EASTERN NILE IRRIGATION AND DRAINAGE STUDY/FEASIBILITY STUDY
DINGER BEREHA IRRIGATION PROJECT

ANNEX 7: HYDRAULICS AND IRRIGATION ENGINEERING

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1. IRRIGATION SYSTEM

1.1 General Description

1.1.1 Rationale for Major Elements

Selection of Diversion

A gravity diversion has been selected for the proposed project because comparison of costs of options have shown that the partly gravity-partly pump option is more cost effective than the alternative of full pumping from a point on the Didessa nearer to the command area.

Canals

Trapezoidal concrete lined canals with a gradient of 0.10-0.20 m/km have been chosen for all canals because of the high permeability of the soils. A steeper canal gradient was examined but the additional pumping costs together with the loss of production exceeded the saving in canal construction cost.

Night Storage Reservoirs

Since irrigation will only take place during the 12 daylight hours, it is necessary to store the nighttime flow in the canals at a location close to the irrigated area. The provision of night storage reservoirs simplifies the management of the scheme and enables the scheme to make efficient use of the diverted irrigation water.

Inverted Siphons

Inverted siphons are used to cross deep valleys along the canal route for reasons of cost; an inverted siphon will cost less than a long section canal with a cross-drainage culvert.

Side Weirs

Side weirs are used to control the water levels in the Primary Canals so that outflows from the canal can be accurately controlled. Side weirs are found to be both less costly and less subject to mechanical failure than mechanical devices such as an AMIL gate. These are very robust devices so there is also less opportunity for interference and vandalism.

Distribution System

Because of the terrain, conveyance and distribution of water perpendicular to the contours has to be mainly through piped networks, because a system with open lined canals coupled to a huge number of large drop structures would be very expensive and would require large land take in a densely populated and cultivated area.

Additional advantages of piped systems are increased distribution efficiency, large reduction in water theft, better control of quantities delivered to users and the possibility to use highly efficient localised irrigation systems with full and transparent control of water quantities delivered.

1.1.2 Layout of the Project

Water is taken from the Didessa River by a headworks made up of a mass concrete weir with a flushing channel with sediment excluder and main canal offtake on the left bank (see map no TM10 for location, drawing no HW01 for lay-out and Chapter 2 for details). Immediately downstream of the primary canal offtake there is a settling basin, from which the settled sediment can be flushed back to the river.

The main canal is about 15 km long up to the boundary of the command area and follows the Didessa River closely for most of its length. At the south eastern extremity of the Project area, the primary canal has gained sufficient command to be able to irrigate the area between the canal and the river. The minimum water level necessary to ensure the submergence of the offtakes to the secondary pipelines in this first section of the Main Primary Canal is maintained by three side weirs. The flow to the secondary pipelines is "on demand", that is by downstream control so that precise regulation of the water level in the canal is not necessary.

The primary canal crosses several deep watercourses in inverted siphons and finally discharges into a Night Storage Reservoir (R1) which serves as night storage for the irrigated area upstream. Downstream of R1 the irrigation scheme forms an H shape and the four Primaries are designated after their geographic location, i.e. NE, SE, NW and SW. The irrigation water is pumped from R1 to these four primaries, the water for the NE and SE primaries being pumped via individual rising mains whereas the water for the NW and SW primaries is pumped via a single rising main which divides after traversing the valley of the Duna Sera stream, which separates the eastern part of the project area from the western part. At the end of each of the four rising mains there is a night storage reservoir.

The reservoir serving the NE part of the command area (R2) has no canals downstream but instead feeds directly to the secondary pipelines which serve surrounding command area. Certain of the secondary pipelines for this area will be taken directly off the P1 rising main. Each one of the pumped Primary Canals (SE, NW and SW) has a reservoir at the downstream end to provide night storage and to provide operational flexibility. The larger command areas served by these three Primary Canals will have their own off-stream Night Storage Reservoir. The flow from the Primary to the off-stream reservoirs will be accurately regulated by a Baffle Distributor and close control of the canal water level by a side weir. Water will be taken from these night storage reservoirs by the secondary pipe system "on-demand", i.e. with downstream control.

The Primary Canals will traverse the deeper valleys in inverted siphons which will have GRP barrels and will either be buried or installed above ground, according to the pipe size and geotechnical conditions. Although most of the scheme will be irrigated by secondary pipelines feeding directly from the reservoirs or Primary Canal, contour secondary canals are required in a number of locations. These will be supplied with irrigation water from the reservoirs using an "AVIO" type gate and a modular baffle distributor to ensure that the correct flow is delivered.

From the primary network, including the canal and the night storage reservoirs, water is conveyed at the head of each standard block by secondary canal and/or a buried pipe network. The Project area is divided into 15 Command Areas corresponding to the main interfluves. For each command area, blocks are demarcated, taking into account the site geomorphology, slope, streams and gullies.

The tertiary system comprises buried pipe networks connected to the secondary pipes or canals. The tertiary systems feed flow control hydrants each serving an area of about 4-6 ha. The flow from the hydrant is controlled by a flowlimiter and distributed by a permanently buried PVC or PE pipes to a number of field outlets. Each farmer has its own field outlet and the full flow from the hydrant is rotated amongst the farmers united around the hydrant. The tertiary network and the hydrants are operated at maximum 12 hours/day when water requirements peak.

The on-field irrigation system is connected to the individual outlets and comprises gated pipes or other improved surface irrigation equipment such as hoses or simple HDPE pipe that can be dragged from farm to farm. Where sufficient pressure is available at the hydrant, farmers can install localised irrigation systems such as drip and sprinkler (draghose system).

1.1.3 Operation of the Project and Night Irrigation

Irrigation will only take place during the 12 hours of day light but the primary canal, Primary Canals and pump stations will operate at a constant flow, appropriate to the irrigation demand, for the whole 24 hour period. During the night the Night Storage Reservoirs will be filled and during the day the reservoirs will be emptied.

It is strongly advised not to extend the duration of irrigation beyond 12 hours as experience in Ethiopia and in the rest of the world has shown that for the for the on-farm distribution method adopted for the project (gravity, with furrows and basins) water use efficiency decreases rapidly when darkness sets in. An irrigation duration of 12 hours would require the farmer to be present in the field for at least 12 hours. With one to two hours travelling between fields and homestead he would have to be 13-14 hours away from home. He would have to leave in the dark between 5-6 am in the morning and return during darkness between 6 and 7 pm. Travelling in the dark would not be such a problem when using a torch, but the suggestion to irrigate in the dark with a torch can not be considered as a realistic option. Increase of the duration from 12 to 14 hours (15%) could only reduce the investment costs of the pipe networks, because the rest of the system is operating 24 hours per day. The equivalent reduction of the overall investment cost is about 2%, if on-farm irrigation efficiency remains unchanged. Operating costs of pumping stations and other irrigation infrastructure will remain the same but management costs will increase because of longer operating hours for the agents of the water supply agency.

1.2 WATER REQUIREMENTS, EFFICIENCIES, DESIGN FLOWS

1.2.1 Climatic Data

For preliminary calculations prior to completion of the hydrological studies, climatic data of the Didessa State Farm station were obtained to determine crop water requirements. The station is on the other side of the Didessa River. The dependable rainfall was calculated on the basis of the rainfall that is equalled or exceeded three years out of four (75% dependable). Subsequent detailed hydrological studies produced values that did not differ very much from the parameters deduced from the original Didessa State Farm values. Table 1.1 shows a comparison, whereas Table 1.2 shows the originally adopted values, valid for Didessa State Farm for the period up to 1996.

Table 1.1: Comparison between originally adopted and later obtained values.

Source	value	Unit	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	total
Abbay MP	daily ETo	mm	3.6	4	4.5	4.6	4.1	3.6	3.2	3.2	3.5	3.8	3.6	3.4	
MCE	daily ETo	mm	*3.7*	*4.1*	*4.6*	*3.5*	*5.1*	*3.6*	*3.0*	*3.1*	*3.8*	*4.0*	*3.8*	*3.4*	
Abbay MP	Mean rainfall	mm	2	5	32	58	183	263	335	271	193	106	20	9	1477
MCE	Mean rainfall	mm	*3*	*6*	*26*	*49*	*158*	*274*	*312*	*277*	*209*	*104*	*28*	*8*	1454

Table 1.2: Climatic Data valid for Didessa State Farm

	ETo	Mean rainfall	Dependable rainfall
	Mm	mm	mm
Jan	3.6	2	0.0
Feb	4.0	5	0.0
Mar	4.5	32	4.8
Apr	4.6	58	10.5
May	4.1	183	100.5
Jun	3.6	263	184.2
Jul	3.2	335	217.4
Aug	3.2	271	206.7
Sep	3.5	193	130.0
Oct	3.8	106	47.1
Nov	3.6	20	2.8
Dec	3.4	9	0.0

1.2.2 Cropping Programme

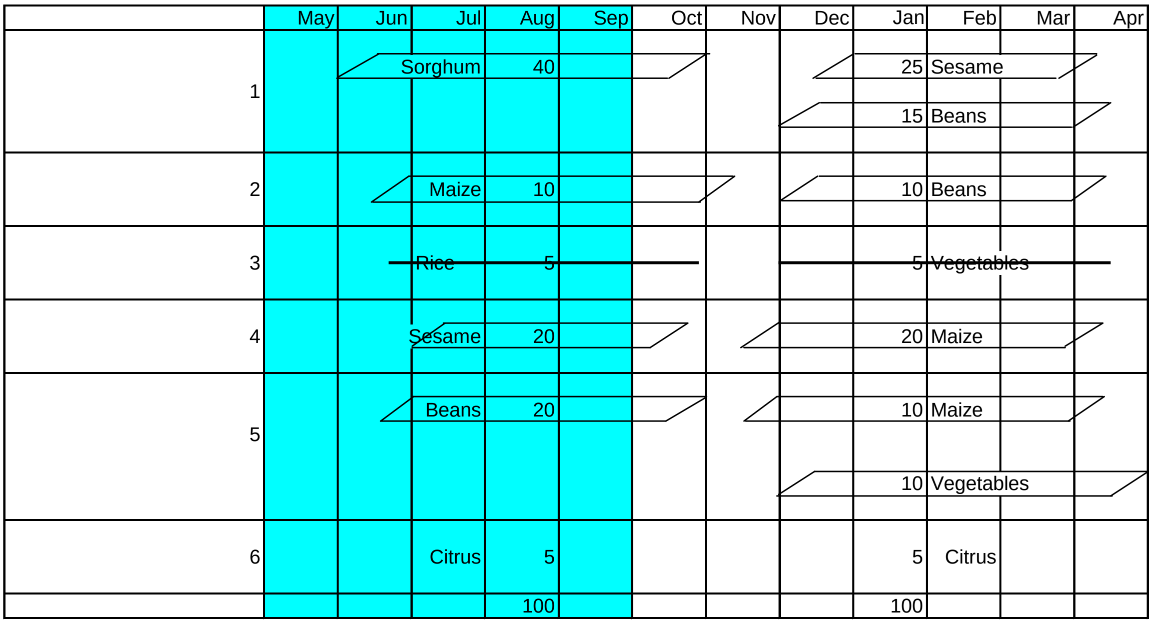
A number of cropping patterns were developed and tested, taking into consideration the climatic and land suitability, subsistence needs, potential markets, as well as farmers' preferences. A representative pattern with a cropping intensity of 200% is shown in Table 1.3. Details on other cropping patterns are presented in Annex 6: Agriculture and Livestock.

Table 1.3: Percentage Planted with Various Crops

Crop	Rainy Season	Dry Season
Sorghum	40	0
Maize	10	30
Rice	5	0
Sesame	20	25
Beans	20	25
Vegetables	0	15
Citrus	5	5

These crops were divided up into five rotations plus perennial citrus as shown in Figure 1.1. This figure also gives the first sowing and harvest date for each crop together with the crop duration. It has been assumed that each crop will be sown over about a 15 day period.

Figure 1.1: Representative cropping pattern with 200% intensity



Crop	Sow	Harvest	Days
Sorghum	01-Jun	14-Oct	135
Maize	15-Jun	28-Oct	135
Rice	20-Jun	28-Oct	130
Sesame	01-Jul	09-Oct	100
Beans	20-Jun	18-Oct	120

Crop	Sow	Harvest	Days
Sesame	15-Dec	25-Mar	100
Beans	01-Dec	31-Mar	120
Vegetables	01-Dec	15-Apr	135
Maize	15-Nov	30-Mar	135

1.2.3 Crop Water Requirements and Adopted Efficiencies

The results of the crop water requirement calculations are presented in Annex 6: Agriculture and Livestock.

Initially, the gross water requirements were calculated on basis of the assumption that the overall efficiency would be about 50%, with an on-farm efficiency of 70% (surface irrigation, furrows and basins), a distribution efficiency of 80% and a main conveyance efficiency of 90%. However, after assessment of the potential performance characteristics of the adopted irrigation system (all lined canals, night storage reservoirs, closed buried pipe distribution systems and pipework for on farm distribution), the overall efficiency was set at 60-65% (on-farm: 75%, distribution through pipe networks that are emptied regularly: 85-90%, main conveyance: 95%).

In order to ensure a maximum flexibility and to allow for losses in the initial years because of lack of experience the overall efficiency has been set at 60%. The efficiency can increase over the years with increasing experience at management and farmers' level, thus allowing decrease of pumped volumes and associated costs. The capacity becoming available can also be used to irrigate areas close to the primary and secondary canals. This is likely to happen when farmers install privately owned small electric or diesel driven pumps along the open canals.

Table 1.4 gives the volume of irrigation water required by a range of dry season irrigated crops that are recommended to be included in the cropping pattern.

Table 1.4: Gross Water Requirements at 60% overall efficiency

Crop	Water requirements (m^3/ha)
Maize, cereals	8,400-8,500
Rice	11,300-11,400
Sesame, other oil crops	8,400-8,500
Sunflower	8,100-8,200
Groundnuts	6,200-6,300
Pulses, beans	6,200-8,400
Onions, other vegetables	8,200-8,900
Citrus	7,200-7,300

1.2.4 Design Capacities and Annual Gross Water Requirements

A range of cropping patterns at 200% intensity and associated water requirements were calculated and it appeared that the maximum unit gross water requirement for recommended cropping patters at 60% overall efficiency would not exceed 0.78 l/s/ha. Taking into account some safety margin a duty of 0.8 l/s/ha has been adopted for all canals, siphons, reservoirs and pipe distribution networks. For intensities of 175% the duty was calculated at 0.6 l/s/ha. As it is not expected that an intensity of 200% over the whole area will be achieved within 10 years, the pumping station will be designed to cater initially for a duty of 0.6 l/s/ha. Capacity will be increased by adding an extension to civil works and by increasing the number of pumps as soon as the water demand increases because of increasing cropping intensity.

Table 1.5 shows the maximum possible diversion flow for an irrigation efficiency of 60% and a cropping intensity of 200%. It can be seen that the main irrigation season will be from the end of October to the third week in April with a peak water requirement in the second half of February.

Table 1.5: Average monthly diversion flows (in m^3/s) at 200% cropping intensity and 60% efficiency

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3.9	6.0	4.7	1.3	0	0	0	0	0.4	0.7	0.3	1.7

The annual gross water requirements at 60% overall efficiency are 44 MCM and 61 MCM, respectively at 175 and 200% cropping intensities, including rice. Without rice and at 200% intensity the requirement would be 58 MCM. The latter value has been adopted for the economic and financial calculations.

1.3 Detailed Description of the Project

1.3.1 Weir and Headworks

Irrigation water will be taken from the Didessa River at a point some 16 km upstream of the North East boundary of the command area near the river. The location of the diversion weir was chosen after reconnaissance surveys confirmed that geological conditions were highly suitable at the site where contour 1240 crosses the river. This site is just upstream of a bifurcation in the river. (see photographs 1.1 and 1.2). A mass concrete weir will be constructed across the river with a crest level of 1246.00 (see Drw nos HW01 and HW02).

The weir will be constructed with a flushing channel at the left end, whereas the offtake to the primary canal will be at right angles to the flushing channel. To minimise the amount of sediment in the water diverted to the primary canal the sill of the offtake will be one meter above the bed of the flushing channel.

Downstream of the canal offtake a 150 m long settling basin will be constructed. This settling basin will be angled towards the river and will have a gate at the downstream end to allow flushing of the settled sediment back to the river. The primary canal will take off the settling basin at right angles. Downstream of the settling basin the canal will be rectangular in section and straight for 50 m followed by a gauging weir designed to ISO 4360. This gauging weir will enable the gates at the headworks to be set to give the flow required to satisfy the irrigation demand.

1.3.2 Main Canal

The length of the main canal between the measuring flume and the command area is 15 km. The canal is crossing through difficult terrain, with many riverine forest areas and some solid rock outcrops. With a gradient of 0.2m/km the fall in waterlevel at full capacity would be 3.0 m. Taking into account 0.4 m additional losses to allow for steeper gradients at sections where volume of excavation and fill has to be minimised (cuts in rock etc, fill at cross drainage structures) the total fall in waterlevel has been fixed at 3.40 m. Hence the waterlevel at the entrance of the command area would be +1241.60.

1.3.3 Other Canals

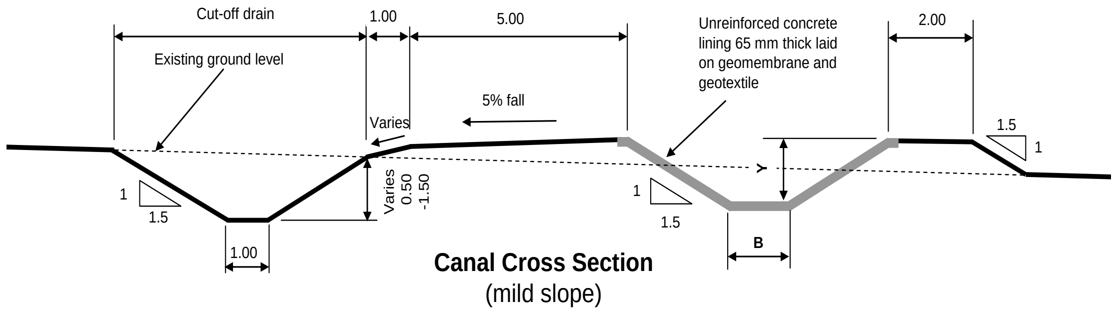
Typical cross sections of the canals are shown in Figure 1.2 and drawings numbers CSF 01 and CSP01. The long sections are presented in drawings numbers LSF01-03 for main canal and LSP01-03 for primary canal. The permeable nature of the soils, the steep transverse slopes and maintenance requirements indicate that the canals should be concrete lined. It is proposed that all canals be lined with a 65 mm layer of unreinforced concrete constructed in alternate bays 3 m long. The side slopes of the canal will be 1.5:1 (H:V), this is found to be the steepest slope on which concrete can be placed and achieve a good finish. The gradient of the canals is 0.0002 whereas the Manning "n" is taken as 0.016. The capacity of the canals is based on a duty of 0.8 l/s/ha, which is sufficient to supply water when the cropping intensity reaches 200%. Therefore, the main canal would carry 6.0 m3/s. To ensure the water tightness of the canal and to obviate the need for sealing of the joints between the lining sections, there will be an appropriate geomembrane laid on the prepared soil surface. On top of the geomembrane there will be a geotextile to prevent the concrete lining sliding on the geomembrane a sit is laid. Alternatively, textured geomembranes may be available, in which case the geotextile can be dispensed with.

An access road 5 m wide is provided on the uphill side of the canal and, on the downhill side of the canal, there is an embankment with a top width of 2.00 m. For the purpose of the feasibility study, a range of standard canal dimensions has been generated for a canal gradient of 0.2 m/km (see Table 1.6). In so far as possible, the canals will be set out such that the cut and fill of the whole cross-section, including the cut-off drain, is in balance to preclude the necessity of either importing fill or disposing of surplus material.

Photograph 1.1: View from weir site in downstream direction of bifurcation

Photograph 1.2: Satellite Image of Weir site

Figure 1.2: Typical Canal Cross-Sections



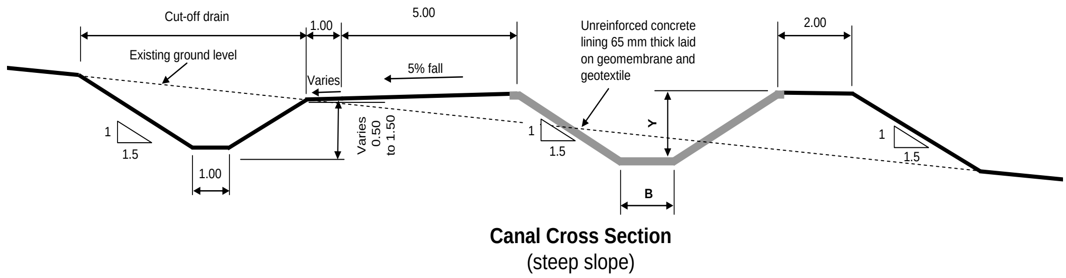


Table 1.6: Primary Canal Dimensions at a gradient of 0.2 m/km

Section Reference	Max Flow m^3/sec	Length m	Canal Section	Bed width m	Section depth m	Water depth m	Freeboard m
PC 1+1a	6.00	7,628	C02	1.75	2.15	2.00	0.35
PC 2	6.00	1,016	C02	1.75	2.15	2.00	0.30
PC 3	6.00	3,845	C03	1.75	1.95	1.93	0.35
PC 4	6.00	1,854	C03	1.75	1.95	1.90	0.30
PC 5	0.40	2,675	C12	0.75	0.80	0.60	0.20
PC5a	0.20	4,509	C12	0.75	0.80	0.45	0.20
PC 6	0.19	1,614	C12	0.75	0.80	0.45	0.20
PC 7	0.45	2,360	C12	0. 75	0.80	0.65	0.15
PC 8	1.04	4,784	C10	0.75	1.10	0.90	0.20
PC 9	2.35	3,034	C07	1.25	1.45	0.92	0.23
PC 10	1.80	1,240	C08	0.75	1.45	1.15	0.25
PC 10a	1.80	2,622	C08	0.75	1.45	1.15	0.25
PC 10b	1.80	0,266	C08	0.75	1.45	1.14	0.25
PC 11	1.16	1.868	C09	0.75	1.25	0.65	0.20
PC 11a	1.16	1,060	C09	0.75	1.25	0.65	0.20
PC 12	0.33	0.883	C10	0.75	0.75	0.60	0.20

Where the canal crosses watercourses, the canal will be constructed on an embankment such that the culvert under the canal has its invert slightly below the invert of the watercourse and there is sufficient clearance between the underside of the canal lining and the top of the culvert, taken as 300 mm minimum.

A higher canal gradient of 0.0003 was examined during the preparation of the draft report but this was found to be economically less favourable than the lower gradients; the value of the agricultural production forgone because of loss of command and the additional pumping cost exceeded the savings in canal construction by a considerable margin of 15.6 million Birr. The calculations were based on a main (feeder) canal flow of 8.3 m^3/s. Appendix 1.3 presents the draft calculations that can be summarised as follows:

- Reduced concrete surfacing area: 27,375 m2, valued at 6.8 million ETB at 250 Birr/m2
- Loss of command area: 93 ha
- Nett present value (NPV) of production foregone: 20.2 million Birr
- Additional pumping head: 2.75 m
- NPV of additional power for 6,500 ha: 2.2 million Birr
- Assumptions to calculate NPV: discount rate = 10% and no of year = 25
- Total additional costs 22.4 million Birr
- Canal savings: 6.8 million Birr
- Net additional costs: 15.6 million Birr

For the currently adopted main canal flow of 6 m3/s the net additional costs will be similar because:

- The NPV of production foregone remains the same
- The NPV of the additional power will be reduced only slightly
- Canal savings will be reduced only slightly.

1.3.4 Cut-Off Drain

There will be a cut-off drain on the uphill side of the canal embankment to intercept surface flow. The cut-off drain will have a bed gradient of 0.001 and a maximum and minimum depth of 1.5 m and 0.5 m. This means that the maximum length of the cut-off drain will be 1,200 m when the flow is in the same direction as the canal flow and 800 m when the flow in the drain is in the opposite direction to the canal flow. This gives a maximum distance between cross-drainage structures or stream crossings of 2,000 m. In most cases the cut-off drains will discharge to a watercourse and, as the canal crosses the valley of the water course, the gradient of the cut-off drain will be maintained at 0.001 such that the cut-off drain will curve away from the canal embankment until it meets the watercourse.

1.3.5 Inverted Siphons

The undulating nature of the terrain means that the canal has to cross many deep valleys. Where the length of the contour canal would exceed the length of the siphon to cross the same valley by a factor of 2.5 or more, experience indicates that the siphon would be less costly than a contour canal with a culvert passing under it. The use of an inverted siphon also obviates the need for a culvert for the cross-drainage and so is particularly appropriate when crossing watercourses carrying large flows. The total number of siphons is 11. The inverted siphons designed for this project have the following characteristics:

1) Reinforced concrete inlet and outlet transitions as shown in drawing number IS01. The inlet and outlet structures are constructed as a reinforced concrete box with mass concrete infill in which a transition from the trapezoidal canal to the circular siphon pipe can be formed.

2) A Glass Reinforced Polyester (GRP) siphon pipe is used. GRP is light and easy to handle and lay with watertight flexible joints and is extremely smooth internally, giving low friction loss in the pipe allowing the use of smaller diameter pipes giving self-cleansing velocities with minimum head loss. No suitable pipes are available on the local market and when shipping and laying costs are taken into account GRP provides the most satisfactory solution.

3) The diameter of the siphon barrel is selected to give a velocity of just under 2 m/s. This velocity should be sufficient to prevent fine particles settling. Higher velocities will give higher friction losses and, particularly, higher losses at the inlet and outlet structures.

4) Where there is sufficient depth of soil the siphon pipe can be buried, see detail on drawing number IS02. Where there is not sufficient depth of soil, the siphon pipe can be supported on concrete cradles above ground.

5) The steep gradient of the streams draining the project area results in the streams having hard rock in the bed. It will be most economical therefore for the siphon to cross the stream supported on concrete cradles, see detail on drawing number IS02. For longer crossings it may be necessary to construct additional cradles in the watercourse. An advantage of crossing the stream on cradles is that a drain valve can be installed on the underside of the pipe which can be used to empty the pipe.

6) A trash rack is installed at the entrance to the siphon to prevent people, animals and trash being swept into the siphon.

Where there is sufficient soil depth, siphon pipes up to and including 1.00 m diameter will be buried. Siphon pipes greater than 1.00 m diameter will be installed above ground on concrete cradles. A schedule of the inverted siphons is given in Table 1.7. Many similar inverted siphons have been in operation for a number of years on the main canal of the Finchaa Sugar Project.

Photograph 1.3 shows an inverted siphon with a steel pipe supported on concrete cradles.

Figure 1.3: Inverted Siphon Outlet Transition (Inlet similar)

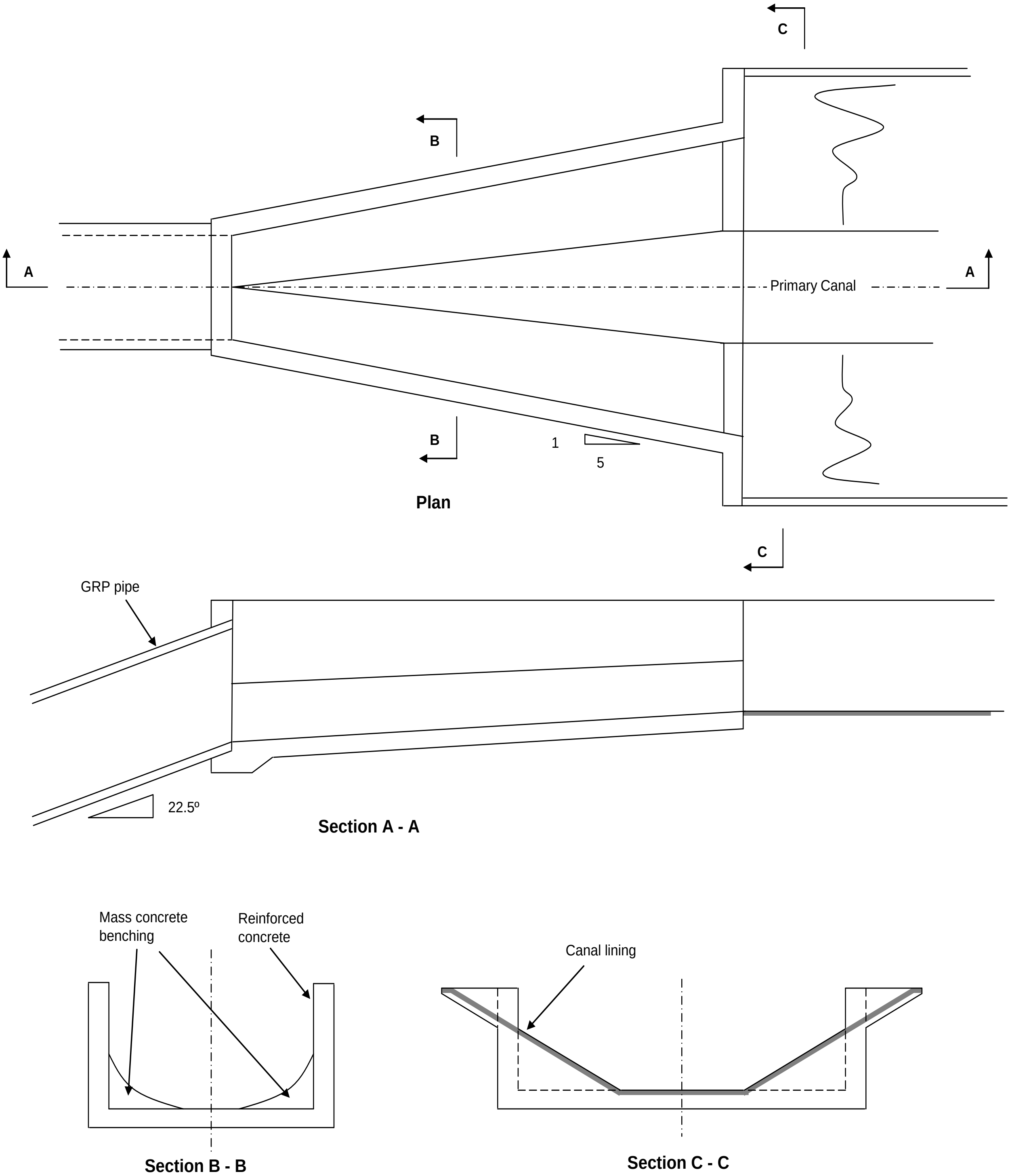


Table 1.7: Schedule of Inverted Siphons (q = 0.8 l/s/ha)

Designation	Pipe length	Q	Pipe Diameter	Velocity	Head Loss
	m	m3/s	m	m/s	m
S1	237	6.00	2.0	1.88	0.55
S2	185	6.00	2.0	1.88	0.50
S3	240	6.00	2.0	1.88	0.56
S4	441	6.00	2.0	1.88	0.77
S5	372	0.45	0.6	1.59	1.42
S6	116	1.04	0.9	1.64	0.48
S7	386	1.80	1.1	1.89	1.16
S8	253	1.16	0.9	1.82	0.95
S9	110	0.33	0.5	1.68	0.75
S10	267	0.66	0.8	1.31	0.57
S11	128	0.66	0.8	1.31	0.35

Photograph 1.3: Inverted Siphon Crossing of River Channel (Dak Lak, Vietnam)

1.3.6 Cross-Drainage Culverts

The canals, in particular the primary canal, pass over a number of water courses which are carried under the canal in culverts. Where culverts are required the canal is aligned such that it is on an embankment high enough to allow the canal to pass over the top of the culvert with a minimum clearance of 300 mm. The culvert itself should be laid with its invert slightly below the bed of the watercourse. The alignment of the canal on an embankment at the crossing of watercourses has the additional advantage of shortening the length of the canal.

The culverts designed for this project are based on a cell 2.00 m high and 2.50 m wide, which can pass a maximum flow of 10 m^3/s. This cell size will allow vehicles, human beings and animals to pass under the canal. The total number of culverts is 19. The layout of typical two cell cross-drainage culvert is shown in Drawing Number CSS01. The calculation of flows are given in Annex 1, section Hydrology and the schedule of the culverts in Table 1.8. The location of the culverts is shown on Map Numbers TM03-TO10.

Table 1.8: Schedule of Cross-Drainage Culverts

Culvert Number	Flow T = 25 (m^3/s)	Number cells 2.00x2.50 m
1	13	2
2	25	3
3	13	2
4	22	3
5	14	2
6	28	3
7	24	3
8	16	2
9	9	1
10	17	2
11	18	2
12	12	2
13	9	1
14	4	1
15	8	1
16	7	1
17	26	3
18	5	1
19	6	1

Where the distance between watercourses exceeds 2,000 m, it will be necessary to install a culvert under the canal on the contour, i.e. not on an embankment. Drawing number CSS02 shows a culvert designed to carry the flow from the cut-off drains under a canal aligned on the contour. The barrel of this culvert is 1.50 m wide and 1.00 m high which is sufficient to carry the flow from 2,000 m of cut-off drain.

Figure 1.4: Cross-Drainage Box Culvert at Watercourse

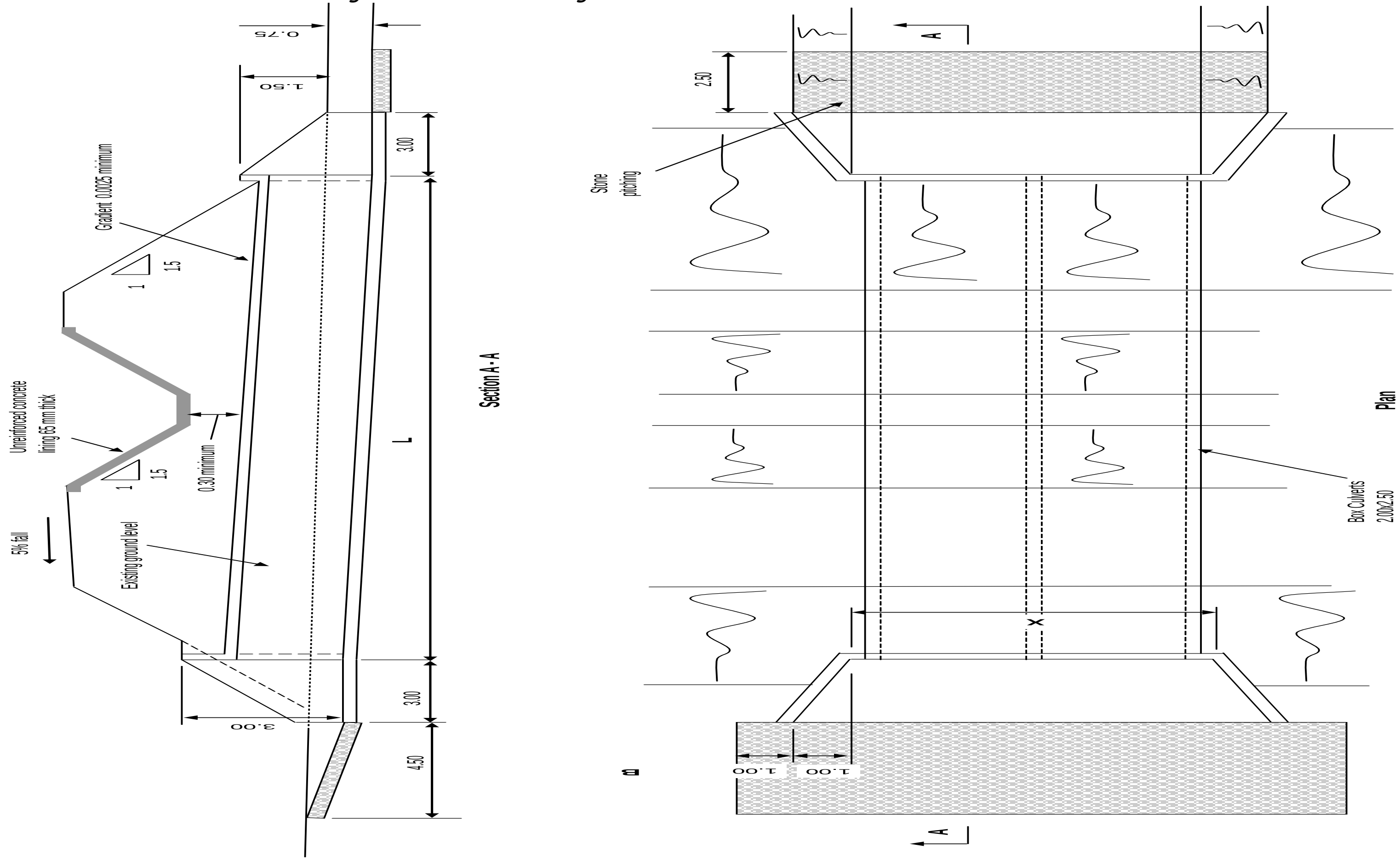
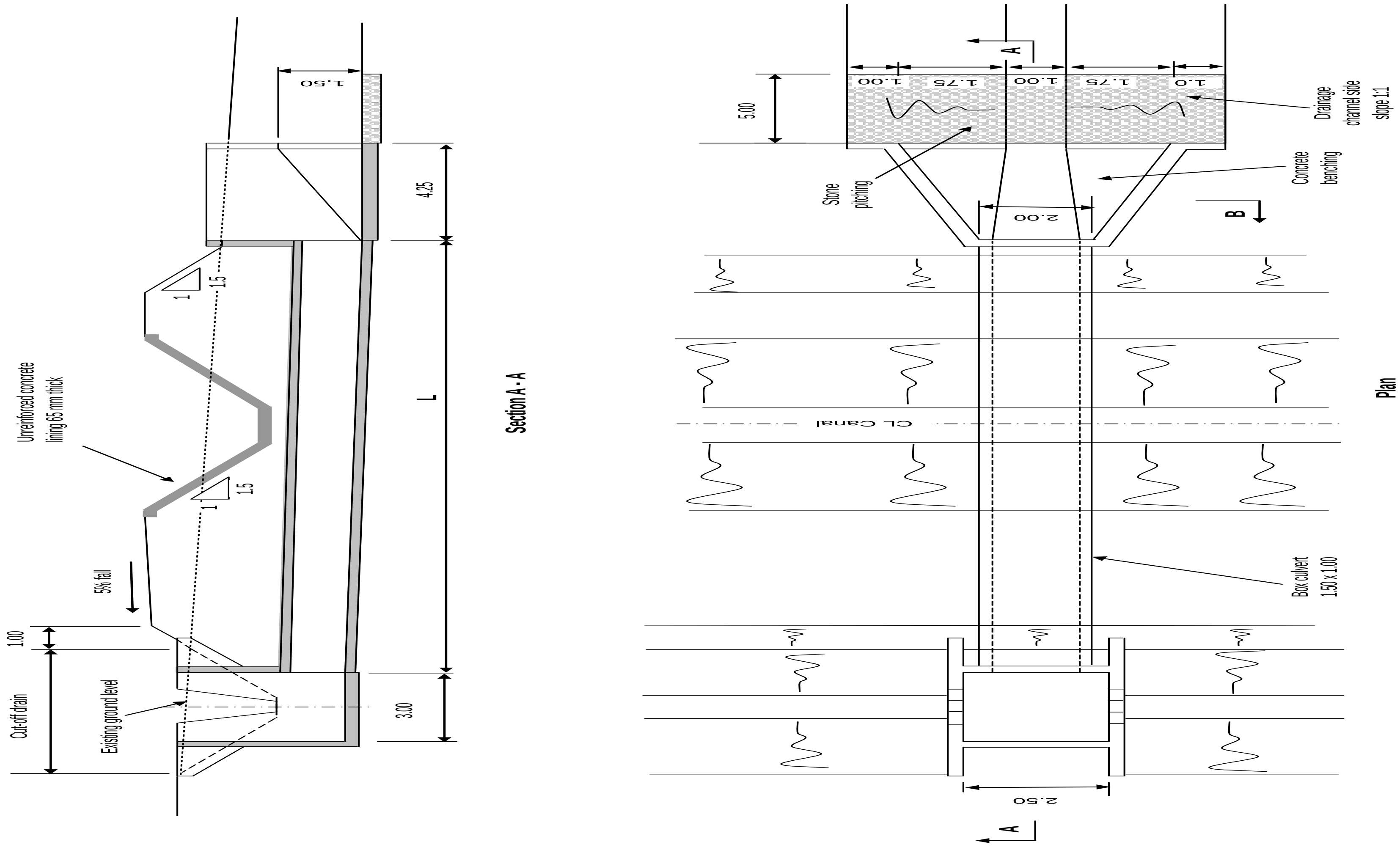


Figure 1.5: Cross-Drainage Box Culvert, no Watercourse



1.3.7 Night Storage Reservoirs

It is anticipated that irrigation will only be carried out during daylight hours; farmers are usually unwilling to irrigate at night and, in any case, irrigation at night becomes extremely inefficient. In order that the primary canal and the pumping station can function at a constant rate over the 24 hour period, night storage reservoirs are provided to store the flow during the 12 hour night time period when no irrigation takes place.

The reservoirs will be of two basic types:

- an on-stream reservoir either at the head of the rising main at the start of the primary canals or at the end of the Primary Canals.
- an off-stream reservoir supplied by an offtake along the canal

The flow into on-stream reservoirs will not be closely regulated; all the water passing along the Primary Canal or out of the rising main will flow into the reservoir and the flow out of the reservoir will either be a regulated flow to a canal or an "on demand flow" to the secondary network. Apart from providing night storage for the irrigation, the on-stream reservoirs will provide a buffer if the flow into the reservoir is too high or too low.

The off stream reservoirs will, on the other hand, have the flow to them carefully regulated so that a constant 24 hour flow appropriate to the irrigation water demand passes into the reservoir. The flow into the reservoir will be controlled by a baffle distributor, see paragraph 1.3.8, which can deliver the flow in units of 10 l/s to an accuracy of +/- 5%. In order that the distributor can provide this accuracy of flow delivery, the water level in the Primary Canal must be regulated within precise limits. This regulation of the water level will be provided by side weirs in the Primary Canal, see 1.3.9. An additional 15% is added to the calculated volume of the Night Storage reservoirs for operational convenience. Layout and details of the Night Storage Reservoirs are given in Drawings Numbers RE01 and RE02. Table 1.9 presents a schedule of the Night Storage Reservoirs. The total number of reservoirs will be 12, covering a total gross storage volume of 298,194 m^3 equivalent to 40 m^3/ha at 7,500 ha net.

Table 1.9: Schedule of Night Storage Reservoirs

Canal	Designation	Flow m^3/s	Net vol. m^3	Gross vol. (+15%) m^3	Reservoir Type
Primary	R1	0.49	21,315	24,512	On-stream
NE	R2	0.86	37,092	42,656	On-stream
SE	R3	0.81	35,054	25,309	On-stream
	R4	0.20	8,513	9,790	Off-stream
	R5	0.10	4,480	5153	On-stream
NW	R6	0.46	19,635	22,580	On-stream
	R7	0.55	23,809	27,380	Off-stream
	R8	0.64	27,632	31,777	Off-stream
	R9	0.83	35,662	41,012	Off-stream
	R10	0.33	14,350	16,502	On-stream
SW	R11	1.04	44,751	38,528	On-stream
	R12	0.26	11,249	12,936	On-stream

There will be outlets from some of the reservoirs to Primary and Secondary canals. It is proposed to control the flow from the reservoir using a AVIO gate and baffle distributors. The AVIO gate maintains a constant water level downstream independent on the water level upstream and then baffle distributors control the flow into the canal. For the larger flows, two baffle distributors will be required, one which gives coarse control of the flow (to the nearest 100 l/s) and the other that gives fine control of the flow (to the nearest 10 l/s). The AVIO gates and baffle distributors required for the reservoir outlets are given in Table 1.10.

Table 1.10: Reservoir Outlets, Gates and Baffle Distributors

From Reservoir	To Canal	Flow m^3/s	AVIO model	Baffle Distributors	
				Coarse	Fine
R3	PC5	0.40	71/40	C_1 1000	XX_2 90
R6	PC9	2.09	140/160	C_1 3000	XX_2 90
R6	SC9-1	0.29	71/40	-	XX_2 420
R10	SC12-1	0.29	71/40	-	XX_2 420
R11	PC8	0.56	90/63	C_1 1000	XX_2 90
R11	SC8-1	0.29	71/40	-	XX_2 420

1.3.8 Canal Offtakes

There will be two types of offtakes to the Secondary and Tertiary irrigation network. Where there is no Night Storage Reservoir the pipe offtake will be connected directly to the canal and the flow into the pipe will be controlled from downstream, i.e. the opening and closing of the hydrants. The only requirement for canal water level is that there should be adequate submergence of the offtake. Where the offtake supplies water to a Night Storage Reservoir, the offtake flow must be controlled precisely to ensure that the appropriate amount of water is supplied to the reservoir to satisfy the irrigation demand. This is done by using a Baffle Distributors, see Drawing Number C001. The flow passed by the Baffle Distributors is adjusted by opening and closing the gates on each section to give the required flow. To give the required range of flow adjustment it will be necessary to provide two Baffle Distributors at each offtake; one to give coarse control of the flow and the other to give the fine adjustment

The schedule of baffle distributors required is given in Table 1.11.

Table 1.11: Reservoir Inlet, Schedule of Baffle Distributors

Reference of adjacent Side Weir	Reservoir or canal supplied	Offtake flow (m^3/s)	Baffle Distributor Model	
			Coarse Adjustment	Fine adjustment
SW4	R7	0.49	C_1 1000	XX_2 120
SW5	R8	0.57	C_1 1000	XX_2 120
SW6	R9	0.73	C_1 1000	XX_1 120
SW7	SC12-2	0.29	-	XX_1 420
SW9	R4	0.18	XX_1 180	X_2 90

The water level in the canal upstream of the Baffle Distributors is controlled within the required limits by side weirs, see paragraph 1.3.9.

Apart from the offtakes from the Primary Canals to the Night Storage Reservoirs, there will be a number of offtakes to secondary pipelines directly from the Primary Canal. The irrigation supply to CA1 will require four direct offtakes and CA11 will require 15 direct offtakes to give a total of 19 offtakes.

1.3.9 Side Weirs

Side weirs have a number of advantages over other methods of controlling upstream water levels. Compared to mechanical devices such as the AMIL gate, a side weir has the following advantages:

1) No moving parts, not subject to mechanical failure.

2) Not subject to interference and vandalism.

3) Low cost

4) Simple to construct and repair; similar to canal lining.

5) Low foreign exchange component.

The Consultant has found that the simple side weir is cheaper to construct than a duckbill weir, which requires more reinforced concrete.

The layout of the proposed side weirs is given on Drawing Number SW01 together with a schedule of standard designs for various flow ranges.

Table 1.12 presents a schedule of the 10 side weirs required.

Table 1.12: Schedule of Side Weirs

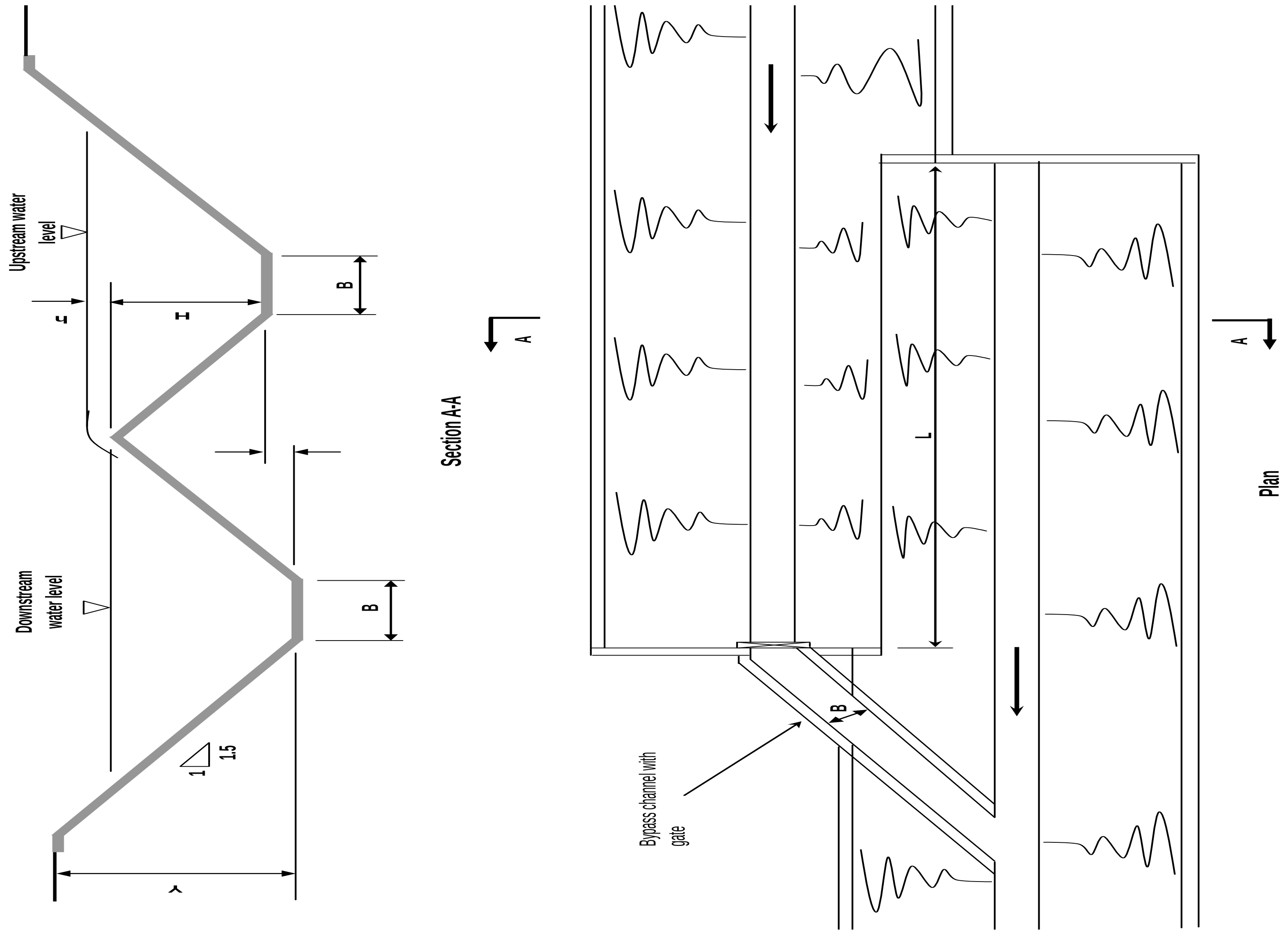
Reference	Canal bed width B	Canal section depth Y	Head on weir h	Height of weir crest H	Length of weir crest L
SW1	1.75	2.15	0.25	1.90	28
SW2	1.75	2.15	0.25	1.90	28
SW3	1.75	1.95	0.25	1.70	28
SW4	0.75	1.45	0.20	1.25	12
SW5	0.75	1.45	0.15	1.30	105
SW6	0.75	1.25	0.10	1.10	7.5
SW7	0.75	0.80	0.10	0.70	7.5
SW8	0.75	1.10	0.10	1.00	12.5
SW9	0.75	0.80	0.10	0.70	7.5
SW10	0.75	0.80	0.10	0.70	7.5

SW01, SW02, SW03 and SW08 are not required to give close regulation of the water level; their purpose is to ensure the submergence of the offtakes feeding directly from the canal to the secondary and tertiary irrigation networks.

Photograph 1.4: Side Weir at Finchaa Sugar Estate

Photograph: Booker Tate

Figure 1.6: Sketch of Side Weir



1.3.10 Pumping Station

The pumping station will lift water from reservoir R1 to the higher level irrigation areas, a static lift of about 28.50 m. The total flow lifted by 10-11 pumps will be 5.5 m^3/s. Each pump will deliver between 0.4 and 0.6 m^3/s. The size of the rising main and characteristics of the pumps are given in Table 1.13.

Table 1.13: Rising Main and Pump Characteristics

Rising main	Pipe D (m)	Length of Rising Main	Pipe class	Total head (m)	Flow (m^3/s)	Number pumps
P1	0.70	2,376	PN16	37.4	0.86	2
P2	0.60	1,016	PN16	35.7	0.81	2
P3	1.40	3,847	PN20	36.6	3.84	7
P4	0.90	1,854	PN10	-	1.04	-

P3 falls in the first part of its length such that the pressure in the pipeline is greater than the pumping head. A higher pressure class of pipe is therefore required for this pipeline. Rising Main P4 branches off P3 and, in fact, functions as an inverted siphon so that a lower pressure class of pipe can be used. The pressure classes of the pipelines takes into account surge pressures.

For this project vertical turbine mixed flow pumps have been selected because of the simple civil works required and their ease of installation. The motors will be weatherproof to obviate the need for a building with the complications of a structure to support a crane. No standby pumps will be installed but a spare pump will be supplied which will be kept in store in case it becomes necessary to remove one of the duty pumps for maintenance or repair.

It is proposed to install all thirteen pumps in a common reinforced concrete structure. This will enable all the pumps to be supplied from a common transformer and to be controlled from a single switchboard, which will be housed in a building close to the pump station.

The manifolds for the different rising mains will be separate but will be connected by a valve that is normally kept closed. In the event of failure of one the pumps supplying either P010 or P020, additional water can be supplied to the manifolds of these rising mains from the manifold of P030.

Each pump delivery will be equipped with an isolating valve and a non-return valve. The delivery pipe work will be connected to the manifold using tied flange adaptors to accommodate any differential settlement between the pump station and the manifold.

The general layout of the pump station is shown on drawing number PS01.

1.3.11 Rising Mains

Details of the Rising Mains are given in Table 1.13 above. The rising mains will be of GRP like the pipes for the inverted siphons albeit of a pressure class to withstand the pumping pressure and surge pressures. The installation of rising mains will also be similar to that of the inverted siphon pipes except that thrust blocks will be required at any bend in the pipe line. All of the rising mains will discharge into reservoirs where an overflow chamber with an area double that of the rising main will be provided to minimise surge in the pipe line. The total length of the four rising mains will be 9.1 km.

2. HEADWORKS

2.1 GENERAL

2.1.1 Location

Dinger Bereha irrigation project is situated in Oromiya Regional State, Illu-Ababor zone, at an distance of 60 km north of Bedele town. The proposed irrigation project is intended to irrigate net area of about 7500 ha of land by diverting part of the base flow of the Didessa River. The command area lies on left Bank of the Didessa River. The proposed weir site on Didessa River is located at grid reference of 203671 E and 983650 N UTM (see coordinates Photograph 1.2).

2.1.2 Objective

The objective is to study and design the proposed diversion weir on Didessa River to divert its base flow to the cite command for the development of irrigated agriculture.

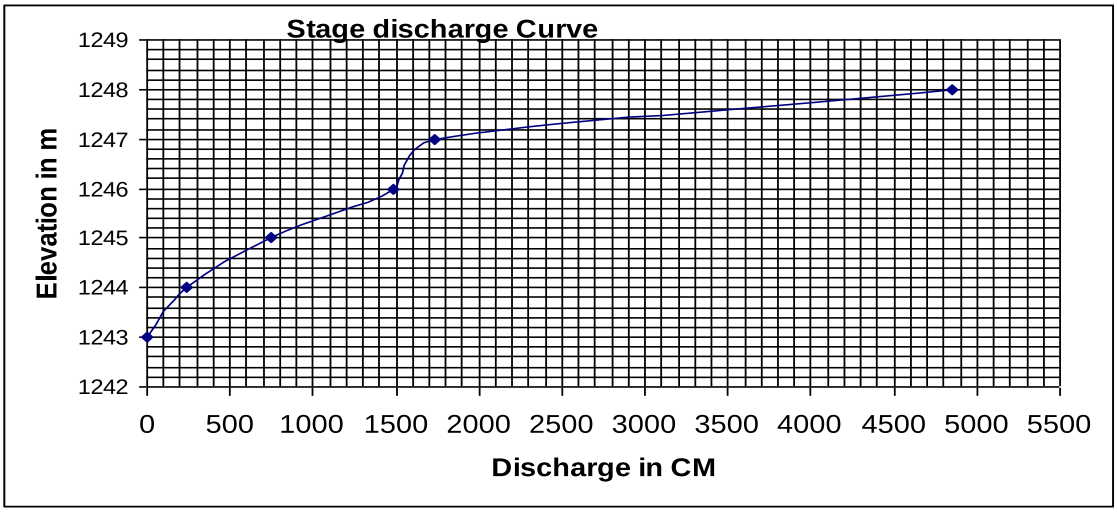
2.1.3 Hydrology

Using the available data from the Hydrology Study of the Abbay Basin Master Plan Study (upto 1996), the value of the peak flood discharge computed by statistics is 1,582 m^3/s for 100 years return period and 1,999 m^3/s for 1,000 years return period. The intended diversion weir is designed for 100 years return period and the embankment height is checked for 1,000 years return period. The hydrological study (see Annex 1) recommended to adopt values of 1,160 m^3/s and 1,235 m^3/s for the flood discharges with return periods of 100 year and 1,000 year respectively. As the reliability of the data of the period after 1996 is questionable it is recommended to retain the higher values as shown in the Abbay studies, especially as deforestation in the catchment area progresses. As will be shown in chapter 2.2.2 below the difference in embankment and wall crest levels in only 0.6 m. The corresponding difference in costs is very small compared to the overall costs of the project and therefore the higher values have been adopted for design calculations.

2.1.4 Tail water depth

Based on the weir site topographic map, river cross-sectional and longitudinal profiles were produced. Using these profiles the stage discharge curve was computed and plotted as shown below. It helps for knowing the tail water depth after construction of weir and enables to decide the arrangement of the weir and protective structures.

Figure 2.1: Didessa Stage Discharge Curve



2.1.5 Foundation conditions

The hydraulic design of the diversion weir, type and arrangements including pertinent structures is determined by the foundation conditions. The weir site foundation composition and depth of overburden, the quality of the underlying and exposed rock is thoroughly discussed in the Geotechnical report.

The soil of the valley is alluvial deposits consists of sand, silt and clay with varied thickness of 4-1.8 meter. Most part of the project area is covered by residual soil derived from the underlying crystalline basement rocks and tertiary basalt. The second layer has an average thickness of 2 meter and the river bed is covered with this rock. The layer is considered as moderately weathered granite gneiss. The layer below this is a fresh massive basement rocks/granitic gneiss.

2.2 DESIGN OF DIVERSION WEIR

Diversion weir is designed to raise the water level in the river sufficiently to the desired level to divert to divert the water in full or in part through regulator into the main supply canal for the development of irrigated agriculture. The weir is designed for both Hydraulic and structural aspects. In diversion weir design, the hydraulic condition under which it is supposed to work must be analyzed first. All the forces acting on it is calculated based on the hydraulic design. The general arrangement of the proposed weir and its main dimension is determined based on the results of hydraulic analysis and then after the structural design follows.

2.2.1 Type of diversion weir

Among various type of weirs, Ogee type weir is selected for this study. The main reason for selecting ogee type weir is that the discharge coefficient for ogee type weir is high and makes the overflow discharge very efficient. The other reason is that the foundation condition for the selected weir site is sound basement rock.

2.2.2 Hydraulic design

2.2.2.1 Fixing weir crest level

At the location of the selected weir site, the river bed is approximately at +1243m above mean sea level (amsl). The weir crest elevation is fixed with reference of river bed elevation considering the following factors.

- The crest level should be set at desired height or level to be able to obtain the required driving head to safely deliver the designed discharge to main canal.
- The weir crest should be set to allow a safely passage of maximum flood discharge within designed weir crest length.
- The bed level of the under sluice should be below sill level of canal head regulator.
- The main canal at the head reach should not be too deep in order to avoid large excavation work, to minimize construction cost and to reduce maintenance and side slopes stability problems.
- The availability of sufficient space for construction of settling basin and the possibility for hydraulic flushing should be considered. There should be sufficient head for flushing operation.

2.2.2.2 Height of weir

The following parameters have been used to determine the elevation of the weir crest and the height of the weir:

- River bed elevation: +1243 average
- Canal bed level at intake: 1.18 m above river bed level so +1244.18
- F.S.L canal depth is 1.61m, so its elevation is +1245.79
- Minimum driving head for full supply discharge : 0.21m
- Total minimum height of weir and crest elevation: 3.00 m and +1246.00 resp.

2.2.2.3 Length of the weir

The length of the weir crest depends on the physical features of the selected weir sit. A weir with a long crest gives a small discharge per unit length and hence, the required energy dissipater per meter of the crest width is smaller than what is needed for a shorter crest length. A weir crests longer than maximum wetted river width causes formation of islands at upstream side of the weir. The formation of island upstream of the weir reduces the effective length of the crest (part of the weir less effective in passing the flood). As a general rule the crest length of the weir including scouring sluice, should be taken as the average wetted width during the flood. If possible the flow per unit width should not exceed 15m3/s/m; this will avoid a relative costly energy dissipation arrangement. Increasing the length of the weir crest to 1.2 times the river width is allowable. Accordingly, the length of the overflow part of the weir is taken as 110 meter.

2.2.2.4 Flow depth over the weir crest and downstream flow profile

The flow depth over the weir crest and the downstream flow profiles have been calculated for two cases :

Q_{100} = 1,582 m3/s, and

Q_{100} = 1,160 m3/s.

Table 2.1 presents a summary of the results whereas sections 2.2.2.5-2.2.2.8 present the detailed calculations. For the larger floodflows the crest of the wingwalls and the embankment would have to be increased by 0.60 m. The costs associated to this increase are very small in relation to the total costs of the project (less than 0.2%) and therefore it is recommended to maintain the values for the larger floodflows in the final calculations of the flood and crest levels.

Table 2.1: Comparison of weir parameters for two floodflow conditions

Item	**Case Q = 1,160 m3/s**	**Case Q =1,582 m3/s**
Q_{100} (m3/s)	1,160	1,582
L weir (m)	110	110
L backwater (m)	1,150	1,000
Max flowdepth over weir (m)	2.67	3.27
Upstream waterlevel (m)	1,248.7	1.249.3
Waterlevel d/s jump (m)	1,246.7	1,247.4
u/s elevation wingwalls and embankment	1,249.4	1,250.0
d/s elevation wingwalls and embankment	1,248.0	1,248.6

2.2.2.5 Flow depth at Q_{100} = 1,582 m3/s

The flow depth over ogee shaped weir crest is determined by the following empirical formula:

$$Q = CL(He)^{3/2}$$

$$He = (Q/CL)^{2/3}$$

where,

Q = peak flood discharge in m3/s = 1,582 m3/s

L = length of weir crest in m = 110 m

He = over flow depth including approaching velocity head in m

C = discharge coefficient which varies 2.0- 2.2 for ogee shaped weir. To start with the C value of 2.2 is selected.

$$H_e = \left(\frac{1582}{2.2 \times 110}\right)^{2/3} = 3.50m$$

Approach velocity:

$$V_a = \sqrt{2g(He - h)}$$

$$V_a = \frac{Q}{(P+h)L}$$

$$\frac{Q}{(P+h)L} = \sqrt{2g(H_e - h)}$$

Where,

h = depth of water over the weir crest in m

Va= approach velocity in m/s

P = height of weir above river bed

$$\left(\frac{3.2469}{3+h}\right)^2 = 3.5 - h$$, by trial and error h = 3.23

Velocity of approach, $V_a = \sqrt{2g(He - h)} = \sqrt{19.62(3.5 - 3.23)} = 2.3m/s$

The corresponding velocity $h_a = \frac{V^2}{2 * g} = \frac{2.30^2}{2 \times 9.81} = 0.27m$ head,

The discharge coefficient C is influenced by a number factors and their effects are checked as follows:

-effect of depth of approach for vertical upstream face,

P/h = 3/3.23 = 0.928 and for He/h = 3.50/3.23 = 1.084→

C/Cd = 0.98 => C = 0.98*Cd = 0.98*2.2 = 2.156

- effect of downstream apron condition,

$$\frac{h_d + d_s}{H_e} = \frac{7.05}{3.5} = 2.014 \Rightarrow C_1 = 1.00$$

- effect of tail water condition

$$\frac{h_d}{H_e} = \frac{2.7}{3.5} = 0.77 \Rightarrow C_2 = 1.00$$

Where,

hd = drop in head between u/s and d/s

ds = tail water depth

Corrected discharge coefficient C = 2.156x1x1= 2.156

$$H_e = \left(\frac{1582}{2.156 \times 110}\right)^{2/3} = 3.54m$$

Depth of water over the weir h = He- ha = He – (Va2/2g), Va = √2g (He-h)

$$V_a = \frac{Q}{(P+h)L}$$

$$\frac{Q}{(P+h)L} = \sqrt{2g(H_e - h)}$$

$$\left(\frac{3.2469}{3+h}\right)^2 = 3.54 - h$$, by trial and error, h = 3.27m

$$V_a = \sqrt{2 \times 9.81(3.54 - 3.27)} = 2.30m/s$$

$$h_a = \frac{V^2}{2 * g} = \frac{2.30^2}{2 \times 9.81} = 0.27m$$

He = h + ha = 3.27 +0.27 = 3.54m

Back water influence,

$$L = \frac{2(h + P - Y_n)}{S_b} = \frac{2(3.27 + 3 - 3.8)}{0.005} = 988 = 1000m$$

Sb = River bed slope and yn normal water depth in the river.

The rise of water level will not affect any existing infrastructure and agricultural land.

2.2.2.6 Flow depth at Q_{100} = 1,160 m3/s

The flow depth over ogee shaped weir crest is determined by the following empirical formula.

$Q = CL(H_e)^{3/2}$

$H_e = (Q/CL)^{2/3}$

where,

- Q = peak flood discharge in m^3/s = 1,160 m^3/s
- L = length of weir crest in m = 110 m
- H_e = over flow depth including approaching velocity head in m
- C = discharge coefficient which varies 2.0- 2.2 for ogee shaped weir. To start with the C value of 2.2 is selected.

$$H_e = \left(\frac{1{,}160}{2.2 \times 110}\right)^{2/3} = 2.84m$$

Approaching velocity

$$V_a = \sqrt{2g(He - h)}$$

$$V_a = \frac{Q}{(P+h)L}$$

$$\frac{Q}{(P+h)L} = \sqrt{2g(H_e - h)}$$

where,

- h = depth of water over the weir crest in m
- V_a= approaching velocity in meter
- P = height of weir above river bed

$\left(\frac{2.381}{3+h}\right)^2 = 2.84 - h$, by trial and error h = 2.666m, 2.67m

Velocity of approach, $V_a = \sqrt{2g(He-h)} = \sqrt{19.62(2.843 - 2.666)} = 1.863 m/s$

The corresponding velocity head, $h_a = \frac{V^2}{2 * g} = \frac{1.863^2}{2 \times 9.81} = 0.177m$

The discharge coefficient C is influenced by a number factors and their effects are checked as follows

-effect of depth of approach for vertical upstream face :

P/h = 3/2.667 = 1.124 and for H_e/h = 2.843/2.667 = 1.066→

C/C_d = 1.01 => C = 1.0*C_d = 1.0*2.2 = 2.2

-effect of downstream apron condition :

$$\frac{h_d + d_s}{H_e} = \frac{2.80 + 3.043}{2.843} = 2.055 \Rightarrow C_1 = 1.00$$

-effect of tail water condition

$$\frac{h_d}{H_e} = \frac{2.8}{2.843} = 0.985 \Rightarrow C_2 = 1.00$$

where,

- h_d = drop in head between u/s and d/s
- d_s = tail water depth

Corrected discharge coefficient C = 2.2x1x1= 2.2

$$H_e = \left(\frac{1{,}160}{2.2 \times 110}\right)^{2/3} = 2.843m$$

Depth of water over the weir h = 2.667

$$V_a = \sqrt{2g(He - h)} = \sqrt{19.62(2.843 - 2.666)} = 1.863m/s$$

$$h_a = \frac{V^2}{2 * g} = \frac{1.863^2}{2 \times 9.81} = 0.177m$$

H_e = h + h_a = 2.667 +0.177 = 2.844m

Back water influence :

$$L = \frac{2(h + P - Y_n)}{S_b} = \frac{2(2.67 + 3 - 2.8)}{0.005} = 1{,}146 = 1{,}150m$$

S_b = River bed slope and y_n normal water depth in the river.

The rise of water level will not affect any existing infrastructure and agricultural land.

2.2.2.7 Downstream flow profile at Q100 = 1,582 m3/s

The proposed weir is sited in high-quality rock, i.e. the river bed is covered with sound basement rock and no need of provision of river bed protection structures, such as stilling basin. The average elevation of the river bed is about 1243 m. By removing the top weathered part of the rock to a depth of 0.5 m, the floor elevation at the toe of the weir becomes 1242.50m.

Discharge per meter of width q = Q/L = 1582/110 = 14.382 m3/s

Upstream total energy height...........................= 3.54 m

Weir crest elevation ....= 1246 m

Upstream total energy level .... = 1249.54m

Total upstream energy above downstream bed level (Eo),

Energy $$E_o = d_1 + \frac{V_1^2}{2 \times g} = d_1 + \frac{q^2}{2 \times g \times d_1^2}$$

$$7.04 = d_1 + \frac{10.539}{d_1^2} =$$, By trial and error d1 = 1.362m

$$V_1 = \frac{q}{d_1} = \frac{14.382}{1.362} = 10.559 m/s$$

$$h_a = \frac{V^2}{2 \times g} = \frac{10.56^2}{19.62} = 5.68m$$

Froude number $$F_1 = \frac{V_1}{\sqrt{g \times d_1}} = \frac{10.56}{\sqrt{9.81 \times 1.362}} = 2.89$$

E1= 5.68 +1.36 = 7.04 = Eo : OK.

Energy at upstream of the jump

$$d_2 = \frac{d_1}{2} \times \left(\sqrt{1 + 8F^2{}_1} - 1\right) = \frac{1.36}{2}\left(\sqrt{1 + 8 \times 2.89^2} - 1\right) = 4.92m$$

$$V_2 = \frac{q}{d_2} = \frac{14.382}{4.92} = 2.92m$$

$$h_{v2} = \frac{V^2}{2 \times g} = \frac{2.92^2}{19.62} = 0.43$$ m

E2 = d2+hv2 = 4.92 + 0.43 = 5.35m

Energy at downstream due to tail water E3 = d3 + hv3 = 3.8+0.46=4.26m, (d3 = tail water depth).

Upstream water elevation = 1249.27

Elevation of the tail water level = 1246.80m above mean sea level

Downstream water level after jump = 1247.42 m above mean sea level

Freeboard at d/s, Fb = 0.1(V1 +d1) = 0.1(10.57+1.36) = 1.2m

Elevation of the d/s wing wall = 1247.42 + 1.20 = 1248.62 m

Elevation of u/s wing walls and embankments = 1250 m

2.2.2.8 Downstream flow profile at Q100 = 1,160 m3/s

The proposed weir is sited in high-quality rock, i.e. the river bed is covered with sound basement rock and no need of provision of river bed protection structures, such as stilling basin. The average elevation of the river bed is about +1243 m. By removing the top weathered part of the rock to a depth of 0.5 m, the floor elevation at the toe of the weir becomes +1242.50.

Discharge per meter of width q = Q/L = 1,160/110 = 10.55 m^3/s

Upstream total energy height...........................= 2.843 m

Weir crest elevation ....= +1246 m

Upstream total energy level .... = +1248.843m

Total upstream energy above downstream bed level (E_o),

E_o = H_e +1246 – 1242.5 = 2.843 + 1246 -1242.5 = 6.343m

Energy upstream of the jump (E_1) = E_o

$$E_o = d_1 + \frac{V_1^2}{2 \times g} = d_1 + \frac{q^2}{2 \times g \times d_1^2}$$

$6.343 = d_1 + \dfrac{5.668}{d_1^2} =$, By trial and error d_1 = 1.033m

$$V_1 = \frac{q}{d_1} = \frac{10.545}{1.033} = 10.2086 m/s$$

$$h_a = \frac{V^2}{2\times g} = \frac{10.2086^2}{19.62} = 5.312m$$

Froude number $F_1 = \frac{V_1}{\sqrt{g\times d_1}} = \frac{10.2086}{\sqrt{9.81\times 1.033}} = 3.2069$

E_1= 5.312 +1.033 = 6.345 = E_o , OK.

Energy at upstream of the jump

$$d_2 = \frac{d_1}{2}\times\left(\sqrt{1+8F^2{}_1}-1\right) = \frac{1.033}{2}\left(\sqrt{1+8\times 3.2069^2}-1\right) = 4.197m$$

$$V_2 = \frac{q}{d_2} = \frac{10.545}{4.197} = 2.513m$$

$$h_{v2} = \frac{V^2}{2\times g} = \frac{2.513^2}{19.62} = 0.322\,\text{m}$$

E_2 = d_2+h_{v2} = 4.197 + 0.322 = 4.519m

Energy at downstream due to tail water E_3 = d_3 + h_{v3} = 2.8+0.46=3.26m, (d_3 = tail water depth).

Upstream water elevation = 1248.67

Elevation of the tail water level = 1246.80m above mean sea level

Downstream water level after jump = 1246.697m above mean sea level

Freeboard at d/s, F_b = 0.1(V_1 +d_1) = 0.1(10.2086+4.197) = 1.44m

Elevation of the d/s wing wall = 1242.5+4.197 + 1.44 = 1248 m

Freeboard at u/s, F_{ub} = 0.61+0.037*1.863*$5.67^{1/3}$ = 0.73m

Elevation of u/s wing walls and embankments = 1248.67 +0.73 = 1249.40

2.2.3 Determination of weir section

The downstream profile of the weir is determined by the following empirical formula. The general equation for downstream profile for all ogee shaped weir is as follows.

$$X^n = kH_d{}^{n-1}y$$

where,

- (x, y) are co-ordinates of the points on the crest profile measured from the apex of the crest.

- Hd = design head excluding velocity head of the approach flow (CHOW)
- K and n = constants depending upon the slope of the upstream face
- = (K= 2 and n = 1.85 for vertical u/s face)

$$X^{1.85} = 2H_d^{\ 0.85} y = 2 \times 3.27^{0.85} y = 5.475y \Rightarrow y = 0.1826X^{1.85}$$

Table 2.2: Crest Profile

X	0.75	1.20	1.50	2.00	2.50	3.00	3.50	3.60	4.00	4.50	4.94
Y	0.11	0.26	0.39	0.66	0.99	1.39	1.85	1.95	2.37	2.95	3.51

The upstream weir crest profile is calculated as follows

R1 = 0.50Hd = 0.50*3.27 = 1.64m

R2 = 0.20Hd = 0.20*3.27 = 0.65m

X = 0. 282Hd = 0.282*3.27 = 0.922 m

In order to create a smooth transition of the flow and to prevent the impact of falling water from scouring the foundation, the surface of the weir toe is design as curved bucket. To be thoroughly effective the bucket should be tangent to the foundation. The radius R of the bucket is estimated by the following empirical formula.

R = 10(v+6.4h +16)/3.6h +64)

Where,

V = velocity at the toe in ft/s

h = Head excluding velocity head in ft

R = 10(1.16285) = 14.55 ft = 4.40 m

2.2.4 Stability analysis

The stability of the structure is checked for two conditions i.e. for condition of high flood level and for condition when water level is at crest level, but no over flow. For the structure to be remained stable the following conditions must be fulfilled:

- The structure must safe against sliding, over turning and resultant force must lie within middle third.
- There should not be tension under the base
- The maximum toe and heel pressures on foundations should not exceed the prescribed safe limits.

Table 2.3 presents the stability analysis for high flood conditions

Table 2.3: Stability Analysis Calculation

Name of force	Symbol	Vertical Force -Downward (+Ve) -upward =(-Ve)		Horizontal Force -towards u/s = +Ve -towards d/s = -Ve		Lever arm (m)	Moment about the toe Anticlockwise (+Ve) Clockwise (-Ve)
1.Weight of weir	W1	1.90*3.5*2400 =	15960			7.05	112518
	W2	4.5*3.5*0.5*2400 =	18900			4.6	86940
	W3	8*1.5*2400 =	28800			4	115200
	∑ W		63660				314658
3.Uplift Force	U1	2.86*8*1000	-22880			4	-91520
	U2	5*0.5*1000*8 =	-20000			5.333	-106660
	∑		-42880				-198180
4. Horizontal hydrostatic & silt pressure	P1			3.27*5*1000 =	-16350	2.5	-40875
	P2			52*0.5*1000 =	-12500	1.67	-20875
	P3			1.4*1.5*1000 =	2040	0.75	1530
	P4			1.52*0.5*1000 =	1125	0.5	562.5
	Ps			32*360*0.5 =	-1620	3	-4860
	∑				-27305		2093, (-66610)

∑V = 63660 -42880 = 20780

∑H = 27305

2.2.4.1 Factor of safety against sliding

(S.F.S) = μ*∑V/∑H = 0.75*20,780/27,305 = 0.57 <1, it is unsafe for sliding. In order to improve sliding conditions the weir should be tied with its foundation by 2Ø 24mm anchor bars per linear width that grouted into holes drilled into the foundation rock and they are placed at a distance of 7&5m from d/s. This will increase the magnitude of safety factor against sliding to1.22 for the worst condition. The additional forces developed by anchored bars are:

Horizontal forces act toward u/s is 6,786kgf and vertical forces acting downward is 12,667kgf,

∑H = 27,305-6786 = 20,529kgf

∑V = 12,667+20,780 = 33,447

(S.F.S) = 0.75*33447/20529 = 1.22 > 1, OK

∑Mr = 314,658 +2,093 = 316751

∑Ma = -198,180 + (-66,610) = -264,790

2.2.4.2 Factor of safety against overturning

S.F = ∑Mr/∑Ma = 316,751/264,790 = 1.2, it is less, however, the anchor bars provided for sliding condition will also improve the factor of safety against over turning. The additional resisting moments developed duet anchored bars are = 76,002kgfm and ∑Mr = 316,751+76,002 = 392,753.

S.F = ∑Mr/∑Ma = 392,753/264,790 = 1.48, it is safe

Location of Resultant(R) force from the toe

Resultant force R = $\sqrt{(\sum V)^2 + \left(\sum H\right)^2} = \sqrt{33447^2 + 20529^2} = 39245$, it is at a distance of $\overline{X}$ from the toe. $\overline{X} = \frac{\sum M}{\sum V} = \frac{127963}{33447} = 3.83m$ and it lies within middle third of the base.

Vertical stress Pmax/min

$$P_{\max/\min} = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right)$$

Where,

B = base width of the weir

e = Eccentricity of the resultant force from the centre of the base. It must be less than B/6 = 8/6 = 1.333m in order to ensure that no tension is developed any in the weir base.

$e = \frac{B}{2} - \overline{X} = \frac{8}{2} - 3.83 = 0.17m$ The resultant force lies at a distance 0.235 m from centre on d/s side.

$$P_{\max/\min} = \frac{33447}{8}\left(1 \pm \frac{6 \times 0.17}{8}\right) = 4181(1 \pm 0.1275)$$

Pmax = 4,181(1.1275) = 4,714 kgf/m2 = 0.47 kg/cm2 < allowable, Ok

Pmin = 4,181(0.8725) = 3,648 kg/m2 = 0.36 kg/cm2 < allowable, Ok, so tension will not develop at the toe.

Table 2.4: Stability analysis for no over flow condition

Name of force	Symbol	Vertical Force -Downward = +Ve -upward = -Ve		Horizontal Force -towards u/s = +Ve -towards d/s = -Ve		Lever arm (m)	Moment about the toe Anticlockwise (+Ve) Clockwise (-Ve)
1.Weight of weir	W1	1.90*3.5*2400 =	15960			7.05	112518
	W2	4.5*3.5*0.5*2400=	18900			4.6	86940
	W3	8*1.5*2400 =	28800			4	115200
	Σ W		63660				314658
3.Uplift Force	U1	1.5*8*1000	-12000			4	-48000
	U2	3.5*0.5*1000*8 =	-14000			5.333	-74662
	Σ		-26000				-122662
	P2			52*0.5*1000 =	-12500	1.67	-20875
	P4			1.52*0.5*1000 =	1125	0.5	562.5
	Ps			32*360*0.5 =	-1620	3	-4860
	Σ				-12995		563, -25735

Forces and moments due anchor are not included in this analysis

ΣV = 63660-26000 = 37660

ΣH = 12995

2.2.4.3 Factor of safety against sliding

(S.F.S) = μ*ΣV/ΣH = 0.75*37660/12995 = 2.17 >1, OK

ΣMr = 314658+563 = 315221

ΣMa = 122662 +25735 = 148397,

2.2.4.4 Factor of safety against overturning

S.F = ΣMr/ΣMa =315221/148397 = 2.12 >1.3 Ok

Location of Resultant(R) force from the toe

Resultant force R = $\sqrt{(\sum V)^2 + (\sum H)^2} = \sqrt{37660^2 + 12995^2} = 39839$, it is at a distance of

$\overline{X}$ from the toe. $\overline{X} = \frac{\sum M}{\sum V} = \frac{166824}{37660} = 4.43m$ and it lies within middle third of the base on heel side of the base centre.

Vertical stress Pmax/min $P_{\max/\min} = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right)$

Where,

B = base width of the weir

e = Eccentricity of the resultant force from the centre of the base. It must be less than B/6 = 8/6 = 1.33m in order to ensure that no tension is developed any in the weir base.

$$e = \frac{B}{2} - \overline{X} = \frac{8}{2} - 4.43 = -0.43m$$

The resultant force lies at a distance 0.345 m from center on u/s side

$$P_{\max/\min} = \frac{37660}{8}\left(1 \pm \frac{6 \times 0.43}{8}\right) = 4708(1 \pm 0.3225)$$

Pmax = 4708(1.3225) = 6226 kgf/m2 = 0.623 kg/cm^2 < allowable, Ok

Pmin = 4708 (0.6775) = 3190 kg/m2 = 0.32 kg/cm^2 < allowable, Ok, so tension will not develop at the toe.

2.3 DESIGN OF HEAD REGULATOR

Design data:

Canal full supply discharge .... = 8.25 m^3/s

Canal bed width at head reach .... = 4.5 m

Full supply depth at the head reach .... = 1.61 m

Canal bed elevation at out let .... = +1244.18 m

Full supply elevation .... = +1245.79 m

Longitudinal slope at the head reach .... = 0.00056

Side slope at head reach within 76 m length... = vertical

Weir data:

River bed elevation .... = +1243

Weir crest elevation .... = +1246

The canal outlet is proposed to take off at an angle 90° with diversion weir axis. The canal bed with is 4.5 m and it is proposed to provide with two bays of 1.5m by 1.90m width & height with one 0.5m thick pier and the discharge is determined by drowned weir formula by neglecting the approach velocity head.

$$Q = \left(\frac{2}{3} C_1 L \sqrt{2g}(h)^{\frac{3}{2}} + C_2 LH\sqrt{2gh}\right) \times 2$$

Where,

Q = Discharge required for irrigation = 8.25 m^3/s

C1& C2 = Discharge coefficient, C1 = 0.557, C2= 0.80

B = width of the gate ……………… = 1.5

h = head difference ……………….. = 0.21

H = depth of water at down stream… = 1.61

$$Q = \left(\frac{2}{3} * 0.557 * 1.5 \sqrt{2 * 9.81} (.21)^{\frac{3}{2}} + 0.8 * 1.5 * 1.61 \sqrt{2 * 9.81 * 0.21} \right) \times 2$$

Q = 2*(0.237 + 3.92) = 8.314 m3/s > 8.25 m^3/s

2.3.1 Under Sluices

Under sluices are proposed to be provided at the left end of diversion weir from where canal takes off. Since only one canal is proposed to take off from the left side of the weir, only one sluice is provided near head regulator. Under sluice shall be aligned in line with axis of weir. The bed elevation of the under sluices is proposed to be kept at river-bed elevation of 1243.00 m. The width of the waterway of under sluices is proposed to be 3.50 m. The under sluices are provide with two openings of 1.5m width by 2m height and separated by 0.50m thick pier.

2.3.1.1 Discharge through under sluice

Discharge through under-sluices for the condition when the upstream water level is at crest level and no water in downstream has been calculated. Under this condition the openings of under-sluices work as free orifice. The discharge through free orifice,

$$Q = C_d \; x \, A \sqrt{2gh}$$

$$= C_d \; x \, L_{e\,ff} \; x \, d \sqrt{2gh}$$

Where,

Q = discharge through under-sluices

Leff = effective length of under-sluices openings = $(L - 0.1nd)$

L = Total leaner water way

n = number end of contraction due to piers

d = depth of opening

Cd = coefficient of discharge = 0.60 (approx.)

h = head over orifice

For under sluice, Leff = (1.5*2-0.1*2*1) = 2.8

Cd = 0.6

d = 2.00

h = 1.00

$$Q = 0.6x\,2.8 * \sqrt{2*9.81*1} = 15m^3/s$$

V= Q/A = 15/(1.5*2*2) = 2.5m/s

When upstream water level is at highest flood elevation i.e. about 1249.27 and downstream water level is also at highest flood elevation of 1246.80 m. Therefore, the under sluices shall work as drowned or submerged orifices. The discharge through drowned orifice shall be obtained by formula:

$$Q = C_d \times L_{eff} \times d\,\sqrt{2g\,(H_1 - H_2)}$$

Where

Q = Discharge through under-sluices

Cd = coefficient of discharge = 0.60

Left= effective leaner water way

H_1 = Depth of water upstream above river bed elevation

d = depth of under-sluices opening

H_2 = Depth of water downstream above river bed

$$Q = 0.60 \times 2.80 \times 2\sqrt{2 \times 9.81 \times (1249.27 - 1246.80)} = 23.39m^3/s$$

2.4 DESIGN OF SETTLING BASIN

At the head works the required flow to an irrigation scheme is diverted from the river. The fine sediments like clay and silt, even medium sands will be transported in suspension and can not be excluded from entering the canal system. Provision of settling basin at the head reach of the main canal will trap the ingress sediment before conveyed far to the downstream. Sediment deposits can be removed by hydraulic flushing methods.

The designflow has been maintained at 8.25 m3/s, the discharge value used in the original calculations. The difference in dimensions caused by a reduction of this flow are minor and associated cost reduction are very small. The resulting overcapacity could allow a substantial increase in command area above the 7,500 net.

Points to be considered for designing settling basin :

- Sufficient head difference should available between the water level in the settling basin and the river at the flushing point.
- The minimum velocity during sluicing when the basin is almost empty should be 1.5-2 m/s or more in case gravel is expected to enter the basin.
- Sufficient storage capacity should be available in the settling basin for storing the sediment entering the basin between periodical cleaning of the basin.
- The bed slope of the settling basin has to be at least equal to critical slope.
- The average velocity when settling basin is filled should be limited to 0.5 – 0.6 m/s in order to avoid unwanted sediment ingress into the canal system.

Length of sand trap

a) Fall velocity (V_f): -The falling velocity of the sediment particles with a diameter of 0.1mm is calculated using Stocke's Law as follows.

$$V_f = \frac{1}{18} \times D^2 \frac{(G_s - G_w)}{\mu} g = \frac{1}{18} \times (0.01)^2 \times \left(\frac{(2.65-1)}{0.0115} \right) \times 981 = 0.78 cm/s = 0.0078 m/s$$

Where,

D: Minimum diameter of deposited materials (0.01cm)

G_s: Specific gravity of deposited materials (2.65)

G_w: Specific gravity of water 15^oC (1.)

g: gravity acceleration (9.81 m/s^2)

V_f: Fall velocity (cm/sec)

μ: Kinematic viscosity of water in 15^{0}C (0.0115 cm^2/s)

Length of sand trap

$$L = \frac{V}{V_f} \times d = \frac{0.65}{0.0078} \times 1.65 = 136 \approx 150 m/s$$

Where

V = average velocity of flow in the basin

d = average mean depth in the basin

Critical depth calculation

$$H_c = \sqrt[3]{\frac{Q^2{}_c}{g \times B^2} \frac{3\sqrt{1+2m(H_c/B)}}{1+m(H_c/B)}}$$

$$H_c = \sqrt[3]{\frac{8.25^2}{9.81 \times 4.5^2} \frac{\sqrt[3]{1+2\times1.5(H_c/4.5)}}{1+1.5(H_c/4.50)}} = 0.65m$$

Critical velocity

A_c = (B+ mH_c)*H_c = (4.5+1.5*0.65)*0.65 = 3.56 m^2

T = B+2mH_c = 4.5 + 2*1.5*0.65 = 6.45

V_c = √ (g*A_c/ T) = (9.81*3.56/6.45)^0.5 =2.33m/sec

$$F = \frac{V_c}{\sqrt{g \times \frac{A_c}{T}}} = \frac{2.327}{\sqrt{9.81 \times \frac{3.56}{6.45}}} = 1$$

Critical hydraulic gradient (I_c)

$$R_c = \frac{A_c}{B+2H_c\sqrt{1+m^2}} = \frac{3.56}{4.5+2\times0.65\sqrt{1+1.5^2}} = 0.52$$

$I_c = \left(\frac{V_c \times n}{R^{2/3}}\right)^2 = \left(\frac{2.33\times0.02}{0.52^{2/3}}\right)^2 = 0.0052$, It is 1m drop in190 m length.

For the settling basin of 150 meter length, the fall is 150*0.0052 = 0.78 m.

However, to make the flow supper critical, a meter of drop in 150 meter length is provided.

Retention time for settlement in a basin is given by $T_R = \frac{Ad_o}{Q}$

Where,

T_R = retention time

A = mean plan area of basin (10.74*150 = 1611 m^2)

d_o = basin mean flow depth (1.65m)

Q = design flow = 8.25 m^3/s

$$T_R = \frac{1611\times1.65}{8.25} = 322.2s$$

Settling time (T_s) under quiescent, non turbulent conditions

$$T_s = \frac{d_o}{V_f} = \frac{1.65}{0.0078} = 212s$$

For effective operation, $T_R \geq T_S$; Hence, the system is very effective.

Calculation for scouring

The bed slope of the settling basin is 1/150 = 0.00666

Mean hydraulic depth at scouring time

$$\frac{Q \times n}{\sqrt{S}} = \frac{\{(b + md) \times d\}^{5/2}}{\left(b + 2d\sqrt{1 + m^2}\right)^{2/3}}$$

Where,

Q = design discharge

S = bed slope

n = Manning's roughness coefficient

m = side slope

b = bed width

d = depth of flow

$$\frac{8.25 \times 0.02}{\sqrt{0.0066}} = \frac{\{(4.5 + 1.5d) \times d\}^{5/2}}{\left(4.5 + 2d\sqrt{1 + 1.5^2}\right)^{2/3}} = 0.42$$

By trial and error d = 0.42m

$$A = (4.5 + 1.5 * 0.42) * 0.42 = 2.1546m^2$$

$$V = \frac{Q}{A} = \frac{8.25}{2.1546} = 3.829 \text{ m/s}$$

Mean Hydraulic gradient at scouring

$$R_S = \frac{(B + m \times d_s)d_s}{B + 2 \times d_s \times \sqrt{1 + m^2}} = \frac{(4.5 + 1.5 \times 0.42) \times 0.42}{4.5 + 2 \times 0.42 \times \sqrt{1 + 1.5^2}} = \frac{2.1546}{6.0514} = 0.3582$$

I_S = (0.02*3.829/0.3582^0.666666)^2 =0.023 > 0.0052 Ok

Size of the Flushing gate

$$Q = 1.5*b*h^{3/2}$$

Where,

Q = discharge for flushing

b = net width of flushing gate

h = depth of flow which is usually the same height of gate

$Q = 2*(1.5*1.2*1.8^{1.5}) = 8.69\ m^3/s$

Two gates with a size of 1.2m width and 1.8m height can be provided for flushing the sediment.

2.5 DESIGN OF EMBANKMENTS

Because of the weir across river, the water level will rise upstream of structure. The structure is designed to function in a restricted width of river. Raised water levels may cause out flanking of riverbanks near the structure if the banks are not above afflux level. To prevent out flanking, earthen protection embankments on the flanks of the weir are provided.

The protection embankments are provided from the abutment of the weir structure and extended up to the point where the required ground elevation corresponding with the top embankment elevation joins the bank. The top level of both embankments is kept at an elevation of 1250m amsl. The length of the right and left embankments are 178 and 160 m respectively. The maximum height is about three meter above ground level whereas the width at the crest will be five meter. Upstream and downstream sideslopes will be 1:2. Hand placed stone riprap with a thickness of 0.3 meter will be provided on the upstream face slope. A cut-off trench of 1.5 meter depth, 3 meter bottom width and 1:1 side slope is proposed in order to prevent seepage underneath the embankment.

3. ON FARM AND TERTIARY UNIT DESIGN

3.1 MAIN CONCEPT FEATURES

During the design of the on-farm equipment and organisation that determines the capacity and operation of the secondary and tertiary systems the following factors have been taken into account:

- The topographical conditions show a general slope > 2% for 90% of the area.
- The numerous gullies and streams cutting the Project area into several main interfluves whereas each interfluve comprises numerous small plateaus.
- The need to minimise the energy requirements by reducing pumping.
- The present social structure and land organisation based on a small farms, and family labour without mechanization.
- The actual farmer's knowledge of water use and management of surface irrigation, when water is available.
- The possibility of future improvement of irrigation efficiencies, with progressive development of modern field irrigation systems without modification of the conveyance and distribution networks and associated water management.

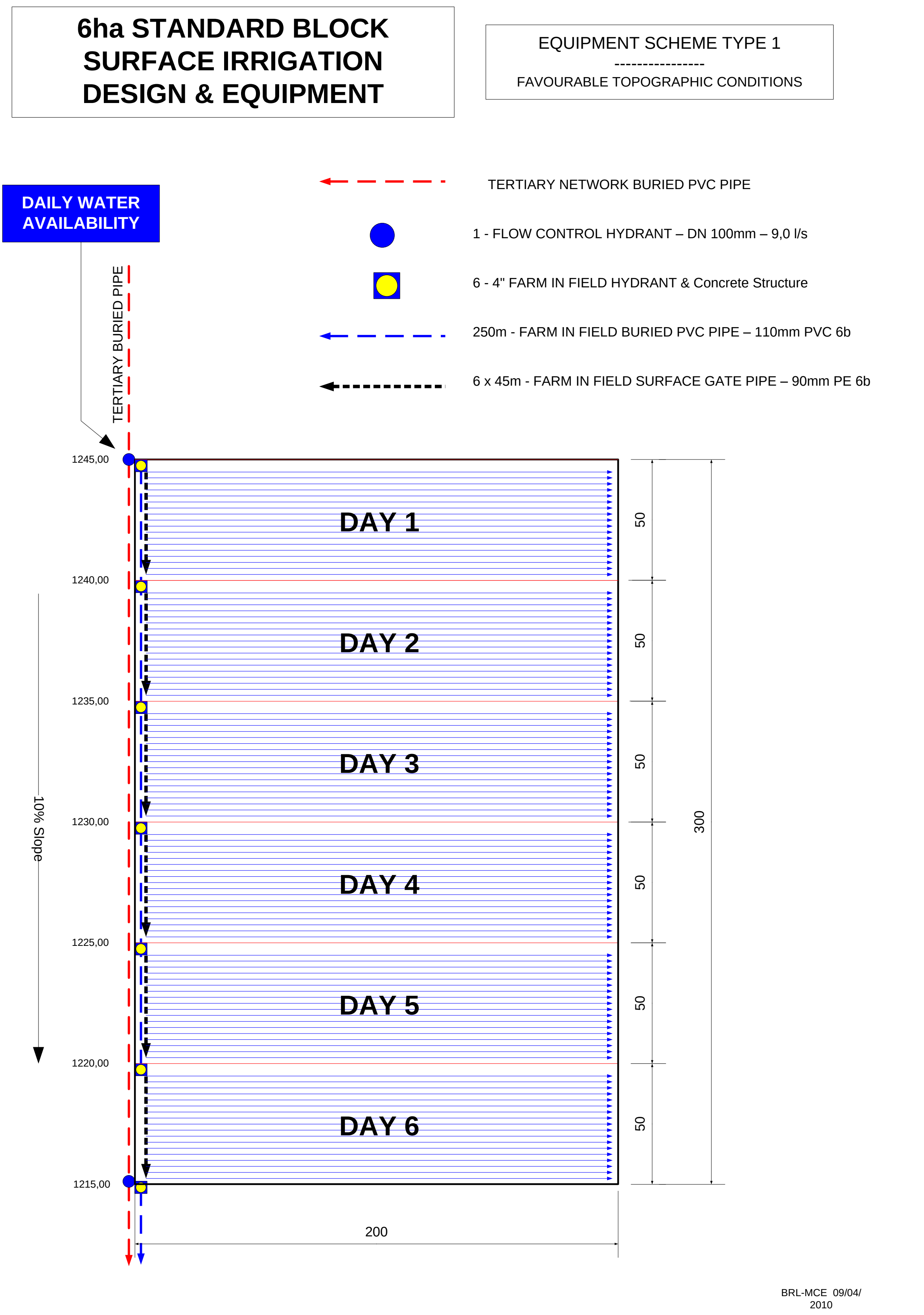
Thus, the design has been based on the principle of permanent water availability during 12 hours of irrigation per day at the head of standard command bloc. The corresponding water discharge is automatically controlled by a flow control device calibrated for Q = S x q0 x 12/ 24, with q0 = 0.8 l/s/ha and S the surface of the standard command bloc. The discharge is suitable with the practice of surface irrigation by furrows, with a minimum of 5 l/s per furrow. The area of the bloc can be divided into several family farms with a daily rotation of the water availability for each farm and a maximum farm watering frequency of 6 days during the peak water requirements period, in order to allow the use of sprinkler or localised irrigation systems. Due to topographical conditions and associated development costs, the only solution for the water conveyance from upstream the primary canal to downstream the head of the standard block is by buried pipes networks.

3.2 THE STANDARD BLOCK

3.2.1 Surface area of the standard block

Considering the topographical conditions of the zone, the surface of the standard bloc is small and reduced to 6 ha (200 x 300). Thus, the length of the contour lines furrows remains feasible (#200m), without excessive earthworks, and they can be earthed by the farmers. When the slope of the bloc is very important, (>8/10%), land should be terraced in order to allow a good watering of the furrows or the basins, and to avoid erosion. Inside the standard bloc, land is organized with 6 farms of 1ha (50 x 200).Due to the difficult topographic and geomorphologic conditions, the carrying out of the contour lines furrows and terraces remains the key point of the present project. The lay-out of the standard bloc with its different equipment is given here below.

Figure 3.1: Lay-out of 6 ha standard bloc



3.2.2 The discharge of the standard block

3.2.2.1 The Flow Control Hydrant

From a 0.8 l/s/ha specific hydro module designed for the project, the theoretical discharge to be delivered at the head of the standard bloc is: 0.8 x 6 x 2 = 9.6 l/s. Two types of flow limiters are presently available from manufacturers of hydraulic equipment.

A TYPE

Using a constant flow valve, EQUIVAR or BOCAR, the A Type flow control hydrant will be used in case of low pressure conditions, when 0.6 mwc < P < 7m. The standard lower discharge of such device is 10 l/s. Nevertheless, and due to the big quantity required for the project, a special manufacture with 9.6 l/s regulated flow is possible. Regulation flow range remains within the ±5% limits when pressure working range is considered. Some pictures and typical installation schemes as well as technical working conditions and parameters issued from manufacturers brochures are given in Figures 3.2 and 3.3.

Figure 3.2: Example of constant flow valve and installation options

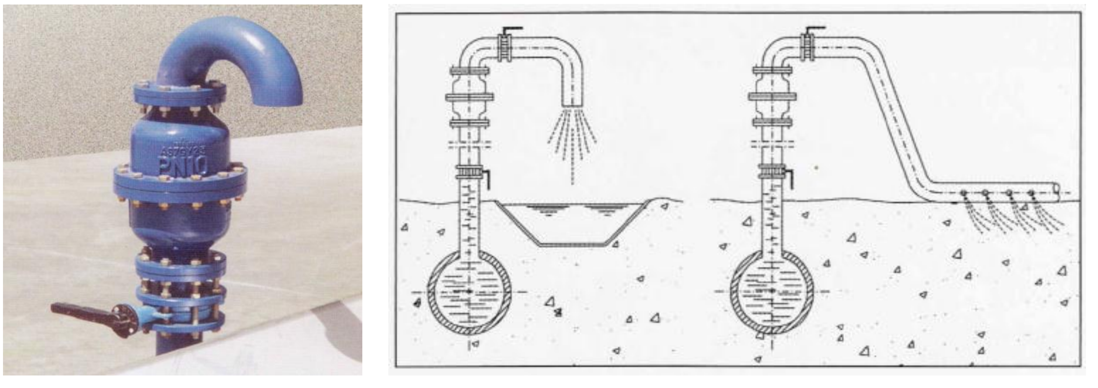


Table 3.1: Main Characteristics of Constant Flow Valves

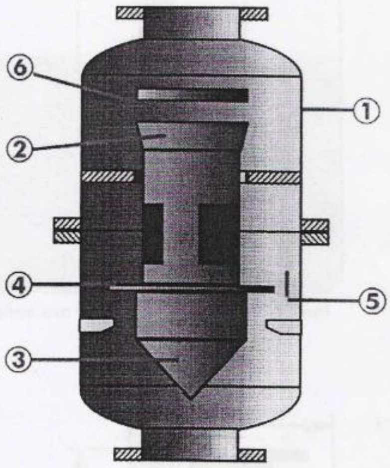
MAIN CHARACTERISTICS TABLE				
MODEL	LIMITED FLOW	MINI HEAD mwc	MAXI HEAD mwc	Nominal Diameter
1	10	0,7	10	ND 100mm
	15	1,1	15	
	20	1,6	35	
	30	3,2	45	
2	20	0,7	10	ND 150mm
	30	1,2	15	
	40	1,8	25	
	50	2,8	35	
	60	4,0	45	
3	40	0,7	15	ND 200mm
	60	1,1	25	
	80	1,6	30	
	100	2,0	35	
	200	3,2	45	

Figure 3.3: Operating Principles of Constant Flow Valves

Installed in pipe, or located under a canal or a reservoir, the EQUIVAR releases at the beginning of the parcels, in pipe or at free discharge, an accurate and invariable flow whatever the pressure of the feeding network.

The EQUIVAR is specially adapted to the low and very low pressure pipes, which take the place in some cases of the tertiary and quaternary canals.

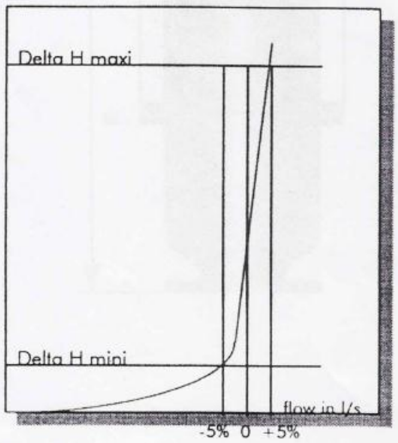
The EQUIVAR range covers flow rates from 10 up to 120 l/s and feeding pressure from 0,6 up to 60 mWC



OPERATING PRINCIPLE

The EQUIVAR is made up with a body (1), divided into a lower room and an upper room.
A moving equipment, consisting of a tube (2) and a ballast (3) is free in motion of vertical transmission.
The water which comes into the lower room is obliged to flow out through the annular passage (5), which causes a head loss and later on an hydrodynamic strain on the plate (4), which increases with the flow rate.
This strain is compensated by the weight of the ballast (3) for a balance flow. If the flow is inferior to the balance flow, the internal moving equipment is unbalanced to the bottom, which opens the annular passage (6) at the periphery of the tube (2) and increases the flow until the balance value. A flow superior to the balance flow rate creates an opposite reaction.

The constant flow valve EQUIVAR is simple, strongly built, reliable and does not require any auxiliary energy.
No special maintenance is to be planned.



The selected device is the type 1, ND 100mm, with a 9.6 l/s limited flow. The device should be installed inside an impregnable shelter with an isolating valve. The isolating valve will only be closed by agents of the water management agency under the following conditions:

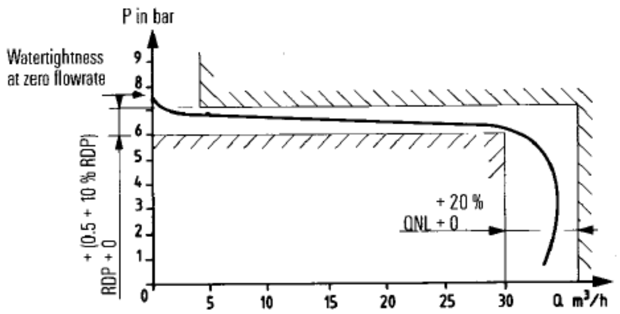
- When maintenance or repair operations are necessary for the equipment inside the command bloc.
- When irrigation times are not followed by the farmers inside the command bloc.
- In case of disputes between farmers of the command bloc.
- If water fees are not paid by the farmers.

The general scheme arrangement is given in Volume Drawings.

B TYPE

When the available pressure is higher than 7 m of water column, standard water flow limiters used for pressurized irrigation networks will be installed at the head of the bloc. As the accuracy of that kind of device is in a range of 0/+20%, a 30m³/h ND 100mm flow limiter is selected with a flow range limits between 8.3 l/s and 10 l/s, i.e. an average flow of 9.2 l/s. An example of the operating range is given below.

Figure 3.4: Example of operating range of flow limiting device



As for the A type, the flow limiter device should be installed inside an impregnable shelter with an isolating valve. The general scheme arrangement is presented in Volume Drawings.

3.2.3 Irrigation of the Farms

From the flow control hydrant located at the head of the block, water is delivered to the 6 farms of the standard block by a buried PVC pipes network. Farms are watered by one hydrant located at the head of each field. When surface irrigation is practiced, only one hydrant is open per standard block and 3 or 6 days rotation is practiced during the peak water requirements period. The same hydrant is installed at all farms.

In case of different farms surfaces, the watering time is adjusted to obtain the project ratio of 60-65 m³/ha/ watering. The hydrant can be opened using a suited elbow key. In order to simplify water management procedures, only one key is allotted per standard block. Consequently, only one farm can be watered in the same standard block, following the water rotation program. Farm hydrants are protected by a concrete structure, which incorporates the stilling basin. The stilling basin is also used as pressure head breaker.

3.2.4 On-Farm Irrigation Equipment

3.2.4.1 Surface Irrigation Equipment in favourable conditions

In case of very favourable topographic conditions and very good irrigation practices by farmers, furrows can be constructed directly from the stilling basin, and specific equipment is not necessary. Nevertheless in order to maximise water efficiency, gated pipes systems are particularly advised for this project when surface irrigation is practiced. Flume hose pipes with adjustable gate outlets are available on the world market (USA, Australia, EU, Iran…). A local solution is possible, using local PE black pipe, as designed here below.

Figure 3.5: Schematic lay-out of flow-control hydrant, buried pipes and individual hydrants

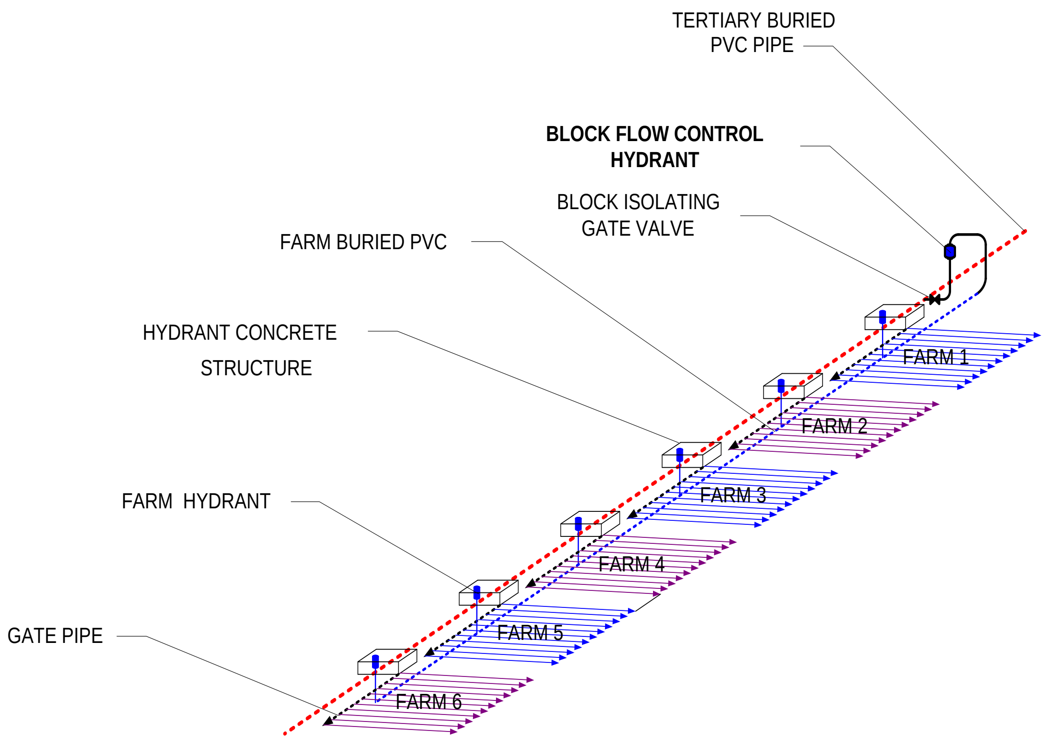
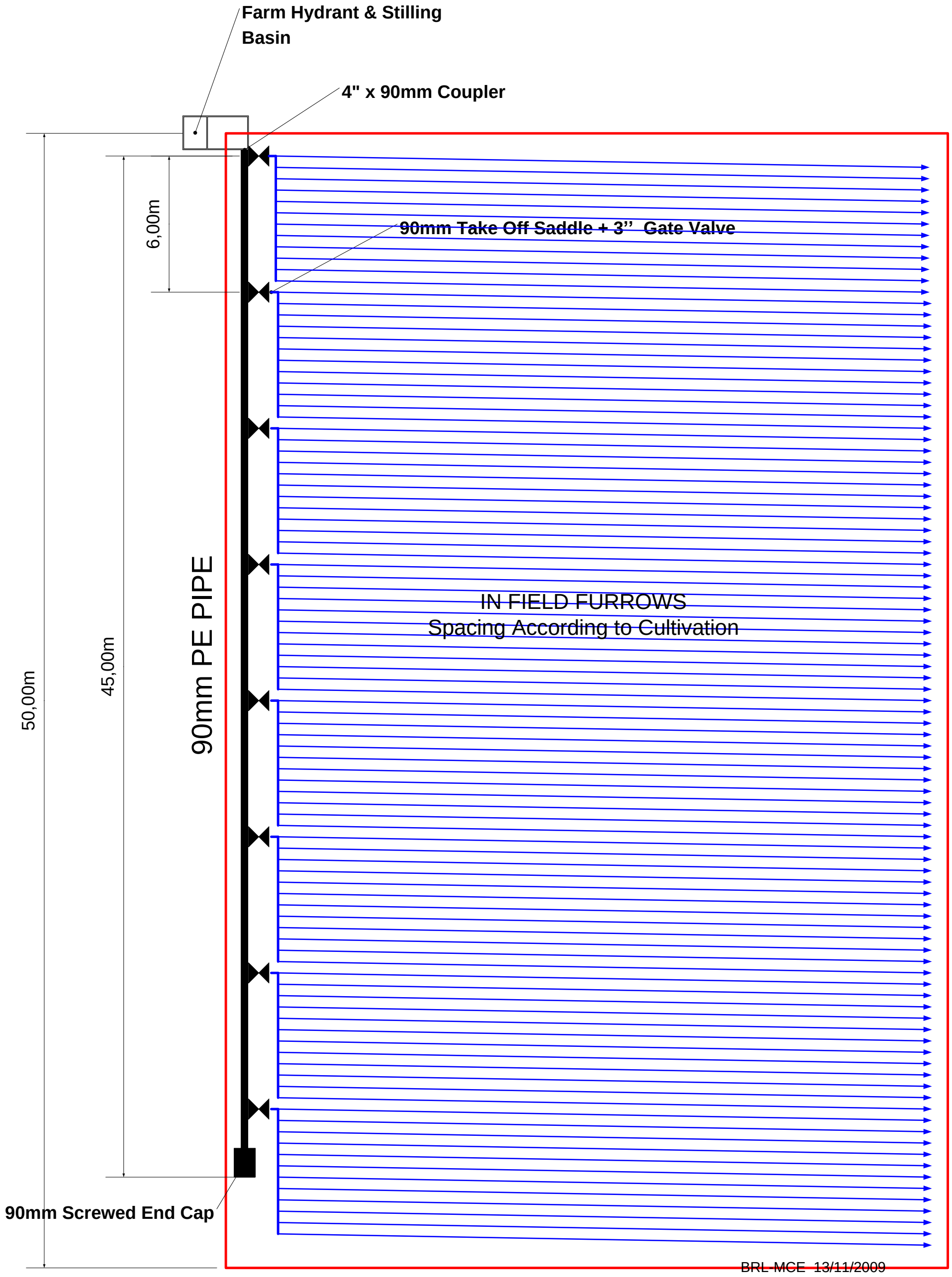


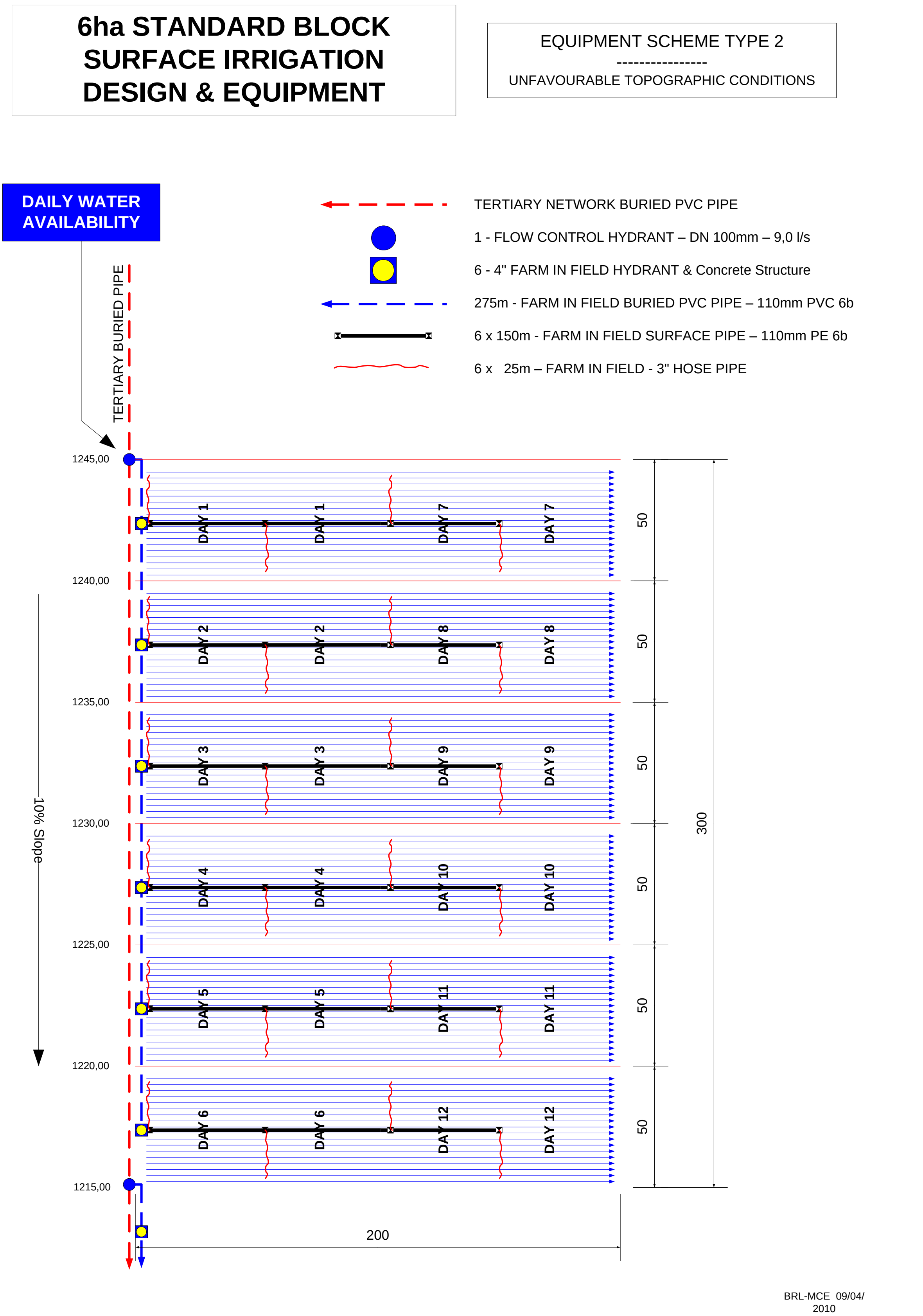
Figure 3.6: Lay-out on a one ha farm using 90 mm PE gated pipe



3.2.4.2 Surface Irrigation Equipment in unfavourable conditions

In case of unfavourable conditions, important slopes (>10%), soils with high permeability and presence of numerous gullies, the furrow length must be reduced and therefore special arrangements have to be adopted as shown in Figure 3.7

Figure 3.7: Lay-out of block with 6 farms and rotation under difficult topographic conditions



3.2.4.3 Sprinkler Irrigation

When pressure is available at the head of the block, (P>2.5 bar), sprinkler irrigation can be used with the following advantages :

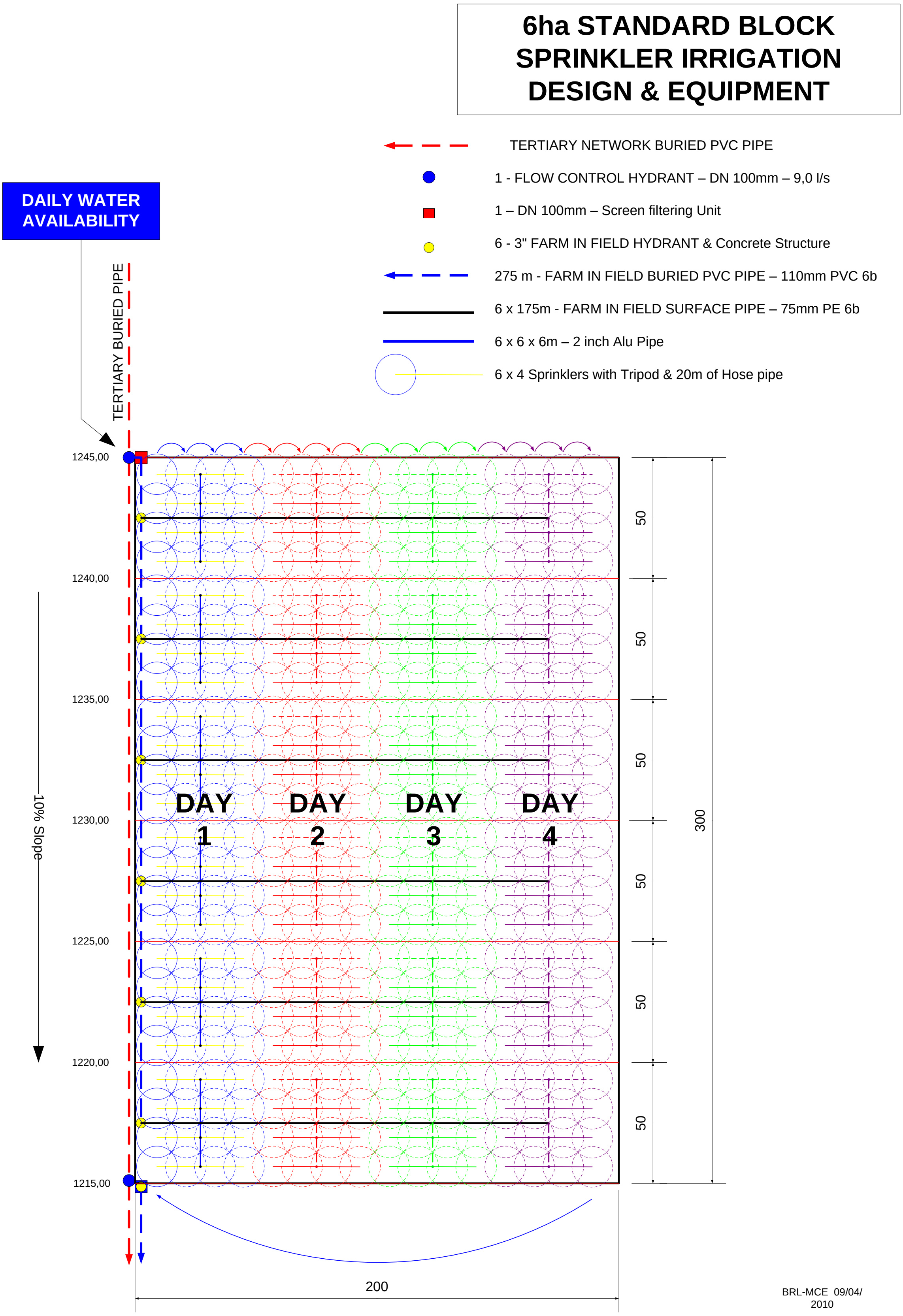
- Improvement of water efficiency
- Decrease of labour requirements for irrigation
- Decrease in earthworks

Drag line systems with small spacing is very suitable for small farms and different crops under difficult topographic conditions. Cost is probably the only limiting factor. For a good and very profitable practice, setting up of such equipment requires a perfect harmony with the farmers of a block. A screen filtering unit is installed at the head of the block, downstream of the flow control device. Block farmers are responsible for cleaning the filter when necessary. The total block discharge (9.6 l/s) is distributed to the farms in the block. The discharge to each farm is limited by the number of sprinklers. Surface pipe networks are connected directly to the farm hydrant. When sprinkler irrigation is working on one farm of a block, all the farms of the same block have to irrigate with the same system.

The main characteristics of the standard block are the following:

- 6 Farms of 1 ha. 6 x 200m x 50m
- Farm allowed discharge 1.5l/s
- Sprinkler mesh spacing 12m x 12m
- Number of sprinklers per farm 4
- Number of Lateral positions 4
- Number of Sprinkler positions 16
- Sprinkler discharge 1.35m³/h ±10%
- Sprinkler hose length 20m
- Sprinkler hose Ø 19mm
- Sprinkler maxi pressure range 2.0b < P < 3.0b
- Main Line Pipe PET pipe 75mm OD
- Alu Lateral Pipe Ø 2 inch
- Standard Irrigation depth 28mm
- Corresponding irrigation time 3 hours
- Corresponding rainfall 9.3mm/h

Figure 3.8: Lay-out of farm with sprinkler irrigation equipment



3.2.4.4 Localized Irrigation

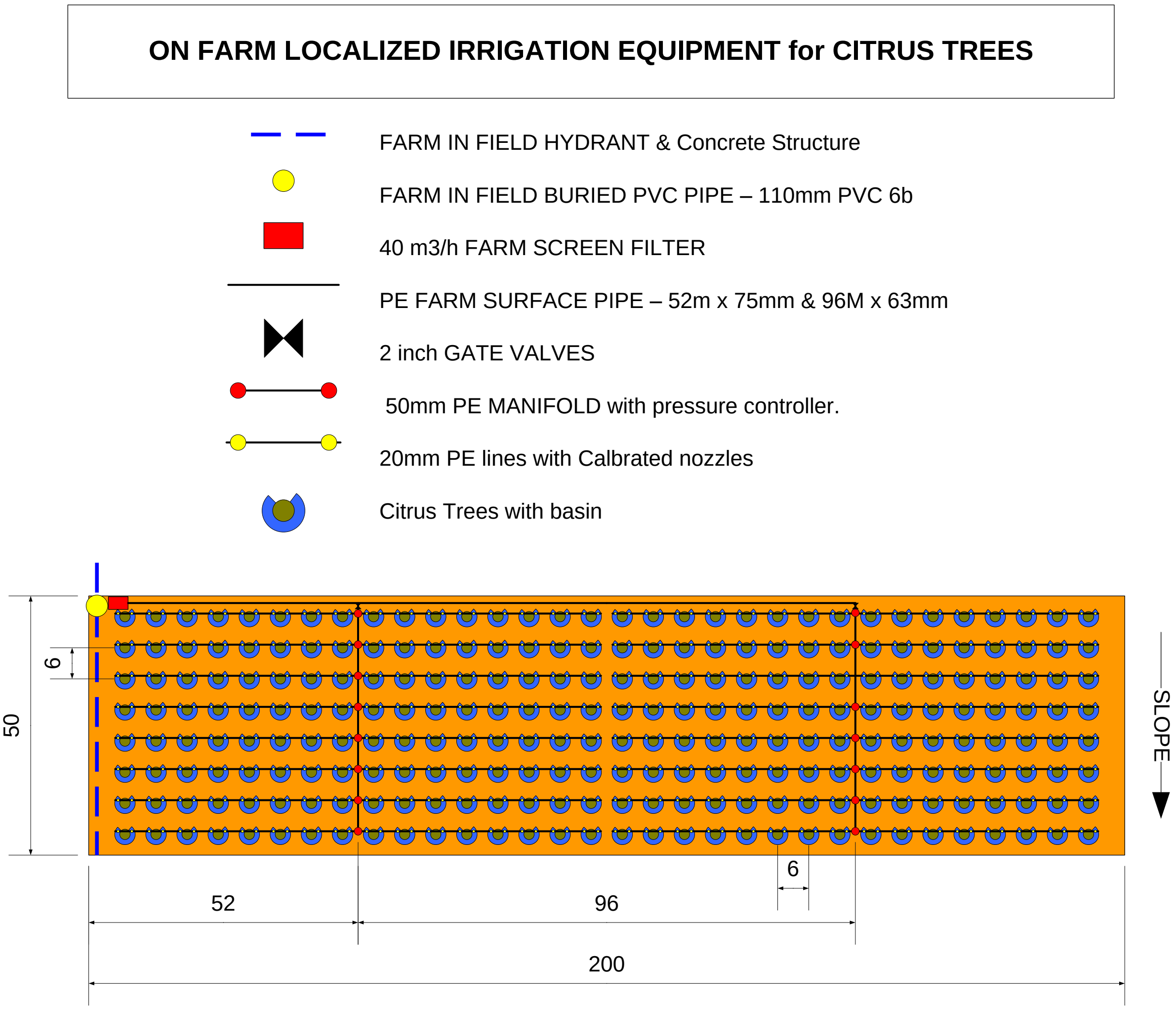
Localized irrigation systems as drip, micro sprinklers, etc. are well suited to fruit trees cultivation. Furthermore these systems require a good technological level mainly for operation and maintenance of the filtration equipment. The only possible system that could be developed is localised irrigation using calibrated nozzles mounted on PE pipelines, a quite simple irrigation system. Basins are constructed and levelled around the trees. Pipes are laid along fruit tree rows and calibrated nozzles are fixed to the PE pipe. For citrus trees like orange, lemon, grapefruit trees… with 6m x 6m spacing, 2 nozzles are required to provide water to each basin. The size of the nozzles is adjusted in order to reach a constant 60 l/h flow ±10% for all the emitters. The total discharge for a complete 1 ha farm planted with 256 citrus trees is 31 m³/h per hydrant controlled discharge.

For adult trees, the watering time for 5mm of daily peak requirement is:

6 x 6 x 5 / 60 / 2 = 1.5 hours

As designed here above, the farm irrigation equipment scheme can be easily inserted in one standard block, without disturbing the overall water management of the block.

Figure 3.9: Layout of 1 ha citrus farm with localised irrigation



3.2.5 Drainage

Apart from important earthworks requirements and hard labour for irrigation by furrows, surface irrigation requires also the digging of a ditches network in order to drain the possible overflow at the end of the furrows. Spoon drains located downstream of the furrows are generally dug by the farmers when tertiary and secondary drains are dug by contractor and maintained by the organization responsible for project management. Nevertheless and due to the particular site geomorphology, with sufficient groundslopes and a dense network of existing streams will be used as drainage network. Consequently drainage works have not been considered for the present project.

4. SECONDARY NETWORK DESIGN

From the primary network, including the canal and the night storage reservoirs, water is conveyed to the head of each standard block by a buried pipe network. The Project command area is divided into 15 Command Areas corresponding to the main interfluves. For a small CA, or part of CA located near the primary canal, pipes are directly connected to the canal, without reservoir.

For large CAs, two networks are designed:

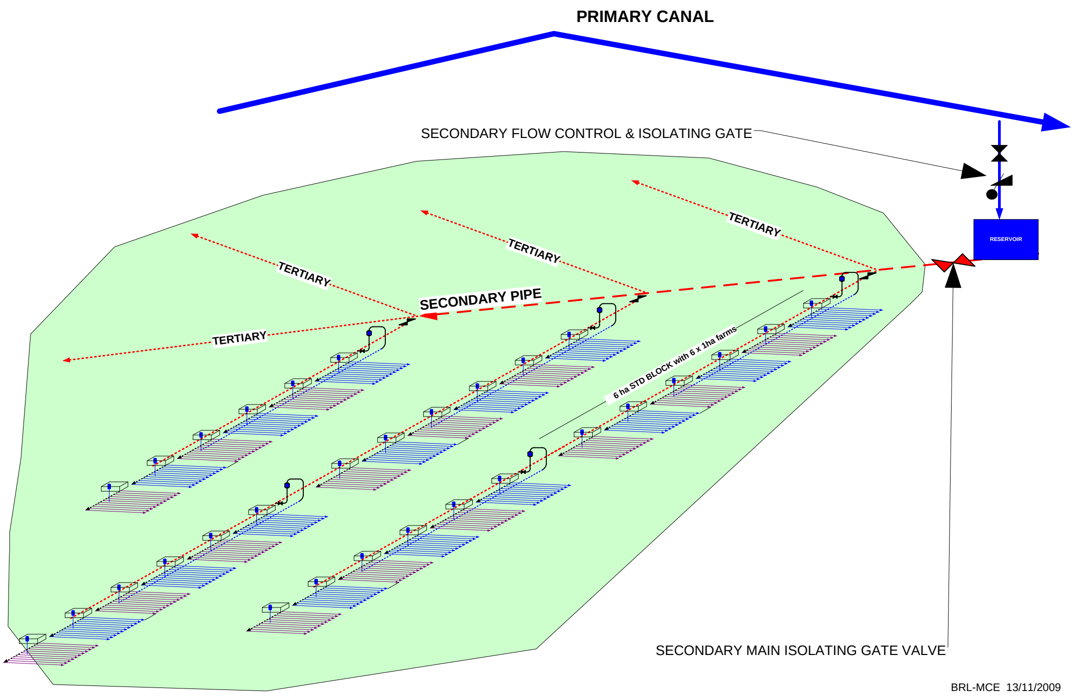
- The secondary network, located at the ridges of the CA
- The tertiary network, watering the blocks from the secondary network

Two kinds of pipes are selected for the present project:

- GRP pipes (Glass Reinforced Pipes), for all diameters ≥ 400mm
- Stiffness class SN 5000 / NWP 6 bar
- PCV pipes for all diameters < 400mm.
- NWP 6 bar

Networks are installed with the necessary fittings and air valves. Appropriate valves are installed at the head of each network branch, to allow proper maintenance and management of water distribution.

Figure 4.1: Schematic lay-out of secondary and tertiary pipe networks



4.1.1 Methodology

In order to obtain a good approach of the network sizing and cost, blocks, tertiary and secondary networks have been designed for 8 CAs, covering a total gross area of 4,357ha i.e. approximately half of the gross irrigable project area.

For each command area:

- Blocks are demarcated for an approximate 6 ha net area, taking into account the site geomorphology, slope, streams and gullies.
- Maximum land slope is around 20%, and maximum furrows length around 200m.
- Inventory of all blocks is established for all the tertiary and secondary branches and project discharges are defined considering that ALL BLOCKS are watered simultaneously with a nominal discharge of 9.6l/s/block.

4.1.2 The studied Command Areas

The main characteristics of the studied command areas are summarised in the following table.

Table 4.1: Main Characteristics of studied Command Areas

CA N°	GROSS AREA	NET AREA	RATIO	N° of HYD	TOTAL DISCHARGE	SECONDARY NETWORK	TERTIARY NETWORK
1	241.3 ha	150.1 ha	62 %	26	250 l/s	1,760 m	4,161 m
4	135.0 ha	124.4 ha	92%	21	202 l/s	2,012 m	1,770 m
6	256.6 ha	212.3 ha	83%	36	346 l/s	3,035 m	5,167 m
9	456.4 ha	377.8 ha	83%	64	614 l/s	3,630 m	8,790 m
11	642,5 ha	412.9 ha	64%	74	710 l/s	3,755 m	8,365 m
12	717.6 ha	567.7 ha	79%	98	941l/s	6,830 m	15,240 m
14	832.8 ha	537.2 ha	65%	92	883 l/s	5,564 m	16,774 m
16	1,074.9 ha	807.4 ha	75%	136	1,305 l/s	19,112 m	17,411 m
	4,357.1 ha	3,189.8 ha	73%	547	5,251 l/s	45,698 m	77,678 m

Summary Inventory Tables for each studied CA were presented in Appendix 4.1 of the Draft Report with for each sheet:

- The total left and right banks blocks areas.
- The corresponding total theoretical discharge.
- The total left and right banks number of flow control hydrants.
- The corresponding total discharges, based on a 9.6 l/s nominal flow.

4.1.3 Hydraulic Calculations

In order to select the secondary and tertiary network pipe diameters a mathematical model has been established using the HAZEN-WILLIAMS formula in Excel.

Hydrant types A & B are automatically selected following the conditions:

- Static Pressure < 0.7 — No solution
- 0.7 < Static & Dynamic Pressure < 10 — Type A Selected
- Static Pressure >7; Dynamic Pressure<7 — No solution
- Other case — Type B selected
- If no solution, diameters must be recomputed as long as solution is finding.
- In case of no solution, position of hydrant must be moved lower.
- Maximum Water velocity: 2m/s (short length excepted). Calculations sheets are given in Appendix 4.1.

Note: last column in table, "Hydrant Pressure" indicates the available pressure downstream of the flow control hydrant, 5m of particular head losses deducted. When the remaining value is > 15m, the cell is coloured in blue indicating the possibility of equipment for all the farms of the block with sprinkler or localized irrigation system. The total number of "blue colour" blocks is 160 for a total of 817, i.e. 20%.

4.2 COSTS OF EQUIPMENT

The investment costs of the equipment are presented in Annex 13: Bills of Quantities and Cost sheets.

4.3 MAINTENANCE & ORGANISATION OF WATER DISTRIBUTION

4.3.1 Maintenance

The standard ratio for maintenance costs of all equipment covering pipes and fittings in the secondary and tertiary networks is 2%. Flow control hydrants, block network and fittings are charged with a 5% maintenance ratio, taking into consideration frequent operation. Farm in field irrigation equipment is from "Surface & mobile equipment" class, for which the standard maintenance ratio is 10%.

4.3.2 Organisation of Water Distribution

The distribution of water will be organized considering the Command Areas, and the number of standard blocs watered inside each area. The control of the water distribution, the respect of its rules, and mainly hydrants opening and closing on time, water availability rotation, reporting of the network failures to the central board and the necessary maintenance works will be performed by 19-20 Water Guards (WGs).

Each WG will have a motor cycle which makes him sufficiently mobile to operate efficiently, mainly at the beginning and at the end of the irrigation day. The organisational structure within which they operate is presented in Annex 11: Organisation and Management.

As a first approach of that task, we propose to charge each WG with the control of 50-70 Standard Blocs, i.e. a total maximum surface of 300-420 ha. From the design of the standard bloc and the general arrangement of the command areas, the distance to be covered on one journey for the control of six blocs is approximately 4 x 600 + 200 = 2,600m, and the daily journey is approximately 80 km, with half in the morning and half in the evening.

The table below gives the list of the water guard requirement for each command area of the project.

Table 4.2 Water Guard Requirement

CA N°	Gross S	Net S	N° WG
CA1	241 ha	150 ha	0.5
CA3	339 ha	249 ha	1
CA4	135 ha	124 ha	0.5
CA5	98 ha	72 ha	0.5
CA6	257 ha	212 ha	0.5
CA7	768 ha	563 ha	2
CA8	110 ha	81 ha	1
CA9	456 ha	378 ha	1.5
CA10	592 ha	434 ha	1.5
CA11	642 ha	418 ha	1.5
CA12	718 ha	568 ha	2
CA13	1,118 ha	820 ha	3
CA14	833 ha	537 ha	2
CA15	433 ha	317 ha	1.5
CA16	1,075 ha	807 ha	2
	7,815 ha	5,730 ha	19

This WG staff will be supervised by Water Guard Supervisors (WGS), with 1 supervisor for 10 WG, equivalent to 2 WGS for the whole project. The WGS are responsible for compiling the daily reports, calculating volumes, and following up on the maintenance operations. At the head, the Water Exploitation Deputy Manager is in charge of the whole staff in front of the General Manager, elected representative of the WUA

APPENDICES

Appendix 1.1: Calculation of Additional Costs of Option Primary Canal with Bedslope S = 0.0003

Calculation of additional costs of alternative S main canal = 0.0003								
	Canal section reduction							
					Surfacing			
	Canal	depth	Length		Area			
	Reference	reduction	m		m^2			
	C1	0.17	27,418	0.62	16,904			
	C6	0.13	4,274	0.47	2,003			
	C7	0.12	2,622	0.43	1,125			
	C9	0.10	2,134	0.36	769			
	C10	0.09	5,404	0.31	1,656			
	C11	0.06	2,870	0.22	621			
	C12	0.07	16,321	0.26	4,296	ETB/m^2		
					27,375	250	**6,843,650**	**ETB**
	Pumping							
	Additional head		**2.75**	m				
	Loss of Command							
						10		
	Canal	Start	End	Length		m fall in		
	Primary	1.68	2.74	11,721	80	103	26.6	
	NW+SC041	0.00	1.47	15,808	300	385	44.7	
	SC050	1.16	1.53	4,492	120	154	9.3	
	SW	0.00	0.72	10,160	150	192	7.0	
	SC020	0.00	0.15	1,549	150	192	0.2	
	SE	0.00	0.72	7,184	150	192	5.0	
							93	ha
		NPV production forgone per ha					218,000	ETB
		NPV production foregone for 93 ha					**20,219,620**	**ETB**
Pump efficiency	Water requirement	Average flow	Total head	Power	Annual consumption	EEPCO tarif	Annual cost ETB	
%	m^3/ha/y	m^3/s	m	kW	kWh/year	ETB/kWh		
80%	6,404	0.000203	2.75	0.007	60	0.5	30	
NPV	$A=(1-(1+r)^{-n})/r$							
r = discount rate		10%						
n = term		25	*years*					
	A =	9.08						
				ha				
NPV additional power =		272	ETB/ha	6,500	**1,769,167**	**ETB**		
				NPV Additional power	1,769,000			
				NPV Production forgone	20,220,000			
				Total additional cost	21,989,000			
			Canal saving		6,844,000			
			Net additional cost		**15,145,000**	**ETB**		

Appendix 4.1: Example of Calculation Sheets Secondary Networks