introduction

Many of the problems which the Estate is presently experiencing are the result of inadequate maintenance in the past and a shortage of trained operating staff. If the rehabilitation works proposed in this report are to be successful, it is essential that these problems are avoided in the future. This will be achieved by three measures:

- increasing the numbers and experience of operating staff and providing suitable training programmes;
- providing adequate and appropriate maintenance plant with suitably trained operators;
- incorporating into the rehabilitation works measures which will enable better control over irrigation and which will reduce the maintenance load (e.g. settling basin, new flow measurement structures, replacement control gates on existing structures, etc. - all as proposed in previous chapters).

The proposal for improving operating staff and maintenance plant are discussed in detail in the following sections.

13.2 Operating Staff

The proposed staffing for the rehabilitated scheme is given in Table I.13.1. Until such time as there are appropriately trained and qualified Somali staff available, it is proposed that expatriate personnel are engaged to fill the posts of head of irrigation section, distribution controller, maintenance controller, and surveyor. Otherwise all staff would be recruited locally. Details of the proposals for expatriate staff are given in Annex IX. The staff structure is illustrated in Figure I.13.1 and the key posts are discussed below.

(a) Head of Irrigation Section

The irrigation section head would report directly to the Agricultural Manager and would be responsible for overall control of all irrigation and drainage functions on the Estate, except field irrigation. This latter function would be directed by the individual farm managers.

The irrigation section would be divided into two main sub-sections, distribution and control, each with its own sub-section head.

(b) Distribution Controller

Under the general direction of the Section Head, the distribution controller would ensure that the irrigation system is operated to a planned schedule to meet the requirements of each farm. He would be in charge of all gate operators (canal attendants) and pump station attendants (both irrigation and drainage).

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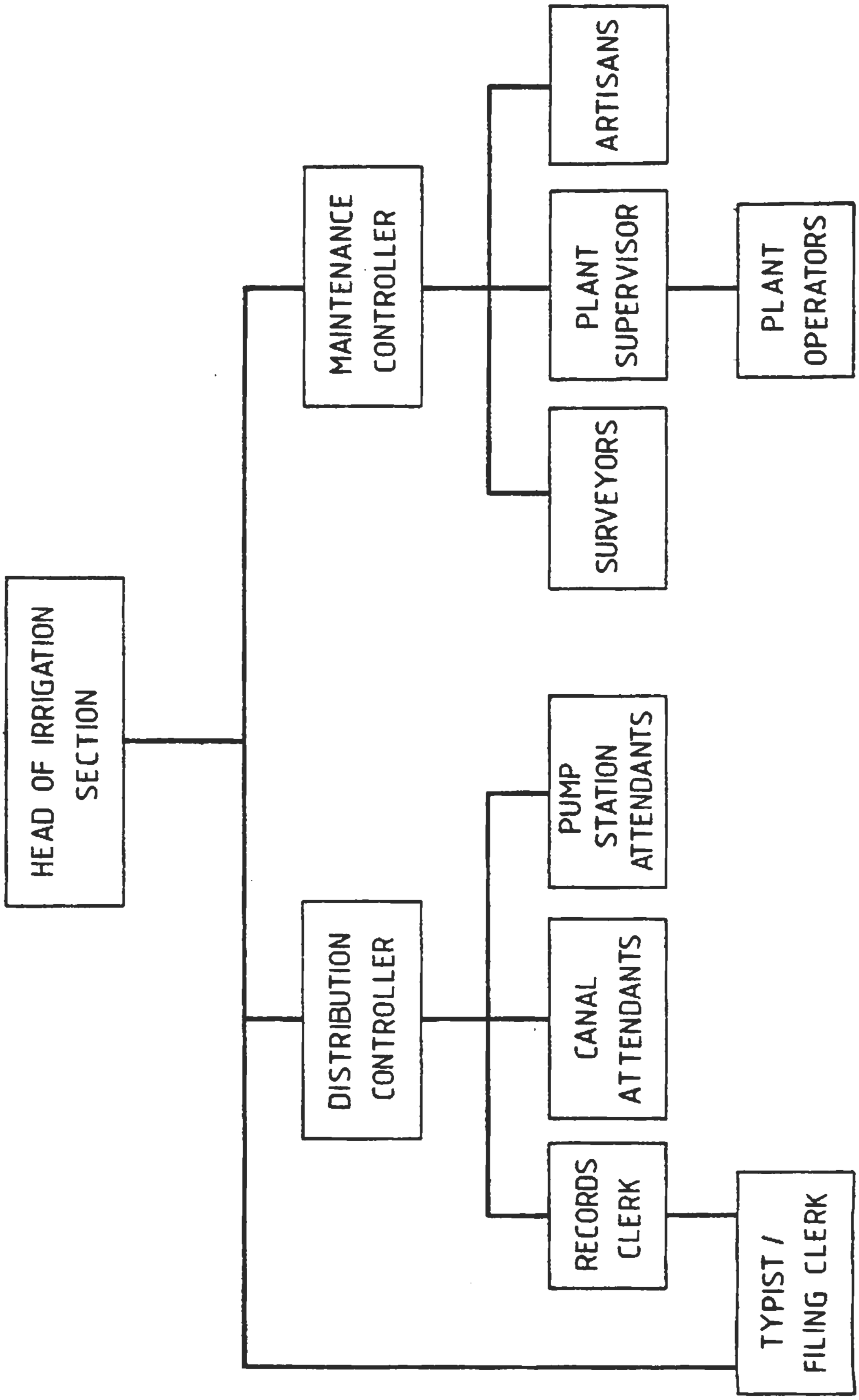
Irrigation and Drainage Operation and Maintenance Staff Requirements

Item	Position	Nr
1	Head of Irrigation Section	1
2	Distribution controller	1
3	Canal attendants	(2 x 9 (2 x 2
4	Pump attendants	
	- irrigation	2 x 3
	- drainage	3 x 1
	- mechanic	1
5	Maintenance controller	1
6	Surveyors	1
	Assistant surveyors	1
	Surveyors' labourers	4
7	Plant operators and drivers	25
	Assistant plant operators and drivers	15
8	Artisans	5
	Artisans' labourers	15
9	Plant supervisor	1
10	Maintenance foremen	6
	Maintenance labour	60
11	Field irrigation supervisors	2 x 6
	Irrigators (for 5 300 ha cane)	2 x 150
12	Records clerk	1
	Typist/filing clerk	1

Figure I.13.1

Irrigation Section
Proposed Staff Structure



He would be responsible for drawing up a new irrigation schedule every two or three weeks and for making adjustments to this in response to water shortages, high water salinity or sediment, and rainfall. He would ensure that full records are kept of river flow, water quality, watertable levels, canal flows, pumping hours and meteorological data.

(c) Maintenance Controller

The maintenance controller would liaise with the distribution controller and would be responsible for all maintenance operations for the irrigation and drainage system. He would have under his control a team of plant operators whom he would direct through the plant supervisor, and a group of artisans responsible for structure maintenance. He would also have a small survey team.

The maintenance controller would draw up a routine maintenance schedule and would supervise its execution, responding to non-routine maintenance work as it arose. He would make regular inspection tours of canals and drains to keep a check on maintenance requirements.

(d) Canal Attendants and Pump Station Attendants

The canal attendants would be responsible for all adjustments to control gates on main and secondary canals and tertiary head regulators. Provision has been made for one attendant for each secondary canal, two for the 21st October canal, and one for each night storage reservoir - a total of nine, with two shifts for each. In addition there will be two shifts of two men through the night to ensure that reservoir filling is supervised.

Pump station attendants would be provided for each permanent station, with two shifts for the irrigation station and three shifts for the drainage station.

A mechanic with sole responsibility for pump maintenance would also be provided.

(e) Surveyors

The surveyors would make regular checks on channel cross sections so as to identify the need for maintenance works and to monitor their progress. Fields would also be surveyed regularly to determine any re-levelling requirements.

(f) Plant Supervisor

The plant supervisor would be responsible for directing the movements and use of maintenance plant in accordance with the schedules drawn up by the controller. He would ensure that all maintenance plant is kept in operating condition by organising routine servicing and preventive maintenance through the workshop.

(g) Artisans

Artisans will be required for building and structure maintenance, and for construction of minor structures. The team should include a mason, bricklayer and carpenter.

Small scale maintenance works on the farms would be carried out by labour gangs (10 labourers for each farm) supervised by foremen. Such works would include hand weeding around canal and drain structures, culvert desilting, and minor repairs to pot-holes and gullies in canal banks and roads. These teams would generally be directed by the farm managers.

The actual process of field irrigation using siphon pipes would be carried out by teams of irrigators under the direction of the farm managers (through field irrigation supervisors).

13.3 Operation and Maintenance Plant and Vehicles

The Estate already has a large plant fleet although many items are presently inoperable. However, when the plant has been repaired and serviced (under the proposed Crash Programme), the Estate will have many of the items required to maintain the rehabilitated scheme. Some additional items will be required. A complete list of the key items of plant is given in Table I.13.2 and the selection is explained below.

(a) Dragline (30 RB or equivalent)

This large dragline is needed at the settling basin to remove the deposited silt. The existing small draglines in the Estate's fleet have insufficient reach and bucket capacity for this task. In fact the calculations presented in Chapter 5 indicate a requirement for two 30 RB draglines, based on the very high assumed sediment loads. However, it is considered that there is insufficient evidence to suggest that the high sediment concentrations experienced in 1980 will persist. In fact there is evidence to the contrary since values measured during the fieldwork and subsequently by Estate staff are very low. It is therefore proposed that only one dragline be purchased initially, with the second machine only being obtained if and when necessary.

(b) Draglines (LS 78)

The medium sized draglines which the Estate already possesses are ideal for desilting work on the deep drains and main canals. These machines can be rehabilitated to good working condition and there are trained operators capable of making good use of them. There will be some 78 km of large canal and deep drain in the remodelled scheme. Assuming a pessimistic rate of cleaning of 1 km/week, two machines would be capable of carrying out a once-a-year channel maintenance programme with time to spare for other tasks.

(c) Hydraulic Excavators (LC 90)

The Estate has seven hydraulic excavators of which four can be brought back to operating condition. These machines will be used on shallow main drains and secondary canals for desilting and weed removal. In fact, since the total length of these channels is only about 61 km, two machines will be adequate in the long term. However it is recommended that the two extra machines are retained until the end of their useful lives to provide additional capability during the first years of rehabilitation.

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Plant Requirements for Maintenance of Irrigation and Drainage Works

Item	Type/size	Nr	Notes
Dragline	30 RB	1*	Sediment basin
Dragline	LS 78	2	Existing machines
Hydraulic excavator	LC 90	4(2)	Existing machines
Bulldozer	Cat D6	2	Existing machines
Front end loader + backhoe	MF 50	2*	For minor earthworks
Grader	Cat 140G	2(1)	Existing machines
Ditcher	Briscoe	1	For tertiary canals
Tipper lorry	6 m^3	2*	
Tractor + trailer		2	
Low loader	For heaviest plant item	1*	
Boom sprayer attachment		1*	
Field drain jetting machine		1*	
Concrete mixer	$\frac{1}{4}$ m^3	2*	
Compressor with tools		1*	
Mobile pumps		2*	Dewatering structures etc.
FWD station wagon	Land Rover	2*	
Pick up		1*	
Motorcycles		15*	

Notes : * indicates new items required

Figures in brackets are long-term requirements, the following items of plant need not be replaced when they reach the end of their useful life : 2 hydraulic excavators and 1 grader.

(d) Bulldozer (D 6)

One bulldozer will be required permanently at the sediment basin spreading and levelling the material deposited by the dragline. Another bulldozer should be available for emergency repairs to embankments and for road maintenance. Two of the Caterpillar D 6 units which the Estate has should be adequate.

(e) Front-end Loader and Backhoe

These are very useful items of plant which can be used for minor earthworks across the Estate. They are mobile and versatile, being capable of both loading and ditching work. Two units are recommended and they should be located at Farms II and VI. They would supplement the work of the hydraulic excavators for smaller localised desilting work (for example at structure groups) and would be used for handling materials (sand, stone, etc.) and removal of deposited silt and weed from canal banks where necessary.

(f) Grader

A grader will be required for routine maintenance of in-field roads, canal banks and shallow surface drainage channels. One of the existing Caterpillar 140 G units will be sufficient for this purpose.

(g) Ditcher

A ditcher will be required for annual cleaning of tertiary canals/header channels. It should be of an appropriate size for this purpose. One unit will be sufficient for the 200 km of tertiary canal/header channel involved.

(h) Tipper Lorry, Tractor and Trailer, Low Loader

Lorries will be required for the transport of materials within the Estate and for transport of silt, weeds, etc. which have been removed from channels where there is no room for local deposition. Two 6 m^3 units should be adequate. Tractors and trailers will be needed for transporting labourers and materials, tools and equipment; two units should suffice.

The low loader is necessary for moving heavy items of plant and equipment (draglines, pumps, engines, etc.). It should be of sufficient capacity to transport the heaviest item (the 30 RB dragline).

(i) Boom Sprayer

In view of the very severe problems experienced with reed and weed growth in the channels, and the rapid regrowth which occurs following cutting of these, chemical treatment is considered essential. A tractor mounted boom sprayer will be required for the application of the appropriate herbicide. The most troublesome vegetative growth in channels is the reed (Phragmites sp.). Two herbicides have been found effective against this, dalapon and glyphosphate, although recent work in the UK suggests that glyphosphate gives the better long-term control. Both chemicals have official clearance in the UK for use in rivers and drainage channels, but care will have to be exercised with use on canals. An initial treatment with herbicide of all channels will have been carried out during the remodelling work. From this initial treatment, the potential effectiveness of both dalapon and glyphosphate can be assessed, as only by site trials can the correct timing of applications and the appropriate dosage be decided.

As a guide, typical application rates and timing of application are given below. For both herbicides, only one application per year should be required.

Herbicide	Trade name	Time of application	Application rate, active ingredient
Dalapon	Dowpon	At or just after flowering	6 kg/ha
Glyphosphate	Roundup	When weeds are fully emerged	2 kg/ha

After the application of herbicide, the weeds will die back and the dead growth should then be removed by hydraulic excavator, dragline or by hand as appropriate.

(j) Other Items

A field drain jetting machine will be required for routine clearing of the limited number of field drains which are proposed. The same device may be useful for cleaning out culverts, particularly those with small diameters which tend to block with silt and debris.

Concrete mixers will be required for minor concrete repair work, and a compressor with appropriate tools should be provided for breaking up old concrete, etc.

Two mobile pumps will be needed for dewatering structures for inspection and maintenance.

(k) Vehicles

A minimum of three vehicles will be required and, in the initial period of rehabilitation, two of these should have four wheel drive. A long wheel-base station wagon should be provided for the head of irrigation section; this vehicle will also be used by the distribution controller. A similar vehicle should be provided for the maintenance controller. The third vehicle should be a pick-up for use by the surveyors, plant supervisor and pump mechanic as required.

The canal attendants will require motorcycles so that they are able to keep a constant check on their canals and make the necessary gate adjustments. Field irrigation supervisors (one per farm) will also require motorcycles.

13.4 Scheme Operation

In the rehabilitated scheme, the distribution of irrigation supplies will have to be much more controlled than it is at present. The inclusion of night storage reservoirs will require greater diligence on the part of the gate operators and pump attendants to ensure that canal flows remain sensibly constant throughout the day. Canal S2 will be the only secondary canal served by gravity from a night storage reservoir. During the day, as the reservoir water level falls, it will be necessary to adjust the canal head regulator gate to compensate. Three or four adjustments during the 12 hour day should be sufficient. With pumped abstraction from a reservoir control of the secondary or tertiary canal flow will be achieved by the number of pumps operating and, to a lesser extent, by varying the engine speed.

At the end of the day all secondary canals will be closed down from the head. The head regulator will be completely closed by the attendant, who will then proceed down the canal closing the offtakes and cross regulators. In this way losses of water will be minimised and the secondary canals will not be empty when the head regulators are opened the following morning.

During the night all flow will be diverted into the reservoirs. At the same time as secondary canal closure is taking place, the main canal head regulator (river intake) and cross regulators will be adjusted for the night-time flow conditions. Two shifts of two supervisors will be in attendance throughout the night ensuring that the reservoirs are filled in the time available and making adjustments to avoid overfilling.

At times of low irrigation demand (e.g. October and November) secondary canals can be operated in rotation. At such times individual canals can be closed for periods of up to one week for inspection and additional maintenance if necessary.

13.5 Operation and Maintenance of the Drainage System

13.5.1 Operation

The drains are to flow by gravity from the fields, through the collector drains, main drains and Intercepting drain, to the drainage pump station and thence to waste. The only operations to be carried out concern the pump station and the disposal system, and these are described in turn below.

(a) Pump Station

The pumps should normally be operated to maintain the water level in the Intercepting drain within a designated operating range. To minimise energy consumption, operating levels should not be kept lower than necessary for satisfactory drainage of the Estate.

To match pump discharge to drain discharge, some intermittent operation may be required, with corresponding storage in the Intercepting drain. However, to keep water level fluctuations to a minimum, intermittent operation should be confined to only one pump, any pumps being run continuously, i.e. for 24 h/d. Frequent large variations or rapid drawdown of drain water level leads to deterioration of the drain side slopes, and must therefore be avoided. It is recommended that the water level upstream of the pump station should normally be kept within a day-to-day operating range of 0.30 m. To ensure adequate submergence of the pump inlets, the pumps should at no time be operated when the upstream water level is less than 94.85 (Jowhar Offstream Storage project datum). The simultaneous operation of all five pumps should only be permitted when the upstream water level exceeds the normal maximum value. Overall pumping duties should be shared equally between all five pumps. The actual scheduling of individual pump operation may be arranged to suit maintenance requirements, staff shifts, etc. However, to ensure that the engines are properly warmed up and to recharge the batteries, pumps should preferably not be run for periods of less than one hour. Each pump should be used at least once a week unless out of order.

Further details of recommended operating procedures for the pump station may be found in the Jowhar Offstream Storage Project Operation and Maintenance Manual (MMP 1981).

(b) Disposal of Drainage Water: Salinity Control

The most important operating decision is whether to discharge the drainage water to the disposal area or to the reservoir. This is determined mainly by the salinity of the drainage water. In view of the many factors involved, neither the build-up in drain flows nor the drainage water salinities can be accurately predicted. From the point of view of reservoir water quality, the important factor is the total quantity of salts, which depends on discharge as well as salinity. Operating rules for the disposal of drainage water will therefore require to be regularly reviewed in the light of operating experience, and in particular with regard to:

- (i) drain discharges
- (ii) amount and variation of drainage water salinity
- (iii) reservoir and river water quality
- (iv) capacity of disposal area

The general objective should be to avoid giving rise to reservoir water salinities greater than an EC of 1 200 micromho/cm.

As a basis for initial operation, it is recommended that the simple rule should be adopted, that drainage water should be discharged to the reservoir only when its salinity is less than an EC of 2 000 micromho/cm (Section 12.8.3).

The disposal of drainage water is controlled by operation of the inlet gates on the pumped drainage outfall culvert and on the inverted siphon. These structures are not designed to enable the flow to be divided between them in a controlled manner, and only one structure should be in use at any one time. Both structures have two barrels with separate gates. To minimise the risks of silting up only one gate should be opened when three or more pumps are running.

13.5.2 Maintenance

(a) Field Drains

Field drains should be cleaned one or two years after installation, to remove any sediment washed in during the initial period after installation before the soil above and around the pipe has consolidated. Subsequently the drains should be cleaned at a frequency of five to ten years (Smedema and Rycroft, 1983).

The jetting machine listed in Section 13.3 (j) would be used for this work. This has a self-propelling nozzle with forward and backward facing jets of pressurised water which clean the pipe.

(b) Collector Drains

With buried collectors the silt traps at the manholes should be cleaned out every five years when jetting the field drains, and if there is evidence of problems the collector drain pipe may also be rodded out at the same time.

Both shallow and deep open collector drains require a high maintenance effort to keep them weed free. As described in Section 13.3 (i) trial applications of dalapon and glyphosphate chemicals, by a boom sprayer, are to be made during the remodelling work and a chemical programme of weed control would be drawn up from the results. The weeds would be removed by the hydraulic excavators with a special bucket or rake.

(c) Main Drains and Interceptor Drain

Weed would be cleaned from main drains and Intercepting drain by the same chemical methods as are proposed for open collector drains.

Siltation of open drains is not expected to be a problem but some reshaping could be done by the hydraulic excavators at the same time as clearing weeds. The LS 170 dragline would be used for clearing weed from the Intercepting drain.

(d) Structures

Maintenance of structures would generally consist of an inspection and rodding out of culverts to be carried out well in advance of each rainy season to enable repairs to be done as required.

More frequent attention will be necessary at the drainage pump station and the disposal works, comprising checking and cleaning, etc. of both civil and mechanised works.

SECTION E

COSTS

CHAPTER 14 - COSTS OF REHABILITATION WORKS

14.1 General Basis of Programme and Costs

The rehabilitation works must be executed with minimal disruption to the cane production activities of the Estate. Programming of the works must therefore fit in with the programme of new planting. Since about 530 ha of new planting will be required each season (twice a year), it will be necessary to adopt a 5 year programme for the introduction of the new field layout over the 5 300 ha cane area. For the main irrigation and drainage works, however, a three year programme is considered more appropriate.

The majority of the rehabilitation works would be carried out by an external contracting agency. The Estate management will be fully occupied in the processes of cane and sugar production and cannot be expected to take on major rehabilitation works, even with the proposed management assistance programme. However, it is proposed that some of the minor improvement works to channels is carried out by the Estate. This will be possible once the Estate's maintenance plant has been restored to operating condition. Such minor rehabilitation works (cleaning out canals and drains, tree and bush removal, minor repairs to structures, etc.) will be similar to future maintenance tasks, which the Estate will have to undertake. This work will therefore provide the opportunity for building up an effective maintenance section and training staff in the appropriate techniques.

The overall programme for the Estate rehabilitation is shown in Figure I.14.1. Civil engineering contract(s) would be let at the end of 1984 and work would commence in the dry season of 1985. By the end of 1988 all work on the main irrigation and drainage system would be complete, but conversion to the new field layout would continue for a further two years.

Rehabilitation work carried out by the Estate would start as soon as the required plant is available, and would continue for three years or so, as required.

Summaries of the work content of irrigation and drainage rehabilitation are presented in Tables I.14.1 and I.14.2.

Cost estimates for the rehabilitation works have been based on the outline designs described elsewhere. Unit rates for the components of the works have been based on those prepared for the Engineer's Estimate for the Mogambo Irrigation Project in Somalia, updated to mid-1983. The Engineer's Estimate total cost was almost identical to the accepted tender total for this project, and was used in preference to actual tendered rates because it was felt the latter would reflect the particular pricing policy of the tenderer. A representative list of the basic rates is given in Table I.14.3.

Allowance has been made for temporary works to maintain flows in existing irrigation and drainage channels during the contract period since it is vital that the rehabilitation works cause minimal interference to the production of sugar cane.

14.2 Programme of Works

In estimating the costs of rehabilitation, works have been allocated to each of the three years for the main rehabilitation programme. The main components are summarised below :

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REHABILITATION OF JOWHAR SUGAR ESTATE

Summary of Irrigation Rehabilitation Works

Description	Unit	Quantity
1 Remodel Existing Canal		
Canal S1	km	3.5
Canal S2	km	7.0
Canal S3	km	6.7
21 October canal	km	19.2
Luigi canal	km	1.7
2 Replace Gates on Existing Structures	Nr	22
3 New Canal Earthworks		
Sediment basin	m^3	60 000
Head reach to reservoir for S1/S2	km	1.8
Head reach for canal S3	km	1.8
Canal S6	km	4.2
Canal S7	km	7.4
Remodel tertiary canals	km	50.0
Header chanel	km	185.0
4 Canal and Reservoir Structures		
Movable weir regulators	Nr	5
Lifting gate regulators	Nr	12
Culverts	Nr	1
Major modifications to existing structures	Nr	3
Tertiary canal head regulators	Nr	22
5 Reservoir and Pumping Stations		
Rehabilitate existing reservoirs	Nr	2
Construct new reservoirs	Nr	3
Pumping stations	Nr	3
6 Land Preparation		
Bush clearance	ha	500
Land levelling	ha	5 300

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REHABILITATION OF JOWHAR SUGAR ESTATE

Summary of Drainage Rehabilitation Works

Description	Unit	Quantity
1. Field Drainage		
Surface drain	km	185
Clean existing collector drain	km	53
Deepen existing collector drain	km	36
Buried field drains	km	14(1) (200)(2)
Intermediate deep collector drain	km	1.2(1) (18)(2)
Surface drain culvert, new	Nr	36
Surface drain culvert, rehabilitated	Nr	17
Surface drain junction	Nr	34
Shallow collector drain culvert, new	Nr	9
Shallow collector drain culvert, rehabilitated	Nr	25
Deep collector drain culverts	Nr	7
Deep collector drain junction culvert	Nr	25
2. Main Drains		
West drain, deepen and extend	km	11.6
W2 drain, deepen and extend	km	7.8
W2/2 drain, deepen and extend	km	2.6
Middle drain, rehabilitate	km	10.9
East drain, rehabilitate and extend	km	15.2
Deep main drain culverts	Nr	9
Shallow main drain culverts, new	Nr	6
Shallow main drain culverts, rehabilitated	Nr	5
Main drain underpass	Nr	6
Main drain junction culvert	Nr	4
Footbridge	Nr	1
Demolish existing structures	Nr	29
3. Interceptor Drain and Drainage Disposal System		
Complete Interceptor drain	)	Not included in project investment See Section 14.9
Complete drainage pump station	)	
Complete drainage disposal channel	)	

Notes : (1) Initial investment programme.

(2) Total requirements.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE

Typical Unit Rates Used in Cost Estimates

Item	Unit	Rate (SoSh)
Bush clearance (light)	ha	3 200
Bush clearance (dense)	ha	4 600
Land levelling	m^3	40
Excavation to form embankments (haul less than 250 m)	m^3	50
Excavation to form emabnkments (haul 250 to 1 000 m)	m^3	66
Excavation to form embankments (haul 1 000 to 2 500 m)	m^3	80
Excavation for foundations of structures	m^3	85
Compacted backfill for structures	m^3	103
Concrete class D blinding (0.1 m thick)	m^2	360
Mass concrete class C	m^3	4 020
Reinforced concrete class A	m^3	4 860
Mild steel reinforcement	tonne	47 300
Shuttering	m^2	1 360
Dry stone pitching and gravel backing	m^2	1 005
Spigot and socket concrete pipe and bedding 0.45 m dia	m	1 685
Spigot and socket concrete pipe and bedding 0.90 m dia	m	5 185
Spigot and socket concrete pipe and bedding 1.20 m dia	m	7 855
Movable weir gate 2.0 m wide	Nr	142 000
Pipe regulator gate for 1.05 m dia pipe	Nr	72 000
Tertiary canal head regulator gate	Nr	45 000
1.0 m^3/s pump and diesel engine	Nr	536 000

Notes : (1) Rates based on estimated rates for the Mogambo Irrigation Scheme, updated to mid 1983, and confirmed by reference to tendered rates for this scheme.

Year 1

- Crash Programme

Year 2

- Rehabilitate 21st October canal to Km 2.5 including the sediment basin.
- Remodel existing canals S1 and S3.
- Construct new canal head reaches for S1/S2 and S3.
- Rehabilitate existing basin off the Luigi canal and construct new reservoir for fields T3 and T4.
- Provide pumping stations for canal S1 and fields T3 and T4.
- Commence West drain earthworks.
- Remodel deep collector drains in West drain area.
- Complete excavation of Interceptor drain to final design section.

Year 3

- Remodel 21st October canal Km 2.5 to Km 7.2, and canal S2 up to Km 4.7.
- Construct new canals S6 and S7.
- Construct new reservoir for canal S7 and 21st October tail reach, and civil works for the pumping station.
- Continue with West drain earthworks and complete branch drains W2 and W2/2.
- East drain earthworks (southern section).
- Continue remodelling deep collector drains and commence remodelling of shallow collectors.
- Purchase field drain trenching machine.

Year 4

- Complete remodelling of 21st October canal (last 12 km) and complete S2 remodelling.
- Rehabilitate existing basin off canal S3, and construct new basin at head of 21st October canal.
- Complete pump station for S7 and 21st October tail.
- Finish West drain earthworks.

- Middle drain earthworks.
- East drain earthworks (northern section).
- Complete collector drain remodelling.
- Install field drains on Field 8II.

The programme of road rehabilitiation would be fitted in to the above programme to make best use of available fill material from the drain earthworks.

Detailed planning of the programme for replanting (and hence introduction of the new field layout) is beyond the scope of this study and will require in-depth consideration before the works commence.

Details of the estimated costs of the irrigation and drainage works are given in Bills 1 to 4 (Tables I.14.4 to I.14.7). The Bills are discussed in turn below.

14.3 Canals and Canal Structures (Bill Nr 1)

The rehabilitation works required for the canal system have been described in Chapters 5 and 7. In Bill Nr 1 these works have been divided into five major components. as described below.

Item 1.1 Remodelling Existing Canals

The rate per kilometre for remodelling work has been assessed from a comparison of the existing channel across sections and the proposed design sections. In most cases the amount of earthmoving is relatively small since the existing canals are large enough for the proposed 12 hour flow regime. However all canals will require reshaping and the removal of weeds, reeds, bushes and trees from the bed and banks.

The remodelling rate for each canal also includes for the demolition of unwanted structures and the repair of structures which will be incorporated into the rehabilitated scheme. Such repairs are generally of a minor nature and include such items as patching concrete and rendering, repointing of rendering brickwork, replacing decaying concrete or brickwork, and channel protection works. Some major repair works have also been allowed for where, for example, brickwork has deteriorated to the extent where much needs replacing. The cost of replacing control gates on regulating structures is included in Item 1.2. New structures required existing canals are included in Item 1.4.

Canal S1 (3.5 km)

This canal is in poor condition and is overgrown. However the existing cross section is generally larger than required, although the banks and to a lesser extent the bed are higher than required. The only significant earthmoving task is thus lowering the bed by up to 0.5 m. In addition the banks will require trimming and the top surfaces grading. Bush and tree growth should be removed.

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BILL NR 1 - CANALS AND CANAL STRUCTURES

Item Nr	Item description	Unit	Rate (SoSh)	Year 2 Quantity	Year 2 Amount (SoSh '000)	Year 3 Quantity	Year 3 Amount (SoSh '000)	Year 4 Quantity	Year 4 Amount (SoSh '000)	Year 5 Quantity	Year 5 Amount (SoSh '000)	Year 6 Quantity	Year 6 Amount (SoSh '000)
1.1	**Remodelling existing canals (including repairs to structures)**												
1.1.1	Canal S1	km	200 000	3.5	700	-	-	-	-				
1.1.2	Canal S2	km	275 000	-	-	4.7	1 293	2.3	632				
1.1.3	Canal S3	km	200 000	6.7	1 340	-	-	-	-				
1.1.4	21st October canal	km	280 000	2.5	700	4.7	1 316	12.0	3 360				
1.1.5	Luigi canal	km	150 000	1.7	255	-	-	-	-				
1.2	**Replace gates on existing structures:**												
1.2.1	Cross regulators	Nr	120 000	4	480	3	360	3	360				
1.2.2	Tertiary head regulators	Nr	45 500	6	273	6	273	-	-				
1.3	**Earthworks**												
1.3.1	Sediment basin	m^3	55	60 000	3 300	-	-	-	-				
1.3.2	Head reach to reservoir for S1/S2	km	540 000	1.8	972	-	-	-	-				
1.3.3	Rehabilitate existing basins	Sum	-	-	325	-	-	-	800				
1.3.4	Canal and reservoirs See Section 14.3			-	1 866	-	9 162	-	1 470				
1.4	**Canal and reservoir structures:**												
1.4.1	Movable weir regulators												
	(a) 3 x 3.0 m	Nr	3 080 000	1	3 080	-	-	-	-				
	(b) 2 x 2.5 m	Nr	1 920 000	2	3 840	1	1 920	-	-				
	(c) 1 x 2.0 m	Nr	675 000	-	-	1	675	-	-				
1.4.2	Lifting gate regulators												
	(a) single	Nr	375 000	-	-	2	750	1	375				
	(b) double	Nr	610 000	2	1 220	1	610	2	1 220				
	(c) triple	Nr	975 000	3	2 925	-	-	1	975				

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE
BILL NR 1 - CANALS AND CANAL STRUCTURES (cont.)

Item Nr	Item description	Unit	Rate (SoSh)	Year 2 Quantity	Year 2 Amount (SoSh '000)	Year 3 Quantity	Year 3 Amount (SoSh '000)	Year 4 Quantity	Year 4 Amount (SoSh '000)	Year 5 Quantity	Year 5 Amount (SoSh '000)	Year 6 Quantity	Year 6 Amount (SoSh '000)
1.4.3	Culverts	Nr	345 000	-	-	-	-	1	345				
1.4.4	Modifications to existing structures	Sum	-	-	-	-	-	-	2 000				
1.4.5	Tertiary canal head regulators	Nr	160 000	5	800	8	1 280	9	1 440				
1.4.6	Pumping stations:												
	(a) For canal S1	Sum	-	-	500	-	-	-	-				
	(b) For fields T3 and T4	Sum	-	-	350	-	-	-	-				
	(c) For canal S7 and 21st October canal tail	Sum	-	-	-	-	3 000	-	1 480				
1.5	Maintaining existing irrigation supplies	Sum	-		500		500		500				
	Totals				23 426		21 139		14 957				

Bill Nr 1 Overall Total = SoSh 59 522 000

Note: Costs do not include contingencies.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE
BILL NR 2 - TERTIARY CANALS AND IN-FIELD WORKS

Item Nr	Item description	Unit	Rate (SoSh)	Year 2		Year 3		Year 4		Year 5		Year 6	
				Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)
2.1	**Canals:**												
2.1.1	Remodel tertiary canals	km	3 000	10	30	10	30	10	30	10	30	10	30
2.1.2	Header channel	km	35 000	37	1 295	37	1 295	37	1 295	37	1 295	37	1 295
2.2	**Land preparation:**												
2.2.1	Bush clearance	ha	3 200	100	320	100	320	100	320	100	320	100	320
2.2.2	Land levelling (including one land planing)	ha	8 720	1 060	9 243	1 060	9 243	1 060	9 243	1 060	9 243	1 060	9 243
2.3	**Drains:**												
2.3.1	Surface drain	km	50 000	37	1 850	37	1 850	37	1 850	37	1 850	37	1 850
2.4	**Structures:**												
2.4.1	Tertiary culvert, new	Nr	120 000	2	240	2	240	2	240	2	240	2	240
2.4.2	Tertiary culvert, re-habilitation	Nr	3 000	6	18	6	18	6	18	6	18	6	18
2.4.3	Tertiary divide structure	Nr	30 000	8	240	8	240	8	240	8	240	8	240
2.4.4	Surface drain culvert, new	Nr	40 000	8	320	7	280	7	280	7	280	7	280
2.4.5	Surface drain culvert, re-habilitation	Nr	3 000	3	9	4	12	3	9	4	12	3	9
2.4.6	Surface drain junction culvert	Nr	50 000	6	300	7	350	7	350	7	350	7	350
2.5	**Roads:**												
2.5.1	Field roads	km	20 000	37	740	37	740	37	740	37	740	37	740
2.6	**Miscellaneous:**												
2.6.1	Siphons	km	95 000	25	2 375				-		-		-
	Totals				16 980		14 618		14 615		14 618		14 615

Bill Nr 2 Overall Total = SoSh 75 446 000

Note: Costs do not include contingencies.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE
BILL NR 3 - DRAINS AND DRAIN STRUCTURES

Item Nr	Item description	Unit	Rate (SoSh)	Year 2 Quantity	Year 2 Amount (SoSh '000)	Year 3 Quantity	Year 3 Amount (SoSh '000)	Year 4 Quantity	Year 4 Amount (SoSh '000)	Year 5 Quantity	Year 5 Amount (SoSh '000)	Year 6 Quantity	Year 6 Amount (SoSh '000)
3.1	**Deepen and extend West drain:**												
3.1.1	West drain earthworks	m^3	50	123 000	6 150	104 000	5 200	37 000	1 850				
3.1.2	W2 drain earthworks	m^3	50			147 000	7 350						
3.1.3	W2/2 drain earthworks	m^3	50			31 000	1 550						
3.2	**Remodel collector drains:**												
3.2.1	Deep collector drains	km	400 000	12.4	4 960	14.8	5 920	9.0	3 600				
3.2.2	Shallow collector drains	km	3 000			27	81	26	78				
3.3	**Rehabilitate and extend Middle and East drains:**												
3.3.1	Middle drain earthworks	m^3	50					35 000	1 750				
3.3.2	East drain earthworks	m^3	50			148 000	7 400	26 000	1 300				
3.4	**Main drain structures:**												
3.4.1	Deep main drain culverts												
	(a) 0.1 - 0.3 m^3/s	Nr	180 000			1	180	2	360				
	(b) 0.3 - 1.3 m^3/s	Nr	430 000	2	860	2	860						
	(c) 1.3 - 2.6 m^3/s	Nr	700 000	1	700								
	(d) 2.6 - 4.0 m^3/s	Nr	940 000	1	940								
3.4.2	Shallow main drain culverts												
	(a) 0.3 - 1.0 m^3/s	Nr	240 000			1	240	1	240				
	(b) 1.0 - 2.8 m^3/s	Nr	570 000			3	1 710	1	570				
3.4.3	Main drain underpass												
	(a) 0 - 0.15 m^3/s	Nr	650 000			1	650						
	(b) 0.15 - 0.5 m^3/s	Nr	980 000			2	1 960	1	980				
	(c) 0.5 - 1.0 m^3/s	Nr	1 440 000			1	1 440	1	1 440				
3.4.4	Main drain junction culvert												
	(a)	Nr	200 000			1	200						
	(b)	Nr	430 000			1	430						
	(c)	Nr	Various			1	1 100	1	960				

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE
BILL NR 3 - DRAINS AND DRAIN STRUCTURES (cont.)

Item Nr	Item description	Unit	Rate (SoSh)	Year 2 Quantity	Year 2 Amount (SoSh '000)	Year 3 Quantity	Year 3 Amount (SoSh '000)	Year 4 Quantity	Year 4 Amount (SoSh '000)	Year 5 Quantity	Year 5 Amount (SoSh '000)	Year 6 Quantity	Year 6 Amount (SoSh '000)
3.4.5	Minor works												
	(a) footbridge	Nr	100 000			1	100						
	(b) demolish existing structures	Nr	20 000	3	60	19	380	7	140				
	(c) clean existing structures	Nr	5 000			2	10	3	15				
3.5	**Deep collector drain structures:**												
3.5.1	Collector drain culvert	Nr	80 000	3	240	4	320	-					
3.5.2	Collector drain junction culverts	Nr	110 000	8	880	9	990	8	880				
3.6	**Shallow collector drain junction culvert:**												
3.6.1	New culvert	Nr	95 000			5	475	4	380				
3.6.2	Cleaning existing structure	Nr	1 000			13	13	12	12				
3.7	**Deep intermediate collector and buried field drains:**												
3.7.1	Trenching machine	Nr	3 000 000			1	3 000						
3.7.2	Deep intermediate collector	km	500 000					1.2	600				
3.7.3	Buried field drains	km	100 000					14	1 400				
	Totals				14 790		41 559		16 555				

Total for Bill Nr 3 = SoSh 72 904 000

Note: Costs do not include contingencies.

Table Nr I.14 .
(cont.

SOMALI DEMOCRATIC REPUBLIC
REHABILITATION OF JOWHAR SUGAR ESTATE
BILL NR 4 - ROADS

Item Nr	Item description	Unit	Rate (SoSh)	Year 2		Year 3		Year 4		Year 5		Year 6	
				Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)	Quantity	Amount (SoSh '000)
4.1	**Form new road embankment:**												
4.1.1	Within 1 km of a drain	km	75 000	-	-	2.0	150	2.0	150				
4.1.2	1-2 km from a drain	km	120 000	7.0	840	4.0	480	4.0	480				
4.1.3	Up to 5 km from a drain	km	200 000	2.0	400	2.0	400	2.0	400				
4.2	**Grade existing road to a cross fall**	km	2 000	60	120	-	-	-	-				
4.3	**Excavate roadside drain**	km	23 000	20	460	25	575	15	345				
4.4	**Construct culverts for roadside drain**	Nr	45 000	6	270	6	270	8	360				
	Totals				2 090		1 875		1 735				

Totals for Bill Nr 4 = SoSh 5 700 000

Note: Costs do not include contingencies.

Estimated costs (based on 1983 survey)	SoSh/km
Deepening section by up to 0.5 m	50 000
Trimming and grading banks (both sides)	75 000
Bush clearance (40 m reservation)	15 000
Sub-total	140 000
	SoSh
Existing structures:	
2 structures to be demolished @ SoSh 25 000	50 000
3 structures, minor repairs @ SoSh 20 000	60 000
Culvert at Km 3.5, major repair to walls, say	100 000
Sub-total	210 000

Hence average remodelling cost (Item 1.1.1) = **SoSh 200 000/km**

Canal S2 (7.0 km)

This canal was partly cleaned out during the fieldwork period but remained in relatively poor condition. Particular problems are silt deposited on left bank (by excavation) and a very overgrown right bank. Cross regulators are relatively crude.

In the upper reaches the canal is wide enough but requires deepening by about 0.5 m. Further downstream the canal tends to be oversized. Silt deposits should be removed from the bank tops and disposed of.

Estimated costs (based on 1976 survey and 1983 observations)	SoSh/km
Deepening section by up to 0.5 m	50 000
Trimming and regrading banks (both sides)	150 000
Bush clearance (30 m reservation)	10 000
Sub-total	210 000
	SoSh
Existing structures:	
Demolish 1 Nr X-reg + 2 Nr tertiary H-regs	50 000
Minor repairs to structures (6 Nr)	120 000
Major works to 3 X-regs to allow fitting of new gates (cost of gates excluded)	300 000
Sub-total	470 000

Hence average remodelling cost (Item 1.1.2) = **SoSh 275 000/km**

Canal S3 (6.7 km)

In the proposed revised canal system canal S3 will serve only three cane fields (the remaining fields being mostly abandoned and not recommended for cane). Nevertheless costs have been allowed for its rehabilitation at the rate of **SoSh 200 000/km.**

21st October Canal (19.2 km)

The 21st October canal will become the main supply canal for the Estate and is therefore of major significance. Canal cross sections were surveyed during the 1983 fieldwork and these have been compared with the design section. It is the first 5 km of canal which requires most remodelling (both widening and deepening) at an average earthmoving rate of 12 m^3/m. From Km 5 to Km 12.5 the section is generally adequate but will require trimming. The pumped tail reach of the canal (6.7 km) is oversized but requires bank raising to suit the increased design water level. On average it is estimated that 2 m^3/m will be required for this. In addition weed and bush clearance is required along most of the canal's length, although bush growth on the banks is less severe than on other older canals.

Estimated costs (based on 1983 fieldwork)	SoSh
Section reforming (bed and banks)	
5 km @ 12 m^3/m	3 000 000
7.5 km @ 1 m^3/m (trimming)	375 000
6.7 km @ 2 m^3/m (bank raising)	670 000
Bush clearance (50 m reservation, 19.2 km)	400 000
Existing structures:	
Demolish three large X-regs	150 000
Repairs to three major structures	150 000
Repairs to 12 tertiary H-regs	240 000
Other items at say 5%	250 000
Total	5 235 000

Average cost of remodelling (Item 1.1.4) = **SoSh280 000/km**

Luigi Canal (1.7 km)

The first 1.7 km of the Luigi canal will be kept open to serve canal S1 when river levels permit. The canal is generally silted up and will require reforming. The costs of this have been allowed for at an assumed rate of **SoSh 150 000/km.**

Item 1.2 Replace Gates on Existing Structures

Many of the existing regulating structures have gates in poor condition or no gates at all. It is important that these are repaired or replaced, as necessary. Repair works have been included under Item 1.1. Replacement cost estimates are described below (gates on new structures are included under Item 1.4).

Canal	Gates needing replacement X-regs	Tertiary H-regs
S1	1	1
S2	3	3
S3	3	2
21st October	3	6
Luigi	-	-
Total	10	12

Cost estimates have been based on a rate of **SoSh 45 500** for a tertiary head regulator gate, and **SoSh 120 000** for a set of two cross regulator gates including frame and gear.

Item 1.3 Canal and Reservoir Earthworks

(a) General

The cost of new canal earthworks is largely dependent on the availability of fill material, some of which will come from the proposed drain improvement works, and some from reservoir excavation.

Volumes of cut and fill for each canal have been calculated using the design cross sections and ground levels from the orthophotomaps. The fill requirement for each canal has then been compared with the availability of excavated material, and haul distances estimated. The results are summarised below.

Canal	Cut (m^3)	Fill (m^3)	Source	Haul requirements (m^3) <250 m	250-750 m	750-1 500 m	1 500-2 500 m
S3 (1.81 km)	1 085	47 240	West drain	-	-	18 550	18 550
			Middle drain	570	1 680	2 255	4 550
S6 (4.18 km)	245	80 035	East drain	12 600	-	-	-
			East drain	4 200	4 200	4 200	-
			East drain	31 520	-	-	-
			Reservoir	-	-	16 150	6 920
S7 (7.40 km)	2 170	160 210	East drain	21 000	21 000	-	-
			East drain	17 600	17 600	-	-
			Reservoir	22 000	20 000	20 000	18 840
Totals	3 500	287 485	-	109 490	64 480	61 155	48 860

For material obtained from drainage channels, the cost of 'excavate and place in embankments' is already included in the drain excavation rate. The additional cost of hauling fill material has been estimated adopting the following rates:

Haul distance (m)	Additional cost (SoSh/m^3)
0 - 250	nil
250 - 750	13
750 - 1 500	19
1 500 - 2 500	32

For material obtained from reservoirs the cost of excavation and haul has been shared between the canal and the reservoir as explained below.

Two new reservoirs are required, a small one serving fields T3 and T4, and a much larger one for canal S7 and the tail of the 21st October canal. The site of the larger reservoir has been chosen so that fill requirements for the reservoir banks are minimised and thus a large volume of excavated material from the reservoir bed is available for canal fill. The volumes of cut and fill required for the reservoir are as follows:

Reservoir for	Area (ha)	Cut (m^3)	Fill (m^3)
Fields T3 and T4	4.0	100	5 250
Canal S7, 21st October	26.6	141 330	18 685

Thus the total cost of canal and reservoir earthworks (excluding the cost of earth material in drains except for the cost of transporting this when it is used for canal banks) is estimated at:

	SoSh
Excavate and form canal and reservoir embankments:	
144 930 m^3 @ SoSh 50/m^3	7 246 500
Haul of material:	
64 480 m^3 @ SoSh 13/m^3	838 240
61 155 m^3 @ SoSh 19/m^3	1 161 945
48 860 m^3 @ SoSh 32/m^3	1 563 520
Site clearance for canal/reservoir reservations say	
80 ha @ SoSh 2 500/ha	200 000
Total	**SoSh 11.01 million**

The cost of the head reach for canal S1/S2 has been estimated separately. This canal follows the alignment of old canal S5 but involves more than remodelling because canal S5 is considerably smaller than required. Having been abandoned for some years it is also very overgrown. From the 1983 survey the volume of earthmoving required has been estimated at 19 363 m^3 for the 1.81 km of canal. Allowing for bush clearance gives a total cost of **SoSh 972 000** at an average remodelling rate of SoSh 540 000/km.

The cost of the sediment basin at the head of the 21st October canal has been estimated from the 1983 survey. The basin will have a 12 m bed width and a length of 700 m (expansion of the bed width can be carried out during the project life if this proves necessary - see Section 5.2.6 herein). A total volume of excavation of around 60 000 m^3 is required. The total cost estimate of **SoSh 3.3 million** is based on the slightly higher rate of SoSh 55/m^3 to allow for working in an operating canal.

The cost of the proposed new reservoir for plant cane at the head of the 21st October canal has been estimated using survey levels frm the 1983 photomaps. The reservoir is approximately triangular in plan with two sides being formed by 21st October canal and the new head reach for canals S1/S2 (old canal S5). Only about 760 m of new embankment are required, but the existing banks will require raising and the inside bank slope flattened to 1 in 3. A total fill requirement of 53 350 m^3 has been estimated. This material can be obtained from the sediment basin excavation with a haul distance of 1 000 to 2 000 m. The additional cost of haul of SoSh 25/m^3 gives an earthworks cost of SoSh 1.33 million, to which must be added the cost of bush clearance on 35 ha. The cost of this reservoir has therefore been estimated at **SoSh 1.47 million.**

Rehabilitation of the existing storage basins (for use as plant cane reservoirs) will require bush clearance and some minor works to the reservoir embankments. A rate of SoSh 5 000/ha has been allowed for this (based on reservoir plan area), which for the 225 ha to be rehabilitated, gives a total cost of **SoSh 1 125 000.** The overall total cost for canal and reservoir earthworks (Item 1.3) is thus estimated at **SoSh 17.9 million.**

(b) Costs Attributable to Night Storage Reservoirs

The night storage reservoirs have been sited in high areas to minimise the fill requirement for embankments and to maximise excavation. The excess excavated material can then be used to form canal embankments. If there were no storage reservoirs, the high demand for fill in canal embankments would have to be met by excavating borrow pits elsewhere. Thus the real cost of the storage reservoirs is very small, as is shown below:

Volume of cut in reservoirs	141 230 m^3
Volume of fill for reservoirs	23 935 m^3
Surplus	117 295 m^3
Volume used for canal embankments	103 910 m^3
Net surplus	13 385 m^3

Hence net cost of reservoirs = (23 935 + 13 385) x 50
= SoSh 1.87 million

The structures required for the reservoirs comprise canal head regulators and pumping stations. Canal head regulators would be required even if there were no night storage reservoirs (to control flow from main canal to secondary canal). The total estimated capital cost for the pumping stations is SoSh 5.33 million, but some SoSh 850 000 of this would be required even without night storage because some fields cannot be served by gravity from minimum river level.

The other small element of cost attributable to night storage is the rehabilitation of the existing basin off the Luigi canal, which will provide night storage for canals S1 and S2. The cost of this is estimated at SoSh 325 000.

The total cost directly attributable to night storage is thus **SoSh 6.7 million,** equivalent to some 3% of the total investment in irrigation and drainage.

Item 1.4 Canal and Reservoir Structures and Pumping Stations

Costs have been estimated for all major structures, and for minor structures of which there are significant numbers. Canal regulators have been subdivided according to size, as described in Chapter 7. The costs include all engineering works and assume construction by contract, including the supply and installation of control gates and gear. The costs have been obtained by applying the unit rates to bills of quantities prepared for similar structures on other schemes.

The new structures include those on existing canals as well as those on the proposed new canals. A complete list of main and secondary canal structures is included as Appendix C to this annex.

Item 1.4 also includes a sum for major modifications to existing structures on the 21 October canal (tail reach). These structures are too low for the revised canal design and will therefore require substantial additional superstructure. This will be less expensive than demolition and replacement by new structures.

New pumping stations are required to pump water from the night storage reservoirs into canals S1 (seasonally), S7 and the 21st October tail reach, and into the tertiary canals serving fields T3 and T4. New pumping plant is required only for the S7 and 21st October tail reach, the other stations will utilise plant which the Estate already has. Cost estimates for pumping plant have been based on up-to-date quotations from manufacturers. The cost of civil works has been estimated by reference to similar installations elsewhere.

It has been assumed that diesel powered pumps will be used throughout, and the costs based on this. However, there will be electric power available when the factory has been rehabilitated and this could be used for pumping (see Annex IV). Electric power would, of course, require the extension and upgrading of the existing transmission line which presently serves the pumping station on the middle drain, with a spur to the west drain pumps. The viability of this merits further study since it could significantly reduce the operating costs. Its use has not been assumed for this study because reliability of the power supply would be of paramount importance.

The use of electric powered pumps for canal S1 (at times of lower river level) should be seriously considered since the existing power line runs very close to the pump station site. The electric powered pumps from the middle drain could be relocated here.

Item 1.5 Maintaining Existing Irrigation Supplies

Sums have been included for the maintenance of existing irrigation supplies. This is of major importance in rehabilitation work since it is vital that the Estate continues to produce cane. It will of course be necessary to take certain areas out of production in order to construct the new works, especially the changeover to the new field layout.

The sums allowed are intended to cover the cost of temporary culverts, canal diversions, pumping stations, etc.

14.4 Tertiary Canals and Infield Works (Bill Nr 2)

The rehabilitation works of the tertiary canals and the establishment of a new field layout has been discussed in Chapter 6. In Bill Nr 2 the works have been divided into six components as described below. Lengths of canals, drains and roads are derived from Table I.6.3.

(a) Canals (Item 2.1)

The existing tertiary canals, where required, will be cleared of weeds and a new section made, capable of taking the design discharge of 150 l/s. This operation has been costed as entailing two consectutive operations.

The header channels are new and should be formed using spoil material from the land-levelling operation to lay down a 4 m wide formation some 0.2 m above ground level. A V-ditcher is to be drawn along this formation to obtain the required section, see Figure I.7.1.

(b) Land Preparation (Item 2.2)

The rehabilitation works include bringing back into cultivation some areas that are presently abandoned. An allowance has been to clear this area (estimated as 500 ha) of weeds, bushes and other vegetation before the proposed field layout is constructed.

Land levelling quantities have been based on the sample areas surveyed, see Section 6.2.3. The costs assume laser controlled equipment and operated by an experienced contracting organisation at estimated contractors' rates throughout.

After three years, when the main engineering works would be complete, it might be possible for the Estate to take on the task of land levelling for the remaining 2 000 ha. This possibility should be considered when preparing the tender documents for the rehabilitation works.

Land preparation costs have been calculated assuming new equipment is purchased. Some reductions could be made if existing equipment is employed. The Estate's land levelling plant (Cameco scrapers) could be used by the Contractor for the five year programme of conversion to the new field layout. The plant would require the addition of laser control equipment for accurate levelling.

(c) Drains (Item 2.3)

New surface drains will be made along the foot of the furrows to the section shown in Figure I.12.6. The material generated should be used to form the adjacent field road. Costs of construction have been apportioned between the surface drain and field road, assuming the excavation cost applies to the surface drain and levelling of the road formation applies to the road.

(d) Structures (Item 2.4)

The costs have been estimated for the minor structures and include all engineering works assuming construction by contract, including the supply of control gates.

Rehabilitation of existing structures allows for cleaning out of silt/earth, minor repairs to concrete or brickwork and fitting with new gates. Where a structure has collapsed it is costed as a new structure and the cost of removal of the existing structure is included (in the cost estimate of the new structure).

(e) Roads (Item 2.5)

Field roads have been included as a separate item but their construction will use excavated material from the surface drain, as described in 14.3 (c).

(f) Miscellaneous (Item 2.6)

Siphons are to be supplied in year 1 as a bulk order. The theoretical requirement is 90 siphons of 50 mm diameter per 70 ha, but an additional allowance of 25% has been made to allow for easier distribution around the Estate and for wastage. Each siphon is assumed to be 2.5 m long.

14.5 Drains and Drain Structures (Bill Nr 3)

The rehabilitation works required to the drainage system have been described in Chapters 11 and 12. In Bill Nr 4 the works have been divided into seven major components as described below.

The costs include for all the main drain works necessary for serving the whole Estate, including outstanding works on the outlets into the Interceptor drain, but not including outstanding works on the Interceptor drain itself (see Section 14.9). They also include the collector drain and field drain works needed for the 5 300 ha net selected cane area but exclude field surface drains (these are included in Bill Nr 2).

(a) Deepen and Extend West Drain (Item 3.1)

The earthworks quantity for deepening the existing West drain has been estimated by comparing the outline design (Table I.12.8) with the cross sections surveyed in 1975 (Drawing Nr SOM 197). For the West drain extension, W2 and W2/2 branch drains, no existing cross sections are available and quantities have been estimated by comparing the outline designs (Tables I.12.9 to I.12.11) with the 1983 survey contours. Where the drain is to be constructed by deepening an existing collector a saving of 2 m^3/m length has been assumed.

(b) Remodel Collector Drains (Item 3.2)

The rate per kilometre for deepening existing collector drains is derived from an earthworks estimate of 8 m^3/m based on an average depth of 2.5 m below ground level with a 2 m^3/m saving from the existing collector.

The rate for shallow collector drains assumes weed cutting by flail mower and weed removal and minor reshaping by hydraulic excavator. It is about 2.5 times the estimated cost of routine drain maintenance by these methods.

The collector drain lengths are derived from the estimated field dimensions (Table I.6.1).

(c) Rehabilitate and Extend Middle and East Drains (Item 3.3)

The earthworks quantities have been estimated by comparing the outline designs with the existing cross sections as surveyed in 1975, or with the 1983 survey contours. This is the same procedure as used for the West drain (Item 3.1).

(d) Main Drain Structures (Item 3.4)

The numbers and sizes of structures required have been identified from the proposed layout and the outline design of the main drains, and standard bills of quantities have been prepared for typical sizes, see Table I.14.8. Costs have been allowed for construction of complete junction culverts at the outlets of the West and Middle drains into the Interceptor drain, but nothing has been included for the junction culvert on the East drain; this has already been completed under the Jowhar Offstream Storage Project.

(e) Deep Collector Drain Structures (Item 3.5)

The number of structures required has been estimated from the proposed layout. Quantities were estimated for each standard structure to give a rate for the structure.

(f) Shallow Collector Drain Junction Culverts (Item 3.6)

It is estimated that 25% of the existing structures will need replacing and the cost of new culverts was calculated as for Item 3.5 above. The remaining 75% will require cleaning and a nominal rate of SoSh 1 000 per structure has been allowed for this to cover labour and minor repairs.

(g) Deep Intermediate Collector and Buried Field Drains

The costs allow for the contractor to supply a trenching machine in Year 2 which would be capable of being used both for field drains and for intermediate buried collector drains. The rate per km for the intermediate collector is for an open drain rather than the lower figure for a buried drain (see Section 12.6.5). The rate per km for field drains covers the costs of fuel, maintenance, support machines, materials, labour, supervision, contractor's overhead and profit. The quantities in Year 3 are for the works required on field 8II.

14.6 Roads (Items 4.1 to 4.4)

The cost of road improvement works has been divided into four items. The most significant cost item is associated with the raising of road levels to avoid waterlogging. New road embankments would be constructed using material excavated as part of the drain improvement works. This material would have to be hauled up to 5 km for use as road embankment, but this is cheaper than excavating in an adjacent borrow area. Some material would also be available from the excavation of roadside drains, but this would be insufficient in itself for the new embankments.

All existing primary and secondary roads will require regrading to a cross fall. For the new road embankments the cross fall will be incorporated during compaction. Road culverts (or in some cases fall structures) will be required to

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Drainage Structures Required for Phase I Development

Chainage	Existing structure	Work required	Cost ($SoSh \times 10^3$)
C.3.1 East Drain			
15.2	None	New underpass	980
10.8	None	New underpass	1 440
9.1	None	New culvert	240
8.3	Tertiary underpass	Demolish	20
7.5	Culvert	Rehabilitate	5
6.7	None	New underpass	1 440
5.4	Tertiary underpass	Demolish	20
5.0	Culvert	Demolish	20
4.7	Tertiary underpass	Demolish	20
3.9	Culvert	Demolish	20
3.9	Culvert	New culvert	570
3.1	None	New culvert	570
1.6	Tertiary underpass	Demolish	20
0.9	Culvert	Demolish	20
0.9	Culvert	New culvert	570
0.0	Junction culvert	Rehabilitate	5
	Sub-total		5 960
C.3.2 Middle Drain			
9.4	Road/rail culvert	Rehabilitate	5
8.6	Tertiary underpass	Demolish	20
7.0	Tertiary underpass	Demolish	20
7.0	Road/rail culvert	New culvert	240
6.4	Tertiary underpass	Demolish	20
5.8	Tertiary underpass	Demolish	20
5.1	Tertiary underpass	Demolish	20
4.3	Road/rail culvert	Rehabilitate	5
3.5	Tertiary underpass	Demolish	20
1.7	Secondary underpass	Demolish	20
1.7	Pump station, culvert	Rehabilitate	5
1.65	Road/rail culvert	Demolish	20
1.65	Road/rail culvert	New culvert	570
0	None	New junction culvert	960
	Sub-total		1 945

Chainage	Existing structure	Work required	Cost (SoSh x 10^3)
West Drain			
11.3	None	New culvert	180
10.1	None	New culvert	180
9.2	None	New culvert	430
6.75	Road/rail culvert	Demolish	20
6.75	Road/rail culvert	New culvert	430
4.0	Road/rail culvert	Demolish	20
4.0	Road/rail culvert	New culvert	700
3.8	None	W2, junction culvert	(see W2)
2.4	Pump station	Demolish	20
2.4	Pump station	New culvert	940
1.6	Cross regulator	Demolish	20
0.5	Wooden footbridge	New footbridge	100
0	None	New junction culvert	1 100
	Sub-total		4 140
W2 Drain			
7.8	Underpass	Demolish	20
6.9	Tertiary underpass	Demolish	20
6.3	Tertiary underpass	Demolish	20
5.6	Tertiary underpass	Demolish	20
5.3	Road/rail culvert	Demolish	20
5.3	Road/rail culvert	New culvert	430
4.15	None	W2/2 junction culvert	(see W2/2)
4.10	Underpass	Demolish	20
3.7	Tertiary underpass	New underpass	980
2.35	S2 Underpass	New underpass	980
1.2	None	New culvert	430
0	None	Junction culvert	430
	Sub-total		3 370
W2/2 Drain			
2.6	Road culvert	Demolish	20
2.2	Tertiary underpass	Demolish	20
2.2	Tertiary underpass	New underpass	650
1.5	Road culvert	Demolish	20
1.2	Road/rail culvert	Demolish	20
1.2	Road/rail culvert	New culvert	180
0	None	Junction culvert	200
	Sub-total		1 110
	Total		**16 525**

connect the roadside drain at intervals to an adjacent main drain or collector drain.

14.7 Cost of Water Storage Options

The cost of providing storage reservoirs for plant cane irrigation is relatively small and has already been included in Bill Nr 1, since these are recommended for inclusion in the rehabilitation works. The costs of major water storage options of Duduble and Jowhar reservoirs, as described in Chapter 3 herein, are discussed below.

(a) Duduble Reservoir

It has been concluded that a reservoir of between 95 and 130 Mm^3 gross capacity would satisfy the Estate's demands more than 4 years in 5.

The outline design of the reservoir embankment and inlet and outlet structures has been prepared for a storage volume of 130 Mm^3, and quantities of the major earthwork and structural components calculated on this basis. Costs are summarised below.

	SoSh million
Embankment and inlet and outlet channels	67
Intake structure	20
Outlet structure	13
TOTAL	110

The outline designs have been based on those prepared for the JOSR, which has been successfully constructed, and therefore can be considered reasonably accurate. However certain assumptions have had to be made regarding site conditions. In particular, suitable fill material for the embankment has been assumed to be available from the reservoir area, generally close to the embankment line. Since no site investigaton work has been carried out this assumption cannot be supported by data on soil suitability. When the reservoir area was inspected in July 1983 the soils appeared more sandy than those in the Jowhar area. A site investigation programme would therefore be an essential prerequisite before any more detailed design work commences.

Costs of other reservoir volumes have been estimated from the work carried out for the 130 Mm^3 case. The results are summarised below :

Reservoir Volume	Estimated Cost (SoSh million)
95 Mm^3	93
150 Mm^3	120
200 Mm^3	145

It must be emphasised that these costs are preliminary estimates only. Costs could be much higher if site investigation reveals a shortage of suitable embankment material in the reservoir area.

(b) Pumping from Jowhar Reservoir

The major cost items involved in this option are raising the reservoir banks by 0.5 m, providing a supply canal to the head of the Estate, and the two pumping stations required.

Earthwork volumes have been estimated for the bank raising and for the new supply canal. Several routes for the supply canal have been considered; the best option is probably a straight line parallel and adjacent to the West drain. In this way material from the drain deepening works can be used to provide fill for the canal.

Costs of the two pumping stations have bveen based on the cost of a similarly sized station for the Mogambo Irrigation Scheme.

The main cost items are summarised below :

Item	Cost (SoSh million)
Raise reservoir embankment	17
Increase capacity of supply canal	4
Supply canal earthworks	10
Supply canal structures	12
Pumping station	19
TOTAL	SoSh 62 million

Recurrent costs of pumping have also been estimated. The computer output has been examined for the 22 year record to determine pumped volumes and average pumped head for each 5 day period. From these an average cost of fuel and oil (diesel powered pumping) has been estimated. A total average annual costs, including maintenance and replacement, of SoSh 1.1 million has been estimated.

14.8 Recurrent Costs

Recurrent costs for the irrigation and drainage system have been estimated as follows:

14.8.1 Pumping Costs

(a) Irrigation Pumping

The pumping requirements have been estimated assuming an annual pumped volume of 24 300 m^3/ha with an average pumped head of 2 m. A pump efficiency of 60% has been taken giving an annual power requirement of 221 kWh/ha. At SoSh 6.7/l of fuel plus 15% allowance for oil and at 0.3 l fuel/kWh an estimated annual cost of SoSh 500/ha/year is given.

The 5 300 ha development includes 1 385 ha that require to be pumped all the year and 500 ha of part-time pumping when river levels are low. Thus with an equivalent pumped area of 1 620 ha the annual fuel and oil costs for irrigation pumping amount to SoSh 810 000. This will apply from year 5 onwards. A linear build-up of costs will apply from year 2 to year 5.

Pump and engine maintenance costs have been estimated at 6.5% of the plant capital cost.

(b) Drainage Pumping

The major pumping requirement is at the drainage pump station to pump drainage water to the disposal area.

It has been assumed for costing purposes that the present requirement is to remove only the 93 mm/year ineffective rainfall from the total gross field area of 8 300 ha. This amounts to a volume of 7.7 Mm^3 per year which is to be pumped against a total head of 7.28 m. The total cost of this in diesel and lubricants would be SoSh 0.6 million per year (diesel costed at SoSh 6.7 per litre). This cost is assumed to apply for year 1 (1985) when the new works will have had little effect.

The drainage requirements after development are estimated as follows :

- 1.8 mm/d sub-surface drainage over 2 100 ha gross cane area on the West drain;

- 0.6 mm/d regular surface runoff from irrigation over the 5 900 ha gross cane area;

- 93 mm/year runoff from ineffective rainfall over the 8 800 ha gross total field area.

This gives a total volume pumped of 34 Mm^3 per year and an annual diesel and lubricant cost of SoSh 2.6 million. This is assumed to apply from year 5 onwards. A linear build up of pumping costs has been assumed from year 1 to year 5.

Pump and engine maintenance costs have been estimated on the same basis as for irrigation pumps.

14.8.2 Operation and Maintenance Plant and Vehicles

For each item of plant the average annual hours of use or average annual distance covered have been estimated.

Costs have then been estimated using appropriate rates which include fuel, oil, operators, drivers, servicing and maintenance. Details are presented in Annex VII.

Replacement costs for plant have been based on the assumed useful lives of the various items (see Annex V).

14.8.3 Materials for Operation and Maintenance

An annual allowance of 0.5% of the rehabilitation capital cost has been included for materials such as cement, sand, gravel, grease, paint, herbicide, etc. An additional allowance has been included for replacing siphon pipes, tools, stationery, etc.

14.8.4 Staff

The cost of operation and maintenance staff has been based on the proposed staff numbers and recommended salary rates as given in Annex VII.

14.8.5 Field Drain Installation

It is anticipated that from year 5 onwards a programme of field drain installation will be required as explained in Section I.12.5. The capital cost for the drain laying machinery is included in Table I.14.6, but the investment cost estimated at SoSh 2 million per year for field drain installation when required, has not been included as explained in Section 3.9 of the Main Report and Annex VII.6.

Recurrent costs are detailed in Table I.14.9.

TABLE I.14.9

Summary of Recurrent Costs for Irrigation and Drainage

Item	Annual Costs Year 5 onwards (SoSh)
Irrigation pumping :	
- fuel and oil	810 000
- maintenance	210 000
Drainage pumping :	
- fuel and oil	2 600 000
- maintenance	285 000
O and M Plant and Vehicles	3 903 420
Materials	1 030 000
Siphons, tools, stationery, etc.	400 000
Staff	7 052 268
Total	**16 290 688**

Note : (1) Table excludes replacement costs.

14.9 Cost of Completing the Outstanding Drainage Disposal Works

As described in Sections 8.5 and 12.8, construction of the Jowhar Offstream Storage Reservoir (JOSR) has disrupted the Estate's previous drainage disposal arrangements and the replacement works included under the JOSR project are still incomplete. These should be completed urgently to prevent further deterioration of conditions on the Estate and to serve the proposed main drains.

The outstanding works comprise:

- completion of the main drain outlets from the West and Middle drains;
- deepening of the Interceptor drain and drainage disposal channel to the JOSR design cross-section (this is also appropriate for the proposed Estate drainage system);
- delivery and installation of pumps in the drainage pump station.

The design of the main drain outlets may need revising to suit the proposed main drain designs, and therefore it is proposed to include these in the Estate rehabilitation project. The costs of new outlets from the West drain and Middle drain are included in Item 3.4.4(c) of Table I.14.6 above.

The remaining works on the Interceptor drain, drainage disposal channel and drainage pump station should be completed under the JOSR project urgently to the original designs. The earthworks required have been estimated from the 1983 survey (drawing 12700/8) and the estimated costs at mid-1983 prices are detailed in Table I.14.10. It should be noted that the JOSR contractor, the Water Development Agency, has already been paid for this work.

TABLE I.14.10

Estimated Current Costs of Outstanding Drainage Disposal Works (SoSh '000)

1.	Interceptor drain earthworks (117 000 m^3)	5 850
2.	Disposal channel earthworks (14 500 m^3)	725
3.	Drainage pumps supply	5 700
4.	Drainage pumps installation	200
TOTAL		12 475

APPENDIX A

CANAL STRUCTURE AND CHANNEL INVENTORY

APPENDIX A

Canal Structure and Channel Inventory

21st October Canal

Chainage	Structure	Notes
		Intake channel from river. Suffers from heavy sediment deposition. At present clear of weeds/ reeds. Not large enough for effective sediment removal. At present is desilted by dragline about 3 times each year.
0.000	Head regulator	Constructed 1970. Reasonably sound condition although top reinforcement in bridge deck is exposed - needs new concrete surface. All gates and lifting gear in good condition. Gate size 2.4 m wide x 4.0 m high, three number.
2.750	Offtake on left	1.25 m wide rising spindle steel gate. Reasonable condition. (Note : structure clear width = 1.0 m.)
3.350	X-reg	Regulator completely drowned. Four steel gates 1.7 m wide, badly corroded - need replacing. Canal banks very overgrown - reeds and bush. Offtake to right across old S5 canal, 1.3 m gate.
4.050	Offtake	Offtake to right across old S5 canal. No head regulator, pipe under canal bank not apparent, 0.7 m wide gated X-reg just downstream, gate and structure poor.
4.500	Offtake on left	Ditto Km 2.75. Bridge on offtaking canal reasonable also.
4.850	Road/rail bridge	Water level only 0.08 m below deck level. Visible parts of structure in reasonable condition. Canal clear of growth.
5.600	Offtake	Offtake to right across old S5. No head regulator, pipe under canal bank not apparent, 1.0 m wide gated X-reg just downstream in sound condition.

21st October Canal (cont.)

Chainage	Structure	Notes
5.750	Offtake on left	2.0 m wide rising spindle gate. Poor condition. Offtaking canal very overgrown. Desilting of the 21st October Canal has resulted in the offtake head regulator being almost buried in silt.
7.200	X-reg/bridge	Gates (4 x 1.8 m wide) and lifting frame need replacing. Structure drowned. Offtake to left - gate needs replacing. Offtake to right reasonable (single gated X-reg 1.0 m wide downstream in reasonable condition).
8.200	Pump	Diesel engine driven, supplying tertiary canal on left bank. Similar units to those at end of Duchessa Canal (q.v.). Pump and engine dated 1975. Heavy reed growth in canal.
10.300	Road bridge	Good condition generally. Some problems with settlement behind abutments leaving approaching slabs unsupported.
11.200	X-reg	Structure completely drowned. Three gates raised out of water, gantry and gates fair condition. 1.0 m wide gated offtake to right in fair condition. Canal clean of growth in centre, but edges heavily overgrown with silt berms.
13.500	Offtake to right	Inaccessible (surrounded by water). 1.0 m gate in reasonable condition.
14.100	Offtake to right	1.0 m gate on right bank - poor condition. Brushwood weir across 21st October Canal.
14.800	X-reg	Four 1.7 m wide steel gates, all corroded, need replacing. Offtake on left bank ungated (serves areas outside Estate). Offtake on right bank, gate damaged. Canal very overgrown.
16.350	X-reg	No gates (space for 2 x 4 m gates/stop logs). Offtake on left bank ungated. Offtake on right bank reasonable condition. Canal completely overgrown with reeds. Very little flow.
17.800	Offtakes to right and left	

21st October Canal (cont.)

Chainage	Structure	Notes
18.400	X-reg	4 x 1.6 m wide gates, all corroded. WL at structure top. Offtake on right, 1.0 m wide gate in reasonable condition. Canal clear of growth.
19.200	Tail escape	Good condition. Back flow from supply canal observed when supply canal flow is high. Deep silt deposits in channel and structure upstream and in the structure downstream. Flap gates on outlet appear sound.

General Comments

Silt deposition in the canal, coupled with heavy reed growth in many reaches, has caused a significant reduction in conveyance capacity. In order to get maximum flow down the canal it is necessary to operate with a very high water level which completely drowns some of the structures and leaves very little freeboard on the remainder.

Some clearance of reeds and silt had been carried out by dragline before the field study, but this machine was standing idle on the canal throughout the field work (broken down).

In some reaches of canal where clearance has taken place, only a relatively small channel in the centre has been desilted and there are well established silt berms on both sides with extensive reed growth.

Canal banks are generally overgrown, sometimes with heavy bush on the outside, making vehicular access dfficult and preventing routine inspection of the canal. Roads along the canal banks are normally reasonable but could be improved by removal of vegetation and by grading to a cross fall.

In the main west - east reach of the canal, offtakes to the south are served either by passing water into the old S5 canal (which runs parallel and adjacent) and using the offtakes from S5, or with new offtakes from 21st October Canal which cross the old S5 canal.

Structures on 21st October Canal generally appear to be sound, but the very high water levels and extensive silt deposits made inspection of the sub-structure impossible. All gates have suffered from corrosion - some need immediate replacement. Gate frames, especially on offtakes, are often insecurely mounted and distorted.

APPENDIX A (cont.)

Canal Structure and Channel Inventory

Canal S3

Chainage	Structure	Notes
0.000	Head regulator	See notes on Duchessa Canal.
0.800	X-reg	Brick structure, fair condition 4 x 1.2 m bays (effective width 1.0 m). No gates, lifting gantry on two brick piers in reasonable condition but pier stability dubious. Offtakes to left and right, right ungated. Left offtake leads to division structure to supply two tertiary canals.
2.400	X-reg and road/rail bridge	No gates on X-reg - 4 x 1.2 m wide bays, fair condition. Offtakes to left and right. Right offtake has no head regulator. Offtake to left has 1.0 m gate. Brick arch culvert 2 x 2.3 m wide openings, silted up, generally sound but headwalls need repair.
3.100	X-reg	Structure reasonable. No gates, 4 x 1.2 m bays. Offtakes to left and right. Left bank offtake needs new gate and frame. Right bank offtake reasonable but gate corroded.
3.700	X-reg	Crude broken wooden gates, 4 bays 1.0 m wide x 2.1 m high. Gate gantry sound. Structure in good condition but structure sill under 0.6 m of silt. Offtake to left, 2 x 1.0 m dia. pipes, both silted up to half depth. Only one gate, corroded but usable.
4.400	X-reg	Structure reasonable. 4 x 1.0 m bays, only one with wooden gate in poor condition. Needs repair to gate frames. Gantry is intact with pulley lifting block. Offtake to right serves storage basin. Offtake to left. Both offtakes have 0.9 m wide gates in poor condition but usable.
5.200	Road/rail bridge X-reg	Brick X-reg in reasonable condition but two of the three piers for gate lifting gantry missing. Four bays (1.0 m wide) all gates missing. Gated offtakes to left and right, 1.0 m wide in fair condition. Road bridge, sound, 4.8 m wide central span with side spans silted up.

Canal S3 (cont.)

Chainage	Structure	Notes
6.000	X-reg	Brick structure sound, 4 x 1.0 m bays. No gates, lifting gantry intact but pier stability dubious. Offtakes to left and right, both 1.0 m wide. Right offtake has no gate or gate frame, left has gate in poor condition and frame loose.
6.800	X-reg	Brick structure with 4 x 1.2 m wide gates (3 missing). Steel gantry with gate lifting pulley block. Sound condition. Canal S4 connects just upstream (q.v.)
7.950	Aqueduct over drain	
8.050	Culvert	2 x 1.0 m openings visible, possibly one more (obscured by silt). Temporarily blocked off upstream. Fair condition.
9.100	Offtake	Not observed.
10.050	Offtake	Offtake to right. Comprises upstream headwall with 1.0 m wide ungated opening (provision for gate/stoplogs), exposed 0.70 m dia. pipe over drain (drain not well defined), road culvert, downstream headwall. All in good condition.
10.450		Canal turns north
11.150	Offtake	1.0 m wide ungated offtake to right.
11.450		Canal ends

General Comments on Canal S3

This canal was cleaned out by hand during the field work, the task being undertaken when the canal was dry. It is a wide canal in reasonable condition, the bed is silted up between 0.5 and 1.0 m and the banks are overgrown on the outside. Structures are generally sound and can be rehabilitated quite easily. Most of the cross-regulator gates are missing and offtaking gates are often poor.

Canal Structure and Channel Inventory

Luigi di Savoia and Duchessa D'Aosta Canals

Chainage	Structure	Notes
		Intake channel from river. Suffers from very heavy sediment deposition. No reed growth. Not large enough for effective sediment control - needs desilting on a regular basis (can fill completely with sediment in three weeks). Is presently desilted by dragline on average 12 times a year (i.e. each month).
0.000	Head regulator	Three 2.1 m wide gates and provision for stoplogs. Structure generally sound. Gate lifting gear and frames reasonable. Gates need replacing. Gates are undershot and hence do nothing to reduce sediment quantity passing into the canal. Sill level is about 2 m below Jowhar weir crest level.
0.450	Road bridge	In reasonable condition but silted up - hence high water level.
	X-reg	Four gates : - 2 outer each 1.35 m wide, rising spindle. Frames loose. New gates needed. - 2 inner each 2.0 m wide. Raised by pulley on lifting frame. In fair condition. Brick structure reasonably sound. Presently operating with very high water level.
	SI offtake	2 x 1.0 m wide gates in fair condition. There is also a pump upstream of the head regulator which is used when gravity supply is not possible.
1.750	Duchessa canal head regulator	Although nearly 60 years old this brick structure is reasonably sound. There are four gates - 2 steel 1.0 m wide and 2 wooden 1.2 m wide, all raised by pulley from overhead gantry. Wooden gates should be replaced by steel gates. Canal bed is badly silted up both upstream and downstream.

Luigi de Savoia and Duchessa D'Aosta Canal (cont.)

Chainage	Structure	Notes
1.750	Offtakes upstream on left bank	One gated outlet to canal S5 (abandoned) and one 1.5 m gate (frame loose) supplying the water storage basin.
	Offtake to canal S2 (right bank)	Direct branch offtake upstream of Duchessa Canal head regulator. Badly silted up.
3.000	Wooden footbridge	Bridge completely collapsed into channel and is restricting flow.
3.350	Offtake to left	1.0 m gate and frame need replacing. Structures completely blocked. Canal very shallow and overgrown.
3.900	Road/rail bridge	In reasonable condition (steel girder construction). Water level presently very high (only 0.10 m below bridge deck). Canal clear of reeds downstream.
4.900	Offtake on left	
5.300	Footbridge	Wood. Collapsing and restricting flow.
5.650	Offtake to left	Heavy reed growth in canal.
6.300	Road/rail bridge	Brick in reasonable condition. Damage to headwall.
6.350	Tail group Start of canal S3	Brick X-reg reasonably sound. Four wooden gates (one missing) 1.3 m wide x 1.6 m deep. New steel gates required. Concrete support pillars for gate gantry in poor condition. Two offtakes, one to left one to right, both have 1.5 m wide steel gates in fair condition. Offtake structures on skew hence effective width about 1.0 m. Canal S3 heavily overgrown with reeds.
	Pumps	Located between road bridge and tail group. Two diesel driven pumps lifting water from drain in to end of Duchessa (to supply canal S3). Pump rated capacity 1 240 m^3/h at 12 m head. Engines 70 kW rated. One engine 1977 vintage, other not known. Pumps and engines from GDR. Actual static lift about 5.0 m.

Luigi de Savoia and Duchessa D'Aosta Canal (cont.)

General Comments

Extensive sediment deposition and heavy reed growth has reduced the channel capacity to a fraction of its design value.

Access on banks is generally good, but growth on banks obscures view of canal and makes inspection difficult. Standard of tertiary offtakes is generally poor in particular gates and gate frames need replacing/repair.

The problems of silt deposition were graphically illustrated when, in late June, the river water level started to fall. In mid-June a river level of approximately 105.0 m was sufficient to ensure flow down the Luigi Canal. However, when this dropped by about 0.35 m (to 104.65) the level of silt deposited in the intake was such that all flow to the Luigi Canal ceased. Desilting by dragline commenced immediately (30th June) and the intake was cleared to some extent by 5th July (say, 5 working days). The dragline then continued working down the Luigi Canal.

APPENDIX A (cont.)

Canal Structure and Channel Inventory

Canal S1

Chainage	Structure	Notes
0.000	Head regulator	Offtakes from Luigi canal through head regulator with 2 x 1.0 m wide gates in poor condition. Head regulator and canal downstream silted up. In addition there is a pump positioned next to the head regulator which is used to augment supplies.
0.300	Road/rail culvert	Silted up but appears sound
0.600	Offtake to left	Structure has no head regulator or gate.
1.800	X-reg	Single gated structure of straight wall type. Gate 0.85 m wide, corroded but usable. Structure sound (rendered brick?). Canal overgrown upstream and downstream.
2.400	Offtake to left	1.0 m wide gated offtake. Gate corroded but operable. Canal very overgrown with reeds and bush.
2.500	Culvert and X-reg	Single gated brick structure with reinforced concrete culvert roof slab. Gate 0.9 m wide by 1.7 m high, in poor condition. Head walls in poor condition, bottom reinforcement in culvert roof exposed.
3.250	X-reg	Single gated structure, gate 1.2 m x 1.2 m corroded but in fair condition, frame bent but operable. Structure reasonable, orifice 1.13 m wide x 0.95 m deep. Canal scoured downstream 0.4 m below structure sill. Offtake on left in poor condition, 1.15 m wide gate (needs replacing), no frame.
3.500	Culvert with X-reg downstream	Culvert has 2 x 1.1 m wide x 1.15 m high openings. Reinforced concrete deck on brick walls, one brick wall falling in, otherwise sound. X-reg single gated structure in fair condition. Gate crude but workable, opening 1.0 m wide x 1.1 m high.

Canal S1 (cont.)

Chainage	Structure	Notes
3.950	Culvert	Canal turns east and passes under culvert, now in fact only a tertiary canal. Culvert upstream headwall has collapsed into canal. Brick walls, reinforced concrete roof slab, reinforcement exposed and corroding. Structure needs replacing. Canal overgrown upstream and downstream. Downstream canal has various offtakes and crude footbridges/cross regulators of brushwood.
4.800	Culvert	Poor structure, blocked off upstream by mud and sticks. No sign of downstream headwall. Canal ends.

General Comments on Canal S1

This is the smallest secondary canal and is in poor condition, generally overgrown and silted up. There are obviously problems in supplying it by gravity in its present state.

APPENDIX A (cont.)

Canal Structure and Channel Inventory

Canals S2 and S4

Chainage	Structure	Notes
		Canal S2 offtakes from Luigi canal immediately upstream of Duchessa head regulator. No head regulator for S2 on Luigi canal.
0.000	Road/rail bridge	Appears in reasonable condition but presently silted up to just below soffit (1.0 m depth of silt).
0.100	X-reg	Brick structure, condition not too good. 4 x 1.0 m wide bays, all gated but only one gate in reasonable condition - others badly corroded. Outer gates jammed closed. Inner gates only have crude lifting frame. Two 1.0 m wide offtakes - structures, gates and frames all in poor condition.
0.450	X-reg	Simple brick structure 0.4 m thick. Three visible openings 1.0 m wide with provision for stoplogs upstream. Reasonable condition but not very effective as cross-regulator. Offtake to left, 1.0 m wide gate, fair condition.
1.200	X-reg	Same as that upstream, 3 x 1.0 m openings, no provision for stoplogs. Poor condition - brick has deteriorated. Offtake to left, 1.0 m wide gate, fair condition.
1.600	Offtake	Offtake to right, 0.5 m wide gate in poor condition. Culvert has collapsed at downstream end - access across difficult.
1.800	X-reg	Same as those upstream but 4 x 1.0 m openings visible. Good condition. No provision for stop-logs. Offtake to left, 1.0 m wide gate, frame and gate need replacement.
2.200	Offtake	Offtake to right, 0.6 m wide, poor condition. Canal overgrown but clearing/desilting in progress downstream.
2.500	Culvert (road/rail)	Has 3 or more 1.0 m wide openings, completely silted up (culvert is 10 m long). Brick in poor condition.

Canals S2 and S4 (cont.)

Chainage	Structure	Notes
2.550	X-reg	Same as those upstream, 3 x 1.0 m wide openings, provision for stoplogs. Poor condition. Offtake to left, 1.0 m gate, gate poor, structure fair.
2.950	Offtake	Offtake to right, 1.0 m wide, needs new gate, otherwise fair.
3.200	X-reg	X-reg as those upstream, 3 x 1.0 m wide openings, poor condition. Offtake to left has no head regulator - appears to be just a pipe through canal bank. Flow being diverted into abandoned field 11 II.
3.750	Drain underpass	Structure not visible. Drain overgrown. Drain bed level about 3.7 m below canal bank top level. Road on canal right bank very overgrown. Canal very badly overgrown with reeds (cleaned on one side by end of June 1983.)
4.100	X-reg	Offtakes to left and right. Left bank offtake 1.0 m wide gated head regulator, poor condition; flow being diverted into abandoned field 1 III. Right bank offtake also poor. X-reg as others upstream.
4.750	X-reg	Structure as others upstream, 3 x 2.0 m openings, fair condition. Offtake to left, 1.0 m wide, poor condition. Offtake to right has no gate.
5.800	Offtake	Offtake to right serving maize outside Estate - no structure.
6.300		Brushwood and mud temporary X-reg. Various small offtakes to south (new field).
7.000	Road bridge and X-reg	Structure originally had three gates, now only one (1.0 m wide). Condition of structure fair. Silted upstream. Offtake on left bank - head regulator has 1.0 m wide gate with 2 x 0.80 diameter pipes downstream (one blocked off?)
7.500		Brushwood and mud temporary X-reg.

Canals S2 and S4 (cont.)

Chainage	Structure	Notes
7.950	X-reg	Three 1.0 m wide openings, provision for gates/ stoplogs, fair condition. Offtake to right 1.0 m wide, fair condition.
8.100		Offtake to left.
8.600		Canal discharges into West Drain. Pumps lift water from West Drain (upstream) and discharge into downstream drain which is used for irrigation. Water can be pumped into canal S4 using another pump.
8.650		Start of canal S4 (canal completely overgrown).
9.850	Culvert	Brick construction, bricks deteriorating but generally sound. Three 1.0 m wide openings blocked off temporarily upstream. Canal very overgrown.
11.00	Joins S3	Tail structure substantial, brick construction. Four 1.0 m wide openings. Joins S3 upstream of brick X-reg.

General Comments on Canals S2 and S4

One side (the left) of S2 has been cleared using a hydraulic excavator during the field work. Average rate of progress of clearing was quite good at about 350 m per day, but the silt has been deposited on top of the left bank making access impossible. The right half of the canal remains silted and very over-grown. Access to the right bank is possible but extensive reed and bush growth make inspection of the canal difficult. Cross regulation is crude and the majority of the structures were drowned prior to the canal cleaning. Many of the offtakes are in poor condition. Temporary brushwood weirs are used at the lower end of the canal to maintain command. Canal S4 is very overgrown.

Canal Structure and Channel Inventory

Canal S5 (Abandoned)

Chainage	Structure	Notes
0.000	Head regulator	In reasonable condition. Takes off from left bank of Luigi canal upstream Duchessa head regulator.
0.700 + 0.900		Two offtakes on left bank serving areas outside Estate. Canal very overgrown, especially for first 500 m.
2.850	X-reg	Buttressed structure in poor condition. Two 1.2 m wide openings but no gates or lifting gear. Housing for left gate broken. Silted up. Offtake to right.
3.550		Outlet from 21st October canal into S5 canal.
3.650		Canal ends - channel downstream totally overgrown. Offtakes to south now served from 21st October canal (q.v.).
5.700	Culvert	Canal still unused, very overgrown. All offtakes downstream served by 21st October canal (q.v.).
9.350		End of old S5 canal.

General Comments

Canal S5 has not been used for some years. Tertiary canals previously offtaking from S5 are now served by the 21st October canal. Downstream of Km 3.650 the canal is totally overgrown with bush and reeds and could not be recommissioned without major reforming and the construction of new control structures.

APPENDIX B

DRAIN STRUCTURE AND

CHANNEL INVENTORY

APPENDIX B

Drain Structure and Channel Inventory

West Drain

Chainage	Structure	Notes
0.000		Proposed connection into Interceptor drain. Present situation: old west drain passes over Interceptor drain on bund and discharges through a culvert (presently blocked) in the northern bund of the offstream storage reservoir. Alongside and to the east there is a 125 m long section of the new alignment not connected in.
0.500	Footbridge	Wooden footbridge across drain giving access to village. New structure required when drain is deepened.
1.600	X-reg	Drain serves as a canal. X-reg on drain has 1.0 m wide single gate. Offtake to left has 2 x 0.8 m gates (one missing). Structures generally sound. West drain is much deeper downstream. Water level at about field level and drain very overgrown with reeds.
2.400	Pump station	The West drain discharges through a culvert into a concrete basin from whence it is pumped into the higher downstream reach of the west drain which serves as a canal. Collector drains from east and west discharge into the same basin. Pump diesel driven unit rated at 1 240 m^3/h at 12 m head. Engine rated at 70 kW. There is electric power but this is not used.
4.000	Road/rail culvert	Brick structure with 4 x 1.0 m dia. pipes. Pipe inverts high (observed above upstream and downstream water level). Drain bed level about 0.5 m below pipe invert. Brick in poor condition, downstream headwall collapsing into drain. Drain downstream relatively clear. Drain upstream overgrown by weeds. Unidentified remnants of structure immediately upstream and downstream of the culvert on right bank could be the inlet and outlet of a lower level by-pass.

West Drain (cont.)

Chainage	Structure	Notes
5.500	-	Drains on both banks. Drain from west on right bank (drains Farm 1 area) connects directly into west drain. Drain very overgrown.
6.150	-	No sign of incoming collector drain culverts. West drain clear downstream but banks very overgrown with bush. Drain dry.
6.750	Road/rail culvert	Brick structure, 3 x 1.0 m dia. pipes. Pipe invert 2.4 m below road level. Structure reasonably sound. Drain downstream overgrown with reeds and bush, drain upstream clear of reeds but overgrown with bush and trees on banks.
9.100		Start of drain (very overgrown).

APPENDIX B (cont.)

Drain Structure and Channel Inventory

Middle Drain

Chainage	Structure	Notes
0.000		Proposed connection into interceptor drain. Present situation: old Middle drain passes across the Interceptor drain on a bund and discharges through a culvert in the northern bank of the offstream storage reservoir. Alongside and to the west 250 m of a new drain has been excavated; this is not connected in and is not along the designed alignment for Contract J01 of the Jowhar Offstream Storage Project. The tail end of the old 21st October canal connects with the Middle drain just upstream.
1.650	Culvert	In reasonable condition but number and dimensions of pipes not identified, present bed level to road level is 3.2 m; structure headwall 1.5 m below road level. Headwalls 11 m wide, length of structure 7.5 m. On 10.7.83, with upstream pump working, water was flowing through the culvert, downstream heavily reeded.
1.700	Pump station	Water pumped from middle drain into aqueduct carrying canal S3 over the drain. Culvert brings in water to pumping sump from fields to the west.
2.800	-	Collector drain
3.500	Underpass	Structure impossible to examine in detail (submerged), slight flow of water. Outlet downstream 5.0 m wide between side walls. Estimated depth of culvert from soffit to sill = 1.0 m. Middle drain upstream is very small in section.
4.300	Culvert	Drain passes under road/rail and tertiary canal. Bed level of drain about 5 m below road level. Structure appears sound. Inlet width = 4.5 m, size of pipes not observed. Collector drain connects in from east (south of culvert).
5.050	Underpass	Not observed, assumed similar to that at Km 8.700.

Middle Drain (cont.)

Chainage	Structure	Notes
5.800	Underpass	Not observed, assumed similar to that at Km 8.700.
6.400	Underpass	Structure inspected but not visible (drowned). Mature tree growing where inlet should be, appears same as at Km 8.700.
7.050	Road culvert and Underpass	Road culvert appears to be blocked - no sign of flow. Headwall length upstream = 6.4 m, pipes could not be seen. Underpass under tertiary silted up. Drain downstream overgrown. Drain upstream has only small cleared section in middle, otherwise overgrown.
6.000	Underpass	Probably comprises 2 or 3 x 2.0 m dia. pipes. Structure looks sound but appears to be set too high. Outlet width between side walls = 3.6 m. Drain downstream totally overgrown. To the north a new drain has been excavated by the Estate, with a new culvert passing under the road just to the east of the end of the Duchessa canal. This culvert conveys drainage water from the fields between the Duchessa canal and 21st October canal, passing the water into the newly excavated drain. Unfortunately this drainage flow (which during the field work was quite substantial) was unable to pass through the underpass at Km 8.7 on the middle drain. Instead it flows eastwards along a collector drain and spills out onto field 2 VI.

Drain Structure and Channel Inventory

East Drain

Chainage	Structure	Notes
0.000	Outfall	East drain outfalls into the old 21st October canal on the southern boundary of the Estate.
0.900	Culvert	-
1.600	Underpass	Drain passes under tertiary canal. Upstream the drain is used as a canal (the two are not connected).
3.100		Drain re-starts again near village (south-eastern corner of field 12 V). Downstream the drain, if ever constructed as such, is used as a canal. Water enters this 'canal' from the tertiary immediately to the north. The canal continues south and links into the tail end of S3.
3.900	Culvert	Silted up almost to soffit, overgrown. Probably comprises one or more 1.0 m dia. pipes but impossible to see. Parts of headwalls visible appear sound. 0.3 m diameter collector drain culvert (pipe only) enters just downstream of east drain road culvert on left bank. Flow appears to go northwards up the East drain in the wrong direction.
4.650	Underpass	Drain goes under tertiary canal. Structure overgrown and silted up - not possible to determine size or condition.
5.000	Road culvert	Road crosses from west side of drain to east side. Culvert comprises 0.50 diameter pipe (recently installed) with soffit only about 1.0 m below road level - totally ineffective, being set too high, too small and filled in at downstream end. East drain shallow and dry upstream and shallow, dry and overgrown downstream. Collector drain from west has no culvert connecting into East drain.
5.400	Underpass	Drain goes under tertiary canal. Appears that drain culvert has collapsed at upstream end but structure could not be located. East drain downstream is quite large and deeper than upstream with a little water (upstream about 1.5 m deep, downstream about 3.5 m).

East Drain (cont.)

Chainage	Structure	Notes
6.000		Drain bends through 90°. Drain depth about 2 m. Collector drain from west may have pipe culvert but pipe fragments observed in nearby excavated material.
6.100	Underpass	Drain bends through 90° and passes under a tertiary canal. Drain very overgrown and structure obscured. Structure comprises exposed 1.0 m dia. concrete pipes carrying tertiary supported on drain pipe structure. Pipe for drain seems to be set too high. Some cracking in structure but generally sound.
6.800	Culvert	Drain bends through 90°. Drain depth about 1.8 m. Drain and structure overgrown - not possible to inspect. Collector drain from east does not appear to connect in.
6.900		Drain bends through 90°. Drain upstream very overgrown with reeds.
7.750	Culvert	Culvert not located. Drains from left and right - no sign of junction culverts. East drain fairly well defined downstream.
8.550	Underpass	Drain passes under tertiary canal. Similar to structure at Km 3.000. Drain depth about 1.5 m, overgrown. Soffit level of underpass only about 0.2 m above drain bed level. East drain does not appear to be connected to the next reach upstream.
9.250		Start of east drain. Drain very poorly defined and overgrown. Drain stops after 300 m at village, restarts south of village. East-west running collector drains at head of East drain do connect in through pipe 0.5 diameter.

Summary

The East drain cannot at present be considered to operate as a drain. It is a discontinuous series of ill-formed channels of inadequate depth, currently overgrown. It is unlikely that any of the structures would be incorporated into a rehabilitated drain.

APPENDIX C

CANAL DETAILS

Notes :

1. Canal structures shown in **bold type** are new structures.

2. Net cane areas quoted are for the proposed new field layout (3% less net area than existing layout).

3. Field numbers refer to the old numbering system.

CANAL S1

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	1.490/1.414	**H-Reg**	1	1I	35.1
0.300	1.409/1.409	Road/rail culvert			
0.600	1.404/1.094	**X-Reg**	1	2I, 3I, 4I	144.0
1.800	1.076/1.076	X-Reg (to be demolished)			
2.400	1.066/0.799	-	1	6IN, 7IN, 5I	124.3
2.500		**X-Reg culvert**			-
3.250	0.789/0.492	X-Reg	1	8I, 6IS, 10IN, 7IS, 11IN	138.0
3.500	0.489	Culvert X-Reg	1	9I, 10IS, 11IS, 12I, 13I	227.3
TOTAL					668.7

CANAL S2

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	-/4.404	**H-Reg**			
1.800	4.351/-	Into storage reservoir			
		Reservoir outlet regulator			
0.000	2.851/2.717	Bridge and X-Reg (X-Reg not needed)	1	3II, 4II	62.2
0.350	2.708/2.421	X-Reg	1	5II, 6II	133.6
1.100	2.404/2.164	X-Reg	1	7II, 8II	111.6
1.700	2.151/1.880	X-Reg	1	9II, 10II	126.2
2.450	1.865 1.618	X-Reg	1	11II, 12II	114.8
2.500		Culvert			
3.100	1.606/1.365	X-Reg	1	13II, 14II	112.0
3.650	1.355/1.355	Drain underpass to be demolished or filled in			
4.000	1.349/0.898	X-Reg	2	1III, 2III, 3III, 4III, 14I	209.9
4.700	0.888/0.529	X-Reg	2	15I, 5III 6III	166.8
5.350		**Drain underpass**			
6.000	0.521/0.415	-	1	7III	49.5
7.000	0.400/ -	Tail	2	8III, 9III, 10III, 11III	186.2
TOTAL					1 272.8

CANAL S3

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	- /3.492	**H-Reg**			
1.810	3.442/3.125	Bridge and X-Reg	1	3IV, 4IV	147.6
2.580	3.105/2.777	X-Reg	1	5IV, 6IV	152.8
4.180	2.737/2.406	X-Reg/Bridge	1	7IV, 8IV	154.0
4 840	2.391/1.774	X-Reg	2	9IV, 10IV, 11IV, 12IV 1V, 2V, 3V, 4V	379.0
5.480		X-Reg	-	-	-
6.220	1.747/1.549	X-Reg/Bridge	-	-	-
6.980	1.535/1.225	X-Reg	1	5V, 6V	144.3
7.780	1.212/0.896	X-Reg	1	7bIII, 8bIII	147.1
8.520	0.886/ -	X-Reg/Tail	2	12III, 13III, 14III, 15III, 16EIII, 16WIII, 17III, 18III, 18bIII	412.3
TOTAL					1 537.0

CANAL S6

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	- /0.976	**H-Reg**			
1.810	0.950/0.660	**Culvert**	1	1VI, 2IV	135.1
2.580	0.651/0.334	**X-Reg**	1	3VI, 4VI	147.3
4.180	0.320/ -	**Tail**	1	5VI, 6VI	148.8
TOTAL					431.2

CANAL S7

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	- /2.394	**H-Reg**			
1.750	2.354/2.354	**Drain under-pass**			
2.400	2.339/1.833	**X-Reg**	1	9VI, 10VI 7VI, 8VI	243.1
3.100	1.819/1.677	-	1	11VI, 12VI	68.3
3.800	1.663/1.236	**X-Reg**	1	9VN, 10VN 9V, 10V	205.5
4.500	1.224/0.930	**X-Reg/Culvert**	1	11V, 12V	141.3
5.400	0.917/0.605	**X-Reg**	2	13V, 14V 15V	150.2
7.400	0.582/	**Tail**	1	16bV, 18V 17V, 17bV, 20bV, 20VS, 20V, 21V	279.9
TOTAL					1 088.0

CANAL 21st October Tail (Pumped from Reservoir)

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	1.489/1.445	**H-Reg**	1	6bVIE	20.0
1.100	1.426/1.290	-	1	8bVI	63.4
1.700	1.280/1.077	-	1	10bVI 12bVI	94.7
2.450	1.065/0.815	X-Reg (to be modified)	1	10bV 10bVN	116.2
4.000	0.794/0.322	X-Reg (to be modified)	1	12bV, 14bV, 16V	215.0
5.450	0.325/0.213	-	1	19V	49.5
6.100	0.209/ -	X-Reg (to be demolished)	1	22V, 23V	97.3
6.900	-	Tail escape (to be modified)			
TOTAL					656.1

CANAL 21st October Design Flows

Chainage (km)	Design flow upstream/ downstream (m^3/s)	Structures (existing/**new**) Regulators and culverts	Offtakes	Fields served	Net area served (ha)
0.000	9.201	River intake structure			
			2	M4, 2II, 2bIII	253.4
2.600	9.124/6.694	**X-Reg**	1	Canals S1, S2	1 941.5
			1	1II, 1bII	167.4
3.350		X-Reg (to be demolished)			
			1	2IV, 2bIV	149.5
4.500			1	M2, M3, M5	314.6
4.850	5.896/4.687	**X-Reg** **Bridge**	1	Canal S3	1 537.0
5.600		-	1	1IV	198.9
5.750		-	1 2	M1, T1 T2, 16VI	160.9 199.4
7.200	4 612/4 204	**X-Reg/Bridge** **Drain under-pass**	1 2	Canal S6 BNE, pump to T3 and T4	431.2 364.3
10.300		Bridge	1	2bIV	76.5
11.200		X-Reg	1	4bVI, 6bVI	122.6
12.000	4.034/ -	**Reservoir**		Canal S7 + 21st Oct tail	1 744.1
TOTAL					7 661.0

APPENDIX D

DETERMINATION OF WATER-STABLE AGGREGATES IN SOILS

APPENDIX D

Determination of Water-stable Aggregates in Soils

Introduction

No standard method has yet been established to determine water-stable aggregates in soils. The method developed in Canada and recommended by the National College of Agricultural Engineering, UK has been used.

Pretreatment

The field sample is air dried, pushed through a ½ inch mesh sieve, passed through a 5 mm sieve and then re-wetted for the determination. Re-wetting can be carried out in one of the three ways:

(a) complete immersion which causes the greatest disruption of soil clods;

(b) wetting by capillarity;

(c) spraying with an atomiser.

It is generally accepted that wetting under a vacuum is not advisable and Australian workers recommend that a higher degree of discrimination between similar soils is possible after wetting by capillarity than after wetting by complete immersion.

Wet Sieving

The field sample is placed on a bank of sieves, immersed in a tub of water and moved up and down in the water at a rate of 30 strokes a minute for a given number of strokes. Aggregate breakdown increases as the duration of agitation increases until after a certain number of strokes no further appreciable breakdown occurs. For most soils, no further breakdown occurs after 500 strokes and therefore this has been selected as a suitable number for routine sieving.

Method

1. Air dry the field samples but before the soil is completely dry, breakdown the clods along their cleavage planes. Pass the sample through a ½ inch mesh sieve followed by a 5 mm sieve and use the material retained on a 3 mm sieve as the sample.

2. Expose the group of samples to be compared to the atmosphere for 48 hours so that they attain as far as possible a uniform moisture content. (The air humidity can affect the water stability of the aggregates). Determine the air dry moisture content.

3. Cover 50 g of the sample with 100 ml of water and allow to stand for 30 minutes.

4. Tip the soil into the middle of the uppermost sieve. The sieves used are of 2 mm, 1 mm, 0.5 mm, 0.125 mm and 0.100 mm aperture diameter, numbers 8, 16, 30, 60 and 150, respectively.

5. Lower the nest of sieves into the tub at an angle to eliminate air locks in the sieves.

6. Agitate the sieves for 17 minutes keeping the water level in the tub constant throughout - use the overflow device.

7. After agitation, retain all the material greater than 0.5 mm diameter, oven dry and weigh.

8. Disperse this material by puddling in a beaker. After puddling, wash the soil through the 0.5 mm sieve and oven dry and weigh the soil left on this sieve.

Results

The fraction weighed at stage (7) above contains both aggregates and coarse primary particles and allowance has to be made for the latter. This can be done by subtracting the weight of material remaining on the 0.5 mm sieve at stage (8) from the weight of material at stage (7). Express this corrected weight as a percentage of the total weight of soil used. (Approx. 50 g corrected to oven dry moisture content).

This result gives the percentage of soil less than 0.5 mm diameter which is present in the water-stable aggregates greater than 0.5 mm diameter.

Compare the results of the two soils tested.

Note

The soil fraction between 5 mm and 3 mm is used in the test because it has been observed that the initial size of the aggregates influences the amount retained finally during wet sieving and only by having the aggregates within a narrow size range can this source of error be made reasonably small.

The only reason for using the nest of 5 sieves instead of only the 0.5 mm sieve, is to increase the efficiency of sieving and the data for the 5 individual fractions can be used to produce a size distribution curve if required.

Source: National College of Agricultural Engineering, Silsoe, laboratory methods.

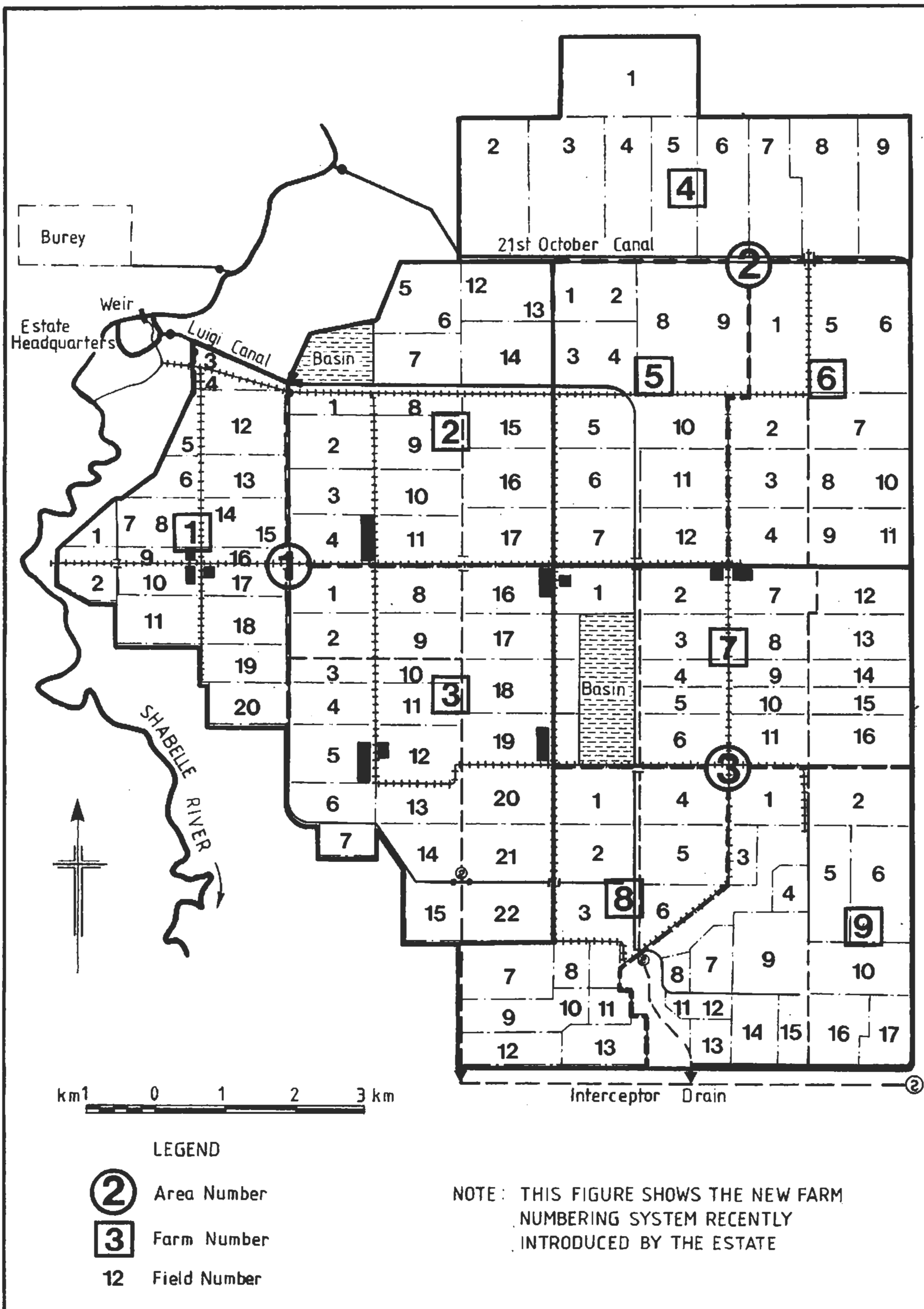
APPENDIX E

NEW FARM NUMBERING SYSTEM

The following figure illustrates the new farm numbering system recently adopted by the Estate, comprising nine farms. It is proposed that this system is discontinued in the future and that the previous system of six farms is re-adopted.

Figure I.E.1

Jowhar Sugar Estate
New Farm Numbering System



APPENDIX F

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