REPUBLIC OF BOTSWANA

Department of Crop Production (DCP)
Ministry of Agriculture

CONSULTANCY SERVICES FOR THE FEASIBILITY STUDY FOR THE ZAMBEZI INTEGRATED AGRO-COMMERCIAL DEVELOPMENT PROJECT

CONCEPTUAL DESIGN REPORT – FINAL REPORT

REVISION				
DESCRIPTION	**DATE** YY mm DD	**BY**	**APPROVED**	**NOTES**
FINAL VERSION	2014 December 19	CP/FA	AG	
AMENDED VERSION (after comments)	2015 March 31	CP/FA	AG	

SGI Studio Galli Ingegneria S.p.A *in Joint Venture with*

Metaferia Consulting Engineers PLC

SGI life, first STUDIO GALLI INGEGNERIA

MCE

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LIST OF ACRONYMS AND ABBREVIATIONS

ARF	Areal Reduction Factor
a.s.l.	above sea level
avg	average
Consultant	Consortium between SGI and MCE
COPAM	Combined Optimization and Performance Analysis Model
CWR	Crop Water Requirements
DEA	Department of Environmental Affairs
DI	Ductile iron
DMS	Department of Meteorological Services
DEM	Digital Elevation Model
dS/m	deci-Siemens per meter
EC	Electrical conductivity
ETo	reference evapotranspiration
FAO	Food and Agriculture Organization
GRP	Glassfibre Reinforced Plastic
ha	Hectare
HC	Hydraulic conductivity
IDF	Intensity–Duration–Frequency
IR	Irrigation Requirements
m.a.s.l	Meters above sea level
MCE	Metaferia Consulting Engineers
MOA	Ministry of Agriculture
NWMPR	National Water Master Plan Review
NOAA	National Oceanic and Atmospheric Administration
pH	Hydrogen potential (measurement of solution acid or alkaline level)
POC	Point of Connection
PVC	Polyvinyl chloride
RUSLE	Revised Universal Soil Loss Equation

SFRM	Several Flow Regimes Models
SGI	Studio Galli Ingegneria SpA
Tc	time of concentration
Tr	Time Return
USLE	Universal Soil Loss Equation
WTS	Water Transfer Scheme
ZIACDP	Zambezi Integrated Agro-Commercial Development Project

0 SUMMARY

This Conceptual Model Report is part of the feasibility study of the Zambezi Integrated Agro-Commercial Development Project (ZIACDP) and has been prepared by the Consultant, Studio Galli Ingegneria S.p.A. (SGI) and Metaferia Consulting Engineers PLC (MCE) after the contract's signature with the Ministry of Agriculture (MoA) of Botswana, in January 2014.

The inception phase has been completed by the middle of March carrying out desk documentation review, kick-off meeting, field visit, data gathering and harmonization, then planning of survey activities and deliverables.

By the 10th of July, the Field Investigation Report has been delivered with results of topographical, geotechnical and soil survey. The current project phase envisages the redaction of the present Conceptual Model Report but also of Agricultural Commercial Business Plan, Financial model, ESIA and EMP. Next deliverables will be the Final Report and Bank able Feasibility Study.

For what concerns the present project (general issues are given in paragraph 1), on the basis of climate regime (paragraph 2), hydrogeology (paragraph 3), soil survey and land suitability maps (see Field Investigation Report) and taking into account market and value added opportunities, the agronomist has identified suitable crops for irrigated production and the production technology required (see Agricultural Commercial Business Plan).

The type of irrigation system is determined according to both irrigation and hydraulic issues (paragraph 4). Once the best alternative has been selected, the conceptual design for the water distribution and irrigation system has been prepared (paragraph 5): this network is on demand pressurized pipelines. Finally the project for access roads and storm water drainage system has been carried out (paragraph 6).

1 INTRODUCTION

1.1 GENERAL ISSUES

An irrigation system is designed to use the available water as efficiently as possible by minimizing the losses in conveyance, distribution and application. The irrigation system consists of the following two sub-systems:

- Agricultural sub-system comprising the cultivated fields with different types of crops, farming system, and agricultural practices including the application of irrigation water and land husbandry.
- Engineering sub-system comprising various structures for storage and diversion of water and pipe networks for water conveyance and distribution.

The ZIACD project has been identified to provide irrigation infrastructure to farmers and entrepreneurs in a large area near the existing Pandamatenga Commercial Farms. To achieve the above objective, the Ministry of Agriculture (MoA) that is in charge of the Project, intends to divert water from the Chobe/Zambezi River for irrigation as well as for domestic use.

The selected area of about 45,000ha is located on the western part of the existing Pandamatenga Commercial Farms at about 110 km South of Kazungulu, in the Northeast of the country.

In line with the aim of improving the country’s food security and livelihood of the rural population, diversify agriculture, contribution to the country’s GDP and creation of employment opportunity through the strategy of development of irrigated agriculture, MoA has initiated the development of large-scale irrigation through the investigation and development of surface water potential of the country. Accordingly, ZIACD Irrigation Project has been planned for implementation using high technology of pressurised irrigation system.

The present Feasibility Study focused on data collection, analysis and preliminary design. The conceptual design of the project is finalized based on the outputs of the feasibility study. The feasibility and conceptual design reports address different sectoral components required for the scheme development and include climate and hydrology, topography, soils, agronomy, livestock, socio-economy, agricultural marketing, value chain, hydraulics and irrigation engineering and economic and financial analysis.

1.2 LAND USE AND LAND COVER

With the exception of the forest and wild life reserve areas, there are no defined land use activities in the project area. However, adjacent to the project area, there is an established commercial farms, Pandamatenga Commercial Farms, that grow sorghum, beans, sunflower and other crops during the rainy period of October/November to February/March. The total area of these farms is about 25,000 ha.

The project area is predominately characterized by bushy/shrub grassland covers but the intensity varies from place to place. In areas where mopane (Coloophosepermum mopane) with scattered big trees like mukuse and moshweshew exist, the area can be described as dense shrub land.

Mukuse and moshweshew are trees with evergreen characteristic that never dry as other vegetations do. Mopane is one of the most typical tree and shrub species found broadly in the project area. It often occurs in silty-sandy soils but it also grows on a large variety of soils ranging from sandy to clayey textures.

1.3 CLIMATE

The climate in the project area is semi-arid characterized by summer rainfalls. Maximum temperatures range between 26°C and 34°C and are experienced between October to July. Minimum temperatures range between 11°C to 20°C occurring between November and July.

Rainfall is highly variable and the annual average is about 538 mm. Most of the rain falls between October and April, with December, January and February being the peak months. The whole year can be subdivided into four seasons including:

- Dry winter season (May to August);
- Rainy summer season (November to March);
- Spring (September to October);
- Autumn (April to May)

The soil climate of the area is characterized by aquatic moisture and isohypertermic temperature regimes (SMSS, 1987 technical monograph No.6). An aquatic moisture regime occurs in poorly drained parts of the lacustrine areas (Soil Mapping and Advisory Services, Gaborone, 1990).

1.4 CURRENT STATE OF IRRIGATION

Presently, irrigation's contribution to overall agricultural production in the country is insignificant. Nevertheless, there is a need to expand production to feed the population and to ensure food security. Raising livestock has long been one of the most important agricultural activities in Botswana. Sheep and goats are said to adapt to the drought condition of the country better than cattle do. Cattle are mostly raised for beef. Dairy and the likes are very limited.

1.5 OBJECTIVES OF THE STUDY

As outlined in the terms of reference (ToR) for the ZIACD Project, the primary objective of the planned intervention is to "establish a viable commercial agricultural development, which will improve Botswana's food security, diversify agriculture, meaningfully contribute to the country's GDP and create direct employment for over 4,000 people. It is also anticipated that the project will create opportunities for Batswana to be involved directly and indirectly as entrepreneurs, therefore increasing the impact of the investment for the country."

Therefore, the principal objective of the infrastructure component of the project is to select the most suitable option - taking into account the criterion of viability and suitability for local conditions - and prepare a corresponding conceptual design and implementation plan. This implies to:

- Carry out an overall final feasibility study and develop bankable business plan;
- Analyze and recommend the best financing options for the project;
- Make recommendations to the size and type of agricultural operations; and
- Conceptual design of the agricultural project utilizing all the available water, which would be delivered to a regulating reservoir at site.

2 CLIMATE AND HYDROLOGY

2.1 GENERAL FRAMEWORK

Geographical location of Botswana and its physiography determine a climate that is arid to semi-arid. In fact, the country lies between Latitudes 18° S and 27° S and Longitudes 20° E and 29° E, besides it is completely landlocked.

The country is largely flat and surrounded by higher plateaus of Zambia to the north, Zimbabwe to the northeast, South Africa to the southeast and south and Namibia to the west, giving it a ‘saucer-like’ physiography. As a result of this, there are no prominent barriers to the flow of moist air and orographic influences on the formation of clouds and precipitation are virtually non-existent.

Briefly the major climatic controls that determine Botswana’s water resources are the rainfall, temperature and evaporation. Over 90% of the rainfall occurs in the summer months and, sometimes, 70% to 90% of the annual total rainfall may occur in only one month. Rainfall tends to occur in wet spells lasting several days at a time: these periods are interspersed with lengthy dry spells. Storm rainfall intensities are usually high but the duration of the storms are short. Rainfall incidence is highly variable both spatially and temporally.

Generally there are high day-time temperatures and high evaporation rates throughout the year. Potential evapotranspiration rates exceed the rainfall total at all times of the year except when extremely heavy storms occur.

2.2 DATA SOURCES AND TYPE

Most of the meteorological measurements has been found in the “National Water Master Plan Review” (NWMPR), Volume 3 “Surface Water Resources”, redacted by Department of Water Affairs (DWA) of Ministry Of Minerals, Energy & Water Resources (March 2006). This data are mainly provided by Department of Meteorological Services (DMS) and already included in the Botswana National Atlas (2003).

The DMS provides data from several stations spread all over the country and generally maintained at schools, police stations and other similar institutions. Often length of time series varies considerably and some stations began recording in the 1920s, even if DMS indicates as reliable data covering the period 1971 - 2000.

Further meteorological information from DMS has been gathered during the present project to get an almost comprehensive dataset for hydrological study of Pandamatenga site. In particular, these measurements are available at 2 sites in Pandamatenga: the Police Meteorological Station (Latitude S 18°33’, Longitude E 25°38’), that is operating since 1961, and Pandamatenga Meteorological Station (Latitude 18°32’, Longitude 25°39’), that is working in the last 15 years.

In some cases needed data are not available at the abovementioned stations therefore hydrological analysis has taken into account measurement recorded somewhere else, as fully described and justified in the next paragraphs.

2.3 DATA ASSESSMENT FOR AGRONOMIC STUDY

The meteorological data, that has been necessary to gather for the present study, consists of: daily rainfall, mean daily maximum and minimum temperature, relative humidity, sunshine hours, mean monthly wind speeds at 2 m and 10 m and mean monthly pan evaporation.

The time series of daily rainfall data measured at Pandamatenga Police Station is quite long (from 1961 to 2007) and almost complete, in fact there are 39 entire years and 3 years with more than 10 month of recording. These 42 years have been considered in the following numerical elaborations.

The annual rainfall within the study area is around 550 mm ranging in the last 40 years from at least 300 mm to around 800 mm in wet years (Figure 1). On average monthly precipitation is almost null from May to September, then the rainfall progressively increases to the maximum value of 136 mm in January; finally mean monthly precipitation decreases with almost the same previous growing trend (Figure 2). There are about 220 rainy days that are distributed according to the graph quoted in Figure 3.

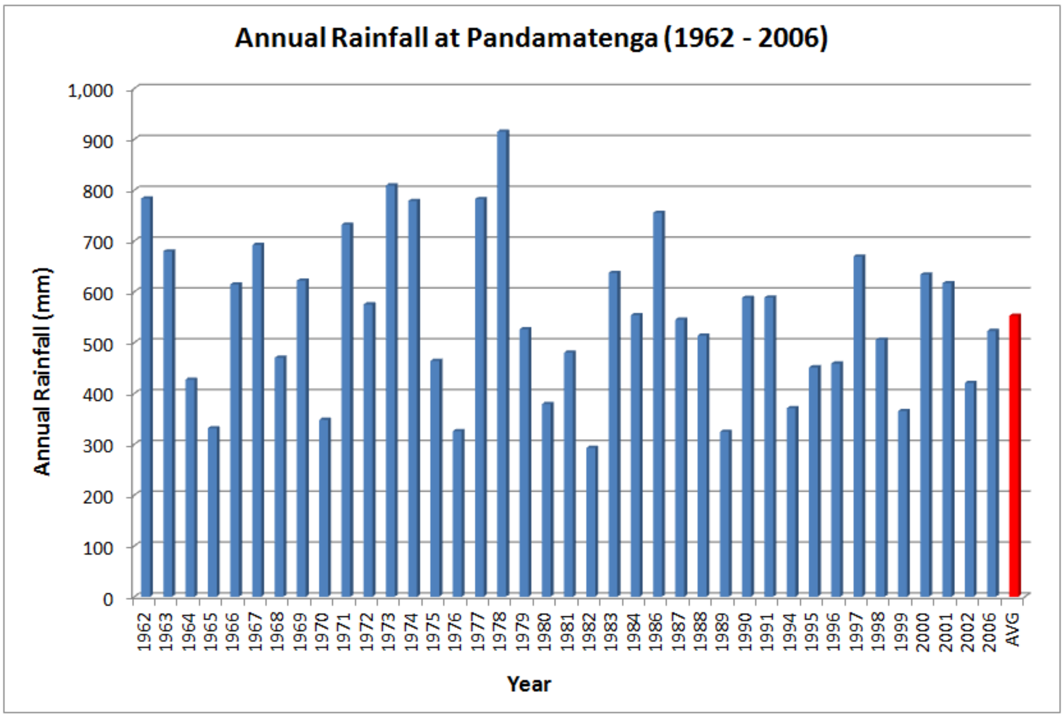


Figure 1. Annual Rainfall at Pandamatenga (Police Station, 1962 – 2006, DMS)

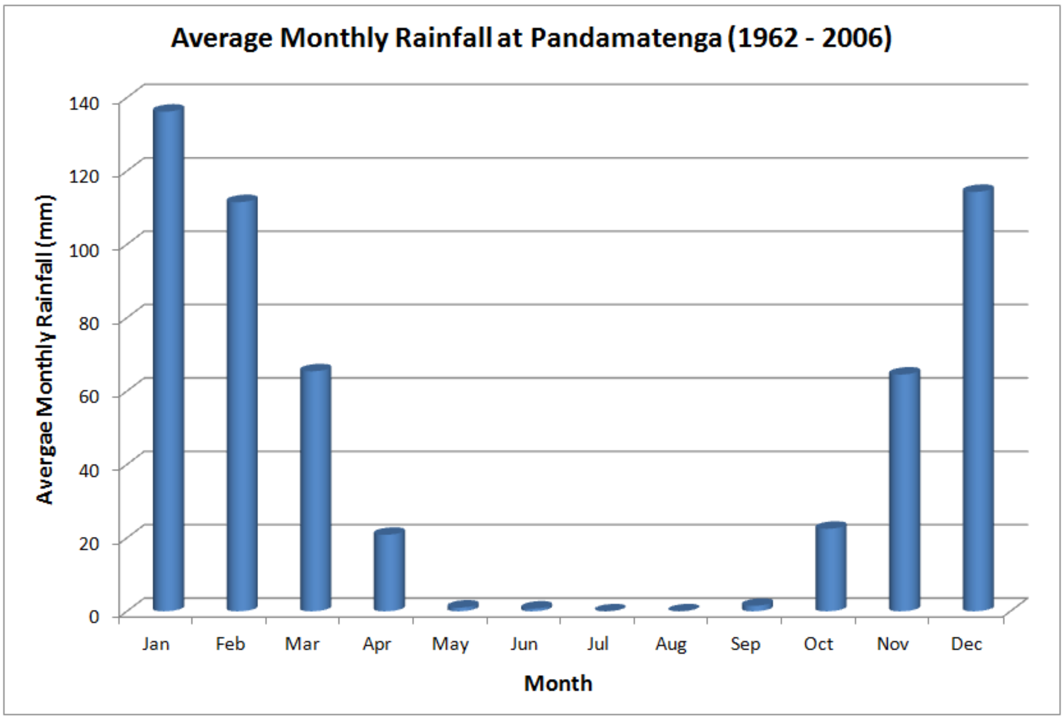


Figure 2. Average Monthly Rainfall at Pandamatenga (Police Station, 1962 – 2006, DMS)

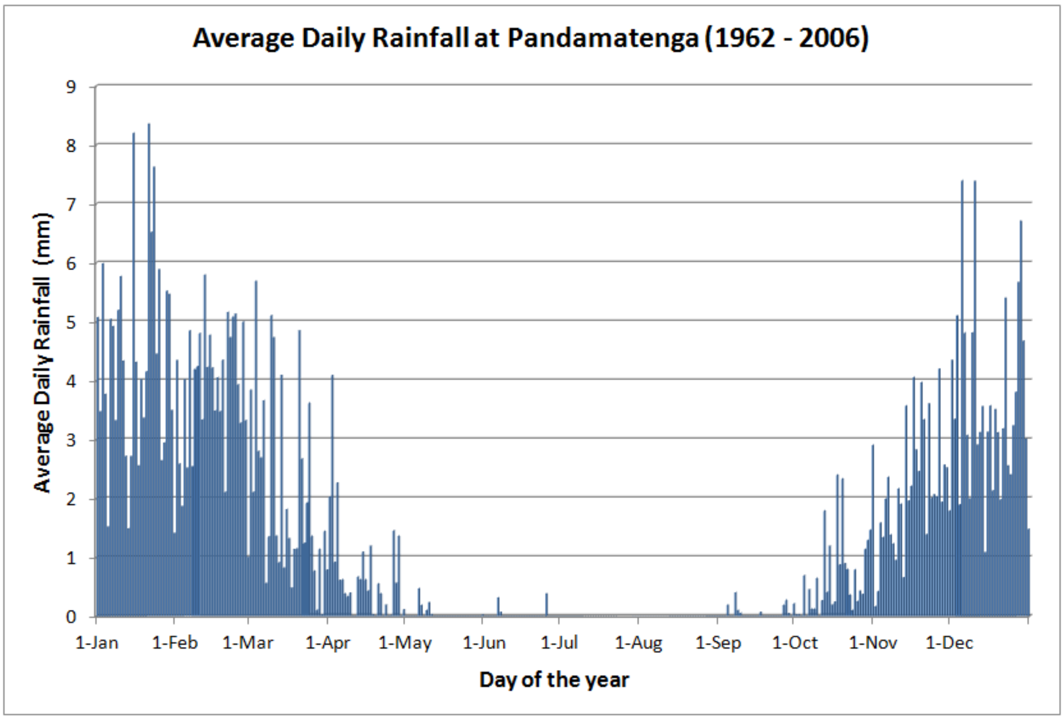


Figure 3. Average Daily Rainfall at Pandamatenga (Police Station, 1962 – 2006, DMS)

The mean annual temperature in the study area is 22.6 °C according to the measurements at Pandamatenga Meteorological Station. Maximum monthly temperature is in October with more than 34 °C while the lower values are in June and July (about 25 °C). The monthly average of minimum temperature is about 8 °C in July and it reaches 19 °C between October and February (Figure 4).

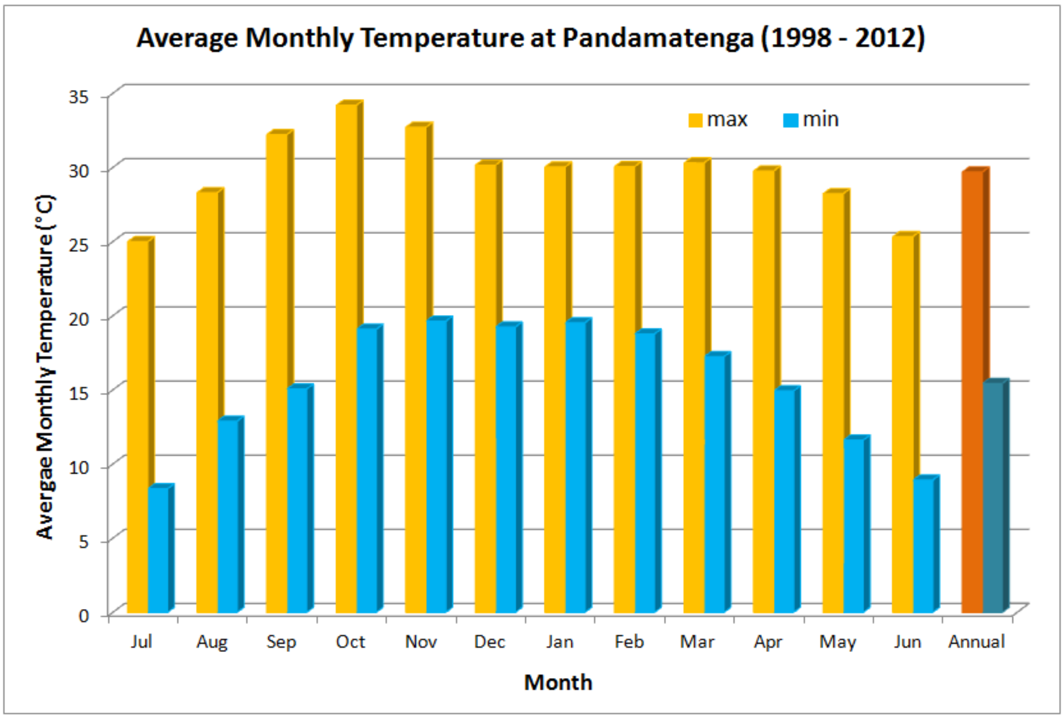


Figure 4. Temperature at Pandamatenga (Meteorological Station, 1998 – 2012, DMS)

The time series of sunshine hours, that have been recorded at Pandamatenga Meteorological Station between 1998 and 2011, show the lowest value in December and January (about 6 hours) and highest in August (10 hours), as represented in Figure 5.

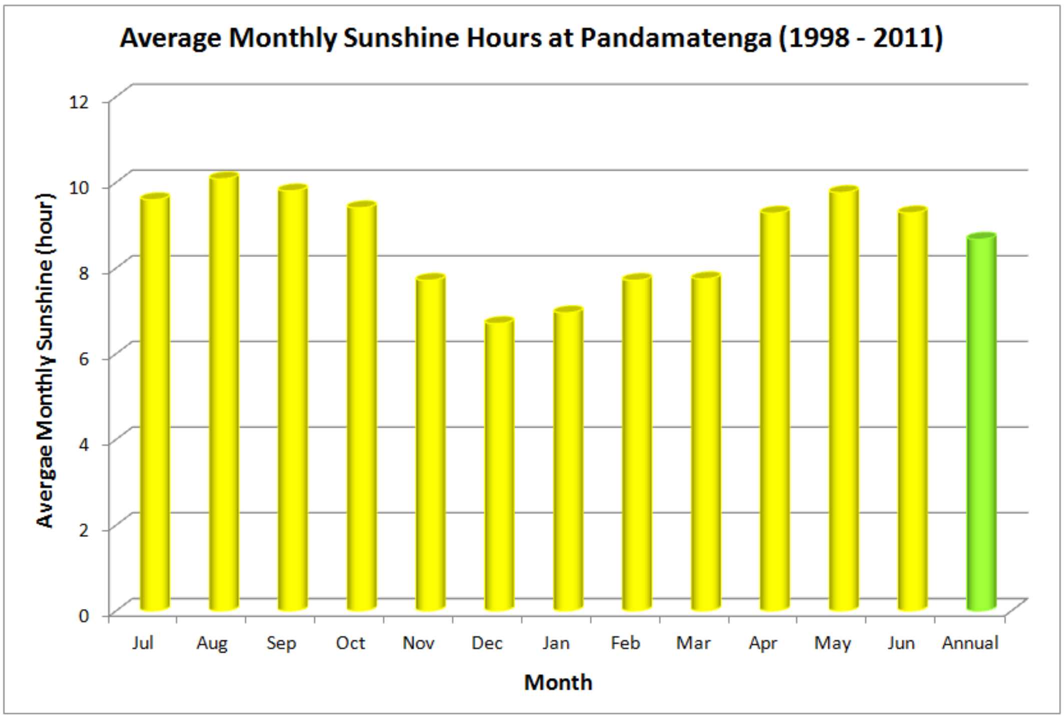


Figure 5. Sunshine hours at Pandamatenga (Meteorological Station, 1998 – 2011, DMS)

For what concerns relative humidity and wind speed, data are not available in Pandamatenga therefore the analysis has made reference to the closest station and verifying that the general meteorological condition can reliably be the same of the study area. These figures and the evaluation of hydrological analogy have been based upon what is redacted within the NWMPR.

In case of relative humidity, there are not relevant differences among the stations in the northern part of Botswana (Maun, Kasane, Shakawe): in fact, the NWMPR includes these station within an unique region when statistical process for estimating missing climatic data needs to be applied (for instance, multiple regression of rainfall data).

For the present study data recorded at Maun has been taken into account (Figure 6). The same monthly trend is shown for measurement at 8 am and 2 pm: during the morning the parameter ranges from 40% in September to 77% in February, while in the afternoon values vary from 20% to 45%.

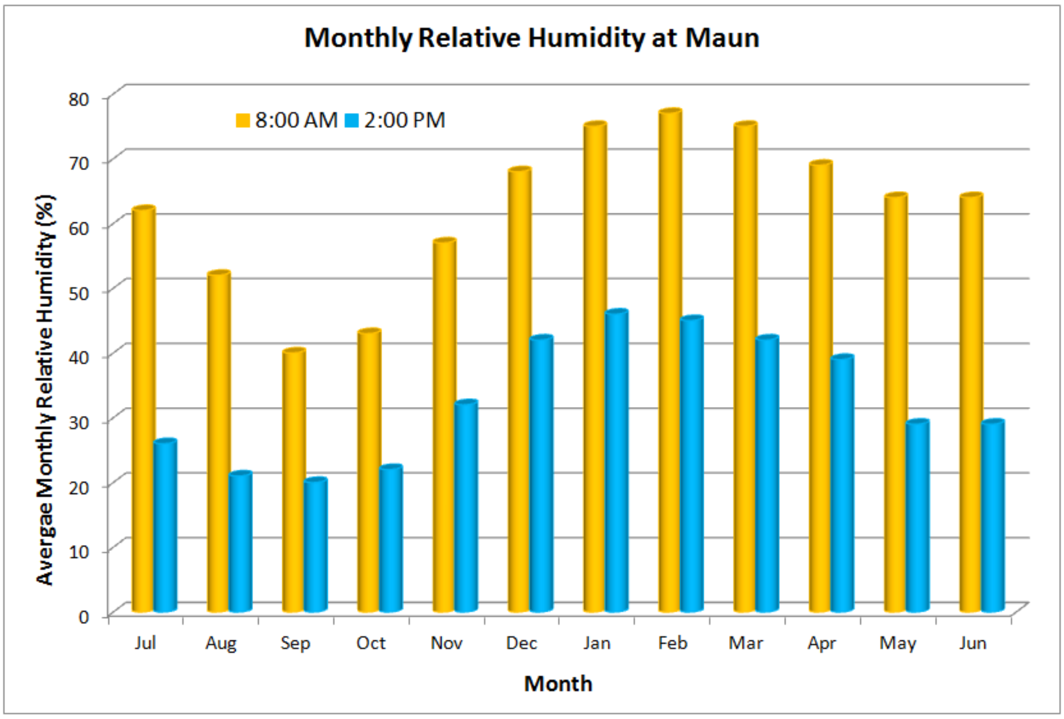


Figure 6. Relative humidity at Maun (Airport Station, NWMPR / Botswana National Atlas)

Even information about the wind speed are not available at Pandamatenga therefore the measurement at Kasane Airport are taken into account: this because, as already mentioned, Kasane belongs to the same statistical cluster of the study area and it is the closest site.

However, in Kasane the velocity is recorded only at 10 m from the ground thus the relationship between wind speed at 10 and 2 m has been analyzed for all other stations (Gaborone, Mahalapye, Francistown, Maun, Shakawe, Ghanzi, Tshane, Tsabong). From this it comes out that the ratio between velocities at two different height is constantly around 63 – 64%: thus on the basis of this the wind speed at 2 m in Kasane has been estimated.

As it can been noted in Figure 7, wind speed is minimum in January (1.3 m/s at 10 m, 0.8 at 2 m) and reaches the highest value in September (respectively 2.5 and 1.6).

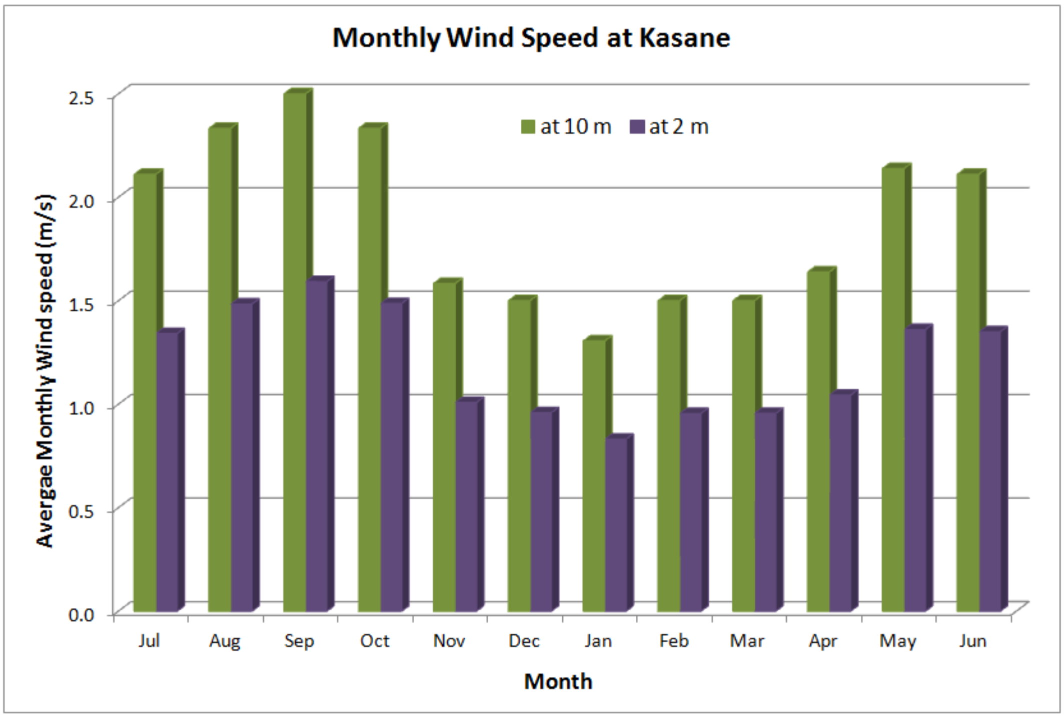


Figure 7. Wind speed at Kasane (Airport Station, NWMPR and estimation)

In Pandamatenga evaporation from pan has been recorder daily since 1997 and the resulting monthly average are shown on Figure 8: measured evaporation ranges between 125 and 170 mm from December to July, then gets higher value having its maximum in October (about 300 mm). The annual cumulative value is about 2,150 mm.

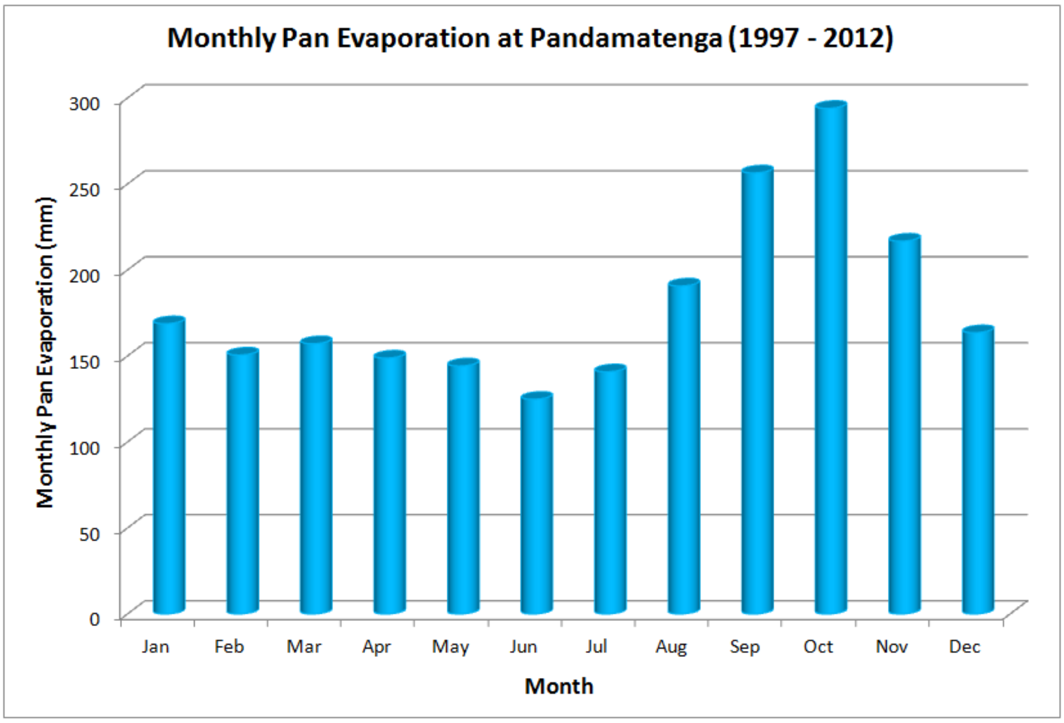


Figure 8. Pan evaporation at Pandamatenga (Meteorological Station, 1997 – 2012, DMS)

The following table summarizes the mean monthly values of meteorological parameters that have been taken into account for agronomic study. Annual values are calculated as monthly average except for precipitation and evaporation, where cumulate amount has been computed.

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Rainfall (mm)	136	112	65	21	1.0	0.8	0.0	0.0	1.5	23	65	114	539
Temperature max (°C)	30.0	30.1	30.3	29.8	28.2	25.3	25.0	28.3	32.2	34.2	32.7	30.2	29.7
Temperature min (°C)	19.5	18.8	17.2	14.9	11.6	8.9	8.3	12.9	15.1	19.1	19.6	19.2	15.4
Temperature avg (°C)	24.8	24.4	23.8	22.3	19.9	17.1	16.7	20.6	23.6	26.6	26.2	24.7	22.6
Sunshine (hour)	7.0	7.7	7.7	9.3	9.8	9.3	9.6	10.1	9.8	9.4	7.7	6.7	8.7
Relative humidity (%) at 8 am	75	77	75	69	64	64	62	52	40	43	57	68	-
Relative humidity (%) at 2 pm	46	45	42	39	29	29	26	21	20	22	32	42	-
Wind speed (m/s) at 10 m	1.3	1.5	1.5	1.6	2.1	2.1	2.1	2.3	2.5	2.3	1.6	1.5	-
Wind speed (m/s) at 2 m	0.8	1.0	1.0	1.0	1.4	1.3	1.3	1.5	1.6	1.5	1.0	1.0	-
Pan evaporation (mm)	169	151	157	149	144	125	141	191	256	294	217	164	2,157

Table 1 Summary of meteorological parameters for agronomic study

2.4 STATISTICAL ANALYSIS OF RAINFALL

The time series of daily rainfall data measured at Pandamatenga Police Station has been considered to identify the maximum height for each year (42 years with complete data between 1962 and 2006). From Figure 9 it can be noted the lower values are around 40 mm; maximum daily precipitation is less than 60 mm for about 50% of the years, while is more than 80 mm for a quarter of considered years.

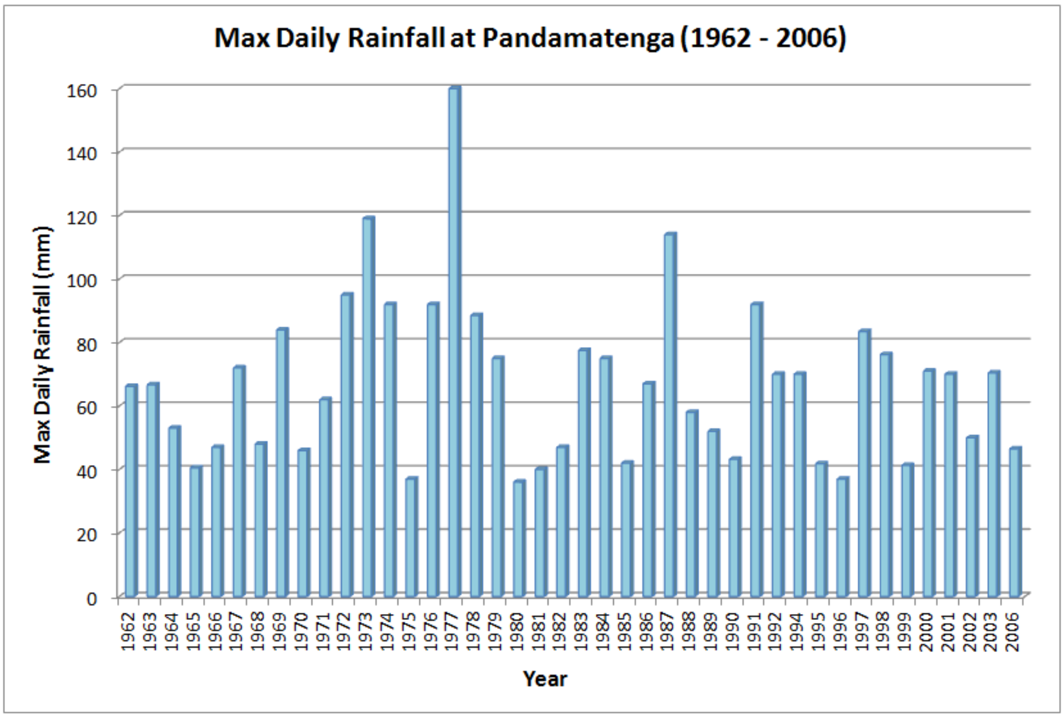


Figure 9. Maximum Daily Rainfall at Pandamatenga (Police Station, 1962 – 2006, DMS)

The Gumbel distribution is used to evaluate the statistical distribution of the maximum yearly values of daily rainfall in order to identify the intensity–duration–frequency (IDF) curves. The 42 values (N = 42) of maximum daily rainfall are listed in descending order thus their statistical probability can be assessed:

$$T_r = \frac{N_{tot}+1}{N_i} \quad ; \quad E_p = \frac{1}{T_r} \quad ; \quad NE_p = 1 - E_p \quad ; \quad Y = -Ln\left(-Ln\left(\frac{T_r - 1}{T_r}\right)\right)$$

Max Daily Rainfall (mm)	N	Time Return (Tr)	Exceeding probability (Ep)	Not exceeding probability (NEp)	Reduced variate (Y)
160.0	1	43.0	0.023	0.977	3.749
119.0	2	21.5	0.047	0.953	3.044
114.0	3	14.3	0.070	0.930	2.627
95.0	4	10.8	0.093	0.907	2.326
92.0	5	8.6	0.116	0.884	2.091
92.0	6	7.2	0.140	0.860	1.895
92.0	7	6.1	0.163	0.837	1.728
88.5	8	5.4	0.186	0.814	1.581
84.0	9	4.8	0.209	0.791	1.449
83.5	10	4.3	0.233	0.767	1.329
77.5	11	3.9	0.256	0.744	1.219
76.2	12	3.6	0.279	0.721	1.117
75.0	13	3.3	0.302	0.698	1.022
75.0	14	3.1	0.326	0.674	0.932
72.0	15	2.9	0.349	0.651	0.846
71.0	16	2.7	0.372	0.628	0.765

Max Daily Rainfall (mm)	N	Time Return (Tr)	Exceeding probability (Ep)	Not exceeding probability (NEp)	Reduced variate (Y)
70.4	17	2.5	0.395	0.605	0.687
70.0	18	2.4	0.419	0.581	0.612
70.0	19	2.3	0.442	0.558	0.539
70.0	20	2.2	0.465	0.535	0.469
67.0	21	2.05	0.488	0.512	0.400
66.7	22	1.95	0.512	0.488	0.333
66.2	23	1.87	0.535	0.465	0.267
62.0	24	1.79	0.558	0.442	0.202
58.0	25	1.72	0.581	0.419	0.138
53.0	26	1.65	0.605	0.395	0.075
52.0	27	1.59	0.628	0.372	0.011
50.0	28	1.54	0.651	0.349	-0.052
48.0	29	1.48	0.674	0.326	-0.115
47.0	30	1.43	0.698	0.302	-0.179
47.0	31	1.39	0.721	0.279	-0.244
46.5	32	1.34	0.744	0.256	-0.310
46.0	33	1.30	0.767	0.233	-0.377
43.2	34	1.26	0.791	0.209	-0.447
42.0	35	1.23	0.814	0.186	-0.520
41.8	36	1.19	0.837	0.163	-0.596
41.3	37	1.16	0.860	0.140	-0.678
40.5	38	1.13	0.884	0.116	-0.766
40.0	39	1.10	0.907	0.093	-0.865
37.0	40	1.08	0.930	0.070	-0.979
37.0	41	1.05	0.953	0.047	-1.121
36.0	42	1.02	0.977	0.023	-1.325

Table 2 Statistical elaboration of maximum daily rainfall at Pandamatenga

Then statistical elaboration consists in calculating the following parameters that lead to find relationship between precipitation heights (h) and return time of storm event:

- Xm : mean value of maximum daily rainfall;
- Sx : standard deviation of maximum daily rainfall;
- Yn : mean value of reduced variate;
- Sn : standard deviation of reduced variate;

$$\alpha = \frac{S_x}{S_n} \quad ; \quad \partial = X_n - Y_n \cdot \alpha$$

$$h = \partial + \alpha \cdot \left(-Ln \left(-Ln \left(\frac{T_r - 1}{T_r} \right) \right) \right)$$

Gumbel parameters					
Xm	**Sx**	**Yn**	**Sn**	**α**	**∂**
67.0	25.7	0.54	1.16	22.2	54.9

Table 3 Parameters of Gumbel analysis for maximum daily rainfall Pandamatenga

Tr (years)	2	5	10	25	50	100
h (mm)	63	88	105	126	142	157

Table 4 Daily rainfall heights related to return time of storm event at Pandamatenga

Being the time series composed of 42 years, rainfall heights with return time of 50 years or more are not reliable as the others. The resulting values are in line with what has been already redacted in the previous study related to the Zambezi Integrated Agro-Commercial Development Project ("Interim Hydrological Report" prepared by TAHAL in 2009) or rather they are slightly higher, that can be considerate as a conservative output.

Statistical Analysis	Return time (years)					
	2	5	10	20	50	100
LN2	59	80.8	95.2	109.1	127.1	140.7
LP3	58.5	80.6	95.8	110.8	131	146.7
Chow	56.7	79.7	94.9	118	157.9	187.9
EVI	59.2	81.6	96.4	110.6	129	142.7
GEV	58.2	80.5	96.3	112.1	133.7	150.9

Table 5 Daily rainfall heights related to return time of storm event at Pandamatenga (TAHAL, 2009)

Measurements of rainfall with duration lower than a day are not available, therefore the relationship between precipitation heights and return time has been conducted making reference to the methodological approach proposed in NWMPR (Volume 3). Within this document it has been redacted values of conversion factor (Table 6) that allow to transform daily rainfall to shorter storm event (Table 7).

Duration	24h	12h	6h	4h	2h	1h
Ratio with daily rainfall	1.00	0.97	0.90	0.80	0.60	0.40
Duration	**1h**	**45m**	**30m**	**15m**	**10m**	**5m**
Ratio with daily rainfall	0.40	0.36	0.30	0.20	0.14	0.08

Table 6 Conversion factor for daily rainfall heights (NWMPR, 2006)

Rainfall height (mm)						
Storm duration	**Return time (years)**					
	2	**5**	**10**	**25**	**50**	**100**
24 hours	63	88	105	126	142	157
12 hours	61	86	102	122	137	152
6 hours	57	79	94	113	127	141
4 hours	50	71	84	101	113	126

Rainfall height (mm)						
Storm duration	**Return time (years)**					
	2	**5**	**10**	**25**	**50**	**100**
2 hours	38	53	63	76	85	94
1 hour	25	35	42	50	57	63
45 minutes	23	32	38	45	51	57
30 minutes	19	26	31	38	42	47
15 minutes	13	18	21	25	28	31
10 minutes	9	12	15	18	20	22
5 minutes	5	7	8	10	11	13

Table 7 Rainfall heights related to return time and duration of storm event at Pandamatenga

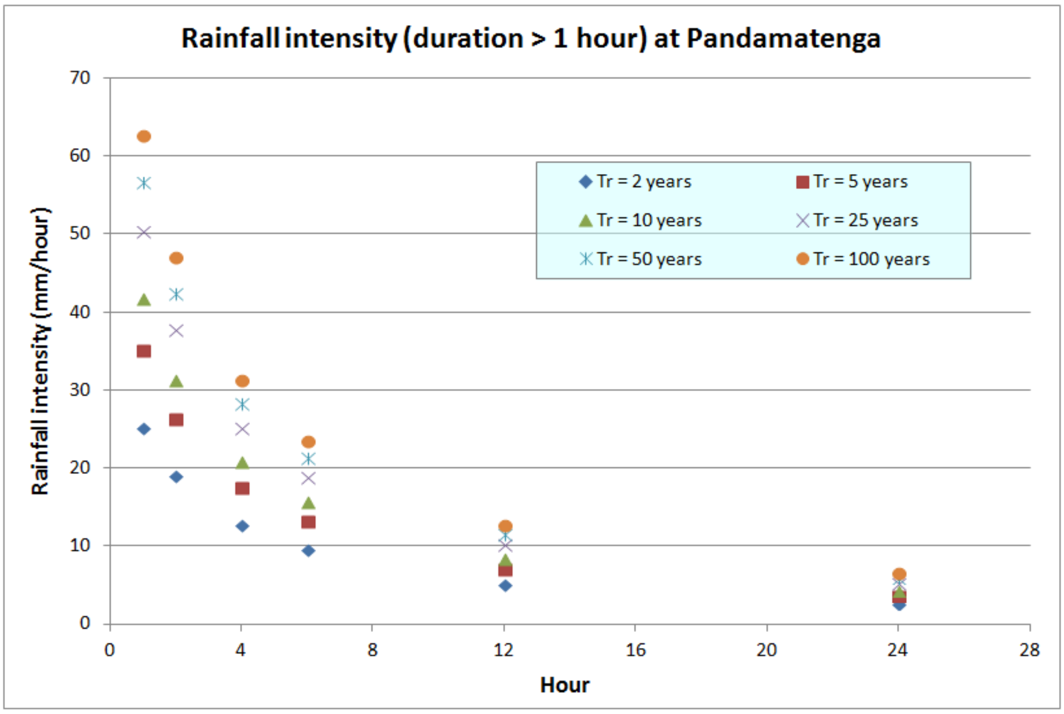


Figure 10 Intensity–duration–frequency relationship for storm event with duration more than one hour

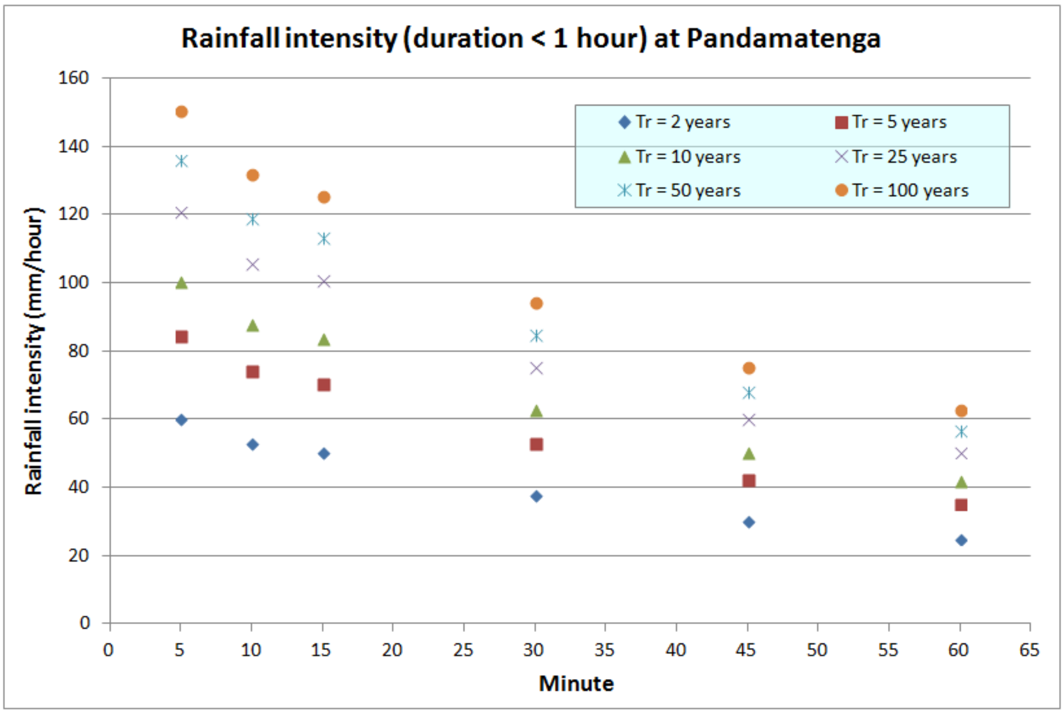


Figure 11 Intensity–duration–frequency relationship for storm event with duration less than one hour

Using the above described relationship, the related storm runoff will be determined and the drainage system will be designed (paragraph 6).

3 HYDROGEOLOGY

3.1 GENERAL HYDROGEOLOGICAL FRAMEWORK

The geology and climate, past and present, are important factors that influence the groundwater resources of Botswana *(Dep. of Surveys and Mapping, 2001).* Groundwater in Botswana is limited, both in quantity and quality and is unevenly distributed over the country. Groundwater collects in aquifers and is abstracted through well fields.

Only a small part of the groundwater resources can be economically abstracted due to high abstraction costs, low yields, poor water quality and remoteness of aquifers in relation to consumers centres *(SMEC et al, 1991, Masedi et al, 1999).* The estimated mean annual recharge is 2.7 mm being zero in western Botswana to 100 mm in the north.

The extractable volume of groundwater in Botswana is estimated to be about 100.000 Mm^3 *(Khupe, 1994).* But only 1% of this amount is rechargeable by rainfall because of the semi-arid climate characterised by low rainfall amount and high rates of evaporation as well as the nature of geology of aquifers *(Ayoade, 2001).*

According to Ayoade (2001) four types of aquifers are found in Botswana:

a) Fractured aquifers, which cover 27% of the country, are found in the crystalline bedrocks of the Archaen Basement in the east and in the Karoo Basalt. These have low yields with the median yield ranging between 2 and 10 ma per hour.
b) Fractured porous aquifers, which cover 37% of the country, are found in Ntane and Ecca sandstones as well as in arkoses in the Karoo Formation. These aquifers have the highest yields.
c) Porous aquifers, which cover 35% of Botswana, occur in sand rivers, alluvium and the Kalahari beds (presumable aquifers existing in the zone of *"Chobe Irrigation Area").* These are usually high yielding and have a median yield ranging between 10 and 300 m^3 per hour.
d) Karstified aquifers occur in the dolomite areas in southwestern parts of Botswana as well as in other areas in Lobatse, Ramotswa and Kanye. Karstified aquifers account for only 1% of the land area of Botswana. These aquifers have a median yield of 4-20 m^3 per hour.

Groundwater is located at great depth except in a few areas receiving regular floods or with permanent water bodies. The depth varies over the country from less than 40 m in the north and east (where is located the *Chobe Irrigation Area*) to well over 60 m in the drier central and south-western parts. The borehole technology has opened up very deep groundwater deposits.

Over a large part of Botswana, borehole yields are poor to fair with average yields being less than 4 m^3 per hour. In only a few areas are the average borehole yields in excess of 8 m^3 per hour.

In eastern and northern Botswana where is located the *"Chobe Irrigation Area",* recharge should increase to between 20 and 100 mm/yr, depending on local geology and geomorphology.

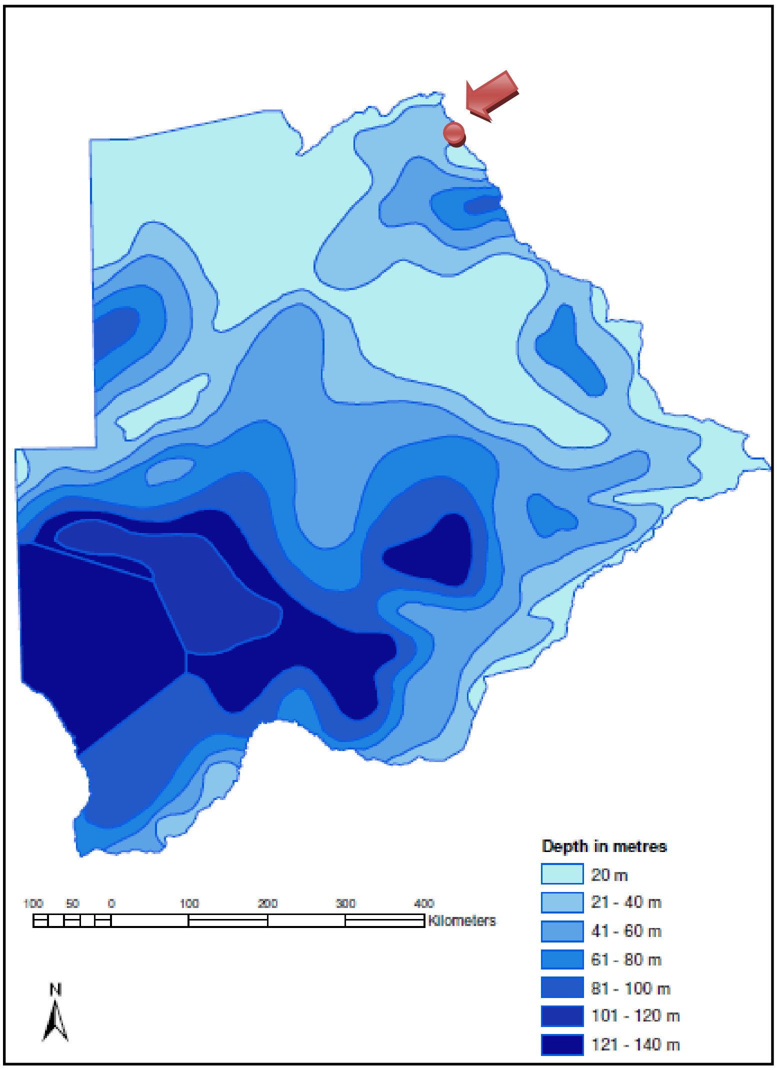


Figure 12. Map of Average depth of Groundwater (Department of Surveys and Mapping, 2001)

Information on groundwater recharge is of fundamental interest for any hydrogeological study, usually displayed as a layer for assigning aquifer productivity, or as an inset map. Recharge is a complex process governed by a number of controlling factors as that are highly variable in space and time as rainfall, evapotranspiration and unsaturated zone.

However, Döll & Fiedler (2008) have developed an algorithm to estimate the diffuse groundwater recharge at the global scale, with a spatial resolution of 0.5°. This algorithm has been adopted to create a recharge layer for the African Hydrogeology Map and for the World Hydrogeological Map.

In Figure 13, it appears the recharge layer of the Hydrogeological map designed for the Southern African Development Community *(SADC Project – Final Report March 2010).* In the zone of Chobe Irrigation Area, the Groundwater potential is classified as: "High, but variable (occasional or no recharge)".

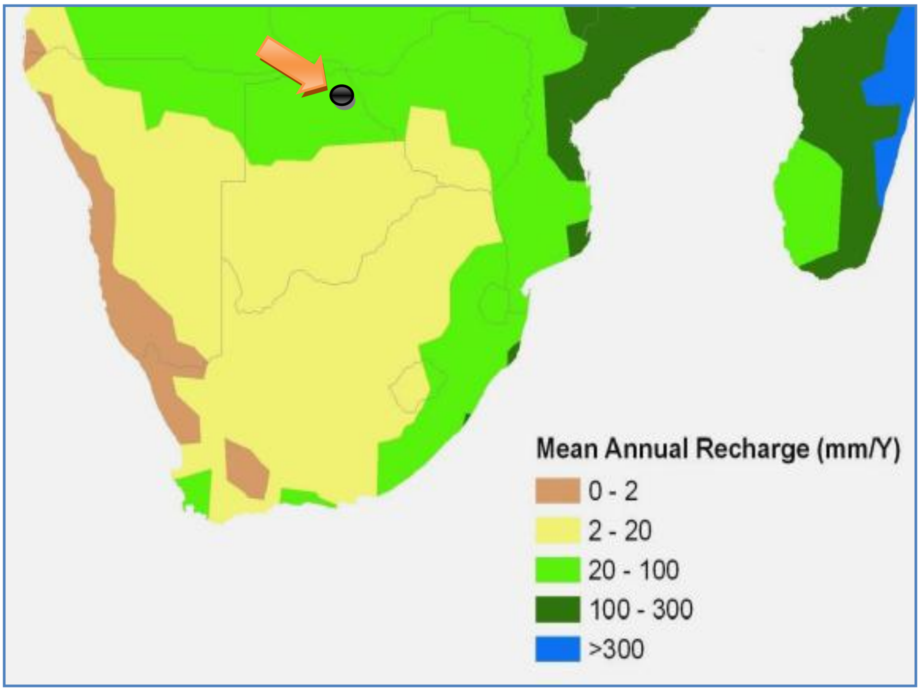


Figure 13. Map of Mean Annual Recharge (Doll and Fiedler, 2008)

3.2 DATA GATHERING AND PREVIOUS STUDY

For the local geological and hydrogeological characterization of Irrigation Area, the Consultant analyzed even the results of previous local studies, as the surveys and shallow pits already performed for *"Geotechnical Investigation for the Preliminary Design on the utilization of the water resource of the Chobe/Zambezi river"* redacted by Geotechnics International Botswana in June 2013, concerning the track of pipeline transfer in the Pandamatenga area for a total length of about 67 km.

This study was consisted of n° 4 boreholes drilled up to 15 m. u.g.l. (DH1, DH2, DH3 and DH4 as represented in Figure 14).

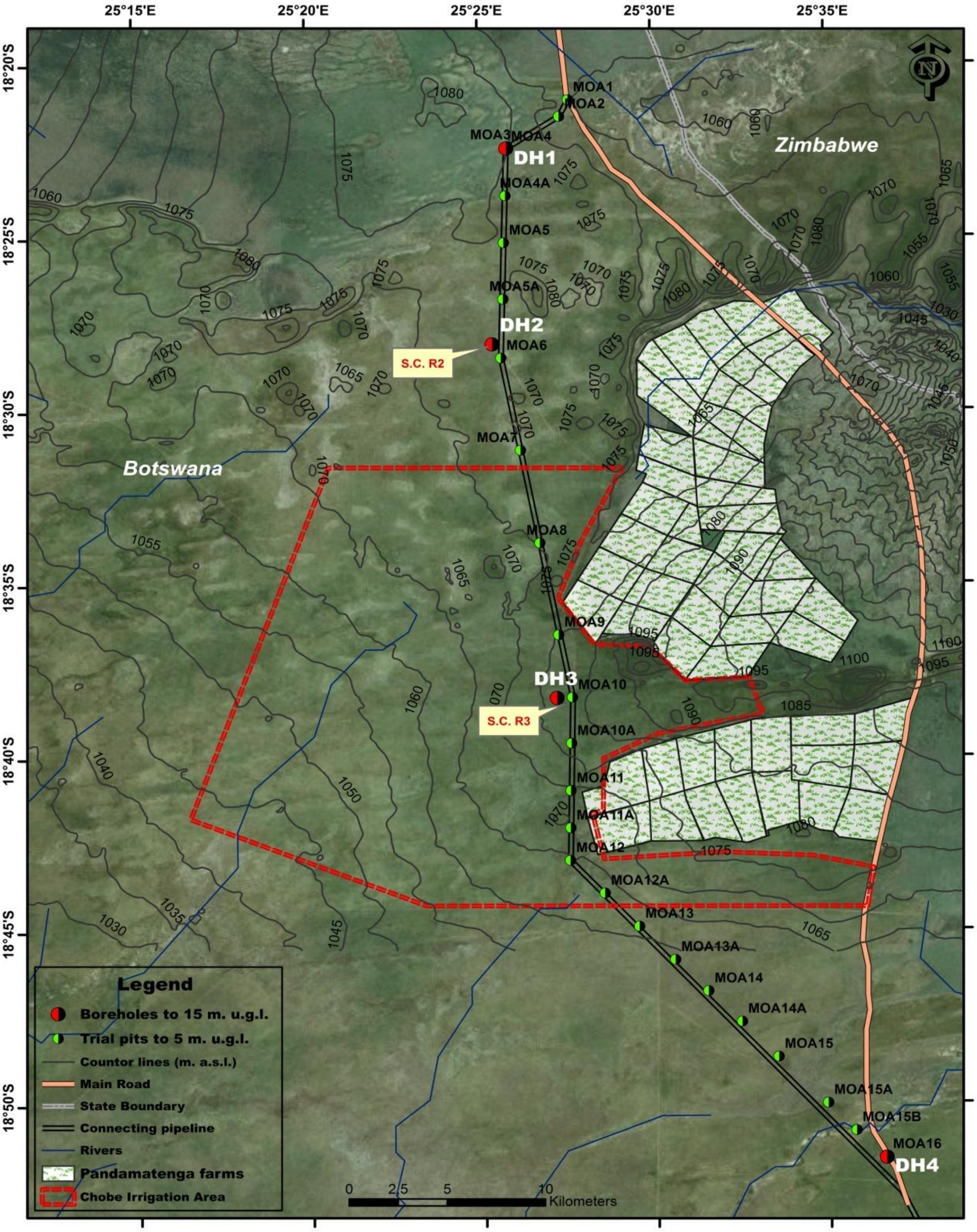


Figure 14. Layout of previous local geotechnical investigations (Consultant's elaboration, 2014)

Depth (m)	Lithotype	Average SPT
(0.0 ÷ 3.0) m.	Black, Stiff, Intact Clay	25
(3.0 ÷ 4.5) m.	Completely weathered, highly fractured, soft Basalt	28
(4.5 ÷ 7.3) m.	Highly weathered, highly fractured, soft Basalt	62
(7.3 ÷ 11.0) m.	Highly to moderately weathered, medium fractured, soft to moderately strong, Basalt	
(11.0 ÷ 15.0) m.	Moderately weathered, medium fractured, moderately strong, Basalt	
Depth of water	***Water table encountered at depth of 8.3 m.***	

Table 8 Stratigraphic description regarding soils in borehole DH1

Depth (m)	Lithotype	Average SPT
(0.0 ÷ 2.0) m.	Sandy Clay	
(2.0 ÷ 10.0) m.	Very dense, slightly cemented silty sand with traces of calcrete gravel	38
(10.0 ÷ 15.0) m.	Very dense, slightly cemented silt sand	25 refusal
Depth of water	***No water table encountered***	

Table 9 Stratigraphic description regarding soils in borehole DH2

Depth (m)	Lithotype	Average SPT
(0.0 ÷ 10.5) m.	Medium dense to very dense silty sand	42
(10.5 ÷ 12.0) m.	Highly weathered, highly fractured, soft basalt	
(12.5 ÷ 15.0) m.	Moderately weathered, medium fractured, moderately strong, basalt	50
(12.0 ÷ 15.0) m.	Moderately weathered, medium fractured	
Depth of water	***No water table encountered***	

Table 10 Stratigraphic description regarding soils in borehole DH3

Depth (m)	Lithotype	Average SPT
(0.0 ÷ 1.5) m.	Silty Sand, aeolian	
(1.5 ÷ 12.0) m.	Medium dense to very dense Silty Sand with traces of calcrete gravel	48
(12.0 ÷ 15.0) m.	Very dense, Sandy Calcrete	
Depth of water	***No water table encountered***	

Table 11 Stratigraphic description regarding soils in borehole DH4

For a better hydrogeological framework of the study area, were examined the considerations of the Hydrogeological Survey about *"Zambezi Integrated Agro-Commercial Development Project"*, where n° 25 exploration boreholes (BH) were drilled in the area that is located midway between Kasane and Pandamatenga, about 30 km to the north of *"Chobe Irrigation Area"* (see figure below).

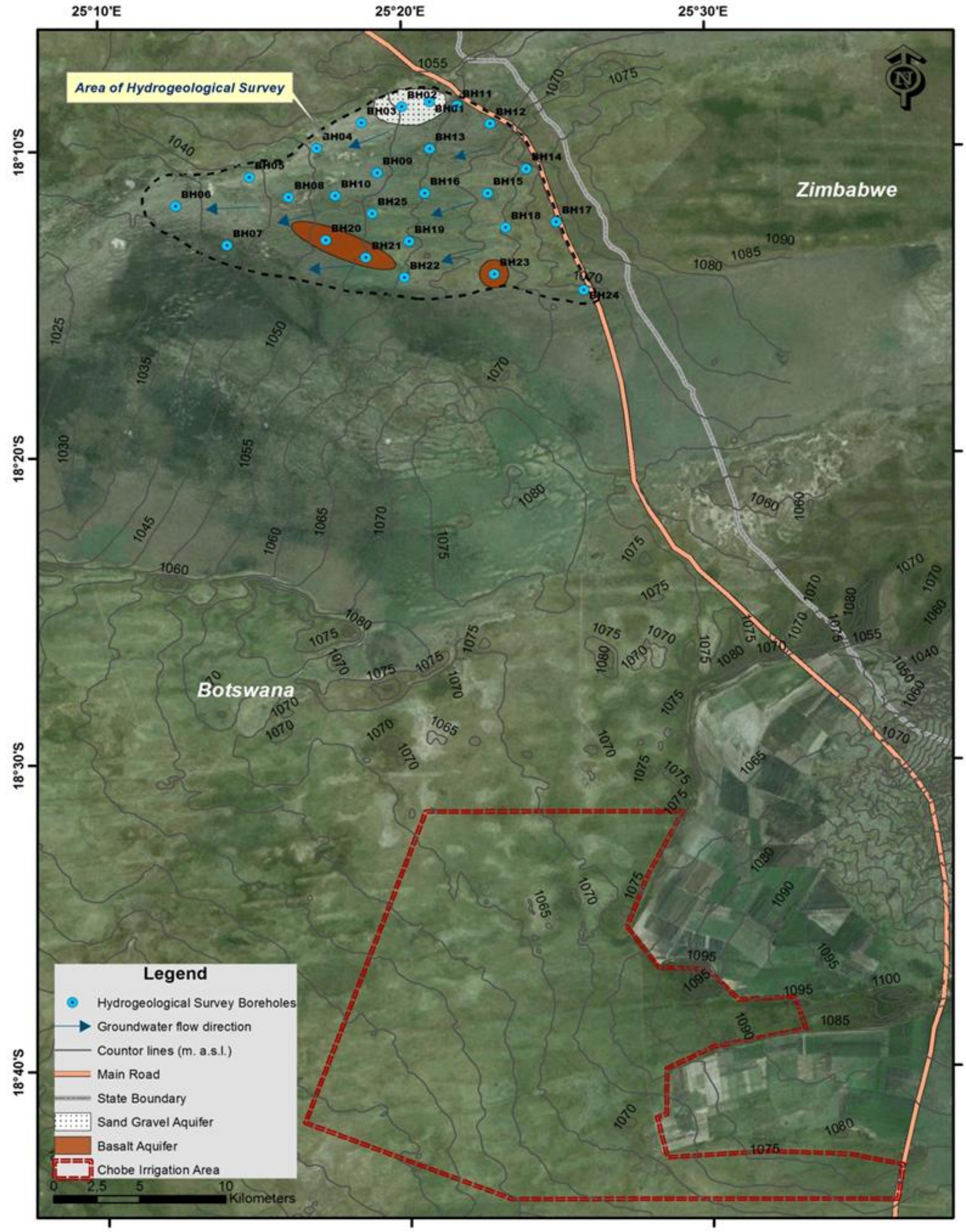


Figure 15. Map of Hydrogeological Survey results (Consultant's elaboration, 2014)

The depth of the boreholes varied from 14 m to 37 m under ground level, furthermore it is realized the execution of water quality laboratory analysis, calculation of groundwater regime, absolute water level and flow direction with indication of the possible exploitable aquifers.

The general results can be summarized in the following tables.

Borehole No.		Depth of Borehole	Depth of W. L*. upon Completion	Depth of W.L (19/20/6/2008) after several days
Sampled	1	30	16.20	16.45 (from pipe)
	2	33	12.60	13.04-0.31=12.73
	3	24	10.00	9.55; 9.50
	4	20	9.82	9.80
Sampled	5	18	10.57	6.98
				from head of pipe 7.04 (0.36m above G.L. (19/6)
	6	30	27.35	10.80
	7	18	12.75	
	8	20	12.93	9.58
	9	19	13.15	13.15
	10	19	12.30	12.25; 12.30
	11	31	17.95	15.55
Sampled	12	31	18.20	19.68-1.44=18.24
	13	31	25.00	15.55
	14	37	-	26.30
	15	30	21.10	21.10
Sampled	16	33	15.75	18.98-1.20=17.78 G.L.
	17	34	31.15	29.20
	18	37	33.50	22.78
	19	25	17.64	17.65
Sampled	20	25	13.55	14.60-1.05=13.55
	21	25	16.92	16.89
	22	30	19.60	19.60
	23	14	perched aquifer	
			3.20	3.30
				well collapsed at 3.50
	24	30	-	24.50-0.72=23.78
	25	13	no water	
				W.L. measured at 19-20/6/2008
				(apparatus starts at 2 m)

* W.L. – water level

Table 12 Depth of Static Water Level upon completion of drilling and after several days (TAHAL Group Hydrogeological Survey, 2008)

Borehole No.	Depth (m)	Thickness of upper soil	Thickness of horizon of concretions or pebble	Thickness of chalk	Thickness of weathered basalt	Depth of fresh basalt
1	30.0	2 m sand + 4 m clay	1 m gravel of flint	17	-	-
2	33.0	3.5 - clay	-	20.5	-	-
3	24.0	1 - clay	-	20	-	-
4	20.0	1.20 - clay	0.80 calcrete	11	8	-
5	18.0	0.60 - clay	2.40 limestone boulders	5 (limst)*	5	12
6	30.0	2 - clay	0.50 calcrete	-	19.5	22
7	18.0	2 - clay		6	6	14
8	20.0	0.5 - clay	1.5 gravel and pebbles of flint	4	12	18
9	19.0	2.5 - clay		-	13.5	16
10	18.0	1.8 - clay	0.20 fragments of basalt	-	10	12
11	31.0	1 - clay	0.20 calcrete, limst*	-	22.8	24
12	31.0	1.2 - clay	-	-	22.8	24
13	31.0	1.7 - clay	0.3 calcrete + basalt	-	22	24
14	37.0	1 - silty clay	2 - boulders, pebbles limst*	-	7	10
15	30.0	1.5 - clay	0.5 calcrete, limst*. boulders	-	22	24
16	33.0	1.8 - clay	0.6 calcrete	-	18.5	21
17	33.5	2 - fine sand	-	-	14	16
18	37.5	2 - fine sand	-	-	12	14
19	25.0	2 - clay	0.5 - calcrete, flint	-	15.5	18
20	25.0	2.5 - clay	0.5 - calcrete+clay	-	16	19
21	25.0	2 - clay	1 - calcrete+clay	-	15	18
22	30.0	2 - sandy soil	1- mixed soil and basalt fragments	-	15	18
23	14.0	2 - sand, 2 clayey sand	1 - pebbles flint, iron, basalt	-	9	-
24	30.0	0.8 - clay	-	-	3.2	4
25	13.5	1 - sandy clay	-	-	11	12
Total	**656.5**					

* limst - limestone

Table 13 The thickness of the various beds from surface downward (TAHAL Group Hydrogeological Survey, 2008)

It can be observed how the depth of groundwater varies between 10 m and 20 m below ground level, the flow direction is from east to towards west vice versa the depth of groundwater level.

3.3 PLANNING OF FIELD SURVEY

In the zone of *Chobe Irrigation Area*, the area looks flat with a slight slope from NE to SW, the location of boreholes has been established choosing them among the ground control point (where ground level will better defined) and taking into account the general trend of water table or rather having 3 points that allows estimating the gradient of water table.

The following image shows the location of the borehole that were realized.

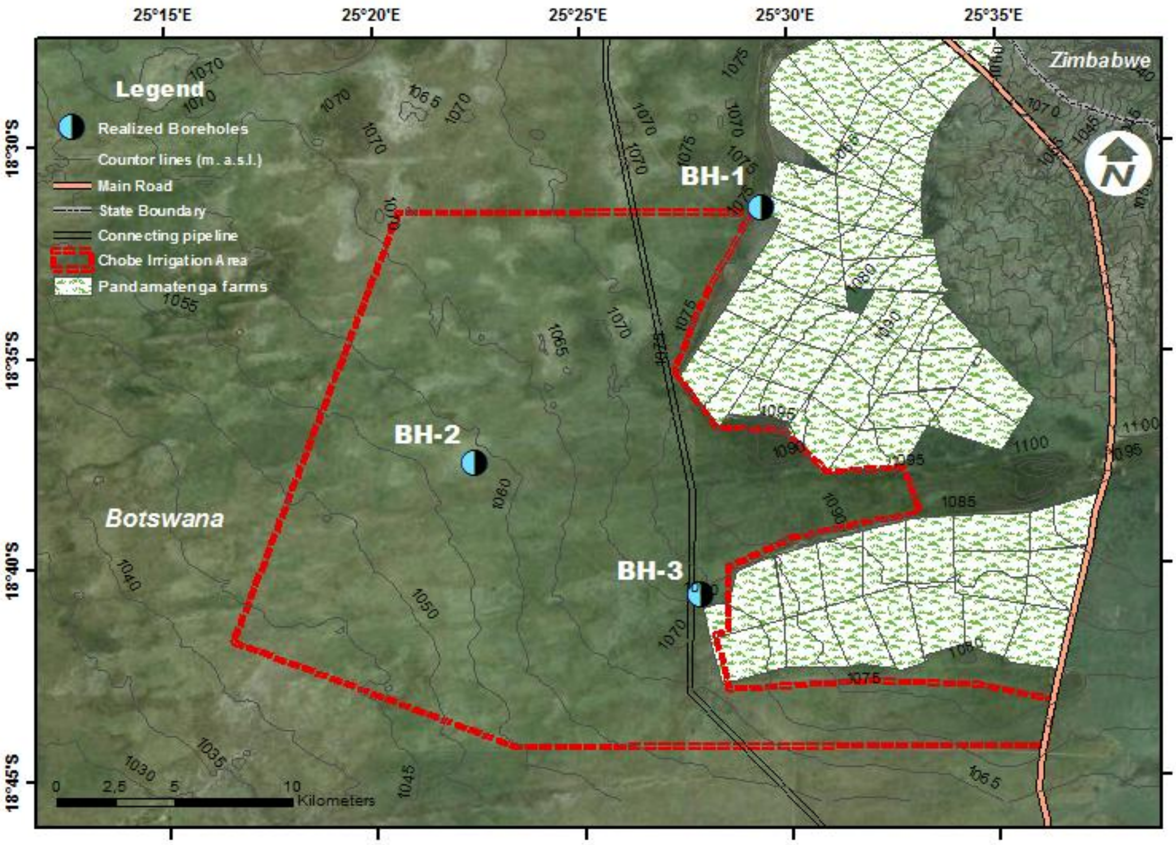


Figure 16. Layout of realized borehole for hydrogeological survey (Consultant's elaboration, 2014)

Thus this investigation was accomplished with n. 3 borehole reaching 40 m under the ground level. The resulting depth of groundwater within the *"Chobe Irrigation Area"* varies between 31.10 m in BH-1 and 36.67 m in BH-3 below surface, while in BH-2 the depth is certainly greater than 40.00 m, because at less depth the hole is resulted dry.

At North of irrigation area, in the borehole DH1 the useful measured depth was 8.30 m below ground level, *(in the boreholes DH2, DH3 and DH4 no water table was encountered to a depth of 15 metros under ground level).*

The depth of the static water level, the elevation of the reference points and the absolute water level are presented in the next table.

Borehole No.	Date of Survey	Depth of S.W.L. (u.g.l.) - m -	Elevation of Ground (a.s.l.) - m -	Absolute Water Level (a.s.l.) - m -
BH-1	June 2014	31.10	1079	1047.90
BH-2	June 2014	> 40.00	1058	< 1018
BH-3	June 2014	36.67	1065	1028.23
DH-1	June 2013	8.30	1075	1066.70

Table 14 Available values of Static Water Level and Absolute Water Level

So, with the available data that are measured in the same autumn period (June 2013 and June 2014), it can be concluded that the flow direction is from north-east towards south-west (see also next figure). Because the water table is so deep, it is reliable that agricultural practises would not cause such a raising to make in contact between surface and groundwater thus avoiding any risk of contamination by fertilizers.

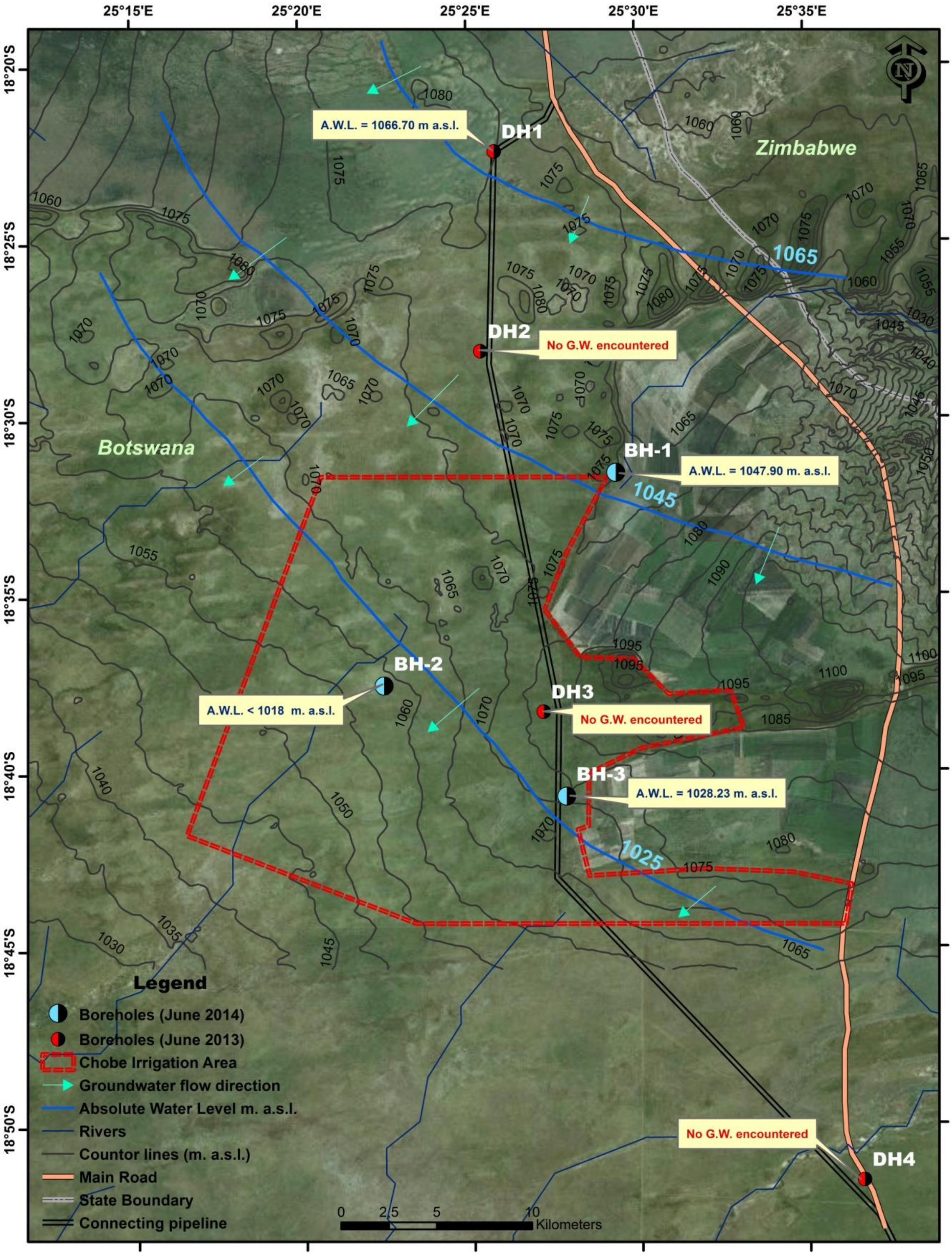


Figure 17. Map of Absolute Water Level and Flow Direction (Consultant's elaboration, 2014)

4 ONFARM IRRIGATION SYSTEM DESIGN

4.1 PLANNING OF IRRIGATION SYSTEM

The step-by-step procedure taken in planning and design of irrigation system are inventory of available resources and operating conditions, topographic map of the area, water supply – source availability and dependability, climatic condition, power source, crop selection and water supply level.

As it was mentioned in the TOR, the key focus of the infrastructure component activities was the planning of water use for irrigation considering the potable water demand as a subsequent step. In addition, it was already noted in the TOR that pressurized irrigation systems could be considered very advantageous for this project.

Based on this and taking several selection criteria, centre pivot sprinkler system was selected for sandy clay loam, sandy loam and loamy sand soils and drip irrigation system for sandy soils (details in paragraph 4.2). The planning and design of these two systems were performed for the water distribution and irrigation system considering the technical feasibility, economic viability, social acceptance and environmental sustainability.

The design of irrigation system was carried out on-demand based of irrigation requirements. Then, the network layout was designed to give inputs on most efficient ways of connecting all the users.

4.1.1 Design criteria

In principle, the first step in the preliminary design phase was the collection of basic farm data. These are topographic map showing the proposed irrigated area, with contour lines, farm and field boundaries and water source or sources, power points- such as electricity lines in relation to water source and area to be irrigated, roads and other relevant general features including obstacles.

Moreover, data on water resources (quantity and quality) over time, the climate of the area and its influence on the water requirements of the selected crops, the soil characteristics and their suitability to the crops and irrigation system proposed, the types of crops intended to be grown and their adaptability to both the climate and the area were collected.

The next step was analyzing the farm data in order to determine the following preliminary design parameters: peak and total irrigation water requirements, infiltration rate of soils to be irrigated, maximum net depth of water application per irrigation, irrigation frequency and cycle, gross depth of water application and preliminary system capacity.

4.1.2 Land resource

As per the results of the soils analysis and based on the field investigation and laboratory results, seven land units have been classified as shown in the following table.

The soil physical and chemical properties used for selection of pressurized irrigation system and design of centre pivot and drip irrigation system are shown in Table 16 and paragraph 9.1.

Mapping Unit	Soil/Land Unit Description	Area	
		ha	%
ZA1/1 1	Flat almost flat, very deep, moderately well drained, very dark gray to gray color, sandy clay loam, developed on lacustrine 0-1.5% slopes: Soil HypereutricVertisols (Vreuh)	5,512	12.13
ZA1/2 2	Flat, moderately deep, well drained, Very dark gray - Dark grayish brown color, sandy loam texture, developed on lacustrine, 0-1% slopes: soils HypocalcicVertisols (VRccw)	6,656	14.56
ZA2/3 3	Flat almost flat, very deep , somewhat excessively drained, dark reddish brown to light brownish gray color, loamy sand texture, developed on sandveld, slopes 0-1.5%, Soils: HypoferalicArenosols (ARflw)	1,903	4.19
ZA2/4 4	Flat almost flat, very deep, excessively drained, dark grayish brown to yellowish brown color, developed on sandveld, deposit, sand texture, slope o-2% soils:	21,300	47.0
ZA5	Flat almost flat, very deep, excessively drained, dark grayish brown to yellowish brown color, developed on sandveld, deposit, sand texture, slope o-2% soils:	10,000	22
ZA6	Settlement	10.8	0.02
ZA7	Quarry Site	4.65	0.01

Table 15 Classified area as per the textural analysis

Soil type	Infiltration (cm/hr)	HC (m/day)	AW (mm/m)
Sandy Clay loam	2.9	2.7	97.2
Sandy Loam	4.73	2.54	85.11
Loamy Sand	10.9	6.37	76.85
Sand	26.57	19.11	61.92

Table 16 Average values of soil physical properties of the study area

4.1.3 Water resource

As per the TOR, total water extraction is estimated at 495 million cubic meters per year. The National Water Authority will use about 150 million cubic meters, with the remaining 345 million cubic meters, being used for the proposed agricultural project. The irrigation water would be pumped from a reservoir with a design capacity of 2.0 million m^3.

As underlined in the National Master Plan for Arable Agriculture and Dairy Development (NAMPAADD), “since water is a scarce resource in Botswana, farmers will be encouraged to use water efficient technologies for irrigation.” In light of this, the Client perceives minimizing water losses in the conveyance, distribution and application as an important consideration for the present project.

Hence, taking into account the latest trends in the irrigation sector as well as indications provided by the ToR, the development of a pressurized irrigation system for the project area has been considered. Moreover, it was given that the design discharge from the pump station to reservoir R2 at Pandamatenga is 23,300 l/s.

4.2 OPTIONS IDENTIFICATION AND ASSESSMENT

The principal objectives of this component of the project are:

- Assess different alternatives for the development of a water distribution / irrigation system in the area,
- select the most suitable option - taking into account the criterion:
  - viability
  - suitability for local conditions
- prepare a corresponding conceptual design and implementation plan.

Irrigation technologies depend on specific chemical, biological and physical conditions of water and soil as well as types of cultivated crops. These, together with the objective of an efficient use of available resources and irrigation system productivity (attained by optimizing investment and operation and maintenance cost), imply setting a number of determinant parameters that guide the assessment of alternatives for the irrigation and water distribution systems development.

Recognizing the pressurized irrigation system suggested by MoA and as per the given TOR and technical proposal, the following two methods of pressurized irrigation systems were selected and evaluated.

- Sprinkler irrigation system
- Drip/trickle irrigation

Sprinkler irrigation system has the following advantages:

- Uniform distribution of water
- Accurate measurement of the applied water, rendering high water use efficiency
- Eliminates excessive losses from deep percolation, surface runoff and conveyance losses
- Land with irregular topography can be irrigated by sprinklers without much leveling and land preparation
- Can be used on soils with low water holding capacity
- Can be used on sloping lands
- Does not require field channels and thus more areas become available for crop production
- Possibilities of fertigation and chemigation
- can be used for almost all crops and on most soils
- Feasibility of frequent, small water amount applications for germination, cooling, frost protection, etc.
- The closed water delivery system prevents contamination of the irrigation water.

Some disadvantage of the sprinkler irrigation system, which will be corrected through design and management, are as follows:

- High initial investment
- Energy cost is usually high as water is pumped under pressure.
- Design, planning and operation of sprinkler system require good technical expertise.
- Sensitivity to wind conditions
- Water losses by evaporation from soil surface and plant canopy, if wetted.
- Induction of leaf diseases in over-head application.
- Hazard of salt accumulation on wetted foliage in overhead application.
- Leaf burns and washout of pesticides from the foliage in overhead application.
- Interference of irrigation with various farm activities like tillage, spraying, harvest, etc.

For what concerns adaptability, some of the conditions, which favours sprinkler irrigation, are as follows.

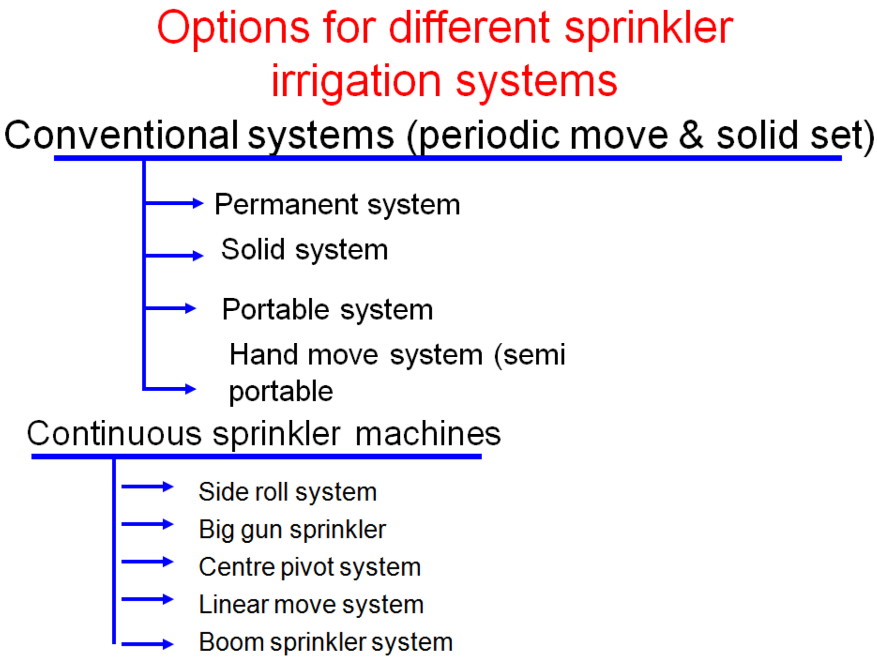
- Shallow soils the topography of which prevents proper levelling for surface irrigation methods
- Land having steep slopes and easily erodible soils.
- Irrigation stream too small to distribute water efficiently by surface irrigation.
- Undulating land too costly too level sufficiently for good surface irrigation.
- Soils with low water holding capacities and shallow rooted crops, which require frequent irrigation.
- Automation and mechanization are practical.
- Higher application efficiency can be achieved by properly designed and operated systems.
- Good clean supply of water, free of suspended sediments, is required to avoid problems of sprinkler nozzle blockage and spoiling the crop by coating it with sediment.

In the options assessment process, the following decision making parameters were used:

- available water sources,
- soils and topography,
- climate and crop,
- capital and labour,
- energy,
- social aspects and policies,
- socio-economic aspects,
- health aspects and
- environmental aspects.

4.2.1 Alternatives of different pressurized irrigation system

Primarily there are two major classifications of sprinkler irrigation systems. These are Conventional systems (periodic move & Solid set) and Continuous sprinkler machines. Different irrigation systems are further classified in the following manner.



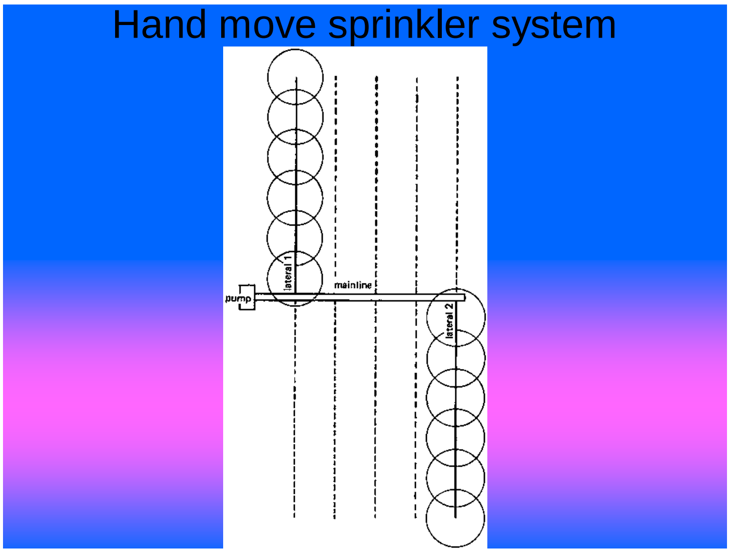


Figure 18 Schematic functioning of hand move sprinkler system

Figure 19 Example of continuous type sprinkler irrigation system

4.2.2 Selection of continuous over conventional sprinkler system

Comparing the conventional sprinkler method with that of the continuous system, the following parameters were envisaged.

- Continuous system is labour-saving as compared to high labour costs to move the conventional set
- Easily automated, fertigation and chemigation is possible- applies chemicals and fertilizers inexpensively, accurately and at any stage of the crop growth.
- Largeness of the command area leads to the use of continuous type of sprinkler system
- Periodic move systems apply water for a set time while stationary before moving to the next position but, the continuous move system apply water while in motion
- Continuous move system is highly efficient and environmentally sound giving less run-off and disposal of water to down stream
- Efficient water usage—minimizes deep percolation loss and evaporation through timely and precise applications.
- Uniform coverage—irrigates uniformly throughout the entire field

4.2.3 Centre pivot irrigation system

Self-propelled sprinkler system rotates around the pivot point and has the lowest labour requirements of the systems considered. The water source for this system, a well or buried pipeline, is located at the centre of the field and delivers water to the pivot arm.

The rotation of the pivot arm results in the pattern of circular irrigated areas. In the case of centre pivot irrigation without corner system, $\pi/4$ = 79% of the square area is irrigated applying irrigation water in circles (vs. squares), as presented in the following figures.

Figure 20 Example of alignment of center pivot

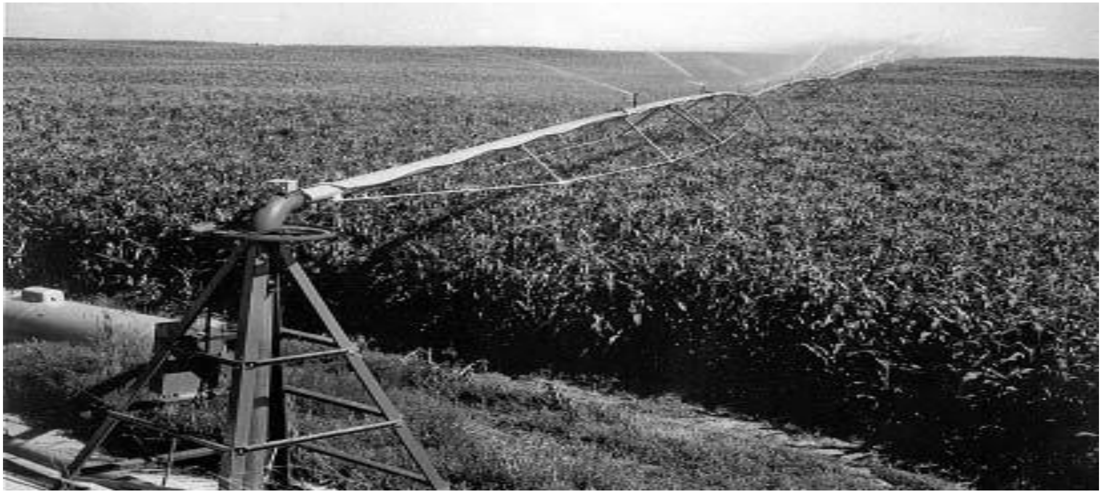


Figure 21 Center pivot sprinkler working in the field

Sprinkler nearest the pivot point may discharge only a fine spray; constant radial velocity but variable tangential speeds (fastest at periphery). The water application amount is controlled by the speed of rotation.

Centre pivots are adaptable for any height crop and are particularly suited to lighter soils. They are generally not recommended for heavy soils with low infiltration rates. Computerized control panels allow the operator to specify speed changes at any place in the field, reverse the pivot, turn on auxiliary pumps at a specified time and use many other features.

The low per hectare cost of large centre pivot systems, the limited labour requirements and the low energy requirements of pivot systems using spray nozzles are the main reasons for the popularity of these systems. Centre pivot systems equipped with nozzles and drop pipes, placing the nozzles just above the crop canopy, are very useful under windy conditions.

4.2.4 Centre pivot with corner attachment and/or end guns

Corner attachment systems that allow irrigation of most of the corner areas missed by a conventional centre pivot system are available. Depending on the method of corner irrigation, pivot systems with corner attachments will irrigate additional area of quarter section.

However, in the case of centre pivot with corner attachment and /or end guns, generally the corner span alone costs about half as much as the rest of the pivot, thereby increasing the capital cost per hectare. It is because of this reason, we planned to take centre pivot without corner or end gun (Figure 22).

Figure 22 Example of center pivot with corner attachment

4.2.5 Linear move sprinkler

The linear move (sometimes called a lateral move) irrigation system is built the same way as a center pivot; that is, with moving towers and spans of pipe connecting the towers. The main difference is that all the towers move at the same speed and in the same direction.

Water is pumped into one of the ends or into the centre. Water can be supplied to the linear move through a canal or by dragging a supply hose that is connected either to a main line or by connecting and disconnecting from hydrants as the linear moves down the field. Field must be rectangular.

Typically gives high application uniformity. Usually fed by open ditch with moving pump, requiring very small (or zero slope) in that direction. It can also be fed by dragging a flexible hose, or by automated arms that move sequentially along risers in a mainline.

It does not have problem of variable tangential speeds as with centre pivot. However, due to the lateral movement, powering a linear with electricity is difficult. Usually, a diesel motor with a generator is mounted on the main drive tower and supplies the power needed to operate the irrigation system.

It requires the source of water to be available all along one edge of the field. Water supply system is more complex for a linear-move system than a centre pivot because the distribution system delivers water along the entire length of one side of the field instead of only at the center (Figure 23).

Figure 23 Linear move supplied to the linear move through a canal

4.2.6 Side role sprinkler

The side roll (sometimes called a wheel roll) system, consists of a lateral, usually a quarter mile long, mounted on wheels with the pipe acting as an axle. Side roll systems also are adapted only to short crops; have medium labour requirements, moderate initial investment, medium operating pressure and generally rectangular field requirements irrigating limited area.

The side roll is better adapted to heavy textured soils than other continuous moving system. This type of system essentially evolved out of a labor shortage to move the hand-move lines. An entire length of side- roll line is moved by a small derive motor installed in the center of the line.

In general, since, the soils of the project area are not heavy textured soil and the area is very large; side roll sprinkler type is not selected (Figure 24).

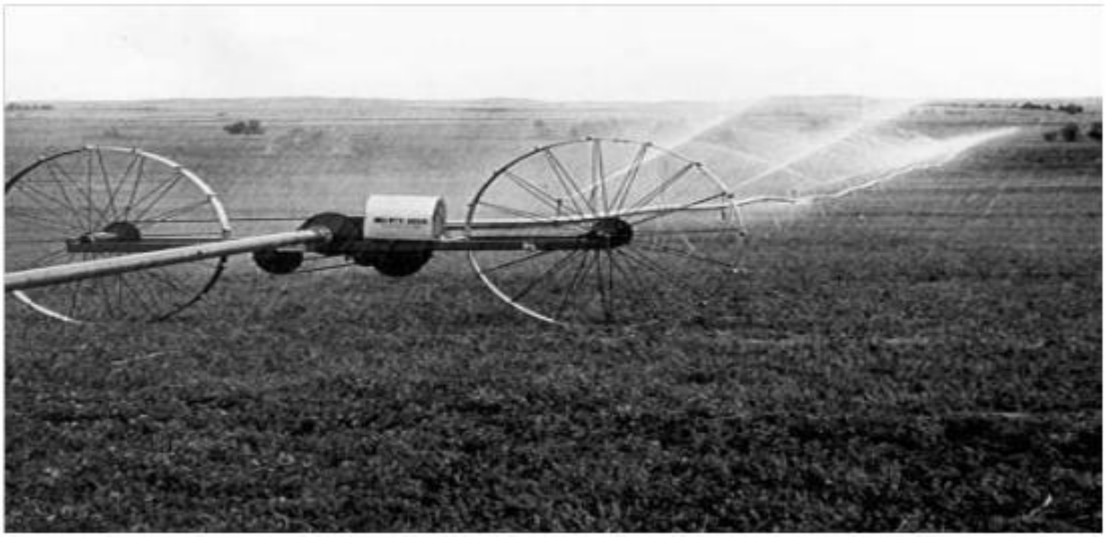


Figure 24 Side roll sprinkler system

4.2.7 Big gun sprinkler

Gun traveller sprinkler is a high capacity sprinkler fed with water through a flexible hose; mounted on self-propelled chassis and travels along a straight line while watering. It has a single large diameter nozzle, which sprays large volumes of water reaching in a circular pattern. Long flexible hose with high head loss may reel up the hose or be pulled by a cable on a trailer.

The travelling big gun system uses a large-capacity nozzle and high pressure to throw water out over the crop as it is pulled through an alley in the field. It is particularly adaptable to various crop heights, variable travel speeds, odd-shaped fields and rough terrain. The big gun requires a moderate initial investment, more labour and higher operating pressures than centre pivots and linear moves (Figure 25).

Figure 25 Big gun sprinkler system

4.2.8 Drip irrigation system

Drip irrigation system is the method of watering at the plant location frequently with volume of water approaching the consumptive use (CU). The spacing of emitters is much less than that of sprinklers and the pipe distribution network is working under low pressure.

In drip irrigation system, not all areas are irrigated and hence, the area irrigated is only accounted for. Moreover, Emitter spacing is not a function of wind in the case of drip irrigation system. It is used on almost any type of soil including Clay soil, marginal/infertile soil and stony soils.

In drip irrigation, system evaporative component is reduced as only limited area of the soil is wetted and the limited wetted area results in reduced weed growth. It gives accurate water distribution resulted in high water application efficiency.

The slow rate of water application improves the penetration of water into problematic soils, keeping the root zone with a high water potential. It is water saving technology with less energy and operating cost (Figure 26).

Figure 26 Layout of drip irrigation system showing the significance of reduced/wetted area as compared to the total area

4.2.9 Final selection of irrigation system

Out of the above five alternative continuous sprinkler irrigation systems, the center pivot has been selected according the following justifications:

1. Since centre pivot is fully / semi automated system with computerized control panels, it is labour saving system.
2. Centre pivot sprinkler system is suitable for light soils with high intake rates, hence the soils of the study area fulfil this requirement
3. The water source for this system, whether a well or buried pipeline, is located at the centre of the field and delivers water to the pivot arm.
4. The cost of the moving centre pivot laterals depends on its length. Long laterals are cheaper (per unit area).
5. The pivot arm is rotated by hydraulic or electric derive motors connected to the wheels at the intermittent towers. Hence, linear move type requires continuous move electricity power, open channel or long hoses to supply water along the lateral, it is rejected to use
6. Linear move requires the source of water to be available all along one edge of the field.
7. Water supply system is more complex for a linear-move system than a centre pivot because the distribution system delivers water along the entire length of one side of the field instead of only at the centre.
8. Gun type system irrigates small plot areas by pulling with additional labour, but the study area is very large which could not be suitable for gun type system.
9. Side roll type of sprinkler is suitable for heavy clay soils and covers small area coverage, hence no clay soil is available and the study area is very large, it is rejected
10. Drip irrigation may be another alternative for fruit crops in a sand soils and is adapted

Therefore, based on the above justifications, the planning and design work of irrigation for the study area was done with centre pivot continuous irrigation system for those identified suitable soils, sandy clay loam, sandy loam and loamy sand. In addition to this, Drip irrigation system was evolved for perennial fruit crop, i.e. mango on sandy soils.

4.3 CROP WATER REQUIREMENTS

Crop water requirements (CWR) encompass the total amount of water used in evapotranspiration. Irrigation requirements (IR) refer to the water that must be supplied through the irrigation system to ensure that the crop receives its full crop water requirements. The four procedural steps involved in the calculation of crop water and irrigation water requirements are:

- Calculation of reference evapotranspiration (ETo) based on meteorological parameters collected from the near-by meteorological stations.
- Crop coefficients for three annual crops, Maize, Wheat, and Soybeans and two perennial crops, Alfalfa and fruit crop (Mango) for full-fledged irrigation during the dry season and also during the wet season, three annual crops (Sorghum, sunflower and beans) for supplementary irrigation.
- Determination of monthly crop water requirements (ETc) depending upon the cropping patterns and local conditions.
- Determination of Irrigation water requirements/demand using calculated effective rainfall of the area

The crop water requirement and irrigation water demand for all selected crops have been estimated using 80% dependable rainfall on a monthly basis and the FAO's CROPWAT computer model (FAO, 1996).

4.3.1 Reference crop evapotranspiration

The estimation of crop water requirement normally needs the analysis of climatic data and agronomic practice of the proposed project area. The reference crop evapotranspiration (ETo), that is analyzed using the modified Penman Method, gives the effect of climate on crop water requirement. The monthly climate data of the study area collected from the following sources.

Parameter:	Rainfall
Source:	Department of Meteorological Services
Parameter:	Temperature (Max and Min)
Source:	Department of Meteorological Services
Station:	Pandamatenga Met. Stn.
Years:	1998 - 2012
Parameter:	Relative humidity (at 8 am and 2 pm)
Source:	National Water Master Plan - Volume 3
Station:	Maun Airport
Years:	1965 - 1998 (approx)
Parameter:	Wind speed (at 10 and 2 m)
Source:	National Water Master Plan - Volume 3
Station:	Kasane Airport
Years:	1982 - 2000 (approx)
Parameter:	Monthly Temperature (Max and Min)

Source: Department of Meteorological Services

Parameter: Evaporation
Source: Department of Meteorological Services
Station: Pandamatenga Met. Stn.
Years: 1997 - 2012
Unit: Mm

Moreover, the climate data used for the analysis of ETo (reference evapotranspiration) using CROPWAT computer model is shown in the following table.

Month	Min Temp	Max Temp	Humidity	Wind	Sun	Rad	ETo
	°C	°C	%	m/s	hours	MJ/m²/day	mm/day
January	19.6	30	61	0.8	7	21.6	4.66
February	19.2	30.1	61	1	7.7	22.2	4.81
March	18.2	30.3	59	1	7.7	20.8	4.5
April	15.1	29.8	54	1	9.3	20.7	4.2
May	10.4	28.2	47	1.4	9.8	18.8	3.87
June	7.1	25.3	47	1.3	9.3	16.8	3.15
July	7.1	25	44	1.3	9.6	17.8	3.2
August	9.9	28.3	37	1.5	10.1	20.6	4.18
September	14.7	32.2	30	1.6	9.8	22.9	5.36
October	18.6	34.2	33	1.5	9.4	24.3	6
November	19.5	32.7	45	1	7.7	22.5	5.25
December	19.2	30.2	55	1	6.7	21.1	4.81
Average	14.9	29.7	48	1.2	8.7	20.8	4.5

Table 17 Input climate data to determine ETo using Penman method of FAO CROPWAT computer model

4.3.2 Effective rainfall

Effective rainfall is part of rainfall, which is effectively used by the crop after rainfall losses due to surface runoff and deep percolation have been accounted for. The CROPWAT Program uses the dependable rain (FAO/AGLW formula) method for computing effective rainfall, which is a function of consumptive use of the crop under consideration and net depth of irrigation applied to the soil. The details are shown in paragraph 9.2.

4.3.3 Cropping pattern

The extent of the command area that could be irrigated by the sprinkled water depends on the types of crops grown and the cropping calendar. As food security, diversified agriculture and contribution to the country's GDP is the main objectives of the project, the cropping pattern is planned to be dominated by food, oil and pulse crops.

However, for sustainability and profitability of the project, the crop mix is also considering high value crops including fruit and forage crops. Besides, it must be noted that:

- the pattern is based on farmers' current practices;
- the total suitable area identified by the soils study and latter adjusted by the irrigation engineers is 15,000 ha (both for rain-fed & irrigated crops);
- the choice of irrigated crops is based on food and cash value mix.

The cropping pattern developed by the agronomist is shown in following table.

Crop Type	Wet Season (Nov-Mar)	% of area coverage	Dry season (May- Sep)	% of area coverage	Remarks
Grain crops	Sorghum	45*	Maize Wheat	40* 30*	
Oilseed	Sunflower	25*	Soybeans	20*	
Pulses	Beans	20*			
Fruit trees	Citrus, Mango	40**	Mango	40**	Grows all year round
Forage	Alfalfa	10*	Alfalfa	10*	Grows all year round
* Area based on suitability ** Area included on improved management basis (40% of 25,000 ha)					

Table 18 Cropping patterns for wet and dry season

Moreover, the cropping patterns for both wet season (Sorghum 6, 750 ha, Sunflower, 3,750 ha and Beans, 3,000 ha) and dry season (Maize, 6,000 ha, Wheat, 4,500 ha, Soybean 3,000 ha, Alfalfa 1, 5000 ha and Fruit, Mango 10,000 ha) along with crop water demand are presented in paragraph 9.2.

4.3.4 Net irrigation requirements

The net irrigation water requirement is determined based on the crop water requirement and effective rainfall. The scheme water requirements are computed as aggregate of the crop water requirement. The irrigation schedules for each crop are computed based on the type of soils and the crop growth stage.

A detailed methodology, calculations, analysis and summary of the crop and irrigation water demand is given in agronomy report of this study. The details of the net irrigation for both supplementary and full-fledged irrigated crops are given in paragraph 9.2.

4.3.5 Irrigation efficiency

The amount of water stored in the root zone is estimated as the net irrigation dose. However, during the irrigation process, considerable water loss occurs through seepage, deep percolation, etc. The amount lost depends on the efficiency of the system.

Irrigation efficiency is the efficiency of the total process of irrigation from the source of the water to the point where the water becomes available in the root zone of the plant. To account for losses of water incurred from the sprinkler and drip systems, an efficiency factor should be included when calculating the gross irrigation requirements. Hence, 80% application efficiency was taken for designing sprinkler and 90% for drip irrigation system in the project area.

4.3.6 Gross irrigation requirements

The gross irrigation requirement is computed by dividing the net irrigation requirement by the respective irrigation efficiency for both centre pivot and drip irrigation systems. Moreover, gross irrigation requirement for all crops under study is shown in paragraph 9.2.

4.3.7 Irrigation duty

The peak requirement in terms of duty in litres/sec/ha was calculated to determine the command area and system capacity of the conveyance and distribution systems based on the estimated monthly gross irrigation demand. Details of irrigation duty calculations are given in agronomy report.

The irrigation water duty was determined from the peak irrigation duty of the peak months of four crops in the case of sprinkler irrigation. Then based on the weighted average area method, the irrigation duty was calculated as 0.71 l/s/ha. This is based on 24 hours irrigation duty of the system. This does not mean that the system is working for 24 hours. However, operation time is 22 hrs in the case of centre pivot sprinkler irrigation system.

The procedure on how the peak irrigation duty for centre pivot irrigation system was determined using weighted average area method is as follows:

Selected crops for center pivot with thir respective area are: Maize (6,000 ha) , Wheat (4,500 ha), Soybean (3,000 ha) and Alfalafa (1,500 ha).. The respective peak irrigation duty during the peak months are , 0.71 l/s/ha on September, 0.62 l/s/ha August, 0.82 l/s/ha September and 0.77 l/s/ha October.. The weighted average area irrigation duty is calculated as:

$$W.A.\text{duty} \quad = \frac{(\,6{,}000\ X\ 0.71\,) + (4{,}500\ X\ 0.62) + (3{,}000\ X\ 0.82) + (1{,}500\ X\ 0.77)}{(6{,}000 + 4{,}500 + 3{,}000 + 1{,}500\,)}$$

Taking the weighted average area irrigation duty of 0.71 l/s/ha, the gross irrigation depth was determined as

$$d_{gross} \;=\; \frac{0.71}{0.116} \;=\; 6.11\ mm/day$$

The net irrigation depth was calculated as

$$d_{net} \;=\; \frac{6.11}{0.8} \;=\; 4.88\ \ mm/day$$

Where,

- 0.116 = conversion factor from l/s/ha to mm/day
- d_{gross} = net irrigation depth, mm/day
- d_{net} = net irrigation depth, mm/day

Details are given on Paragraph 9.2.

Moreover, irrigation duty for drip irrigation system was determined as 0.64 l/s/ha on 24 hrs basis from the peak month of October for Mango crop. However, the operation time for drip irrigation system is designed as 21 hrs. The detail calculation procedure is shown on agriculture report and Paragraph 9.2.

4.4 SPRINKLER IRRIGATION SYSTEM DESIGN

The main field factors to be considered when designing centre pivot irrigation systems are: the seasonal and peak water use rate of the cropped area; soil infiltration and moisture holding characteristics; crop characteristics and their water-versus-yield relationship; anticipated effective rainfall; field topography and boundaries; water supply quality and quantity; equipment and operating costs; and various other economic parameters.

4.4.1 Gross application depth

The net application depth for center pivot irrigation system was determined from the peak irrigation duty of the peak months of four crops during dry season and three crops for wet season supplementary irrigation in the case of sprinkler irrigation. Then based on the weighted average area method, the net irrigation requirement was calculated as 4.88 mm/day.

Before calculating the gross irrigation requirement, the need of Leaching ration, LR has been evaluated using the standard equation

$$LR = \frac{EC_w}{5\,EC_e - EC_w} \qquad \text{(equation 1)}$$

where:

- LR = Leaching ratio
- ECw = Electrical conductivity of irrigation water (dS/m) Zambezi river
- ECe = Estimated electrical conductivity of the average saturation extract of the soil root zone profile for an appropriate yield reduction (dS/m)

Taking the average values of ECe = 0.53 dS/m and ECw of 0.09 dS/m on equation 1:

i.e. 0.9 dS/m / (5 X 0.53 – 0.09) = 0.035

Hence, the calculated leaching ratio comes 0.035 i.e < 0.1, this indicates that the rainfall and applied water is sufficient to leach out and there is no need of additional water for leaching purposes.

Taking the leaching requirement LR ≤ 0.1, the gross application depth was determined using equation 2:

$$d' = \frac{k_f U_d}{E_{pa}/_{100}} \qquad \text{(equation 2)}$$

where:

- LR = Leaching requirement ratio (decimal)
- Kf = frequency factor (Keller and Bliesner, 1990) to adjust standard crop water-use values for high frequency irrigation (decimal). Kf was taken as irrigation frequency

factor of one, taking the small grains under peak season with in the maximum irrigation interval of 4 days

- Ud = the peak-use ET rate of the crop, mm/day. This is the net crop water requirement of 4.88 mm/day
- EPa = 80 % of application efficiency

The resulting gross application depth (d') is 6.11 mm/day.

4.4.2 System capacity

The total system capacity under for centre pivot without corner attachment and /or end guns are determined using equation 3:

$$Q_s = \frac{L^2 d'}{K_1 T} \qquad \text{(equation 3)}$$

where:

- Qs = total system discharge capacity (l/s)
- K1 = (3,600 s/hr) / π = 1,146
- T = Average daily operating time (hrs /day), 22 hrs/day
- L = irrigated radius in circle when no end-gun or corner system is not available (m), 454 m
- d' = average daily gross depth of water application required during peak water-use period (mm), 6.11 mm /day.
-

$$Q_s = \frac{454^2 \text{ X } 6.11}{1146 \text{ X } 22} = 49.95$$

$$Q_s = 50 \text{ l/s}$$

4.4.3 Irrigation scheduling

According to Keller and Bliesner, 1990, scheduling could be based on infiltration rate and soil moisture holding capacity. In the study area, the soil is coarse textured with high infiltration rate and low soil moisture holding capacity.

Under centre pivot irrigation, the soil-water deficit various by ± dn around the average deficit, when dn is small, the cycle time is normally 1 < Tc < 4 days. Moreover, Keller and Bliesner, 1990 recommended that for coarse textured soils with water holding capacity less than 120 mm/m, most of the Wa is readily available, so, MADa = 30%. The net depth of water is calculated with equation 4:

$$d_n = \frac{MAD \text{ } X \text{ } Z \text{ } X \text{ } W_a}{100} \qquad \text{(equation 4)}$$

where:

- dx = Maximum net depth of water to be applied per irrigation, mm

- MAD = % management allowed deficit,
- Wa = Available water holding capacity of the soil, mm/m
- Z = Effective root zone depth, mm

$$d_n = \frac{30 \text{ X } 86.39 \text{ X } 0.8}{100} = 20.73$$

Using equation 4, net depth of water application per irrigation was determined from the soil parameters , given that MAD = 30%, average water holding capacity of the three soil types (Sandy clay loam, Sandy loam and loamy sand) in the project area, Wa = 86. 39 mm/m, average effective root depth of four crops, Z = 0.8 m, dn = 20.73 mm

Irrigation interval is the time that should elapse between the beginnings of two successive irrigations. Hence, irrigation interval is determined with equation 5:

$$f' = \frac{d_n}{U_d} \qquad \text{(equation 5)}$$

Where,

- f' = irrigation interval or frequency, days
- dn = net depth of water application per irrigation, to meet consumptive use requirement
- Ud = conventionally computed daily crop water requirement, or use rate, during the peak use month, 4.88 mm/day

$$f' = \frac{20.73}{4.88} = 4\ days$$

Determining the irrigation interval with equation 5, using dn = 20.73 mm and Ud = 4.88 mm/day, f' = 4 days, which is in the range of maximum irrigation frequency given by Keller and Bliesner (1990).

Net irrigation demand (mm/day)	4.88
Efficiency (%)	80
Gross demand (mm/day)	6.11
Actual fraction of irrigation time (0.9)	0.9
Area (ha)	65
Discharge for one center pivot unit (l/s)	50
Discharge Q, (m^3/ h)	180
Radius of the center pivot unit , m	454
Diameter of spray	8m

Table 19 Summary of design criteria used for design of centre pivot sprinkler package

Center pivot sprinkler	Specifications
Area of center pivot	65 ha
Diameter of pivot	908
Span	54.53 m
Overhang	13.41 m
Net irrigation requirement	4.88 mm/day
Gross irrigation Requirement	6.1 mm/day
Discharge at the pivot inlet	50 l/s
Pressure inlet available at connection with pivot	45 m / 4.5 bar
Pumping hour design	22 hr
Type Of Wheels	Tubeless Tire two wheel
Sprinkler Spacing	2.2 m
Total area is	15,000 ha

Table 20 Specifications of centre pivot sprinkler package

4.5 DRIP IRRIGATION SYSTEM DESIGN

This section of the design report deals with Type plot A and B, for drip irrigation development system of fruit. This typical design of commercial pressurized irrigation system model plot B irrigation site is located in Chobe District, Botswana.

4.5.1 Water requirement for fruit trees

The determination of water requirements for wide spaced tree crops like fruits are different from vegetable crops especially when it is irrigated with drippers. It is calculated litter per day per plan using equation 6:

$$\text{Peak Water Requirement } = \frac{\text{Crop area xETo x Kc x wetted area}}{\text{E}} \quad \text{(equation 6)}$$

Where

- The crop area (m²) = row to row spacing (m) x plant to plant spacing (m). As per the information from Agronomist, the respective spacing between row and fruit tree are taken as 10 m x 8 m and is being implemented in the centre used for design of the dripper system.
- ETo= Potential Evapotranspiration of the location. As calculated considering effective rainfall, a maximum of 5.85 mm/day on October is estimated for the fruit trees.
- Kc= Crop coefficient, as shown in the crop calendar (paragraph 9.2.8) a maximum of 0.85 is estimated for mango trees in the month of October.

- In equation 6, ETo*Kc represent ETc, Etc - Eff. Rain fall= Net irrigation. So our net irrigation is 4.97 mm/day, representative ETo will be 4.97/0.85= 5.85mm/day
- Wetted area is the area which is shaded due to its canopy cover when the sun is overhead, which depends upon the stage of growth of plant. Hence, G.Gupta 2002, stated that wetted area under drip irrigation system ranges from 20% for Pomegranate to 50% for other fruit trees like mango. So for the project area ,48% is recommended for design purposes.
- E = application efficiency, for the project area, 90% is recommended for drip irrigation

Therefore, water requirement = (10m x 8m x 5.85 mm/day x 0.85 x 0.48) / 0.9 = 212 litter/day/plant, Hence, 212 litter / day / plant was used to design the drip system for mango fruit trees.

4.5.2 General layout of fruit field

The general lay out of the drip network comprises the topography of irrigable area with its appropriate scale and carefully planned irrigation water supply system. Besides, the design of layout system includes application pipe arrangements (distribution system), number of plots, required valves and operation schedule (Figure 27). The field layout boundaries have been laid out to suit the prevailing soil, the location of the Point of Connection (POC) to the secondary pipe and topographical condition.

There are about 10,544 ha gross area for Mango production. One field is a square setup having 908m x 908m size having 82.45 ha. This was done to fit the drip alignment with that of centre pivot square field. Since the topography of the project, the field shape and size is almost uniform, design of one typical field with six plots is considered for the evaluation and cost estimate of the system.

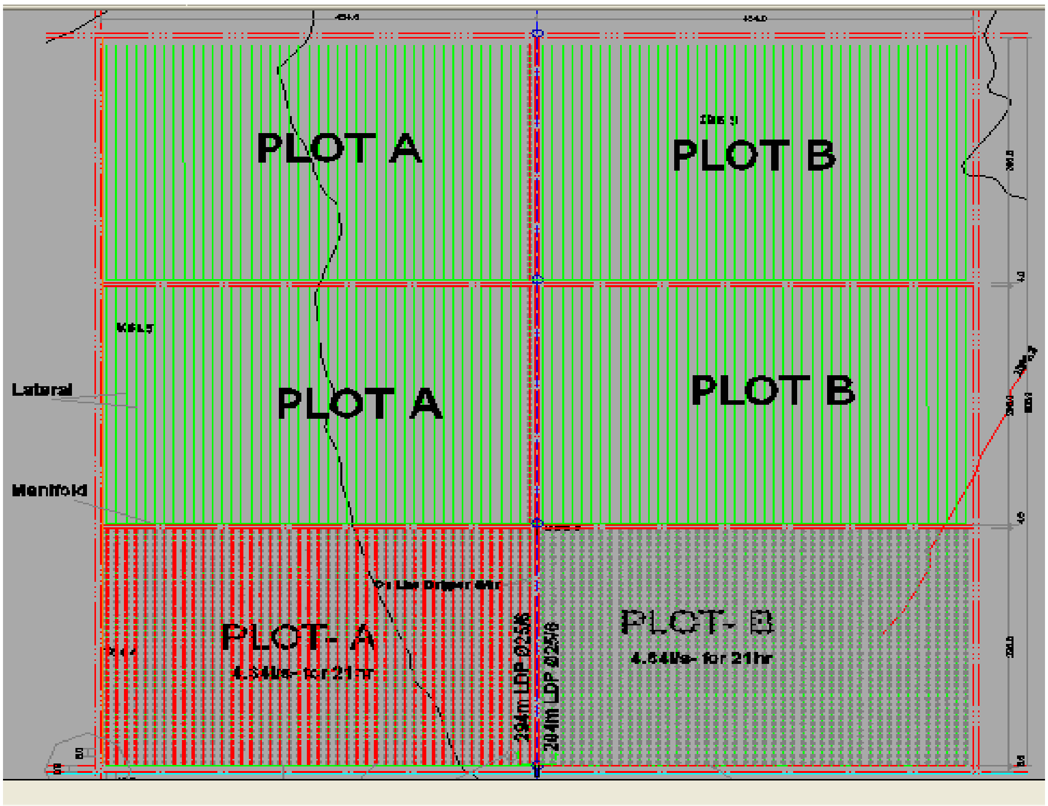


Figure 27 Typical layout of drip irrigation system

There are about 10,500 ha suitable to Mango production. One field is a squire setup having 908m x 908m size having 82.45ha. Since the topography of the project and the field shape and size is almost uniform, design of one typical field with six plots is considered for the evaluation and cost estimate of the system.

4.5.3 Basic design units (BDU) of a cluster

The BDU of cluster, drip type has two plots each with 454 m long and 296 wide having 45 laterals spaced 10 m within net area of 13.38 ha. From the total length of 454 m, the optimum manifold length was designed as 447 m and the remaining are for road and border effects. Moreover, One side of the square size was divided into three taking 296 lateral length each to fit to the tree spacing of 8 m for 37 trees and the remaining are for roads and borders effect.

Shift	Plot area	Lateral (PE)		Manifold (HDPE)		POC
		Length (m)	Diameter (mm)	Length (m)	Diameter (mm)	Discharge (l/s)
Plot A	13.38 ha	296	25	447	110	4.64
Plot B	13.38 ha	296	25	447	110	4.64

Table 21 Design parameters of drip irrigation

As a sample, Basic Design Unit of a cluster selected two plots under one valve having filter, pressure regulator, fertilizer injector and other fittings. These two plots will be operated by the same operating unit and will be supplied each 4.64 l/s water parallel at time making the total discharge at operating unit point of connection as .28 l/s. As the design of shifts is one, within 21hr three operation units having six plots each will be irrigated plot (Figure 28 and Figure 29).

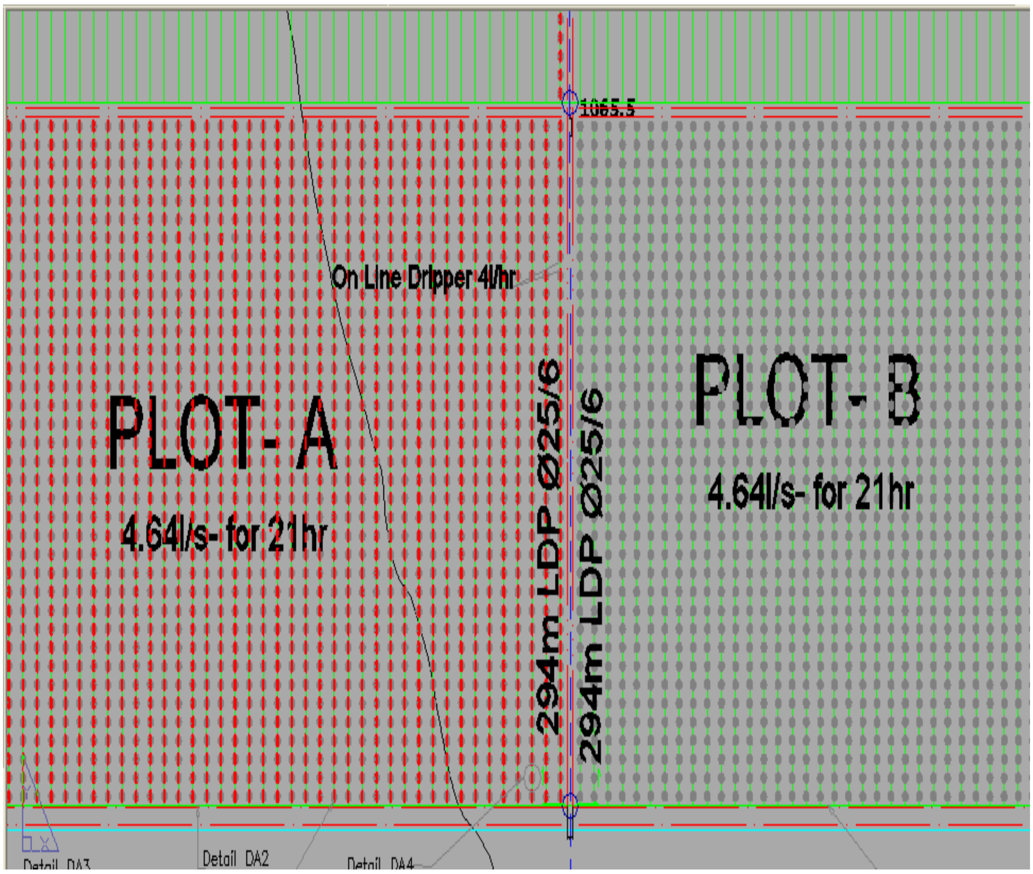


Figure 28 Basic Design Unit of a Cluster, for fruit

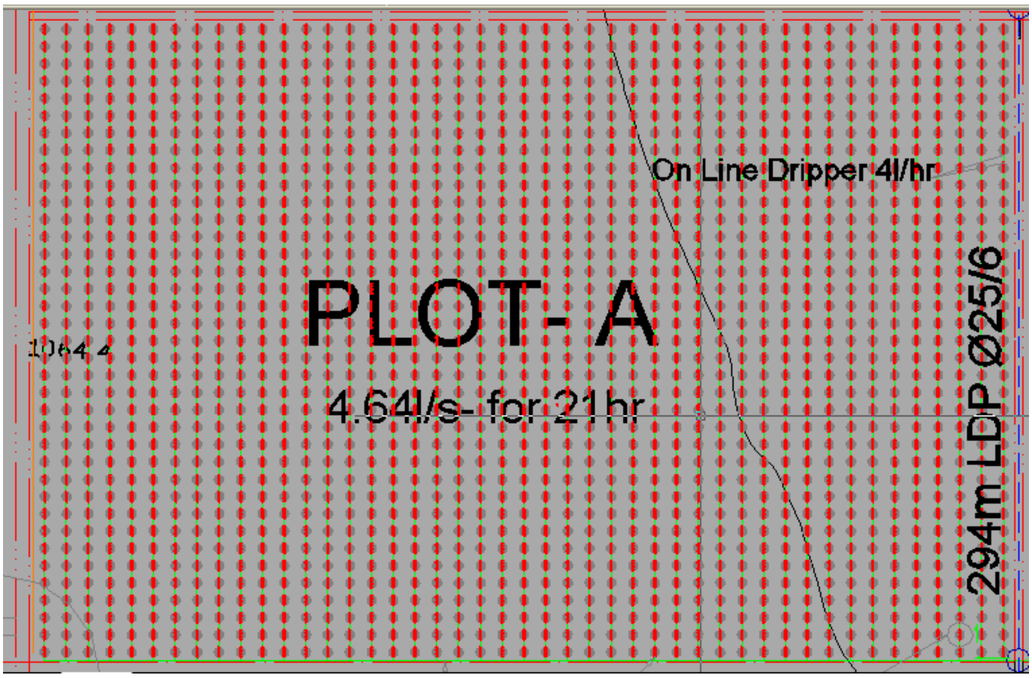


Figure 29 Basic Design of one plot

4.5.4 Selection of drippers, type, spacing and discharge

For fruit trees, dripper type of on line dripper with smooth outlet of pressure compensating type is selected because of attaining the discharge uniformity within the plot of the project unit.

The discharge of an emitter is generally expressed by the power curve equation:

$$q = kH^x \qquad \text{(equation 7)}$$

where:

- H = average emitter pressure head, (m)
- q = average emitter discharge (1/hr)
- x and k = coefficients that characterize a specific emitter.

Usually these are obtained or calculated from the manufactures catalogue or test results. The expected uniformity of application shall be more than 90%. In this specific study, discharge is taken from the manufacturers' catalogue. Assuming that:

- Plant tree is Mango tree, with recommended tree spacing of 10m by 8m;
- One lateral per tree raw;
- Selected dripper type is On-line dripper;
- Drippers each apply 2 l/hr at minimum recommended operating pressure of 1 to 1.5 bar; (discharge pressure relationship from manufacturer's standard)
- Soil is sandy;

Dividing the determined crop water requirement of 212 litter/ day / plant to that of 21 hrs/day irrigation times, 10 litres / hr discharge is to be applied for one plant. Hence, to meet the 10 lit/hr discharge requirement of one plant, 5 drippers are required with a dripper discharge of 2 l/hr.

Therefore, five drippers are designed in a straight line pattern for each tree with a spacing of 1.0 m, which is acceptable for the soil type of present irrigation area. However, if the economy justifies, basin dripper type with 2-meter ring diameter and a spacing of 1m having five dripper of 2 l/hr discharge for each fruit trees is also to be taken as another option.

4.5.5 Design of laterals

The aim of design process is to get uniformity of distribution of the water. This principle is based on the criteria of the 20% rule. It is the maximum difference in pressure between any two emitters, which irrigate in the same operation and should not be more than 20% of the nominal pressure:

$$(dP\ max = <\ 0.2\ Pn)$$

In the case of 10% rule: maximum difference in discharge between any two emitters, which irrigate in the same operation, should not be more than 10% of the nominal discharge:

$$(dQ\ max = <\ 0.1\ Qn)$$

The factors affecting the lateral characteristics considered in the design process includes flow rate, inlet pressure, lateral lengths, and differences in pressure within laterals (due to head losses and due to elevation differences).

The following basic design data have been considered to design the lateral.

- Lateral type – online, single,
- Lateral length – 296 m,
- Lateral internal diameter (I D) – 20.8 mm,
- Emitter sparing /No. of emitters- 1 m spacing / 5 emitters per tree,
- Emitter flow rate - 10 l/hr,
- Average minimum operation pressure at dripper = 10 to 15m,
- Slope of the land – as topography require for each plot.

The discharge at a lateral inlet will be determined by:

$$q_o = n * q_{dripper} \qquad \text{(equation 8)}$$

Where:

- qo= inlet discharge (l/hr),
- N = No of emitters,
- q dripper = Dripper/ emitters discharge (l/hr).

From this, the total number of emitters in a lateral = Number of trees * discharge of a tree = 37 * 10 l/hr/tree = 370 l/hr = 0.37 m^3/ hr. This value of the discharge was taken for design of lateral and head frictional loss analysis.

The head loss is calculated first considering plain pipe. Then the outcome is multiplied by the correction factor F. The value of correction factor F depends on the number of out lets - n, along the laterals pipes and the material from which the pipe is made up of.

Number of outlet	F value	Number of outlet	F value
1	1	12	0.376
2	0.62	15	0.367
3	0.52	20	0.360
4	0.47	24	0.355
5	0.44	28	0.351
6	0.42	30	0.350
7	0.41	40	0.345
8	0.40	50	0.343
9	0.39	100	0.338
10	0.385	>100	0.333

Table 22 Factor F for multiple outlets

The friction head loss are calculated using the Darcy- Weisbach equation:

$$J = 8.38 * 10^6 \frac{Q^{1.75}}{D^{4.75}} \qquad \text{(equation 9)}$$

$$H_f \quad = \quad J * F * \frac{L}{100} \quad \text{......... (equation 10)}$$

where:

- J = head loss expressed by %;
- Q = flow rate (m3/hr);
- D = pipe diameter., mm
- F = factor for multiple outlet given on Table 15.
- L = length of lateral , m
- Hf = frictional head loss, mm

Using the actual topographical contours, the maximum elevation differences have been considered for each lateral operating by the same unit.

The total actual head loss for each lateral is comprised of the friction head loss, turbulent head loss and elevation difference. The whole computation is performed in tabular form and shown in paragraph 9.4.

The final result of the tabular calculation is the approval of the selected lateral diameter, lateral length, minimum operating pressure at lateral inlet and other parameters for the actual irrigation site using 20% rule.

For having uniformity of distribution of the water along the laterals, the criteria of the 20% rule should be fulfilled.

In the present irrigation field case, the maximum pressure variation between two drippers in the same plot area should be less than 20% of the nominal pressure at the outlet of the operation unit.

In the lateral designs as shown in paragraph 9.4, computed and checked the fulfilment of the 20% rule, for all used lateral diameters and lengths.

During computation of each plot and lateral pressure variations, all the actual maximum topography differences in each plots and entire the laterals and manifolds, head losses are considered. The lateral inlet pressure was determined with equation 11.

$$H_i = H_o + \frac{3}{4} H_f + \frac{1}{2} \Delta Z \quad \text{........... (equation 11)}$$

Where,

- H_i = Lateral inlet pressure, m
- Ho = Average operating pressure of dripper, nominal pressure, manufacturer's specifications, m
- H_f = frictional head loss in the lateral, m
- ΔZ = change in elevation in the lateral alignment, m

4.5.6 Design of manifolds

The discharge in each manifold is determined by:

$$qm = (qa * N)\, Nr \quad \text{........... (equation 12)}$$

where:

- qm = discharge in each manifold (l/hr)
- qa = average emitter discharge (l/hr) =2
- N = No. of dripper/ emitters in a lateral =185
- Nr = No. of laterals in a manifold = 45

Using equation 12, the discharge on manifold = 2 X 185 X 45 = 16650 l/hr = 16.65 m3/hr and this value was used to analyse the pressure and discharge in the manifold.

The frictional head loss in the inlet of the manifold was determined using equation 9 and equation 10 and inlet pressure at the manifold is using equation 13.

$$H_m = H_i + \frac{3}{4} H_f + \frac{1}{2} \Delta Z \quad \text{........... (equation 13)}$$

Where,

- H_m = Manifold inlet pressure ,m
- H_i = = Lateral inlet pressure, m
- H_f = frictional head loss in the manifold, m
- ΔZ = change in elevation in manifold alignment, m
- The full analysis and results are presented in paragraph 9.4.

Two main requirements on manifold diameter determination are the following.

First of all, the manifold diameter should be economical. As discussed in the determination of the delivery pipe diameter, the most precise method for determination of the economical diameter is detail computation of the total cost of the pipe at different diameter sizes and select diameter with the least total cost.

Here the total cost of different diameter pipes shall include cost of pipe (conduit) for the whole length, cost of energy, which represent the head losses in the pipe diameter under consideration, other installation, operation, and maintenance costs.

A second issue is that manifold should be quite enough strong against water hammer pressure, which may occur due to sudden closure or sudden opening of valves (mainly at the manifold inlet valve). For this reason, the velocity shall be limited bellow 1.5 to 2.0 m/sec.

The manifold diameter was determined by:

$$Q_m \quad \frac{VD^2}{12.73} \qquad \text{.............. (equation 14)}$$

where:

- Qm = discharge in each manifold (l/hr)
- V = Recommended velocity (m/sec)
- D = Manifold internal diameter

Considering both requirements, the detail analysis of economical diameters, for the manifolds pipes, at each plot is performed in paragraph 9.4.

4.6 MAINTENANCE OF IRRIGATION SYSTEMS

The purpose of an irrigation scheme is to achieve a higher crop production level through appropriate management and planning of water distribution practices. This can be achieved through better and organized project management practices.

Good operation of any irrigation system includes matching the irrigation duration with the rate of application and the intake rate of the soil to maximize the fraction of water stored in the root zone. If a field is under-irrigated, all the infiltrated water could be stored in the root zone, giving apparently high irrigation efficiency even though the water distribution uniformity across the field may be poor.

Conversely, an over-irrigated field will have low irrigation efficiency even if the irrigation appears to be uniform, because of the deep percolation. Thus, knowledge of the soil moisture content prior to irrigation is essential to maintaining high application efficiency while providing sufficient water for optimum crop growth.

Proper maintenance involves anticipating the need for repairs and replacement of worn-out mechanical parts and damaged or broken pipes. Spare parts of commonly needed items should be kept on hand for emergencies. Periodic inspection of supply pipes, mechanical equipment (such as pumps, nozzles, emitters and filters) and distribution systems should be made throughout the irrigation season.

It is important to perform preventative maintenance in the fall, winter, and/or early spring in order to be ready for the next irrigation season. An audit or evaluation of the irrigation system is recommended if the system is not as efficient as it should be. An audit determines the depth of water being applied and distribution uniformity. If a pump is used, it is tested to determine fuel or energy use efficiency.

4.6.1 Sprinkler

Regular maintenance of sprinkler equipment will reduce repair costs, help the system last longer, and keep irrigation efficiency at design levels. Each manufacturer provides guidelines and manuals for equipment operation and maintenance. Such information is the preferred source and should be referenced when performing irrigation equipment repair and maintenance.

Sprinkler systems should be inspected and any necessary repairs completed prior to the start of the irrigation season. All irrigation systems should receive special attention at the end of each irrigation season. During the fall, while water is still available for operation, it is advisable to run the sprinkler system and look for problems. This will allow to plan for any needed maintenance well in advance of the next irrigation season. Check all nozzles for plugging, mismatched sizes, breakage, corrosion or other damage caused by wear and tear. Couplers and connections should be checked for leaks and repairs/replacements should be completed as soon as possible.

If a sprinkler system has been properly prepared for winter storage, spring maintenance is much easier. Often local irrigation supply companies provide a fall or winter tune-up service at a reasonable cost. If the field is used for pasture, careful attention should be given to protecting the irrigation system from livestock damage.

Sprinkler package selection is a major topic when making the original purchase of the center pivot, but it is just the first decision related to managing the center pivot year-in, year-out. To be effective, the

Sprinklers must continue to run properly which means that when wear and tear causes the sprinkler to malfunction, repair or replacement is necessary. Once installed, it is more important to ensure that each sprinkler continues to operate as it was designed.

Field topography and pumping plant performance can have major impacts on the performance of a sprinkler package. Sprinklers may be damaged by a myriad of issues at any time after the original installation. Failures to replace damaged sprinklers or remove materials that may plug nozzle openings allow the water application to be affected in a negative manner for extended periods. Keeping good records on pumping plant performance and performing a simple sprinkler system check on a regular basis will help ensure that the system is operating efficiently.

The design sprinkler flow rate out of each sprinkler orifice is based on the water pressure supplied to the sprinkler inlet. Overall, the discharge delivered by a sprinkler also depends on the system capacity, the distance from the pivot point to a specific sprinkler, and the spacing between sprinklers at that location on the lateral. The goal of the sprinkler package selection or design process is select nozzles that would apply water with over 90% application uniformity.

The nozzle diameter has a big influence on the discharge from the nozzle since the discharge depends on the square of the nozzle diameter. Depending on the construction material of the nozzle and the quality of water being pumped, the nozzle opening could change. If the nozzle opening increases due to wear, the actual flow rate may be vastly different than the original design.

What are the problems associated with center pivot sprinkler operation?

The most obvious answer to this question is that over time various parts of the sprinkler can become worn-out to the point where it no longer distributes water over the same wetted area in a uniform manner. However, in some cases the original installation can be the issue.

It is always good to conduct the inspection just before sunrise and sunset as the angle of light from the sun makes it easier to identify water application problems. Each sprinkler should be operating and look very similar to the sprinkler next to it. If not, the regulator or nozzle opening could be partially plugged.

Each of the issues described above could have been identified by a simple three-part inspection, which is best done in the spring before the crop canopy is present:

1. Verify that the correct flow rate and operating pressure supplies the system,
2. Compare the sprinklers sizes installed to the sprinkler design printout,
3. Verify that the sprinkler is not cracked or broken and that the deflection pads are not worn excessively.

Why is water application uniformity important?

The original sprinkler package design will normally have water application uniformity above 90% when operated under no wind conditions. Reduced water application uniformity means that some areas of the field are not receiving the correct amount of water.

If any of the issues discussed above are present the non uniformity can occur each time water is applied and the accumulative impact is that grain or forage yield can be less than expected. Often times small problems that affect only a few sprinklers may not be noticeable in yield maps while others can easily be seen from the air.

4.6.2 Drip

Flush the system at the beginning of the growing season and check to be sure the emitters are not clogged. Do this by opening the ends of the tube and running clean water through the system, starting with the lines closest to the supply source. Once the tubes have all been checked and sealed again, check for flow from each emitter. Regular flushing of the system throughout the season may be necessary depending on the cleanliness of the water supply and filtering system. This will help remove larger mineral and organic matter particles that can clog emitters. To keep the small openings in low-flow systems from becoming clogged, the water source must be properly filtered. The cleanliness of the irrigation water will determine how often the filters should be checked and cleaned. If continual clogging is a problem, it may be necessary to select finer screens or use a sand filter or chemically treat the water.

4.6.3 Filters

Check the filters regularly and frequently until the best cleaning schedule for the system can be determined. The frequency of cleaning the filters may be greater in the spring when more debris is in the water. Back flushing, or removing the filters and washing them out backwards is the most common way to clean most filters. Replace the filters when they get holes or openings too large to filter out damaging or clogging particles. Organic matter slipping past the filter or algae growing in pipes or fittings may cause serious system problems, especially when the source is a secondary water system. Opening the end of the system and flushing will help remove organic matter. If algae growth is a problem, chlorine can be used to kill the algae. Applying a concentration of 10 to 20 ppm of chlorine for 30 to 60 minutes should solve most algae problems. After the algae have been killed, it will need to be flushed as described above.

Both screen and sand media filters in a drip irrigation system should be checked during or after each operating period and cleaned if necessary. A clogged screen or grooved-disk filter can be cleaned with a stiff bristle brush or by soaking in water. A sand media filter should be back flushed when pressure gauges located at the inlet and outlet sides indicate a five psi difference. Check drip irrigation lines for excessive leaking, and look for large wet areas in the planting area indicating a leaking tube or defective emitter. It is also a good practice to flush sub-mains and laterals periodically to remove sediments that could clog emitters. Systems can be designed with automatic back flushing devices and automatic end line flushing devices, but still require manual checks.

4.6.4 Chemical Control Measures

Unfortunately, filtration alone is not always adequate to solve all water quality problems. Chemicals are necessary to control algae, iron and sulfur bacteria, and disease organisms. Chemicals can cause some materials to settle out or precipitate out of the water while causing other materials to maintain solubility or stay dissolved in the water. Chlorine is a primary chemical used to kill microbial activity, to decompose organic materials, and to oxidize soluble minerals, which causes them to precipitate out of solution. Acid treatments are used to lower the water pH to either maintain solubility or to dissolve manganese, iron, and calcium precipitates that clog emitters or orifices. Potassium permanganate also is used to oxidize iron under some conditions. It is recommended to place the filtration system after the chemical treatment to remove any particles formed. Chemigation protection and injection equipment requirements vary with toxicity class of the injected chemicals.

4.6.5 Bacterial Slimes/Precipitates

Bacteria can grow in the absence of light within the system or in a contaminated reservoir. The bacteria can live on iron or sulfur and produce a mass of slime that quickly clogs emitters and filters. This slime can also act as an adhesive to bind other solids together to cause clogging. They also can cause soluble iron and sulfur to precipitate out of the water.

Bacteria cause iron precipitation by oxidizing soluble ferrous oxide to form insoluble ferric oxide. Iron concentrations as low as 0.1 ppm can be troublesome, whereas levels of 0.4 ppm can be severe. The iron precipitate forms as a red filamentous sludge, which can attach to PVC and polyethylene tubing and completely block emitters.

Sulfur in amounts over 0.1 ppm of total sulfides can be troublesome in irrigation water. Bacteria that live on sulfur can produce white stringy masses of slime, which can completely block the emitting devices. Interactions of soluble iron and sulfur can lead to a chemical reaction forming insoluble iron sulfide. Stainless steel filter screens used in high sulfide water can cause iron sulfide precipitation. Chlorination is the usual treatment to kill bacteria or inhibit their activity. A continuous residual rate of 1 to 2 ppm of free available chlorine at the distant end of the irrigation system or an intermittent rate of 10 to 20 ppm for 30 to 60 per treatment cycle should be effective. The initial injection rate may need to be higher to achieve the desired residual level in the system. Treatment cycles may be required at the end of each irrigation cycle for severe water sources or after every 10-20 hours of irrigation for cleaner water sources.

Sometimes, reservoirs are contaminated with bacteria and shock chlorination is necessary to reduce or solve the problem. This is done by injecting chlorine at a rate of 200 to 500 ppm into the reservoir. The volume of water to be treated must be estimated from the diameter and depth of the reservoir.

4.6.6 Algae and Aquatic Plants

Algae and aquatic plants in surface waters can be great nuisances' because they reproduce rapidly during summertime blooms. They have a tendency to become entangled in screen meshes and clog the surface of sand media filters, resulting in frequent filter back flushing. Algae can be controlled in surface waters by adding copper sulfate or other chemicals in an approved manner. Green algae can grow only in the presence of light, so they do not cause a problem in buried pipelines or black polyethylene. However, algae can grow in the white PVC pipe or fittings used to assemble aboveground pipelines and then be washed into laterals and emitters to cause clogging. Chlorine is used to kill algae within the irrigation system. A chlorine concentration of 10 to 20 ppm for between 30 and 60 minutes is suggested. It is advisable to work section-by-section through the pipeline and flush the dead algae out of the pipes immediately after treatment, to prevent emitters clogging. If significant emitter clogging occurs, a higher concentration may be needed to decompose the organic matter in the emitter.

4.6.7 Chemical Precipitation of Iron

Water with over 0.1 ppm of iron is quite likely to cause a problem in irrigation systems. The problem can be solved by either removing the iron from the water or by retaining the iron in solution.

4.6.8 Chlorine precipitation

Free chlorine will instantly oxidize ferrous iron to ferric iron and take it out of solution as a solid. The iron concentration must be determined, and chlorine must be injected at a rate of 1 ppm for each 0.7 ppm of iron. Some additional chlorine may be needed for other contaminants, such as iron bacteria and bacterial slime. Complete mixing of the chlorine and water is necessary and can be accomplished by creating turbulence in the system before the filter. A sand media filter is the most appropriate choice and should be backwashed frequently, preferably automatically.

If manganese is present in the water source, caution must be exercised, because oxidation of manganese by chlorine occurs at a much lower rate. Care must be taken to precipitate the manganese before the filter, or clogging problems could occur.

4.6.9 pH Control

Iron is more soluble at lower pH values. Acid can be continuously injected to keep the pH low in the irrigation system or can be used periodically to dissolve iron deposits. To dissolve the iron, the pH must be reduced to approximately 2.0 or less for a period of 30 to 60 minutes. The system must be flushed to remove the iron after treatment.

Iron precipitation can be caused by raising the pH. A solution to increase the pH can be prepared by mixing 3 pounds of soda ash (58 percent light grade) with 4 gallons of water. This neutralizing solution can be injected into the water system and can be mixed with chlorine solutions.

4.6.10 Iron Sulfide Precipitation

Sulfur-bearing minerals are common in most sedimentary rocks. A soluble form of sulfate is carried by water. Sulfates are difficult to precipitate and generally remain in solution. Sulfate can be used as a food source by bacteria, which produces hydrogen sulfide gas as a by-product. If sufficient iron is present under moderate reducing conditions, iron sulfides can be precipitated, and a sand media filter is suggested to remove the precipitate.

4.6.11 Precipitation of Calcium Salts

Calcium salts, particularly calcium carbonates, precipitate out as a white film or plating in the system. The salts are soluble at low pH. Acid can be used to maintain a pH of 4.0 or lower for 30 to 60 minutes which dissolves calcium deposits to clean emitters and pipelines. Hydrochloric (muriatic) acid is recommended for treating calcium blockages although sulfuric and phosphoric acid can also be used. Temperature, pH, and calcium concentration are all factors influencing calcium solubility, so conditions can vary throughout the irrigation system. Water sources differ in the amount of hardness and/or pH requiring different amounts of acid to lower the pH.

The most common acid that growers will find available is muriatic acid (20% hydrochloric acid) at hardware and farm supply stores. Make sure that you flush and clean the injector after acid application since the acid may be corrosive to internal parts. Allow the acid treated water to remain in the pipe lines for 30 minute to 1 hour, and then flush with water. Use extreme care in handling acids and always add acid to water.

In general, the maintenance of drip system centers on identification of the factors, which can lead to reduction of the performance of drip system and procedures to mitigate these negative impacts. Factors that can slow or stop flow through the drip system include; suspended material, chemical precipitation, biological growth, root intrusion, soil ingestion and crimping of the drip line. To ensure maximum system life reduces or eliminates the impact of the negative factors (Table 1). This may require water treatment and a systematic program for regular maintenance. In this section, we outline the various potential issues that can adversely affect the drip system and offer procedures to mitigate the potential damage.

Indication	Possible problem
Gradual decrease in flow rate	Dripper plugging Possible pump wear (check pressure)
Sudden decrease in flow rate	Stuck control valve Water supply failure
Gradual increase in flow rate	Incremental damage to dripper line by pests
Sudden increase in flow rate	Broken lateral, manifold, secondary or primary lines Pressure regulator failure
Large pressure drop across filters	Debris buildup in filters Inadequate flushing of filters
Gradual pressure decrease at filter inlet	Pump wear or water supply problems
Sudden pressure decrease at filter outlet	Broken lateral, manifold, secondary or primary pipe line Pressure regulator or water supply failure
Gradual pressure increase at filter outlet	Dripper plugging
Sudden pressure increase at filter outlet	Stuck control valve , Other flow restrictions
Sudden pressure decrease at manifold	Damaged or broken lateral

Table 23 Drip induction and possible problems

4.7 REFERENCES FOR IRRIGATION SYSTEM DESIGN

- Jack Keller and Ron D. Bliesner, 1990Sprinkle and Trickle Irrigation. An avi Book. Published by Van Nostrand Reinhold, New York
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5 ON DEMAND PRESSURIZED PIPED IRRIGATION SYSTEM

5.1 INTRODUCTION

The present feasibility study for an irrigation project in Pandamatenga takes into account what has been already designed in the *"Preliminary Design Report on Utilization of the Water Resources of the Chobe/Zambezi River"* redacted by WRC in November 2013.

In this document the water transfer scheme (WTS), shown in Figure 30, has been described: after the water withdrawal from Chobe River a series of pumping stations, reservoirs and pipes are aligned for about 580 km in order to join the Botswana's North-South Carrier (NSC), that supply drinking water to Gaborone. In particular, this project foresees:

- abstraction of 495 million m^3 of water per annum from the Chobe/Zambezi River System at Kazungula in the Chobe District, Botswana;
- requirement for about 345 million m^3 of water per annum for agricultural purposes, mainly for the present ZIACD project;
- future water deficit in urban centres to be at least 100 million m^3 per annum; conveyance of 150 million m^3 from the Pandamatenga Reservoir through inter alia Nata, Francistown and Tonota to be discharged into the existing North - South Carrier Water Project at Break Pressure Tank (BPT1) located at Moralane area near Selebi – Phikwe.

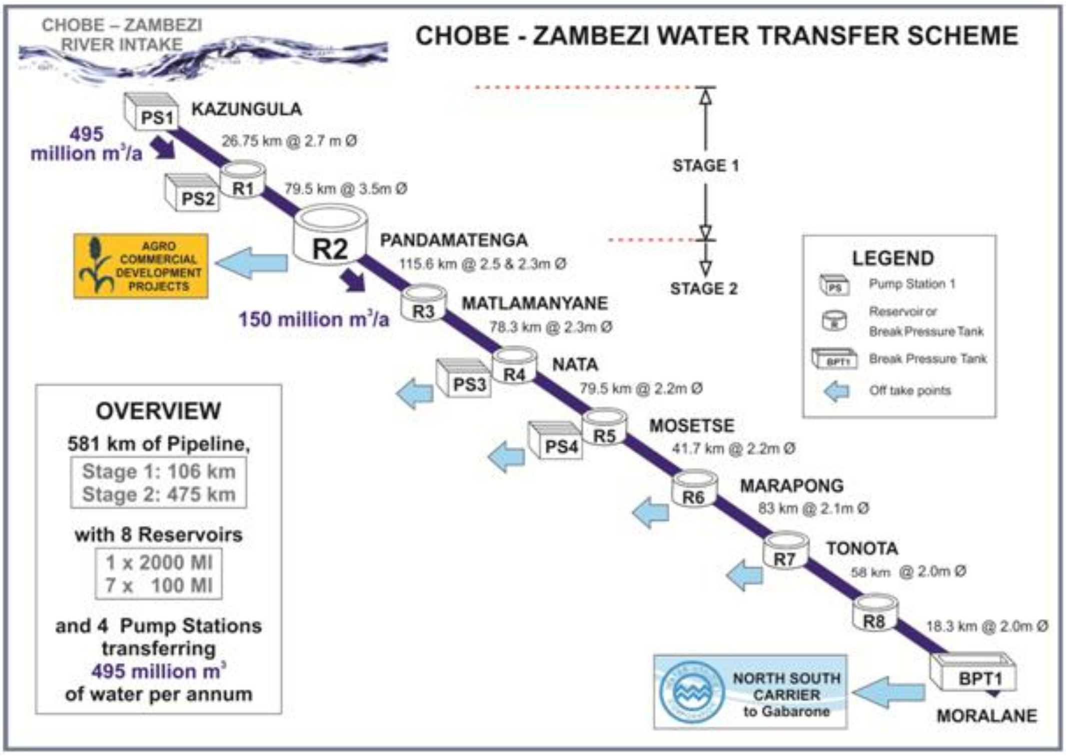


Figure 30. The already designed Chobe – Zambesi water transfer scheme ("Preliminary Design Report on Utilization of the Water Resources of the Chobe/Zambezi River" by WRC, 2013)

The present irrigation project, according to the above figure, takes water from reservoir R2, that has capacity of 2.0 Mm^3 and is supplied by a pipeline with diameter 2.7 m and design discharge of about 23.3 m^3/s.

5.2 IDENTIFICATION OF PIPELINES LAYOUT

The study related to agronomy and on farm irrigation (paragraph 3) have lead to evaluate the water requirements for crop, border the command area on the basis of soil suitability and identify the size and type of irrigation system.

From the topographic point of view, the reservoir R2 has inlet at 1079 m a.s.l. and water top at 1090.20 m a.s.l.. The irrigation block are generally lower than reservoir R2, except for the eastern zone of the study area, both next to the reservoir and in the southern zone: however, this area has already been excluded by the land evaluation study.

The typical layout of pressurized system, that supplies water to the fields, is composed by primary lines (main pipes) and secondary lines (submain pipes) that cross the fields bringing water at the center pivots or drip Point of Connection (PoC). The layout of the primary pipes has been defined taking into account also the following criteria and issues:

- Start from reservoir;
- Shortest lengths to reach farthest part of study area;
- Following the natural slope down of terrain;
- Being as high as possible;
- Verifying the grade of terrain slope (it is fine if it is equal or more than water energy slope, likely 1-1.5 m/km).

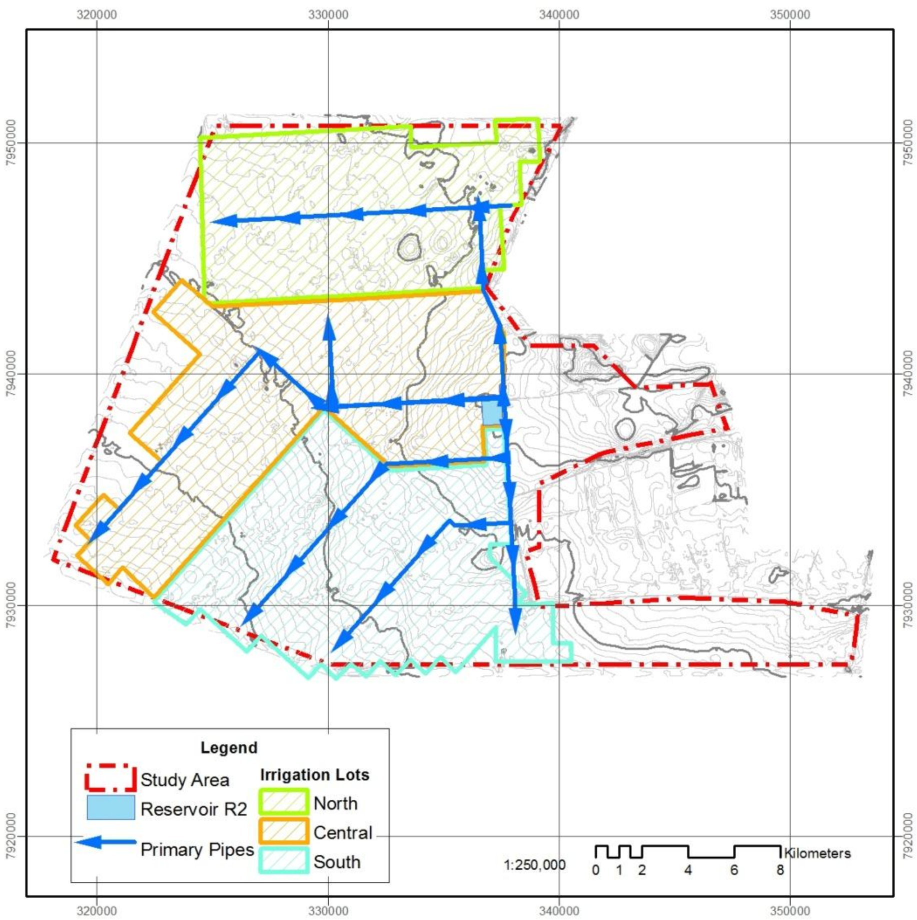


Figure 31. Layout of primary pipes and irrigation lots

The water supply scheme is branch type, instead of being closed, because for the present project it represents the optimal compromise between costs and reliability. For what concerns economic matter there are many facts that lead to money savings, while in terms of system efficiency the main issue is that the agronomic study has envisaged an irrigation cycle of 4 days. This means that crops are watered during a day then there are 3 days of interval for the next watering: during this period any local failure, that might occur, would be solved without interfering with agricultural practises.

About costs, first of all, the branch type allows to reduce the length of the pipes to the lowest value and also the required dimensions of the pipes are the smallest: this because each filed is reached by only one primary system and there is no terminal portion of a pipeline system that need to work also as an intermediate part of another one.

If this latter condition is considered, the loop pipeline could be designed either with a large diameter, causing high cost of installation, or with small size, increasing energy expense and requiring more powerful pumping system: this because head losses would be higher. Then loop system need a larger number of cut-off valves.

The implementation of a closed network for the present irrigation system would need about 30 km of further pipes, beyond the 85 km that are currently envisaged. This would increase of a third the present estimation of cost for primary pipes included in paragraph 5.3.2.

Finally, it has to be noted that the closed system is typical of the potable water supply where consumers have to be constantly served even terms of water quantity but also for what concerns pressure level. In case of agriculture, thanks to 4 days irrigation frequency and not having the constrain of a continuous service, these limitations are not so strict.

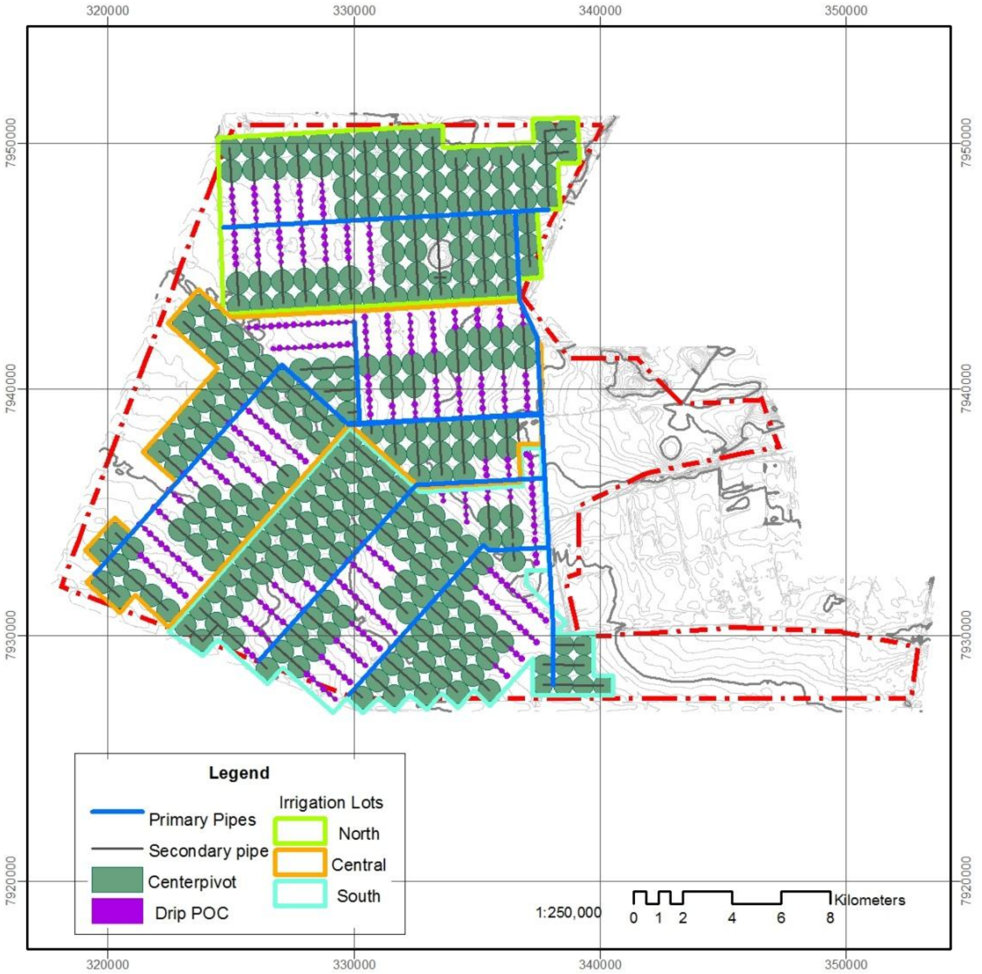


Figure 32. Layout of secondary pipes and irrigation system (center pivot and drip)

The water transfer scheme, described in paragraph 5.1, would cross the irrigation area interfering with infrastructures (irrigated fields, pipes, drains and roads) that are design with the present project that is at feasibility level. Modification to the proposed layout in order to avoid partially or totally interferences would neither affect the concept of the design, nor cause significant variation in dimensioning and cost.

However, if the alignment is required, some fields would be reduced in extension and some secondary pipe would be moved, but it would be worth to evaluate some minor changes even for the track of WTS. Next figures represented the proposed alignment of center pivots, secondary pipes and WTS.

Figure 33. Proposed alignment of center pivots, secondary pipes and WTS

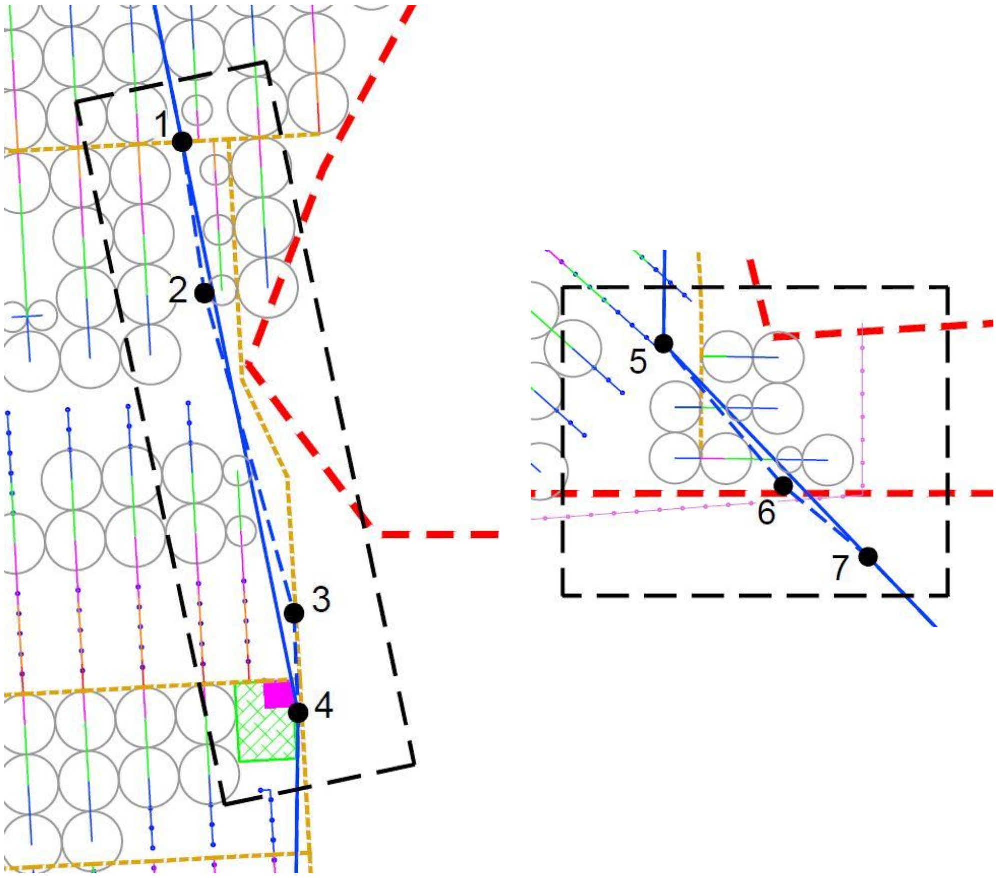


Figure 34. Proposed alignment of center pivots, secondary pipes and WTS (details)

5.3 DESIGN OF PIPELINE NETWORK

5.3.1 General aspects and hydraulic criteria

There are several advantages in using pipelines for conveyance in water distribution projects as well as for distribution on farm. A pressure pipeline system:

- facilitates the use of flexible schedules because of the capacity to transmit pressure;
- is an automated system for transmitting and carrying out precise instructions;
- reduces considerably evaporation and seepage losses;
- can be laid with straight alignments and go up and down hills because it do not have to follow contours;
- can convey water to higher elevation with the use of pumps and lifting plants;
- reduces considerably required right-of way width in comparison with canals.

The selection of the type of pipe, diameters and materials must be done with great care in order to insure that the initial installation is technically and economically acceptable and not limiting for the future. This also because it is very difficult to make changes once pipelines have been installed.

Besides, wrong type of pipe can cause an increase of the construction costs (transportation and installation), a reduction of the system's life, higher annual maintenance and power costs. The diameter is a function of hydraulic issues, like flow rate and friction losses, that then affect energy cost therefore the design is also the sizing of most economic pipe diameters.

Once that water requirements related to crops and pressure issue of irrigation system are set, the design activities proceeds with the definition of the characteristics of a pipe that essentially includes:

- the material the pipe is made of;
- the nominal diameter, which is the one considered for hydraulic calculation;

Hydraulically the material characterizes friction phenomenon through the roughness coefficient, while the diameter determines the flow velocity: both of them are then related to the head losses along the water supply system. These issue are described in the following paragraphs (5.3.2 and 5.4).

5.3.2 Pipe materials

The selection of pipe materials consists in evaluating the characteristic of conveyed fluid and soil where the pipe is supposed to be laid, but also the working conditions or rather the external loads, such as seismicity, working pressure, transient conditions.

Other particular local conditions might be included in this analysis (for instance impervious or densely inhabited areas), however it can be summarized that the water pipes should be analyzed in term of:

- mechanical resistance to internal and external loads;
- resistance of physical, biological and chemical nature of conveyed water and soil;
- flow resistance (smoothness) that must be the lowest possible;
- ease and safety of installation;
- comprehensive optimal cost, considering not only materials and installation but also maintenance and duration.

A rough preliminary evaluation leads to identify small and medium diameters (less than 2000 mm) for the present irrigation system that is characterized by low pressure. In this field of application, in general, pipes can be made of metallic, plastic or composites materials.

<u>Anchoring</u>

It is important to consider the possibility of leakage due to the soil movement and the consequent joint disassembly due to soil settling as well as the effects of the internal pressure that generate longitudinal forces on the pipeline. These forces have to be countered with anchor blocks at vertices, branch offs, reductions and in the majority of cases where the pipeline is tested hydraulically. The need to anchor the pipeline is reduced or does not exist with the steel pipes since the continuity afforded by the welded joint makes them resistant to the longitudinal forces.

<u>Corrosion resistance</u>

Metallic pipes are subject to corrosion both chemical in the ground and from the transported fluid and also due to galvanic corrosion as opposed to GRP that is impervious to corrosion. Corrosion control in steel pipes is made possible with internal and external coatings on the pipes in the factory and during installation as well as during the life of the pipeline with cathodic protection on the external surface. Cathodic protection requires continuous monitoring and in certain areas is even subject to theft.

<u>Resistance to Ageing</u>

The only relevant analysis of the material behaviour is comparing the possible decay of their characteristics with time and with working pressure . For water mains the analysis can be restricted to pressure investigations as the temperature generally remains constant at around 20° C.

The behaviour of metallic materials and GRP are not influenced by the constant application of the pressure forces. Plastic materials display a thermoplastic behaviour, while GRP is designed with higher safety factors to account for ageing, but they are affected by pressure surges that can cause vacuum in the pipeline if it is not protected from it.

Hydraulic characteristics

Steel pipes by virtue of modern manufacturing methods have considerably smooth surfaces. The steel pipes with resin coating have an internal smoothness similar to that of GRP, at least when new. Steel pipes with cement mortar lining have an absolute roughness greater than the GRP pipes and this is important in as much as higher head losses: this has to be accounted for at the design stage. GRP has a polished internal surface and therefore a very low absolute roughness with benefits deriving from better flow, lower sedimentation and incrustation on the internal surface.

Installation

Steel pipes are supplied in random lengths of around 10 m depending on the diameter that determines the weight and therefore the weight limit of the site machinery. The progress is hindered by the welding process and therefore (apart from the high investment in welding equipment) also the continuity of the internal and of the external protection must be restored. Progress is 36 m/day in average.

GRP is lighter to install, does not require any protection, fittings are prefabricated and installed without slowing down the installation, heavy equipment and service roads are not required and with light equipment progress can be in the order of 120 m/day.

In the following table there is a comparison among the mentioned materials considering the main issues related to a pressurized water supply system. It must be noted that given judgments (for instance, lowest) are exclusively relative to the materials that are taken into account, not in absolute.

MATERIAL & PARAMETERS	**Steel**	**Polyvinyl chloride (PVC)**	**Glassfibre Reinforced Plastic (GRP)**
Available sizes *	any	not for mains	any
Laying & jointing	time & experience required	easy & fast	easy & fast
Roughness	low	low	lowest
Pressure resistance	highest	low	low
Corrosion resistance	prone °	resistant	resistant
Maintenance	periodical	periodical	not needed
Durability	low	lowest	high
Basic cost	costliest	low	lowest
Transport cost	high	low	lowest
* Available sizes within the range needed for the present project ° Steel is prone to corrosion: mortar lining and cathodic protection are required			

Table 24 Comparison among materials for pressured water supply networks

The most frequent method used to decide among various acceptable product alternatives is an installed cost comparison. The results of such a restricted focus determination may be misleading since, the installed cost evaluation ignores many other costs which may occur during the lifetime of the pipelines. A reliable comparison must also consider the costs incurred (or avoided) throughout the design life of the irrigation system.

The sum of all costs is called the Life Cycle Cost. This total cost of any item includes costs experienced over the study period to:

- purchase the item
- install it
- operate it
- maintain & repair it
- replace it (if necessary)

The pipe made by Glassfibre Reinforced Plastic (GRP) has the lowest life cost because they:

- are durable and corrosion resistant
- do not produce rust or scale suitable for potable water
- are low in weight (¼ the weight of ductile iron and 1/10 th weight of concrete pipe)
- require no cathodic protection
- require no internal or external coatings
- have flow efficiencies that allow down sizing
- have a design life of 50 years
- have zero maintenance costs
- exhibit low internal friction, resulting into low operating (pumping) costs
- show constant hydraulic characteristics over time
- are suitable for high service pressures and temperatures
- have easy and reliable jointing mechanisms
- are suitable for underground and above ground applications

For all above, the Glassfibre Reinforced Plastic (GRP) might be suggested as the preferable material for pipes for the current project.

5.4 IMPLEMENTATION OF COPAM MODEL FOR HYDRAULIC SIMULATION

The **development process of an irrigation system** follows a systematic chronological sequence comprising the design, construction and management. Instead of seeing this as a "one-way" process, it is important to think about it as an integrated process composed of interrelated phases. The picture below presents key steps of an irrigation scheme development, highlighting the parameters used in options identification and assessment.

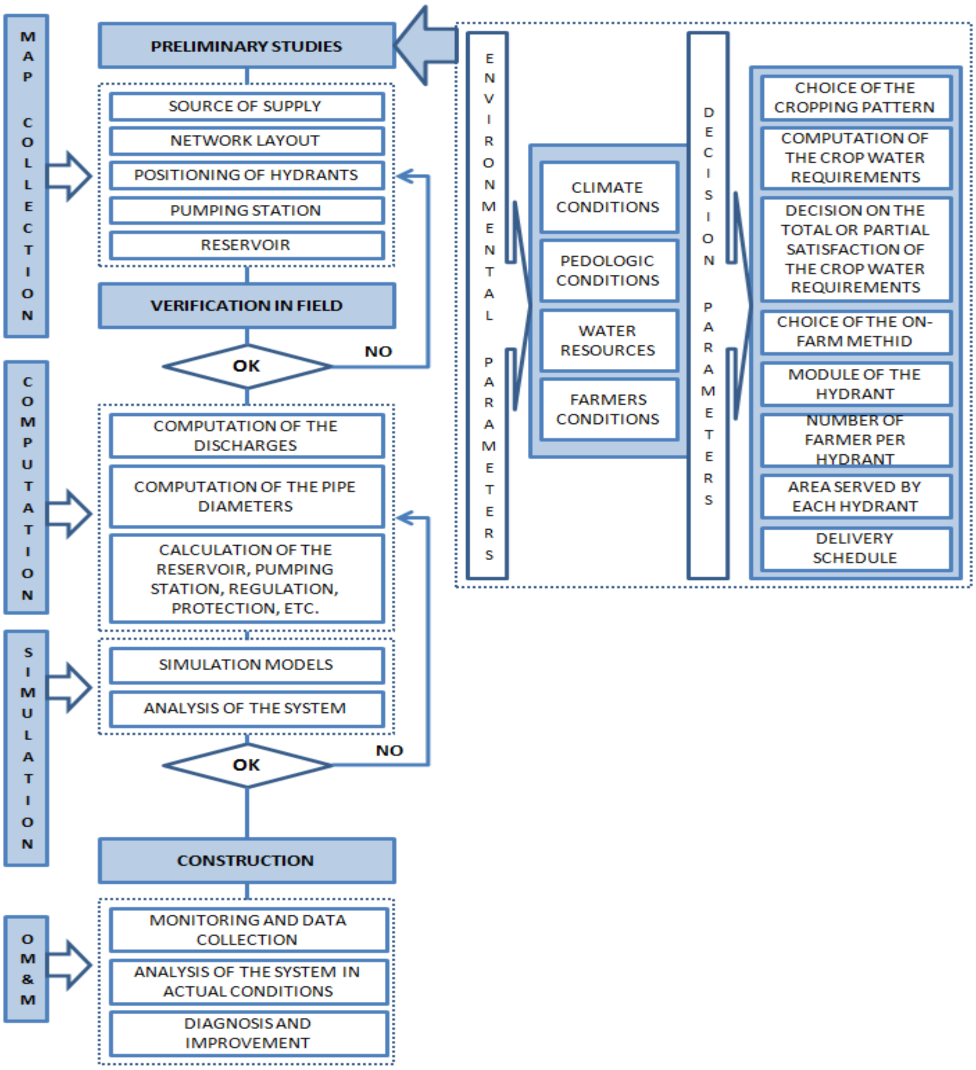


Figure 35. Key steps of an irrigation system development scheme (FAO, 2000)

The software COPAM (Combined Optimization and Performance Analysis Model) provides a computer assisted design mode. One or several flow regimes may be generated and the optimization modules give the optimal pipe sizes in the whole network.

In fact, performance of the resulting design is then analysed according to performance criteria. Based on this analysis, the designer decides whether or not to proceed with further improvements either by a new optimization of the whole system or through implementation of local solutions (such as using booster pumps or setting time constraints for unsatisfied hydrants). The synthetic flow chart of COPAM is presented in the following figure.

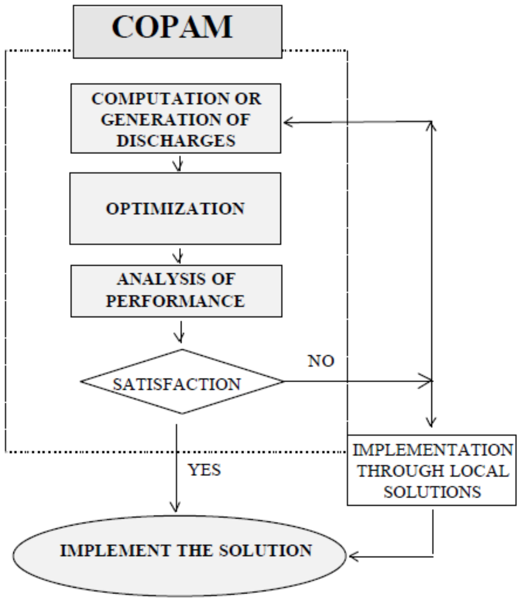


Figure 36. Synthetic flow chart of COPAM software (FAO, 2000)

5.4.1 Theoretical background

The discharges flowing into the network strongly vary over time depending on the cropping pattern, meteorological conditions, on-farm irrigation efficiency and farmers' behaviour. Because of this complexity, empirical methods give only general indications, while the COPAM software implements statistical models aim to compute a single distribution of one or more design flows for each pipe section of the network,

COPAM deals explicitly with calculation of pressurized irrigation systems capacity for on demand operation thanks to a module that implements what has been studied by Clément (1966). In particular, the “first Clément model” is based on a probabilistic approach where, within a population of R hydrants, the number of hydrants being open simultaneously is considered to follow a binomial distribution.

This model, although based on a theory, were extensively used for designing sprinkler irrigation systems in France, Italy, Morocco and Tunisia. In fact, a probabilistic approach for computing the discharges into the sections of an on-demand collective network has been widely adopted because it is not reasonable to calculate the irrigation network by adding the discharges delivered at all the hydrants simultaneously.

Considering hydrants with a different discharge, the total discharge downstream a generic section k is given by:

$$Q = \sum_j R_j P_j d_j + U(Pq) \times \left(\sum_j R_j P_j (1 - P_j) d_j^{\,2}\right)^{1/2} \quad ; \quad P = \frac{q_S \cdot A}{r \cdot R \cdot d}$$

- R: number of hydrants downstream reach j;
- P: probability of the hydrants downstream reach j to be open;
- q_s: specific continuous discharge (l/s/ha);
- A: irrigated area (ha);

- r: coefficient of utilization (for instance r = 0,667 if irrigation is for 16 hours at day);
- d: nominal discharge of hydrants (l/s).
- U defines the "quality of operation" of the network; it normally has a values ranging from 2.324 (99% probability of having efficient system) to 1.645 (risk of 5% of not supplying irrigation water demand). It is the reduced variable for the Gauss distribution law.

The second Clément model (not applied in the present study) is based on some fundamental concepts on the theory of the stationary Markovian processes: the irrigation process is simulated as a birth and death process in which, at a given state j (j hydrants open), the average rate of birth is proportional to (R-j) and the average rate of death is proportional to j.

This hypothesis limits its applicability because it introduce the concept of saturation that can be well applied for designing telephone lines, where if the busy line is engaged the customer has to call later. But for irrigation systems it is not so easy to establish saturation conditions. Furthermore, also when the system is saturated farmers may decide to irrigate with a lower pressure and/or discharge at the hydrant. Finally, the complexity in mathematical approach and the negligible differences in results pushed all designers to apply anytime the first model instead of the second one.

5.4.2 Geometry and input

The pipeline network has been defined in paragraph 5.2 and it is divided in primary and secondary pipes. The two irrigation systems have been set: center pivot and drip irrigation (see paragraph 4.4 and 4.5).

The COPAM assumes the network is of the branching type. Each node (hydrants and/or linking of sections) is positioned by a number. The node numbering is extremely important for the correct execution of the program. It has to be allocated as follows:

- The upstream node (source) must have number 0;
- The other nodes are numbered consecutively, from upstream to downstream. Any node may be jumped;
- The number of the section is equal to the number of its downstream node;
- All terminal nodes of the branches must have a hydrant;
- No more than two sections may be derived by an upstream node. If so, an imaginary section with minimum length (1 m) must be created and an additional node must be considered. This node must have a sequential number;
- No hydrants may be located in a node with three sections joined. If so, an additional node with a sequential number must be added;
- If hydrants with two or more outlets exist in the network, one number for each outlet needs to be allocated by creating an imaginary section with minimum length.

The information that must be input at each node are:

- area irrigated by each hydrant (in hectares); if no hydrant occurs in the node area null has to be typed;
- hydrant discharge (in l/s);
- section length (in m);
- land elevation of the downstream node (in m a.s.l.);
- nominal diameter of the section pipe (in mm). In the design stage, null diameter must be considered.

The list of commercial diameters (in mm) must be inserted, together with data about thickness (in mm), roughness (g, Bazin coefficient) and unitary cost of the pipe. Except for roughness, that is needed for hydraulic calculation, in the present study other parameter are not set because are object of a separate and more detailed evaluation.

5.4.3 Simulation results

The program "Clément" allow the computation of the discharges flowing into the network through the first and the second Clément models. In the present case the first one is selected, and additional parameters have to be typed:

- specific continuous discharge (in l/s/ha): this is according to the values included in paragraph 4.4
- minimum number of terminal open hydrants (5 is the chosen value)
- percentage of uncultivated land (in %): in the present case this parameter is null;
- Clément use coefficient (r) = 0.9167 because functioning is 22 hours a day;
- Clément operation quality, U(Pq) = 1.654, that means 5% of failure.

On the basis of these assumptions, the simulation has given the following results: Table 25 indicates hydraulic characteristic of pipe in relation to the number of fields that are supplied, while Table 26, Table 27 and Table 28 show the dimensioning in terms of diameter and head losses along the primary pipes supplying the 3 irrigation lots.

Number of supplied fields	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
1	65	54	350	1.27	0.56
2	130	106	400	2.38	0.84
3	194	157	450	2.80	0.99
4	259	208	500	2.80	1.06
5	324	259	550	2.61	1.09
6	389	310	600	2.35	1.09
7	453	360	650	2.08	1.09
8	518	411	650	2.70	1.24
9	583	461	700	2.30	1.20
10	648	512	700	2.83	1.33
11	712	562	750	2.36	1.27
12	777	613	750	2.80	1.39
13	842	663	800	2.33	1.32
14	907	714	800	2.70	1.42
15	971	764	850	2.24	1.35
16	1,036	815	850	2.54	1.44
17	1,101	865	850	2.87	1.52
18	1,166	915	900	2.37	1.44
19	1,230	966	900	2.63	1.52
20	1,295	1,016	900	2.91	1.60
30	1,943	1,519	1,100	2.23	1.60
40	2,590	2,021	1,200	2.49	1.79
50	3,238	2,523	1,300	2.53	1.90
60	3,885	3,025	1,400	2.45	1.96
70	4,533	3,526	1,500	2.30	2.00
80	5,180	4,027	1,600	2.13	2.00
90	5,828	4,528	1,700	1.95	1.99

Number of supplied fields	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
100	6,475	5,029	1,800	1.77	1.98
110	7,123	5,530	1,900	1.60	1.95
120	7,770	6,030	2,000	1.45	1.92
130	8,418	6,531	2,100	1.31	1.89
140	9,065	7,031	2,200	1.19	1.85
150	9,713	7,532	2,200	1.36	1.98

Table 25 Hydraulic characteristic of pipe in relation with the number of fields that are supplied

Lot	Reach	Sub Reach	L (m)	Field n°	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
North	Reservoir R2 & Pumping Station								
	A	1	8345	112	7252	5630	1900	1.66	1.99
	B	2	451	100	6475	5029	1800	1.77	1.98
		3	915	93	6022	4678	1800	1.53	1.84
		4	914	86	5569	4328	1700	1.78	1.91
		5	914	79	5115	3977	1600	2.08	1.98
		6	914	72	4662	3626	1600	1.73	1.80
		7	914	64	4144	3225	1500	1.93	1.83
		8	917	56	3626	2824	1400	2.13	1.83
		9	914	48	3108	2423	1300	2.33	1.83
		10	914	40	2590	2021	1200	2.49	1.79
		11	911	32	2072	1620	1100	2.54	1.70
		12	916	24	1554	1217	1000	2.39	1.55
		13	914	16	1036	815	850	2.54	1.44
		14	914	8	518	411	650	2.70	1.24
	C	15	467	12	777	613	750	2.80	1.39
		16	913	6	389	310	600	2.35	1.09

Table 26 Geometrical and hydraulic characteristics of primary pipes supplying North irrigation lot

Lot	Reach	Sub Reach	L (m)	Field n°	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
Central	Reservoir R2 & Pumping Station								
	A	1	541	142	9195	7131	2200	1.22	1.88
		2	911	142	9195	7131	2200	1.22	1.88
		3	915	137	8871	6881	2100	1.46	1.99
		4	908	132	8547	6631	2100	1.35	1.91
		5	902	124	8029	6231	2000	1.55	1.98
		6	922	116	7511	5830	2000	1.36	1.86
		7	923	108	6993	5430	1900	1.55	1.91
		8	910	100	6475	5029	1800	1.77	1.98
		9	444	93	6022	4678	1800	1.53	1.84

Lot	Reach	Sub Reach	L (m)	Field n°	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
	B	10	4526	72	4662	3626	1600	1.73	1.80
		11	915	63	4079	3175	1500	1.87	1.80
		12	911	54	3497	2724	1300	2.95	2.05
		13	913	48	3108	2423	1300	2.33	1.83
		14	915	42	2720	2122	1200	2.74	1.88
		15	914	36	2331	1820	1200	2.02	1.61
		16	915	30	1943	1519	1100	2.23	1.60
		17	914	24	1554	1217	1000	2.39	1.55
		18	914	20	1295	1016	900	2.91	1.60
		19	917	16	1036	815	850	2.54	1.44
		20	915	12	777	613	750	2.80	1.39
		21	889	7	453	360	650	2.08	1.09
		22	937	2	130	106	400	2.38	0.84
	C	23	452	15	971	764	850	2.24	1.35
		24	917	14	907	714	800	2.70	1.42
		25	915	12	777	613	750	2.80	1.39
		26	917	9	583	461	700	2.30	1.20
		27	914	5	324	259	550	2.61	1.09

Table 27 Geometrical and hydraulic characteristics of primary pipes supplying Central irrigation lot

Lot	Reach	Sub Reach	L (m)	Field n°	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
South	Reservoir R2 & Pumping Station								
	A	1	2638	145	9389	7282	2200	1.27	1.92
		2	2838	45	2914	2272	1300	2.05	1.71
		3	3967	8	518	411	650	2.70	1.24
		4	814	7	453	360	650	2.08	1.09
		5	816	4	259	208	500	2.80	1.06
	B	6	540	100	6475	5029	1800	1.77	1.98
		7	908	96	6216	4829	1800	1.63	1.90
		8	914	93	6022	4678	1800	1.53	1.84
		9	920	90	5828	4528	1700	1.95	1.99
		10	911	88	5698	4428	1700	1.86	1.95
		11	1523	87	5633	4378	1700	1.82	1.93
		12	914	79	5115	3977	1600	2.08	1.98
		13	914	71	4597	3576	1600	1.68	1.78
		14	914	63	4079	3175	1500	1.87	1.80
		15	914	55	3561	2774	1400	2.06	1.80
		16	914	47	3043	2373	1300	2.24	1.79
		17	914	39	2525	1971	1200	2.36	1.74
		18	914	31	2007	1569	1100	2.38	1.65
		19	914	23	1489	1167	1000	2.19	1.49
		20	914	15	971	764	850	2.24	1.35
		21	914	7	453	360	650	2.08	1.09
	C	22	536	37	2396	1871	1200	2.13	1.65

Lot	Reach	Sub Reach	L (m)	Field n°	A (ha)	Q (l/s)	DN (mm)	J (m/Km)	V (m/s)
		23	916	36	2331	1820	1200	2.02	1.61
		24	1883	35	2266	1770	1100	3.03	1.86
		25	914	30	1943	1519	1100	2.23	1.60
		26	914	25	1619	1268	1000	2.59	1.61
		27	914	20	1295	1016	900	2.91	1.60
		28	914	15	971	764	850	2.24	1.35
		29	914	10	648	512	700	2.83	1.33
		30	914	6	389	310	600	2.35	1.09
		31	914	3	194	157	450	2.80	0.99
		32	914	1	65	54	350	1.27	0.56

Table 28 Geometrical and hydraulic characteristics of primary pipes supplying South irrigation lot

Figure 37. Supplied discharge along primary pipes for the study area

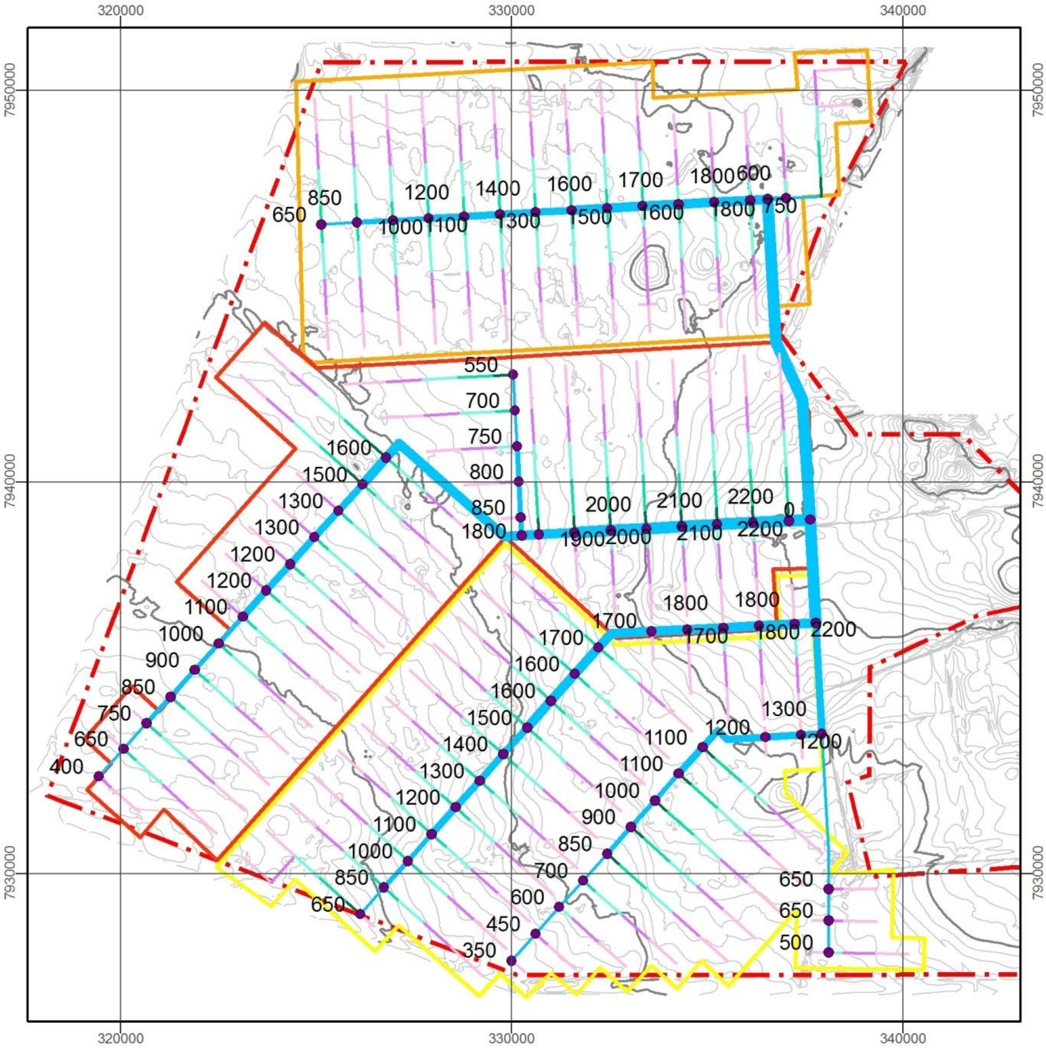


Figure 38. Diameter of primary pipes for the study area

5.4.4 Pumping system and power requirements

On the basis of simulation conducted with hydraulic model the head losses and the related piezometry profile has been defined for each of the 3 irrigation system. Tables below indicate the results of these computation.

Lot	Reach	Sub Reach	Ground Elevation (m s.m.m.)	Piezometry (m s.m.m.)	Pressure Head (m)	Cumulated Head Loss (m)
North	Reservoir R2 & Pumping Station		1081.00	1141	60	79
	A	1	1070.06	1129	59	65
	B	2	1069.91	1128	58	64
		3	1069.17	1127	57	63
		4	1068.00	1125	57	61
		5	1068.00	1123	55	59
		6	1067.99	1121	53	57
		7	1067.86	1120	52	56
		8	1067.39	1118	50	54

Lot	Reach	Sub Reach	Ground Elevation (m s.m.m.)	Piezometry (m s.m.m.)	Pressure Head (m)	Cumulated Head Loss (m)
		9	1066.16	1116	49	52
		10	1065.62	1113	48	49
		11	1065.02	1111	46	47
		12	1064.03	1109	45	45
		13	1063.43	1106	43	42
		14	1062.55	1104	41	40
	C	15	1070.06	1113	43	42
		16	1070.94	1111	40	40
** Data are referred to the downstream node of indicated sub reach*						

Table 29 Head loss and piezometry for primary pipes supplying North irrigation lot

Lot	Reach	Sub Reach	Ground Elevation (m s.m.m.)	Piezometry (m s.m.m.)	Pressure Head (m)	Cumulated Head Loss (m)
Central	Reservoir R2 & Pumping Station		1081.00	1129	48	85
	A	1	1080.39	1129	48	85
		2	1078.81	1127	49	83
		3	1076.15	1126	50	82
		4	1073.03	1125	52	81
		5	1070.19	1124	53	80
		6	1067.31	1122	55	78
		7	1065.54	1121	55	77
		8	1063.41	1119	56	75
		9	1062.51	1119	56	75
	B	10	1059.44	1111	51	67
		11	1057.85	1109	51	65
		12	1056.00	1106	50	62
		13	1055.06	1104	49	60
		14	1054.00	1102	48	58
		15	1052.81	1100	47	56
		16	1052.00	1098	46	54
		17	1050.00	1096	46	52
		18	1049.00	1093	44	49
		19	1047.52	1091	43	47
		20	1046.44	1088	42	44
		21	1045.49	1086	41	42
		22	1044.00	1084	40	40
	C	23	1062.70	1114	51	50
		24	1063.01	1111	48	47
		25	1064.00	1108	44	44
		26	1064.00	1106	42	42
		27	1064.00	1104	40	40
** Data are referred to the downstream node of indicated sub reach*						

Table 30 Head loss and piezometry for primary pipes supplying Central irrigation lot

Lot	Reach	Sub Reach	Ground Elevation (m s.m.m.)	Piezometry (m s.m.m.)	Pressure Head (m)	Cumulated Head Loss (m)
South	Reservoir R2 & Pumping Station		1081.00	1131	50	75
	A	1	1078.85	1127	49	72
		2	1071.15	1122	50	66
		3	1066.00	1107	41	44
		4	1064.00	1106	42	42
		5	1063.41	1103	40	40
	B	6	1078.00	1117	39	68
		7	1075.63	1115	40	67
		8	1074.18	1114	40	66
		9	1072.71	1112	39	64
		10	1070.37	1110	40	62
		11	1066.84	1107	41	59
		12	1065.92	1106	40	57
		13	1063.36	1104	41	56
		14	1061.36	1102	41	54
		15	1059.00	1100	41	52
		16	1058.45	1098	40	50
		17	1056.29	1096	40	48
		18	1054.55	1094	40	46
		19	1053.00	1092	39	44
		20	1050.56	1090	39	42
		21	1048.13	1088	40	40
	C	22	1071.00	1120	49	65
		23	1070.63	1119	48	63
		24	1068.83	1113	44	58
		25	1067.30	1111	44	56
		26	1066.86	1109	42	53
		27	1063.35	1106	42	50
		28	1061.00	1104	43	48
		29	1060.45	1101	41	46
		30	1059.00	1099	40	44
		31	1056.45	1097	40	41
		32	1055.35	1095	40	40

Table 31 Head loss and piezometry for primary pipes supplying South irrigation lot

It must be underlined that along the secondary pipe, serving generally about 5 fields, the head losses are about 10 m, while at the centre pivot at least 30 m of residual pressure must be guarantee, when the scenario with maximum water requirement is simulated. For this reason, in the previous table, each of the 3 irrigation system has a final sub reach where the “cumulated head loss” is 40 m (10 m plus 30 m) or rather the “pressure head” is 40 m.

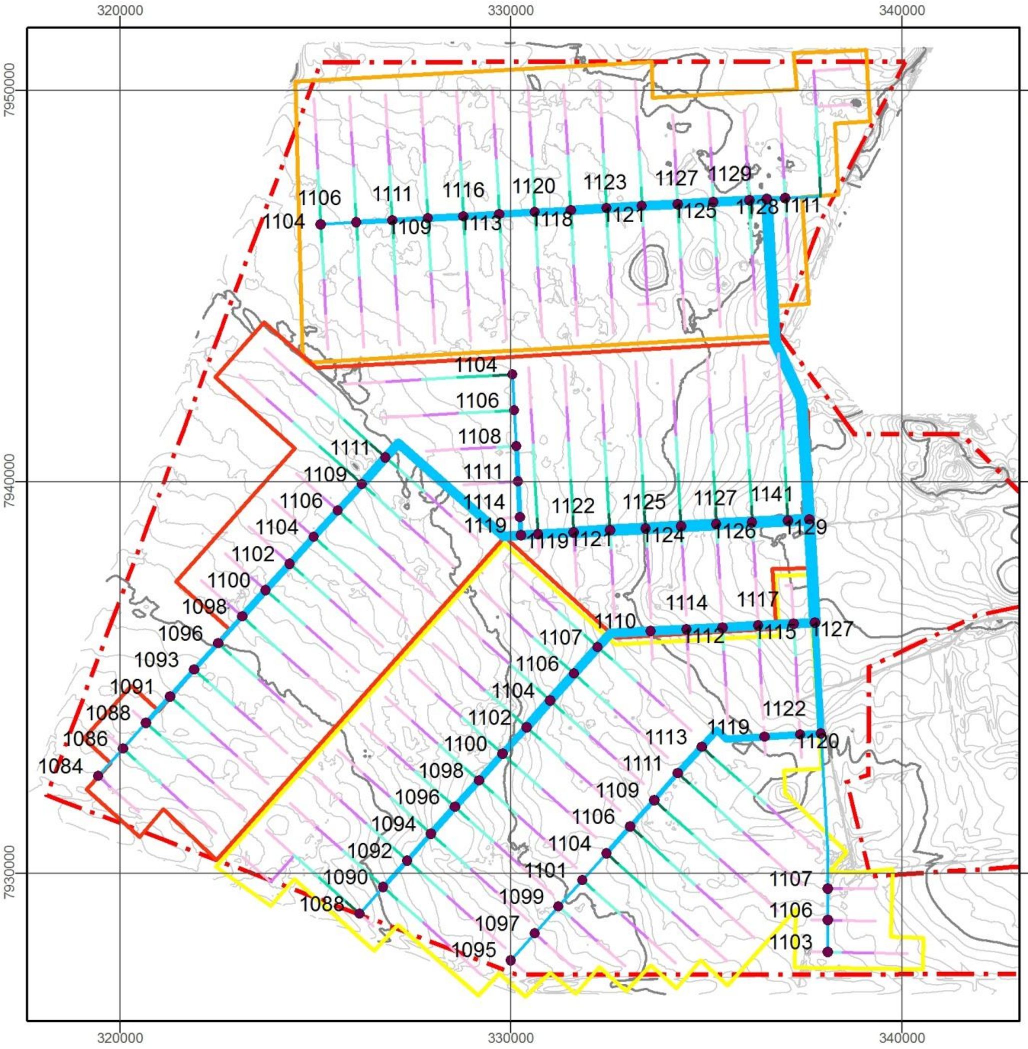


Figure 39. Piezometry along primary pipes for the study area

From this hydraulic computation, considering the reservoir has a bottom at 1081 m s.m.m., the North, Central and South lots need a pumping system with the following characteristics.

Lot	Discharge (l/s)	Head (m)
North	5630	60
Central	7131	48
South	7282	50

Table 32 Main characteristics of the 3 pumping system for the irrigation lots

According to the mentioned characteristics of pumping station, the power requirement is about 12,5 MW, while the annual energy consumption is around 64 million kWh/y.

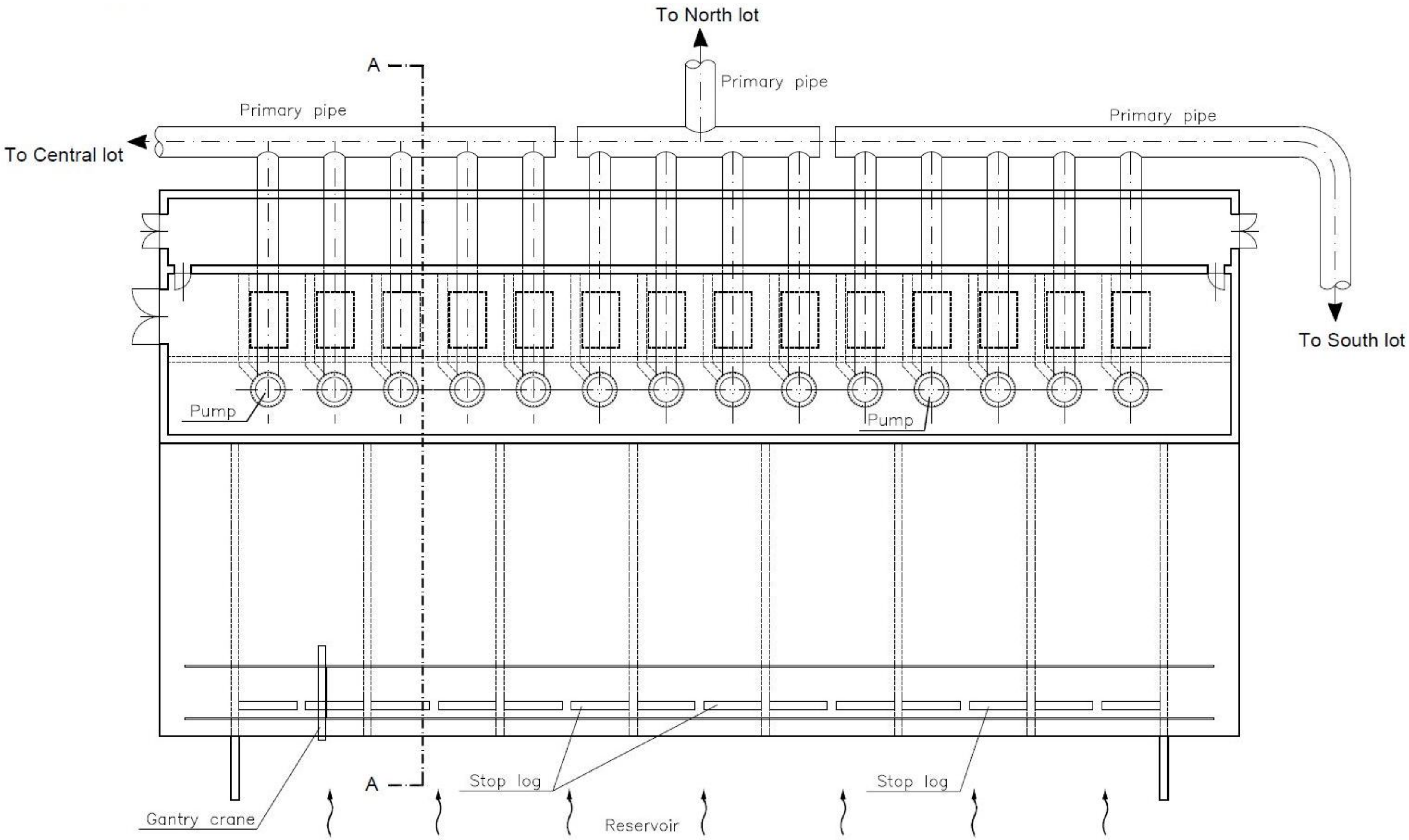


Table 33 Plan of pumping system

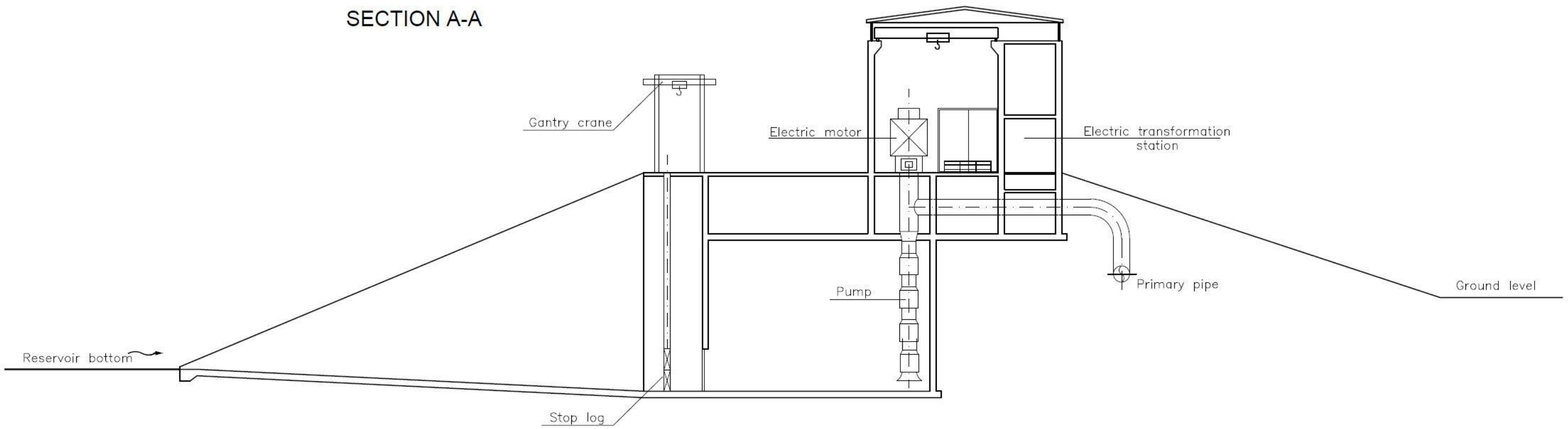


Table 34 Cross section of pumping system

6 DRAINAGE SYSTEM AND ROADS

6.1 INTRODUCTION

The presence of irrigation agricultural practise (paragraph 3) and water supply network (paragraph 5) leads to realize access roads and a drainage systems in order to collect storm water falling within the fields and discharge it outside the study area, preventing that infrastructures would be flooded.

The design of this drainage system starts from statistical analysis of rainfall included in paragraph 2.4 and is based upon the estimation of storm water runoff (paragraph 6.3), but it takes also into account what has been already envisaged for the neighbouring rainfed farms ("Consultancy Services for Construction Supervision of Road Network and Drainage Systems for Pandamatenga Farms – Final Design" by DIWI, October 2011).

In fact, according to this project there are several drains discharging water into the present study area for an overall amount of about 170 m³/s for storm event with a return time of 10 years.

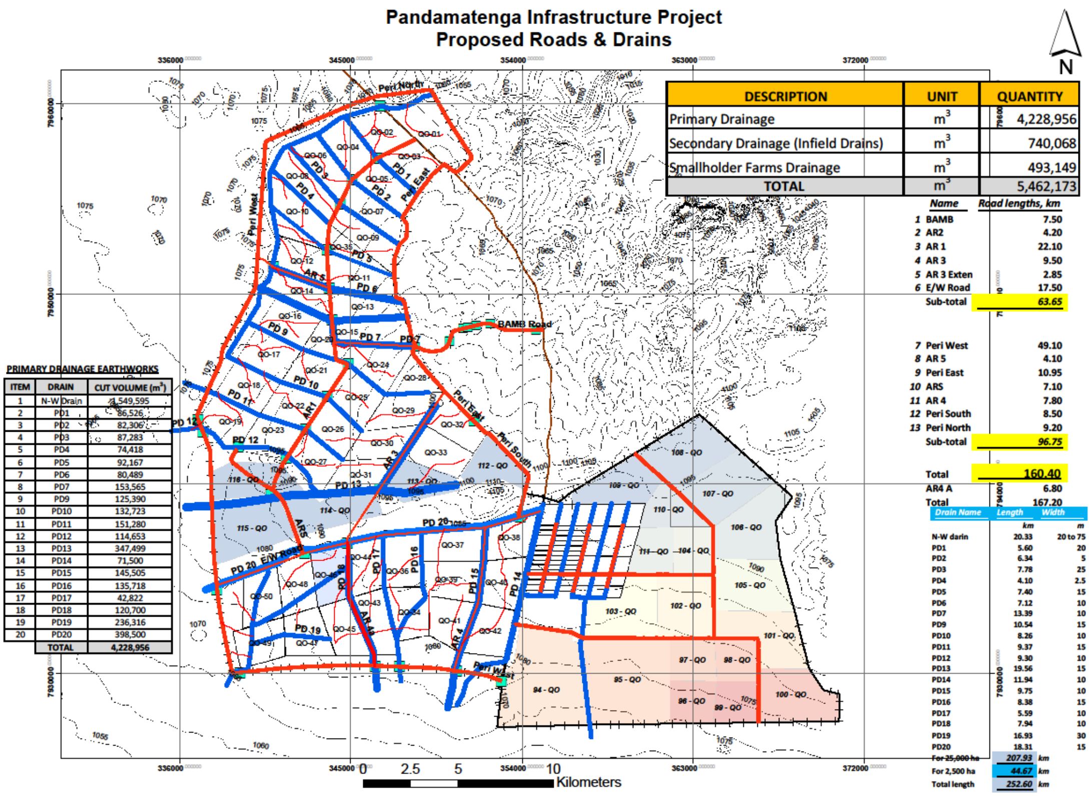


Figure 40. General layout of drainage system and access roads for existing farms ("Consultancy Services for Construction Supervision of Road Network and Drainage Systems for Pandamatenga Farms – Final Design" by DIWI in 2011)

6.2 IDENTIFICATION OF DRAIN AND ROAD LAYOUT

The layout of the water supply system for irrigation has been already described in paragraph 5.2: this pipeline network take also into account as well as the topography of the study area, trying to follow the natural drainage paths.

This is obviously also the main criteria to set up the layout to collect and discharge storm water that falls within the study area. The roads must simply guarantee access to each fields.

Besides, the drainage system has to intercept the runoff from adjacent zones and convey it to the study area: naturally (northern border, see Figure 42) or artificially (drains for the existing Pandamatenga farms, see Figure 40).

As for the pressurized pipeline network, also the storm water system is composed of primary and secondary channels: these latter collect water directly from fields and discharge it into the primary channels that generally are along the main natural drainage paths. Methodological approach and numerical elaboration to dimension this channelization are given in paragraph 6.3 and 6.4.

Figure 41. Layout of primary drains for the present design and drainage system of the existing farms

The drainage system has to guarantee the safety of roads, especially at their intersection, but it also must minimize risk related to sediment transport and wildlife. For these reasons, some considerations for drain management are given in paragraph 6.6.

6.3 ESTIMATION OF STORM WATER RUNOFF

6.3.1 Identification of watersheds

The calculation of runoff starts from delineation and characterization of the watersheds that are within the study area: first of all, these zones must be identified according to the drain and road layout (paragraph 6.2), then they can be geometrically described.

For the primary drainage system the following image defines the watersheds whose geometrical characteristics are extracted from the Digital Elevation Model (DEM) that has been implemented during the present feasibility study, combining information from acquired high resolution stereo pair images (with 50 cm of pixel dimension) and results of ad hoc topographic field survey (more details in the Field Investigation Report).

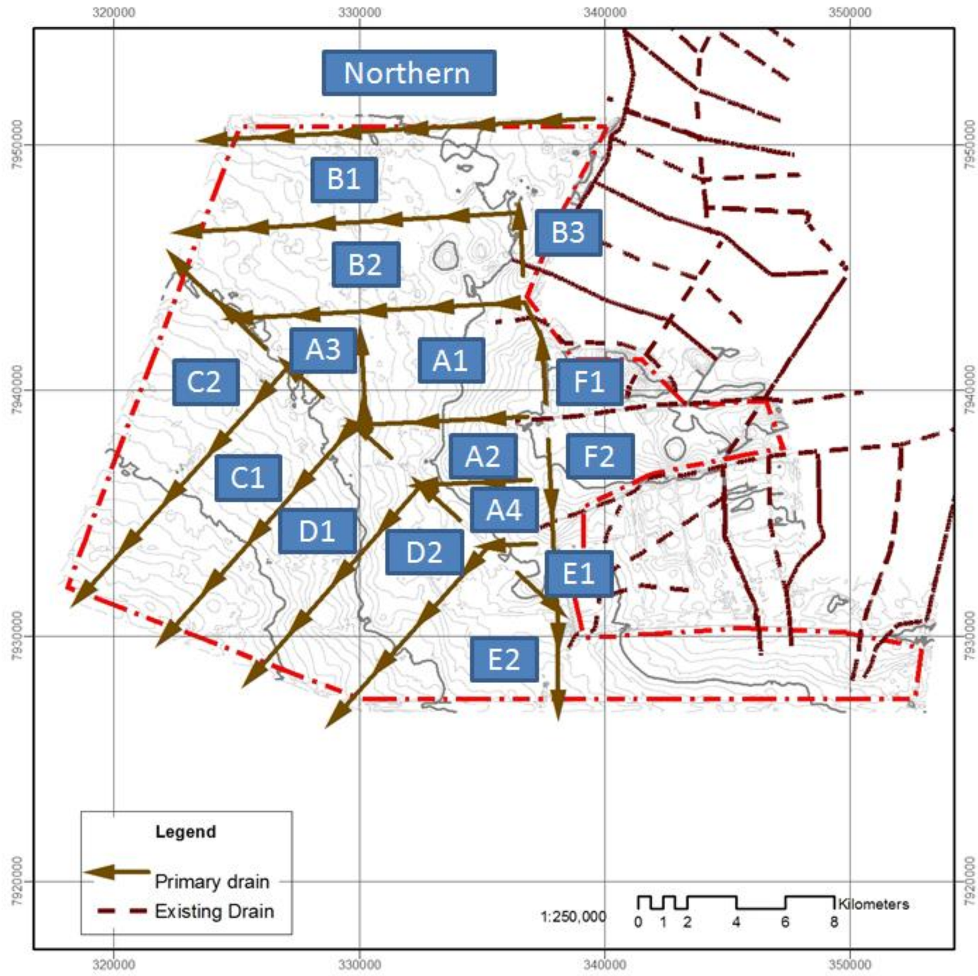


Figure 42. Main watersheds of drainage system

As mentioned earlier, the secondary channels collect water directly from field therefore the evaluation of their watershed takes into account the number of fields that they drain. According to the definition of irrigation system, each field is a square with sides of 908 m thus the total extension is 82.4 ha.

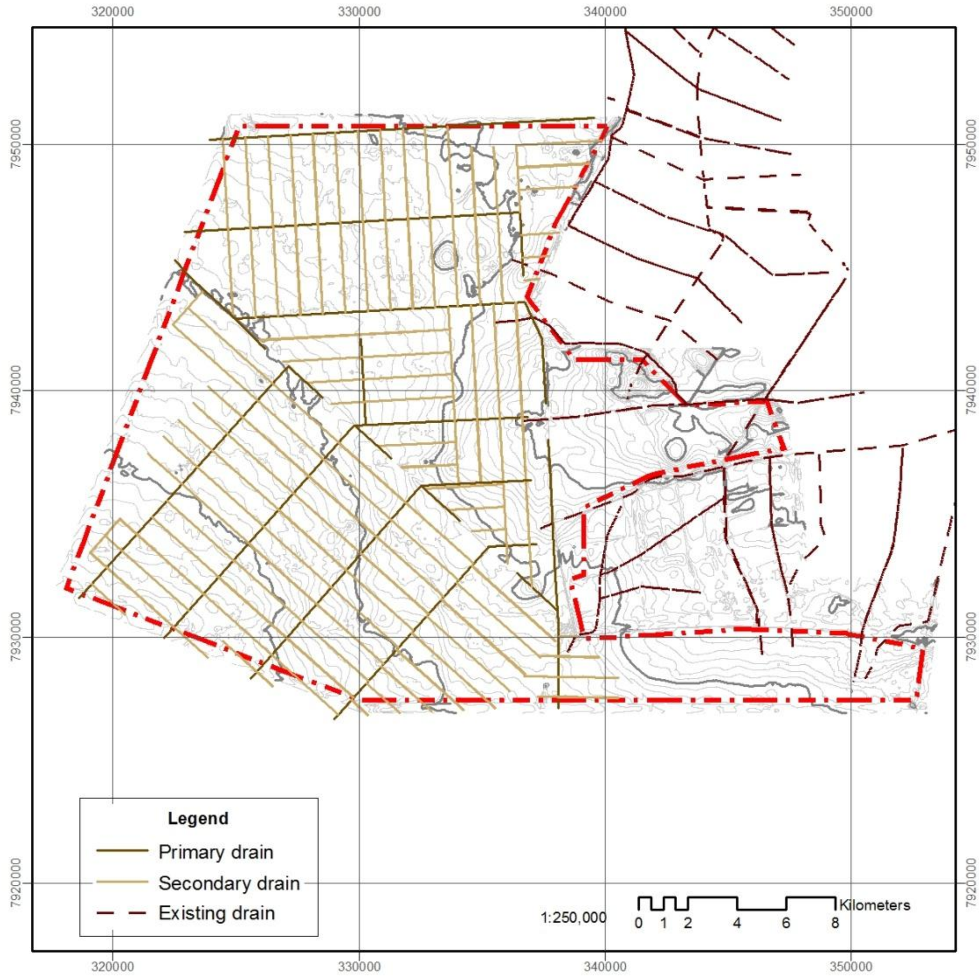


Figure 43. Layout of primary and secondary drains for the present design and drainage system of the existing farms

The following image indicates the schematic draining system for fields: according to the irrigation layout, the secondary channel might drain from 1 to maximum 5 fields.

The geometrical characteristics of watersheds related to primary and secondary drains are summarized in Table 35 and Table 36.

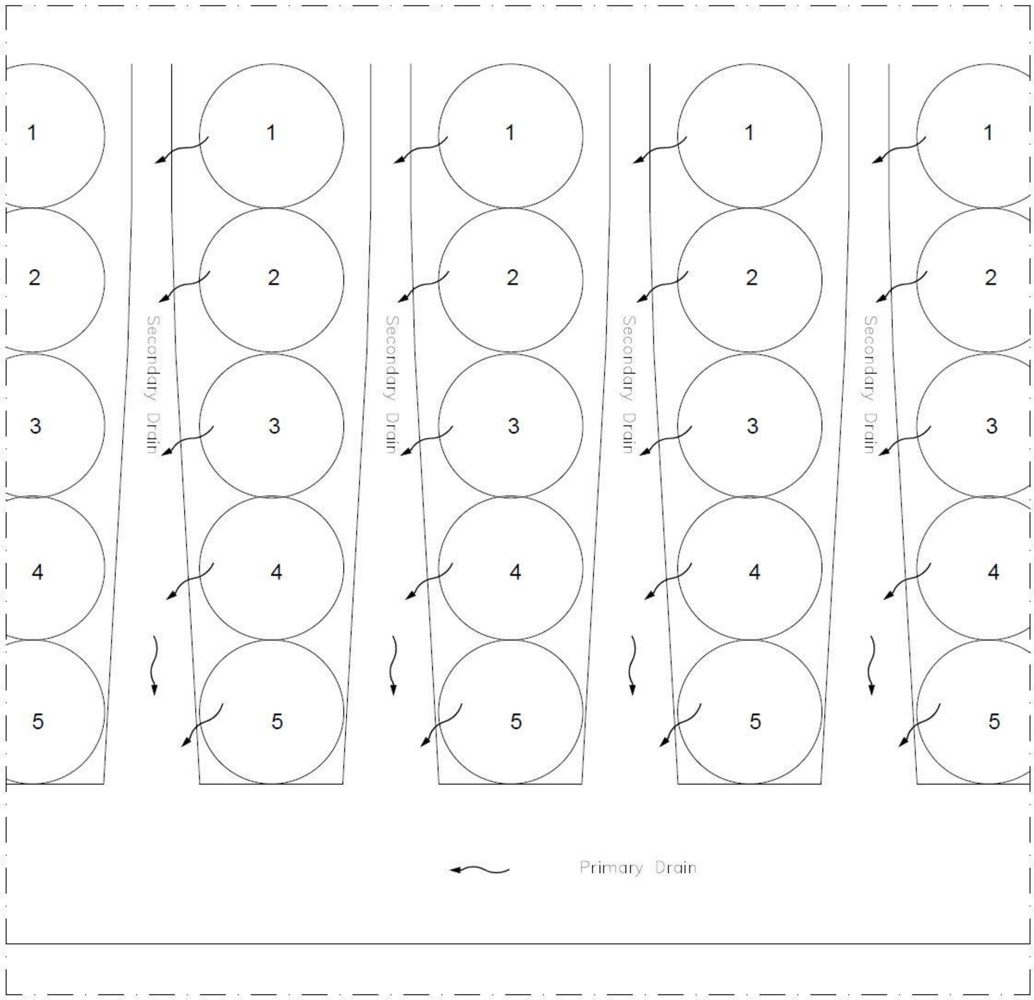


Figure 44. Schematic layout of drainage system for fields towards the secondary drains

6.3.2 Time of concentration

The catchments are finally characterized from the hydrological point of view by the time of concentration (t_c) that measures the response of a watershed to a rain event. Generally the procedures used to estimate t_c depend upon few watershed characteristics. To accurately determine t_c for a watershed, the hydraulics of each part (overland and channel) of the flow path can be distinguished. In some cases the two flow path can be taken into account in the same formula or the computation can be done separately.

This parameter can be estimated with several formulas that are generally valid for a certain extension of catchment. The formula from Aronica – Paltrinieri (1954) is applied when the watersheds are smaller than 170 km² and it is derived from Giandotti's formula (1934).

$$t_c = \frac{4A^{0.5} + 1.5L}{0.8(h_m - h_0)^{0.5}} \qquad \text{(Giandotti, 1934)}$$

$$t_c = \frac{\frac{1}{Md}\sqrt{A} + 1.5L}{0.8(h_m - h_0)^{0.5}} \qquad \text{(Aronica e Paltrinieri, 1954)}$$

Symbols included in formulas mean:

- t_c = time of concentration (hour);
- A = basin area (km²);
- L = flow path (km);
- h_m = sub-basin average elevation (m a.s.l.);
- h_0 = sub-basin lowest elevation (m a.s.l.);
- M = coefficient related to vegetation coverage (ranging from 0.667 if bared to 0.167 when permanent grass);
- d = coefficient related to soil permeability (ranging from 1.27 if semi impervious to 0.69 when high permeable).

The chosen values for coefficient M and d are respectively 0.2 (partially cultivated area) and 0.9 (moderately permeable soil). The geometrical characteristics of watersheds and the related time of concentration are summarized in the following tables.

As regards the secondary channel, the watersheds have been related to the number of drained fields as represented in Figure 44. On average drainage slope of fields (i) is about 1 m/km, minimum elevation is factitiously set equal 0 and mean and maximum elevation is calculated considering the diagonal length of field.

The following table indicates the concentration time in case of channels that drain from 1 to 5 fields. Calculation of concentration time for primary drain is referenced to basin drawn in Figure 42.

Drained fields	**Area (km²)**	**L (km)**	**H^*_{min} (m)**	**H^*_{avg} (m)**	**H^*_{max} (m)**	**i (m/m)**	**Tc (h)**
1	0.82	1.3	0	0.6	1.3	0.0010	9
2	1.65	2.0	0	1.0	2.0	0.0010	11
3	2.47	2.9	0	1.4	2.9	0.0010	12
4	3.30	3.7	0	1.9	3.7	0.0010	13
5	4.12	4.6	0	2.3	4.6	0.0010	13

Table 35 Time of concentration for watersheds drained by secondary channels

Basin	**Area (km²)**	**L (km)**	**H_{min} (m a.s.l.)**	**H_{avg} (m a.s.l.)**	**H_{max} (m a.s.l.)**	**i (m/m)**	**Tc (h)**
North	98	20.0	1062	1071	1080	0.0009	31
B1	58	18.1	1061	1067	1072	0.0006	33
B2	45	13.8	1061	1066	1070	0.0007	30
B3	4	2.1	1070	1073	1075	0.0024	9
A1	33	7.7	1064	1072	1080	0.0021	16
A2	17	5.7	1066	1073	1080	0.0025	13
A3	12	2.5	1061	1063	1064	0.0012	20
A4	9	3.6	1069	1073	1076	0.0020	13
C1	46	12.8	1043	1052	1060	0.0013	21
C2	45	12.8	1043	1052	1060	0.0013	21
D1	39	10.1	1048	1057	1066	0.0018	18

Basin	Area (km²)	L (km)	H_{min} (m a.s.l.)	H_{avg} (m a.s.l.)	H_{max} (m a.s.l.)	i (m/m)	Tc (h)
D2	34	10.1	1048	1057	1066	0.0018	17
E1	3	2.2	1068	1070	1071	0.0014	12
E2	34	8.9	1055	1062	1068	0.0015	19
F1	24	9.3	1080	1088	1095	0.0016	16
F2	35	9.3	1080	1088	1095	0.0016	18

Table 36 Time of concentration for watersheds drained by primary channels

6.3.3 Spatial analysis of rainfall

After having statistically analyzed the frequency of rainfall (paragraph 2.4), it is necessary to evaluate their spatial variability. In fact, the reduction of the precipitation depth from a design storm for a punctual to an effective (mean) depth over the entire watershed is essential in order to not overestimate the volume of precipitation and to fit the design of hydraulic structures as correct as possible.

Generically, this evaluation has been conducted introducing a parameter: the Areal Reduction Factor (ARF) is defined as the ratio between the average areal depth of precipitation and the average point depth. It ranges from 0 to 1 and is a function of storm characteristics (intensity and duration), as well as basin characteristics (size, shape and geographic location).

The National Weather Service of the National Oceanic and Atmospheric Administration (NOAA) has suggested a relationship to estimate the reduction of punctual rainfall (Figure 45) according the extension of the catchment and the duration of storm event (Technical Report NWS 24, "A Methodology for Point-to-Area Rainfall Frequency Ratios").

This type of relationship has been also included in the Botswana Road Design Manual, provided by Department of Roads , with reference to the intensity of precipitation (Figure 46): this means taking into account also the time of return of storm event.

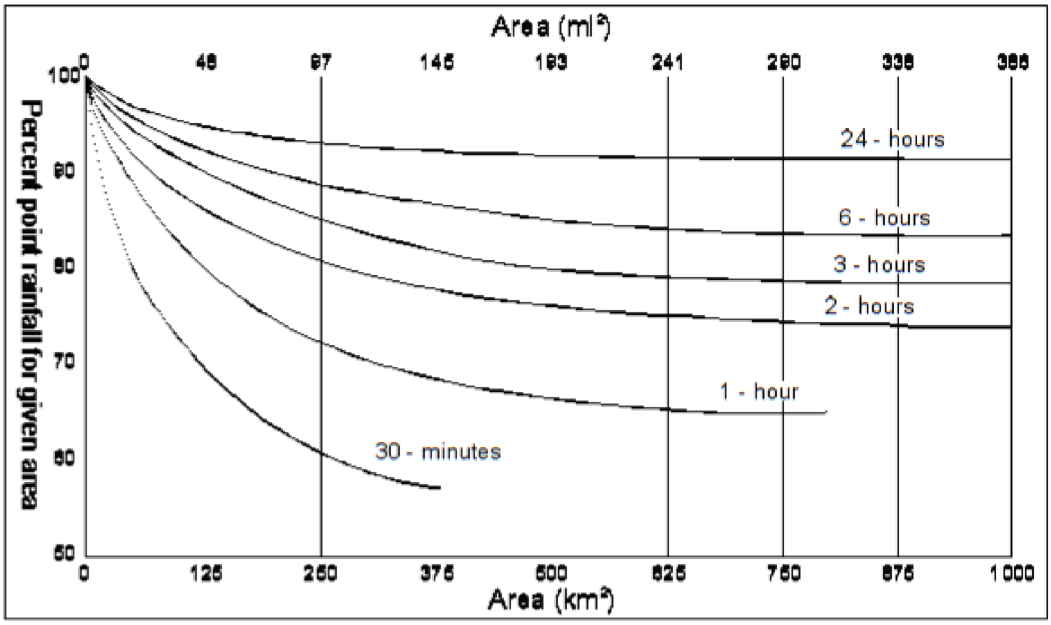


Figure 45 ARF related to storm duration and catchment area (Technical Report NWS 24, NOAA)

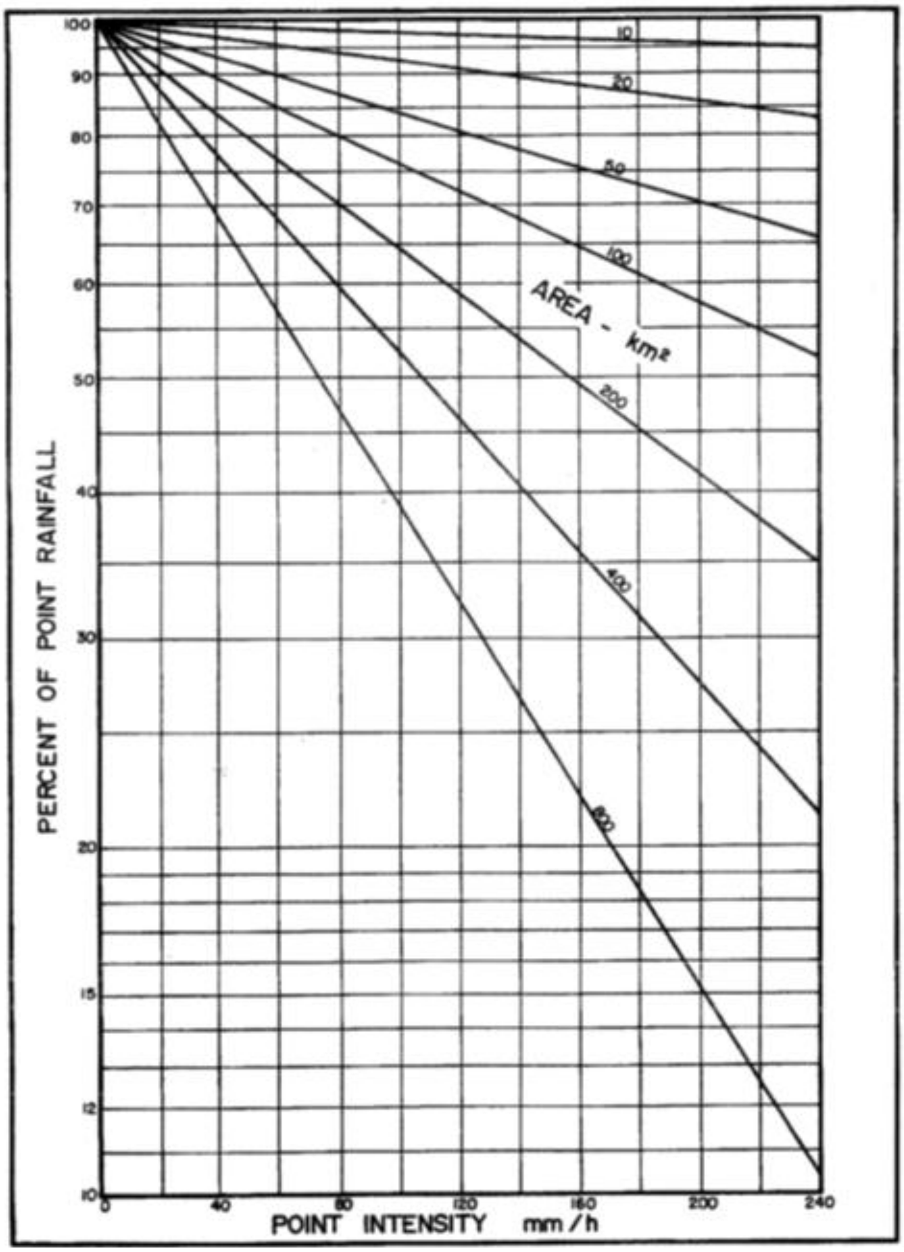


Figure 46 ARF related to rainfall intensity and catchment area (Botswana Road Design Manual)

The extension of the watershed included within study area is generally smaller than 50 km², the critical storm durations (time of concentration) are often longer than 12 hours and the intensity of rainfall is lower than 10 mm/h, therefore the ARF is always close to 1.

However, in the present analysis the formulation proposed by U.S. Geological Survey has been adopted:

$$ARF = 1 - \exp(-1.1 \cdot d^{0.25}) + \exp(-1.1 \cdot d^{0.25} - 0.0386 \cdot A)$$

- d = duration of storm event (hour);
- A = basin area (km²).

The resulting ARF for each watersheds are been quoted in Table 39 and Table 40.

6.3.4 Runoff peak and hydrograph

The runoff is the result of the movement of rainfall excess over the watershed surface to its outlet. This calculation is generally done applying a deterministic hydrological model that allow the runoff generation to be evaluated: the precipitation event is first described in terms of total volume, time and area of distribution. Losses, consisting of interception and infiltration, are simulated and subtracted from the precipitation, resulting in direct runoff or rainfall excess. Direct runoff is transformed into a direct runoff hydrograph usually by unit hydrograph methods.

The runoff hydrograph is expression of a flow rate at a certain point of drainage network over time: synthetic hydrographs are predictions based on watershed characteristics and assumed rainfall intensities: the shape of a runoff hydrograph can be specified by the time to peak flow and time for recession flow.

Among the procedures that are proposed in the technical literature to calculate the hydrograph runoff, the Soil Conservation Service (U.S. Department of Agriculture, 1986) specifies a triangular shape obtained using the so called rational formula (Wanielista, 1990). According to this method, hydrograph time base (t_b) and peak discharge (Q_p) might be calculated as:

$$Q_p = KCiA \quad ; \quad t_b = t_p + xt_p$$

where:

- K = peak attenuation factor (see Table 37);
- C = runoff coefficient;
- I = average intensity of rainfall;
- A = watershed area;
- t_p = time to peak, associated to time of concentration (see paragraph 6.3.2);
- x = (1291/K) - 1 , if A is measured in square miles.

General Description	Falling Limb Factor, x	Peak Attenuation Factor, K^a	
		A (mi^2)	A (acres)
Rational formula shape	1.00	645	1.00
Urban, steep slopes	1.25	575	0.89
Typical SCS	1.67	484	0.75
Mixed urban/rural	2.25	400	0.62
Rural, rolling hills	3.33	300	0.47
Rural, flat slopes	5.50	200	0.31
Rural, very flat	12.00	100	0.16

Table 37 Hydrograph attenuation factor (Wanielista, Yousef "Stormwater Management", 1993)

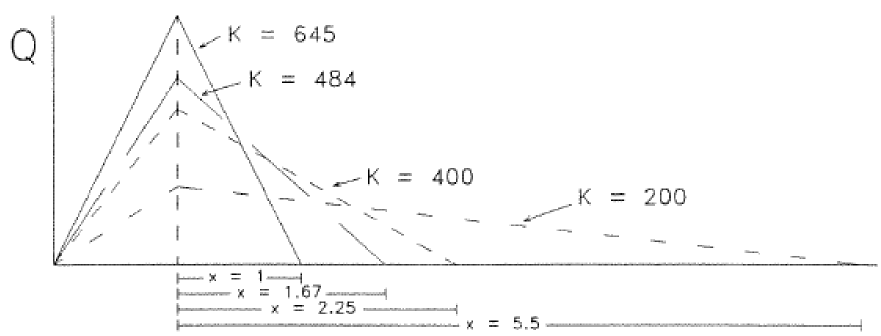


Figure 47 Triangular shaped hydrograph related to hydrograph peak reduction factor included in Table 37 (Wanielista, Yousef "Stormwater Management", 1993)

Description of Surface	C
Natural surface types	
Bare impermeable clay with no interception channels or run-off control	0.70
Bare uncultivated soil of medium soakage	0.60
Heavy clay soil types:	
• pasture and grass cover	0.40
• bush and scrub cover	0.35
• cultivated	0.30
Medium soakage soil types:	
• pasture and grass cover	0.30
• bush and scrub cover	0.25
• cultivated	0.20
High soakage gravel, sandy and volcanic soil types:	
• pasture and grass cover	0.20
• bush and scrub cover	0.15
• cultivated	0.10
Parks, playgrounds and reserves:	
• mainly grassed	0.30
• predominantly bush	0.25
Gardens, lawns etc	0.25
Developed surface types	
Fully roofed and/or sealed developments	0.90
Steel and non -absorbent roof surfaces	0.90
Asphalt and concrete paved surfaces	0.85
Near flat and slightly absorbent roof surfaces	0.80
Stone, brick and precast concrete paving panels:	
• with sealed joints	0.80
• with open joints	0.60
Unsealed roads	0.50
Railway and unsealed yards and similar surfaces	0.35
Land use types	
Industrial, commercial, shopping areas and town house developments	0.65
Residential areas in which the impervious area is less than 36% of gross area	0.45
Residential areas in which the impervious area is 36% to 50% of gross area	0.55

Table 38 Run-off coefficients for use in rational and modified rational methods ("Hydrological and Hydraulic Guidelines, New Zeland", 2012)

According to the data included in the previous tables and figure, the present study has adopted conservatively 0.4 as runoff coefficient (cultivated soil), while the chosen peak attenuation factor is the typical SCS (K = 484, x = 1,67): this is because the rational method (K = 645, x = 1) would probably overestimates the peak discharge.

The following tables include calculation related to SCS hydrograph runoff (Wanielista, 1990) for secondary and primary drains on the basis of watersheds identified in Table 35 and Table 36. The storm event that has been considered has a return time of 10 years.

Drained fields	Area (km^2)	tc (h)	tb (h)	Rainfall intensity (mm/h)	ARF	Q (m^3/s)	u ($m^3/s/km^2$)	V (Mm^3)
1	0.82	9.3	60.5	10.2	1.00	1.0	1.26	0.03
2	1.6	10.9	70.6	8.7	1.00	1.8	1.08	0.06
3	2.5	11.8	76.6	8.0	1.00	2.5	0.99	0.09
4	3.3	12.5	81.3	7.5	1.00	3.1	0.93	0.12
5	4.1	13.1	85.3	7.2	1.00	3.7	0.89	0.16

Table 39 Runoff hydrograph with 10 years return time for watersheds drained by secondary drains

Basin	Area (km^2)	tc (h)	tb (h)	Rainfall intensity (mm/h)	ARF	Q (m^3/s)	u ($m^3/s/km^2$)	V (Mm^3)
North	98	31	200	3.4	0.97	40	0.41	4.0
B1	58	33	211	3.2	0.98	23	0.39	2.4
B2	45	30	194	3.5	0.99	19	0.43	1.9
B3	4	9	60	11.1	1.00	5	1.37	0.1
A1	33	16	107	6.2	0.98	25	0.76	1.3
A2	17	13	83	8.0	0.99	17	0.98	0.7
A3	12	20	127	5.4	1.00	8	0.66	0.5
A4	9	13	82	8.0	1.00	9	0.99	0.4
C1	46	21	137	5.0	0.99	28	0.61	1.9
C2	45	21	136	5.0	0.99	27	0.61	1.8
D1	39	18	117	5.8	0.99	28	0.72	1.6
D2	34	17	111	6.1	0.99	25	0.75	1.4
E1	3	12	75	8.8	1.00	4	1.09	0.1
E2	34	19	125	5.4	0.99	23	0.67	1.4
F1	24	16	106	6.2	0.99	18	0.76	1.0
F2	35	18	120	5.7	0.99	25	0.70	1.5
Total	**537**	**-**	**-**	**-**	**-**	**-**	**-**	**22**

Table 40 Runoff hydrograph with 10 years return time for watersheds drained by primary drains

As mentioned in paragraph 6.1, there are some existing channels discharging water towards the present study area; in particular some of them would interferes with the irrigation fields: those are listed in the following table.

Because for these existing drains only the peak discharges are known, the storm water volume is roughly estimated taking into account the ratio between volume (V) and peak discharge (Q) that has been found for watershed within the study area (Table 43). On average this value is about 20 hours.

Existing drain code	Q (m^3/s)	V (Mm^3)
PD11	16.8	1.2
PD12	15.9	1.1
PD13	26.4	1.9
PD19	43.3	3.1
PD20	23.3	1.7
Total	**126**	**9**

As above indicated, the overall storm water volume for a 10-years event is about 31 Mm^3 from which 18 Mm^3 are generated within the study area, 4 Mm^3 from the outside northern basin and 9 Mm^3 are brought by drains collecting water in neighbouring existing farms.

6.4 DESIGN OF GRAVITY DRAINAGE SYSTEM

The channels are designed to collect storm water falling within the fields and drain it outside the study area. This system will allow agricultural practise and related infrastructure - including roads and their intersection with channels (bridge and culvert), which are a critical issue from a hydraulic point of view - to be preserved from floods. representing the most.

6.4.1 Design criteria

The dimensioning of the drainage system must make reference to different issues that are analyzed within the present paragraph.

Time of return and freeboard.

First of all, the design must consider the probability of having a storm event that exceeds the capacity of the drainage system and, therefore, causes floods within the study area. Obviously the higher the chosen time of return, the greater will be the cost of realization.

The selection of this frequency of failure consists in a sort of risk assessment that is generally related to the importance of the infrastructure to be protected. Typically national and international standards suggest and/or prescribe certain return time but also, in some cases, also the safety margin to be considered.

In fact, the freeboard between the water level (during the design storm event) and the top of bank drain is normally governed by considerations about the channel itself (size, velocity, etc...) but also about the risk of flood (importance of the surrounding property, structures, etc...). Return period and freeboards that might adopted according to the traffic volume and type of roads are indicated in the following table.

Road type	Bridge standard	Culvert standard
Major road	Passage of the 100-year return period flood with minimum clearance of 0.6 m normally but with up to 1.2 m where large trees can be transported in the river.	− Passage of the 100-year return period flood by heading up to a maximum 0.5 m below road and adjacent house floor levels, and − Passage of the 10-year flood without heading up.

Road type	Bridge standard	Culvert standard
Rural road	Passage of the 50-year return period flood with a minimum clearance of 0.6 m	− Passage of the 50-year return period flood by overtopping the embankment to a maximum depth of 0.2 m, and − Passage of the two year return period flood with no heading up
Remote road	Passage of the 20-year return period flood with a minimum clearance of 0.3 m.	− Passage of the 20-year return flood with 0.3 m freeboard, and − Passage of the two year return period flood with no heading up.
Access tracks	Passage of the 10-year return period flood with a minimum clearance of 0.3 m.	− Passage of the 10-year return period flood by heading up to a maximum 0.3 m below road level.

Table 41 Time return and minimum freeboard for bridge and culvert according to road type (“Hydrological and Hydraulic Guidelines, New Zeland”, 2012)

For the present project it has been adopted to make reference to flood event with 10-year return and guarantee at least 50 cm as freeboard: this safety margin is also to prevent potential reduction of geometrical cross section of the channel because of sediment deposition.

<u>Material</u>.

Fields survey and analogous experiences, as the agricultural zone next to the study area, suggest to design unlined canal for the drainage network. This is mainly because eventual seepage losses are not affecting the functioning of irrigation system, while concrete structures would be costly for both construction and maintenance.

Therefore open channel will be excavated and shaped to the required cross section in natural earth or filled without special treatment of the wetted surface. Compaction of bank or fill material for the purpose of stabilization is not considered as a lining operation.

<u>Geometry of cross section</u>.

The trapezoidal cross section can be adopted for the primary and infield drains, primarily because it is the simplest way to construct.

As flow is a function of cross sectional area the less the drain depth the greater the drain width will need to be. The cross section selected for a canal should be such as to carry the design storm discharge.

The ratio of bottom width to depth usually ranges from 2 : 1 for small channels to 6 : 1 for the bigger drain, the side slopes of a canal depend upon the stability of the material in which it is constructed. Inside slopes of 2 : 1 (horizontal to vertical) are practically standard for earth canals under ordinary conditions.

Longitudinal slope and flow velocities.

The slope of the primary and infield drains is chosen according to the natural ground slopes but also operation and maintenance problems should be considered in the selection of canal cross-section characteristics.

This issue is mainly related to suggested values of flow velocity. In unlined canals, the velocity should be such as to prevent erosion of the canal bottom (maximum allowable) or deposition of soil that might be transported with water flow over fields (minimum allowable).

When the velocity of the flow is such that there is no silting or scouring action in the canal bed, then that velocity is known as critical velocity. Generally the critical velocity depends on the nature of the soil formation in which the water flows. Values of flow velocity that are commonly suggested are from 0.3 to 0.6 m/s for sandy soil, 0.6 - 0.9 m/s for black cotton soil and 0.9 to 1.15 m/s for firm clay and loom.

For what concerns the capacity of moving sediment downstream along the channels, the evaluation can be done considering the formulation proposed by Shields that relates flow dynamics to mean diameter of sediment (see paragraph 6.6.2).

6.4.2 Hydraulic dimensioning of primary and secondary drains

On the basis of all above mentioned criteria and the storm discharges resulting from hydrological analysis (Table 39 and Table 40), the drainage system has been hydraulically designed according to Manning's formula, that allows to calculate flow capacity of channels once that their geometrical and roughness characteristics are fixed:

$$Q = V \cdot A = \left(\frac{1}{n} R_H^{\ 2/3} i^{1/2} \right) \cdot A$$

where:

- Q = flowing discharge;
- V = water velocity;
- A = area of the cross section;
- n = roughness coefficient (Manning's number);
- i = longitudinal slope;
- R_H = hydraulic radius, ratio between area and wetted perimeter.

Roughness is related to the material and condition of channel bottom and slopes: drain are excavated, thus earth made, generally straight and with uniform cross section. According to Chow, V. T. (Open Channel Hydraulics, 1959) value of Manning's coefficient might be 0.022 if clean surface and 0.027 when short grass grows along the channel: for design purpose it has been chosen the mean between the mentioned values (0.025).

The longitudinal slope is chosen according to the natural topography of the terrain where the channel have been tracked. For what concerns the secondary drains, it has been considered they have a common longitudinal slope of 0.5 m/km.

The cross section of drain is trapezoidal with inside slope 2 : 1. The dimensioning of each drain consists in finding the bottom width and water depth for which resulting flow capacity is equal, of even higher, than the related storm discharge.

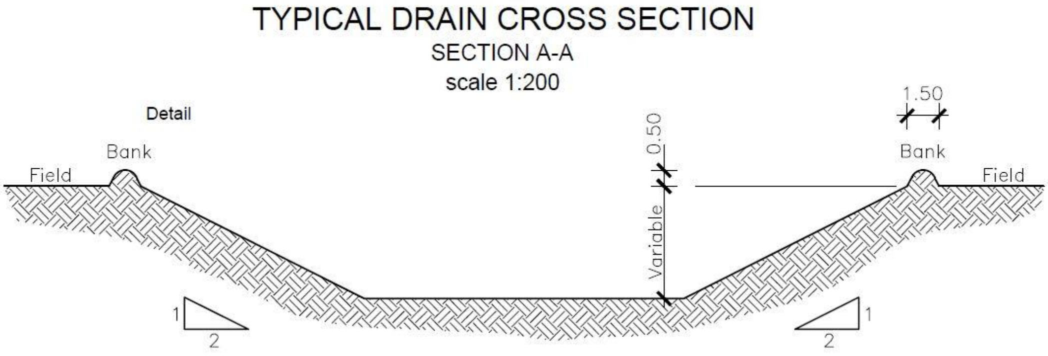


Figure 48 Typical cross section of drains

The followings tables quote results of hydraulic calculation for secondary drain (Table 42) and primary drains (Table 44). These latter channel have been designed considering cumulated discharges included in Table 43: in this table the parameter "Q_{ext}" indicates flow generated in the existing Pandamatenga farming area and then discharged towards the present study area

Figure 49 Layout of primary drains and their code

	Hydrology		Hydraulics				
Drained fields	**A (ha)**	**Q (m^3/s)**	**i (m/m)**	**b (m)**	**d (m)**	**V (m/s)**	**Q (m^3/s)**
1	0.8	1.0	0.00050	2.5	0.60	0.51	1.1
2	1.6	1.8	0.00050	3.0	0.70	0.56	1.9
3	2.5	2.5	0.00050	4.0	0.75	0.61	2.5
4	3.3	3.1	0.00050	5.0	0.75	0.62	3.0
5	4.1	3.7	0.00050	6.0	0.75	0.64	3.6

Table 42 Hydraulic dimensioning of secondary drains

Drain code	Reach code	Q_{ext} (m^3/s)	Basin	A (km^2)	Q (m^3/s)	A_{tot} (km)	Q_{tot} (m^3/s)
D1	A	0	F1/2	12	9	12	9
	B	16	-	0	0	12	25
	C1 - C4	0	B2	45	19	45	44
D2	A	0	B3	3.7	5.0	4	5
	B	17	-	0	0	4	22
	C1 - C4	0	B1	58	23	62	45
D3	A	0	A1/2+A2/3	22	18	22	18
	B	0	A1/2	17	13	17	13
	C	0	A2/3	5.8	5.7	6	6
	D1 - D4	0	C2/2+D1	62	42	62	78
D4	A	0	A3/2	6.1	4.0	6.1	4.0
	B1 - B4	0	C1+(C2/2)	68	41	74	46
D5	A	0	A2/3	5.8	5.7	5.8	5.7
	B	0	A4/2	4.7	4.7	4.7	4.7
	C1 - C2	0	D2	34	25	44	36
D6	-	0	-	98	40	98	40
D7	A	0	A4/2	4.7	4.7	4.7	4.7
	B	0	E2/2	17	11	22	16
D8	A	26	F2/2	18	12	18	39
	B	23	E1/2	1.6	1.8	19	64
	C	0	E1/2	1.6	1.8	1.6	1.8
	D	43	E2/2	17	11	56	77
D9	A	0	A3/2	6.1	4.0	6.1	4.0
	B	0	-	0.0	0.0	68	49

Table 43 Cumulated discharges along the primary drains

Drain code	Reach code	i (m/km)	b (m)	d (m)	V (m/s)	Q (m^3/s)
D1	A	1.70	3	1.25	1.42	10
	B	1.70	5	1.75	1.82	27
	C1 - C4	1.10	9	2.00	1.70	44
D2	A	0.70	3	1.25	0.91	6.3
	B	0.70	7	1.75	1.22	22
	C1 - C4	0.70	12	2.00	1.40	45
D3	A	3.20	5	1.25	2.09	20
	B	0.34	8	1.50	0.80	13
	C	1.76	3	1.00	1.28	6.4
	D1 - D4	1.67	11	2.25	2.29	80
D4	A	1.00	3	1.00	0.97	4.8
	B1 - B4	1.40	9	2.00	1.92	50
D5	A	1.97	3	1.00	1.36	6.8
	B	0.37	6	1.00	0.64	5.1
	C1 - C2	1.90	7	1.75	2.01	37
D6	-	0.31	14	2.25	1.01	42
D7	A	1.00	6	0.75	0.90	5.1
	B	1.40	5	1.50	1.52	18
D8	A	1.66	9	1.75	1.94	42
	B	1.80	11	2.00	2.23	67
	C	1.42	3	0.75	0.99	3.3
	D	1.37	12	2.25	2.09	78
D9	A	1.00	3	1.00	0.97	4.8
	B	2.00	11	2.00	1.66	49.8

Table 44 Hydraulic dimensioning of primary drains

In case of intersection between channel and road, the flow is conveyed by one or more concrete box culverts that has the overall width of incoming channel.

6.5 DESIGN OF ROADS

For what concerns the road system, three different types have been identified. The following figures show their layout, their typical cross section and detail about their composition.

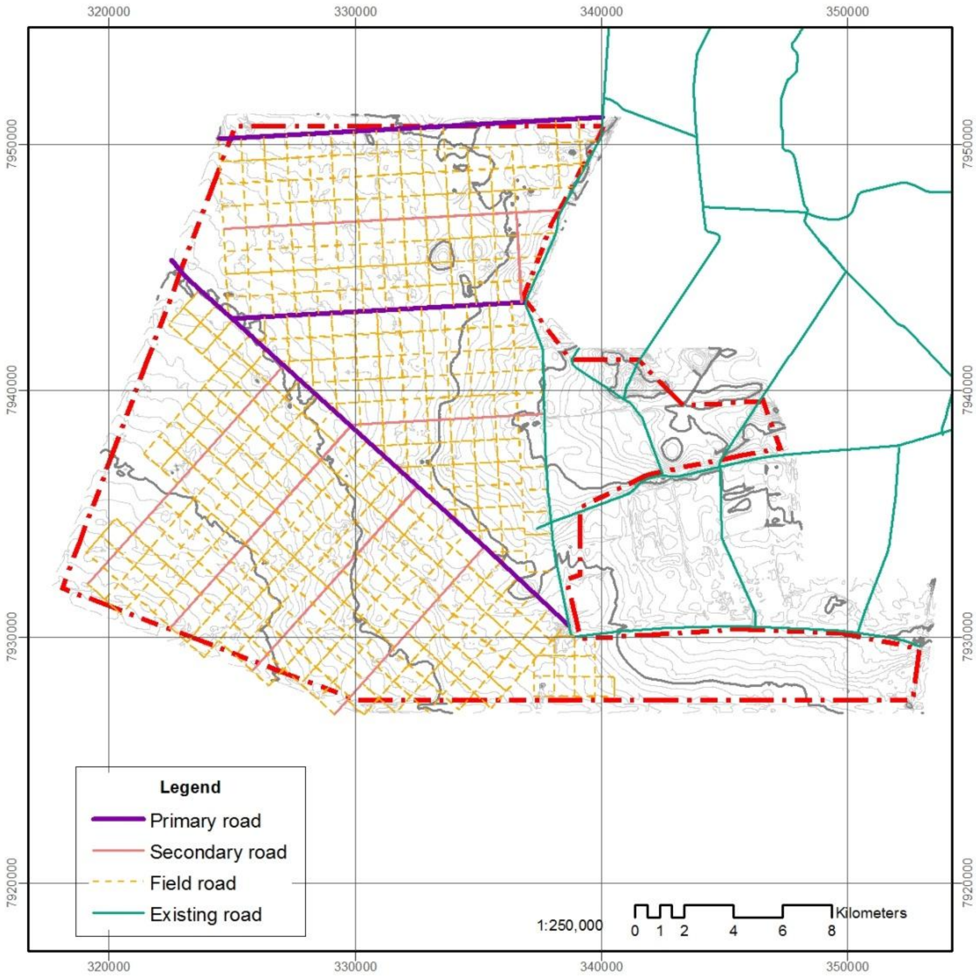


Figure 50 Layout of roads

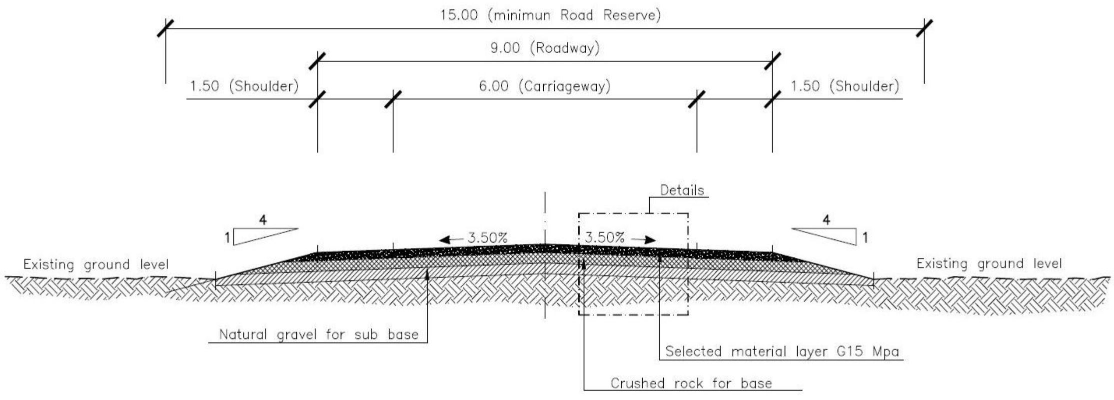


Figure 51 Typical cross section of primary road

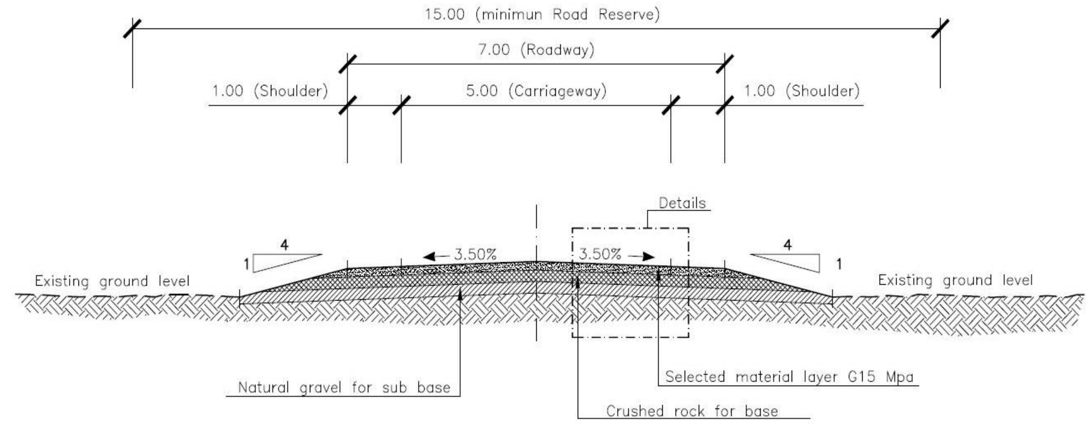


Figure 52 Typical cross section of secondary road

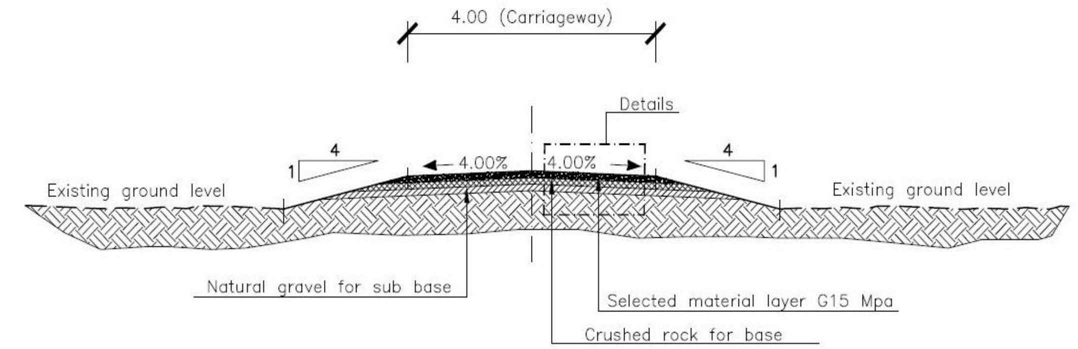


Figure 53 Typical cross section of field road

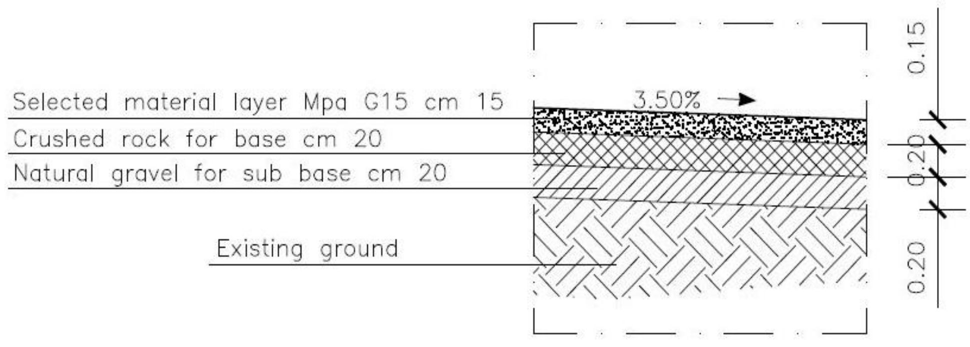


Figure 54 Detail about the composition of primary and secondary road

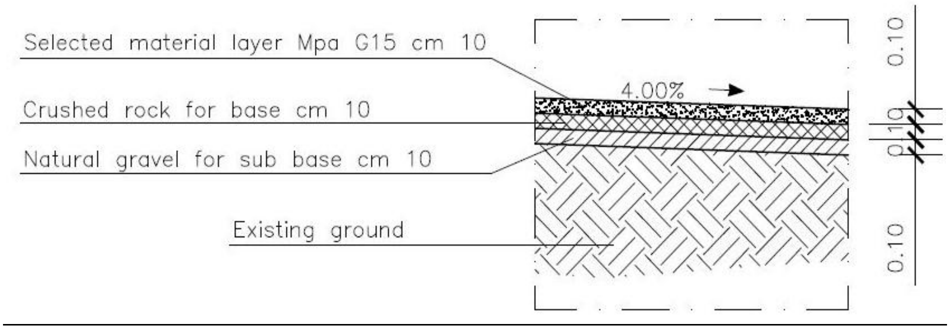


Figure 55 Detail about the composition of field road

6.6 STORM WATER AND SEDIMENT MANAGEMENT

6.6.1 Evaluation of soil losses

Erosion is the release of soil particles by wind, ice, water, etc..., a phenomenon that in rural areas might have relevant negative impact on the irrigation system. First of all, it causes loss of soil available for agriculture but it also might reduce the discharge capacity of storm water system and consequently increase the probability of floods.

Generally, in the rural storm water management this issue is faced applying the water yield model (Williams and LaSeru, 1976) or the universal soil loss equation USLE (Wischmeir and Smith, 1972), that has been also "revised" (RUSLE). This latter approach has been chosen because it has shown in numerous applications internationally that is an excellent compromise between applicability, in terms of input data for the model, and the reliability of the results.

The results of the model are significantly valid in areas in which the phenomenon of erosion is of an interill type, that is, where erosion takes place mainly in streams with the formation of small cuttings in the land. That is the case of Pandamatenga area for what can be seen in the existing farming area.

The current standard for estimating annual erosion is the Revised Universal Soil Loss Equation (RUSLE), a variant of the original USLE. The Revised Universal Soil Loss Equation is as follows:

$$A = R \cdot K \cdot L \cdot S \cdot C \cdot P$$

where:

- A is the mean annual quantity of soil eroded per unit surface area, expressed in tonnes/ha/year;
- R is the erosion factor for rain and surface runoff that takes into account the aggressiveness of rainfall events. The unit of measurement for the metric system is (feet * tonfeet * inch/acre * hour * year);
- K is the factor that allows for the erosive tendency of the soil, thus it allows estimating the detachment and transportation of land brought about by weather agents. This factor, which is a function of the chemical-physical characteristics of the land, is expressed in [t / (ha·R)];
- L,S are two topographical factors, termed "Length" and "Slope" respectively;
- C is the factor for soil coverage and soil use that takes into account the protection offered by the vegetation layer;

- P is the conservation factor for anti-erosion practices and expresses the influence on the soil loss of the particular arrangement of the cultivation and territorial arrangement works carried out (not considerate in the model).

The parameter R might be calculated as:

$$R = 8.12 + 0.562 \cdot P$$

where P is the annual rainfall (539 mm in Pandamatenga).

The parameters L and S are estimated according to the following formula:

$$LS = \left(\frac{\lambda}{72.6}\right)^m \cdot \left(\frac{430x^2 + 30x + 0.43}{6.57415}\right)$$

in which:

- λ is the field length in feet (2960 feet = 900 m);
- m = 0.3 if field slope is less than 3% (this is on average in the study area);
- x = SINΘ, where Θ is the slope angle (0.115° = 0.002 m/m on average).

The assigned values for parameter K, C and P are respectively 0.3, 0.08 and 0.5 on the basis of information proposed by US EPA and included in the following tables.

	Organic Matter Content		
Texture Class	< 0.5% K	2% K	4% K
Sand	0.05	0.03	0.02
Fine sand	0.16	0.14	0.10
Very fine sand	0.42	0.36	0.28
Loamy sand	0.12	0.10	0.08
Loamy fine sand	0.24	0.20	0.16
Loamy very fine sand	0.44	0.38	0.30
Sandy loam	0.27	0.24	0.19
Fine sandy loam	0.35	0.30	0.24
Very fine sandy loam	0.47	0.41	0.33
Loam	0.38	0.34	0.29
Silt loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy clay loam	0.27	0.25	0.21
Clay loam	0.28	0.25	0.21
Silty clay loam	0.37	0.32	0.26
Sandy clay	0.15	0.13	0.12
Clay	0.25	0.23	0.19

Table 45 Values for parameter K to apply RUSLE method (US EPA, 1973)

Land Use	C Factor
Cropland	0.08
Pastureland	0.01
Forestland	0.005
Urbanland	0.01

Land Use	P Factor
Cropland	0.50
Pastureland	1.0
Forestland	1.0
Urbanland	1.0

Table 46 and Table 47 Values for parameter C and P to apply RUSLE method (US EPA, 1973)

According to the above described method and applying the mentioned values, the resulting mean annual quantity of soil eroded per unit surface area (A) is around 0.85 tonnes/ha/year. Taking into account that the study area is totally 45,000 ha and soil has a relative density of 2.2 tonnes/m³, the overall soil loss is more than 17,000 m³/year.

6.6.2 Sediment transport

The amount of eroded sediment is surely collected by the drainage system, therefore it would be crucial to guarantee its downward transport: this can be verified taking into account the hydrodynamic condition along the channels while draining storm waters.

For a fluid to begin transporting sediment, the boundary (or bed) shear stress exerted by the fluid must exceed the critical shear stress for the initiation motion of grains at the bed. One of the most prominent and widely used incipient motion criteria is the Shields diagram (1936) based on shear stress.

Shields assumed that the factors in the determination of incipient motion are the shear stress / velocity (τ / u_*), the difference in density between sediment and fluid (ρ_s and ρ_f, respectively 2.2 and 1.0 tonnes/m³), the diameter of the particle (d), the kinematic viscosity (ν), and the gravitational acceleration (g). The relationship between these two parameters is then determined experimentally:

$$\theta = f\left(\frac{du_*}{\nu}\right) = \frac{u_*^2}{g\frac{\rho_s - \rho}{\rho}d}$$

where $u_* = \sqrt{\tau_o / \rho}$ and $\tau_o = \gamma R_H i$ in which R_H is the hydraulic radius that is defined as the ratio of the channel's cross-sectional area of the flow to its wetted perimeter.

This formula has been applied in order to calculate the maximum diameter of sediment that can be moved downward by the water current, if all the other parameters are known: results are included in the following table (hydrodynamic data are taken from Table 42 and Table 44).

Drained fields	A (ha)	Q (m³/s)	V (m/s)	Rh (m)	i (m/m)	τ_0 (N/m²)	d (mm)
1.00	0.82	1.0	0.51	0.43	0.00050	2.1	3.0
2.00	1.65	1.8	0.56	0.49	0.00050	2.4	3.4
3.00	2.47	2.5	0.61	0.56	0.00050	2.8	3.9
4.00	3.30	3.1	0.62	0.58	0.00050	2.9	4.1
5.00	4.12	3.7	0.64	0.60	0.00050	2.9	4.2

Table 48 Sediment transport along secondary drains

Drain code	Reach code	Q (m³/s)	V (m/s)	Rh (m)	i (m/m)	τ_0 (N/m²)	d (mm)
D1	A	40	1.01	1.73	0.00031	5	7
	B	9.2	1.42	0.80	0.00170	13	19
	C1 - C4	25	1.82	1.16	0.00170	19	27
D2	A	44	1.70	1.45	0.00110	16	22
	B	5.0	0.91	0.80	0.00070	5	8
	C1 - C4	22	1.22	1.24	0.00070	9	12
D3	A	45	1.40	1.53	0.00070	10	15

Drain code	Reach code	Q (m^3/s)	V (m/s)	Rh (m)	i (m/m)	τ_0 (N/m^2)	d (mm)
	B	18	2.09	0.89	0.00320	28	39
	C	13	0.80	1.12	0.00034	4	5
	D1 - D4	5.7	1.28	0.67	0.00176	12	16
D4	A	78	2.29	1.66	0.00167	27	38
	B1 - B4	4.0	0.97	0.67	0.00100	7	9
D5	A	46	1.92	1.45	0.00140	20	28
	B	5.7	1.36	0.67	0.00197	13	18
	C1 - C2	4.7	0.64	0.76	0.00037	3	4
D6	-	36	2.01	1.24	0.00190	23	33
D7	A	5	0.90	0.60	0.00100	6	8
	B	16	1.52	1.02	0.00140	14	20
D8	A	39	1.94	1.30	0.00166	21	30
	B	64	2.23	1.50	0.00180	27	38
	C	1.8	0.99	0.53	0.00142	7	10
	D	77	2.09	1.68	0.00137	23	32
D9	A	4.0	0.97	0.67	0.00100	7	9
	B	49	1.66	1.50	0.00100	15	21

Table 49 Sediment transport along primary drains

As it can be seen in previous tables, secondary drains can convey sediments with diameter lower than 3 mm, while the primary drains have an higher capacity that generally allow greater soil particles to be transported. This evaluation should guarantee the main part of sediment eroded from fields do not deposit along the drainage system.

6.6.3 Banks, sediment traps, detention and storm water ponds

The erosion of soil has the double negative effect of reducing agriculture production and discharge capacity of storm water system. This issue appears to be very critical in the study area, especially considering the current condition of Pandamatenga soil and previous experience in the neighbouring farming area.

Among the potential structural and non – structural interventions that might be proposed, it seems very cost effective to realize small banks along the field side that discharge into the drainage network. This would have several benefits:

- intercept runoff before it concentrates into a high velocity erosive force that forms rills and gullies;
- trap sediment from soil erosion;
- slow down runoff velocity (and storm flow peak) and increase infiltration, especially if crop or standing stubble are planted along the bank.

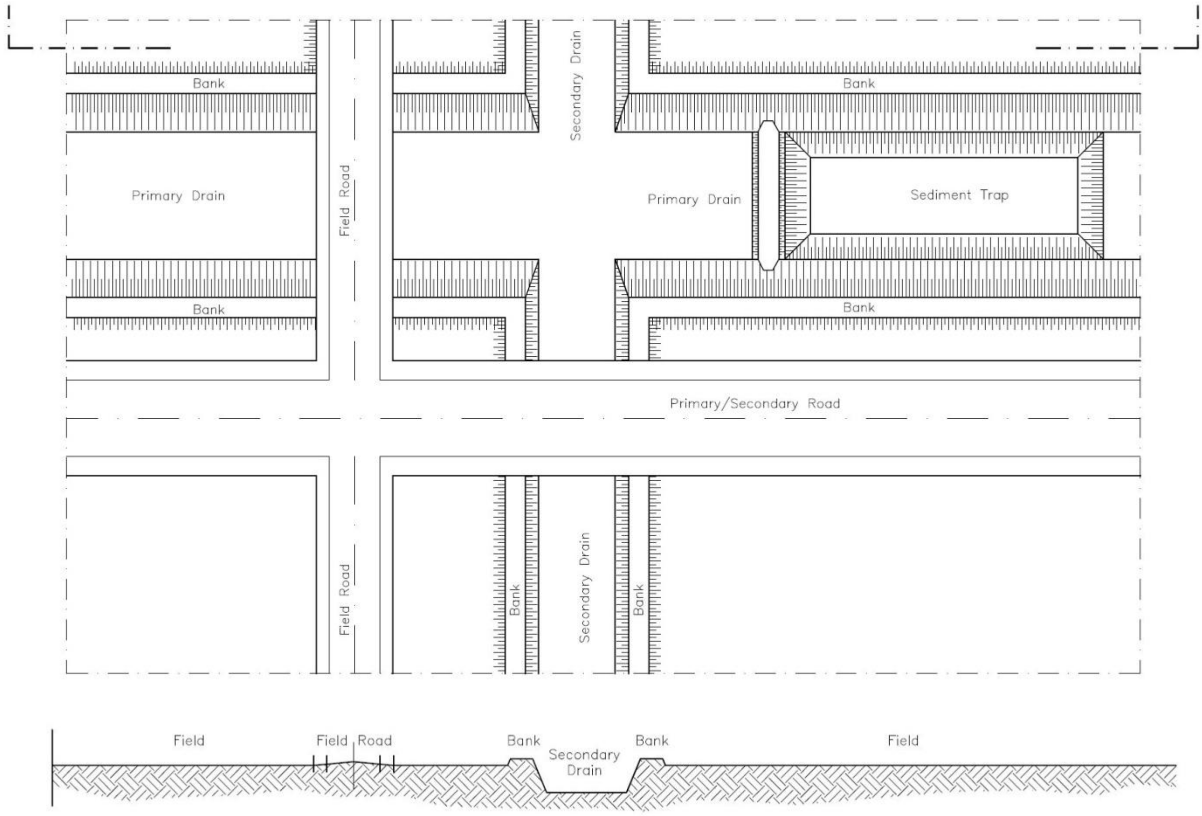


Figure 56 Planimetric view and cross section of fields, drain and road to identify designed banks

This bank could be 50 cm high and it can be interrupted exclusively at the end and in the middle of each fields (or rather every 450 m) for about 20 m.

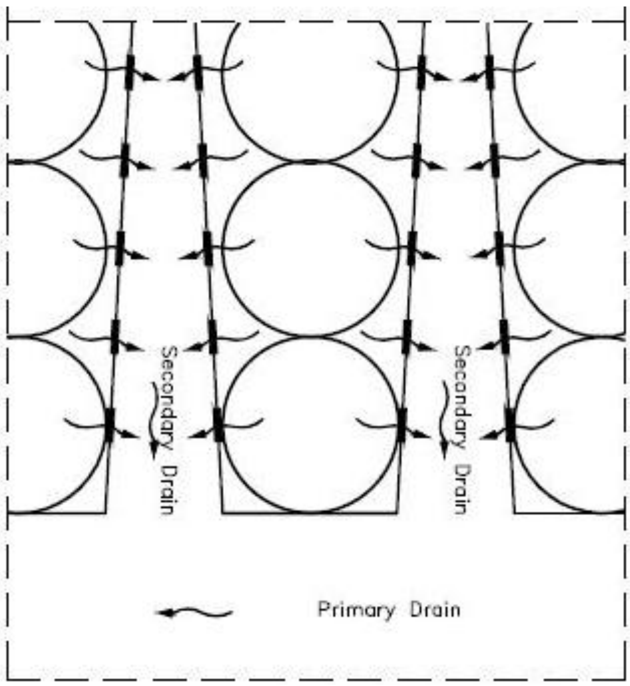


Figure 57 Outlet scheme of storm water from fields

These openings would guarantee that the water flow collected along the banks can be totally and continuously discharged into the secondary drains. In fact, these openings work hydraulically like weirs thus their dimensioning makes reference to the following equation:

$$Q = c_q LH\sqrt{2gH}$$

where:

- Q = flow discharged over the weir (m³/s);
- cq = discharge coefficient, generally supposed equal to 0,385 when broad crested;
- L = weir length (m);
- H = water depth over the weir (m).

For each fields (about 900 m long) the peak discharge for the 10-years storm event is about 1.1 m³/s thus each of these openings, being every 450 m, should be allow about half of this discharge (0.55 m³/s). According to the mentioned formula, this flow is discharged when water depth less than 7 cm is constantly over a weir 20 m long. Runoff collect within an entire field (1.1 m³/s) would be drained with 10 cm as water depth.

In order to prevent erosion and scour when the water flow pass beyond the openings, the drain slope and its bottom are stabilised and protected by gabion mattresses.

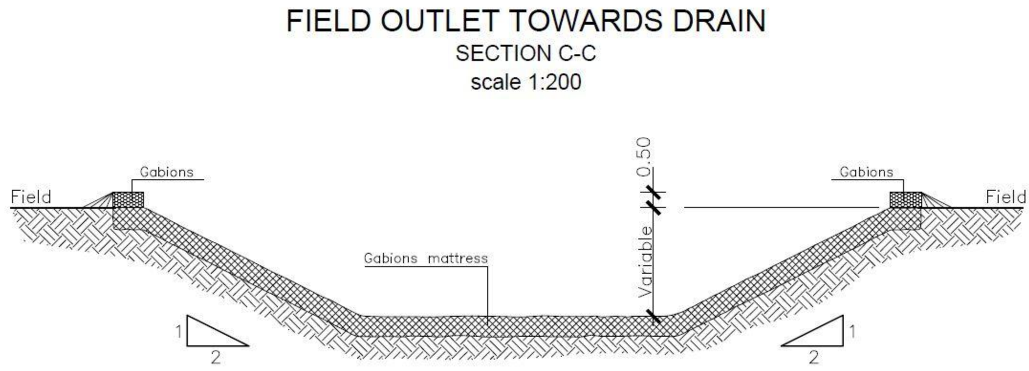


Figure 58 Typical cross section of drain where field outlet is located

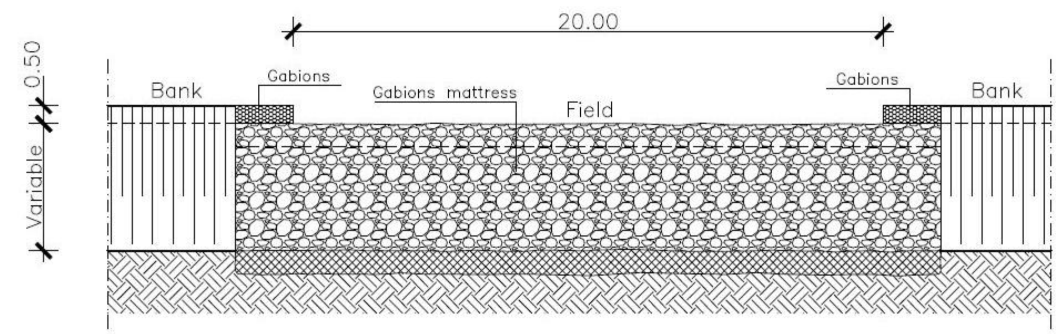


Figure 59 View of the drain slope where field outlet is located

When the storm water accumulates behind the banks, the irrigation fields would be then flooded for a length that is proportional to water depth and terrain flatness. In order to avoid such situations, the final portion of fields (a strip parallel to the bank 2.5 m wide) is properly shaped as shown in the following picture. Therefore, before having the irrigated area flooded, the water depth should be higher than 50 cm that means a discharge over the weir of about 12 m^3/s: this peak flow for a single fields has an huge return time (100-years rainfall events is 50% higher than the 10-years, as quoted in Table 7).

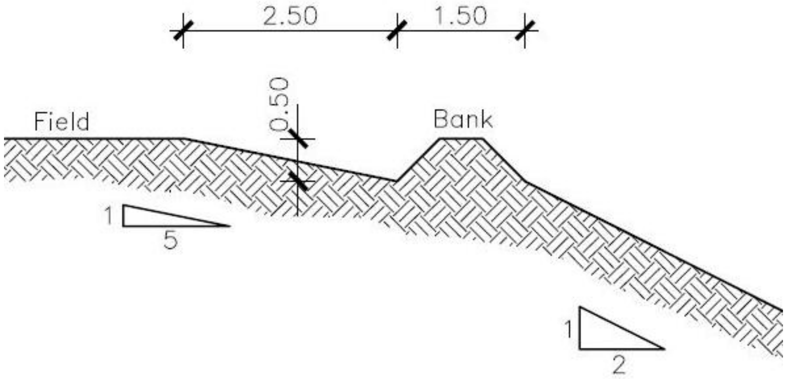


Figure 60 Cross section of designed banks

Besides, along the drainage network some sediment traps can be implemented: these structures allow for collection of sediment at single locations and reduction of downstream maintenance requirements.

Deposition happens when the capacity of sediment transport (the shear stress that moves sediment particles, see paragraph 6.6.2) decreases therefore the ability to effectively capture sediment is influenced by hydrodynamic conditions (flow velocity) and type of sediment particles (mainly size of particles). Thus the kinetic energy in the water column reduces creating a net-depositional area. According to the already mentioned Manning's formula (see paragraph 6.4.2) this effect can be obtained increasing the cross section that is available for the water flow and/or reducing the longitudinal slope.

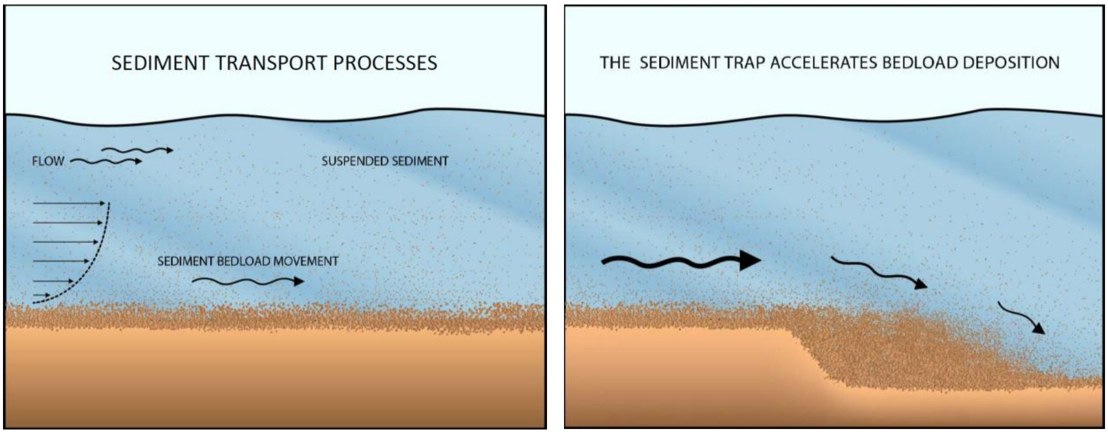


Figure 61 Schematic functioning of sediment traps

The proposal is to create sediment trap with null longitudinal slope and bottom that is 1 m deeper than the incoming channel; at the end of the trap the sediment would be further hindered by a small barrage that realize a sort of weir 50 cm high over the bottom of the channel. This type of structures should be easy to reach and to empty by machinery thus they might be localized at the intersection between channel and roads.

Along the primary drains 26 sediments traps have been envisaged (Figure 62). They are about 25 m long and large as much as the channel where they are located. On average they can be considered full with sediment when the deposition depth is about 1.5 m because their bottom 1 m of lower than the channel and at the end there is a barrage of about 50 cm (Figure 63).

For instance, along the drain D1 4 sediments traps are envisaged: the first one is within the reach B where the bottom channel is 5 m wide, therefore if deposition height is 1.5 m and trap length is 25, the sediment volume would be 5 x 1.5 x 25 = 188 m³. The subsequent reach C of drain D1 has a bottom of 9 m thus 3 traps can accumulate 3 x 9 x 1.5 x 25 = 1,013 m³.

According to these assumptions the overall amount of sediment would be almost 9,000 m³ (see Table 50) that is about half of the annual soil erosion (see calculation in paragraph 6.6.1).

Figure 62 Positioning of sediment traps along the primary drains

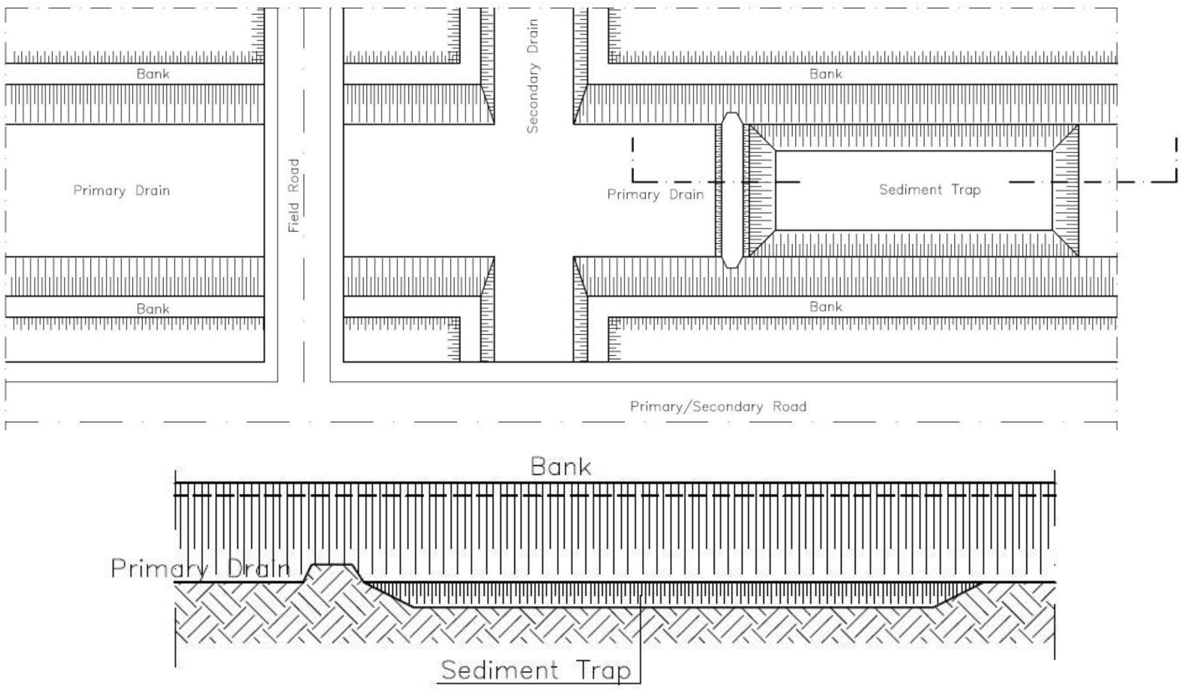


Figure 63 Planimetric view and longitudinal along primary drain to identify designed sediment traps

Drain code	Reach code	Drain bottom (m)	V_{trap} (m^3)	V_{tot} (m^3)
D1	A	0	0	0
	B	1	188	188
	C1 - C4	3	338	1013
D2	A	0	0	0
	B	1	263	263
	C1 - C4	2	450	900
D3	A	1	188	188
	B	1	300	300
	C	0	0	0
	D1 - D4	3	413	1238
D4	A	1	113	113
	B1 - B4	2	338	675
D5	A	0	0	0
	B	0	0	0
	C1 - C2	3	263	788
D6	-	3	525	1575
D7	A	0	0	0
	B	2	188	375
D8	A	1	338	338
	B	1	413	413
	C	0	0	0
	D	1	450	450
D9	A	0	0	0
	B	0	0	0
Total		-	-	**8,817**

Table 50 Estimation of sediment deposition within the sediment traps

Finally, another system has been implemented in order to collect sediments determined by soil losses along the fields: at the end of each primary drain a detention pond has been designed. This structure would receive water and sediment since the beginning of storm event, then its volume would be filled up and its entrance should be closed.

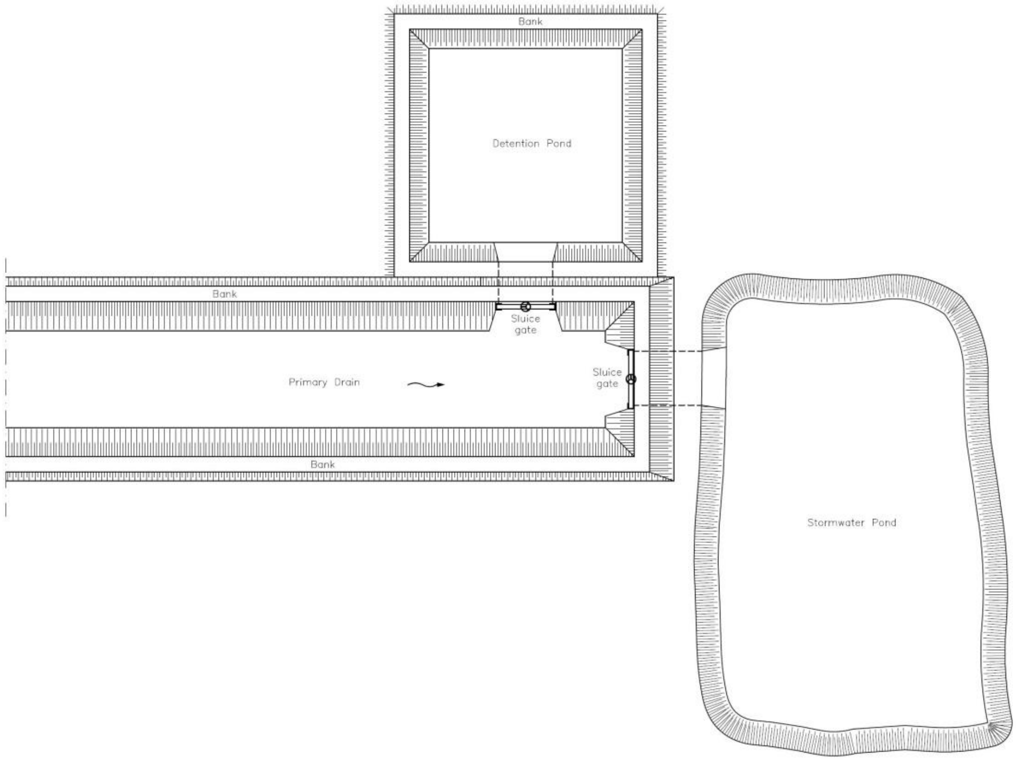


Figure 64 Planimetric view of terminal portion of a primary drain and connection with detention and storm water ponds

Their dimensioning can be roughly done considering the water volume (Vw) that would be produced during the first half an hour of the storm event. With reference to peak discharges (for about 12 hours) included in Table 43, the side length of square detention ponds is calculated considering about 2 m as height. The resulting overall volume would be more than 7,000 m^3.

Drain code	**Q (m^3/s)**	**Vw (m^3)**	**L_{pond} (m)**
D1	1.7	752	19
D3	1.9	834	20
D4	3.3	1,469	27
D5	1.9	853	21
D6	1.5	671	18
D7	0.7	302	12
D8	3.2	1,443	27
D9	2.0	914	21
Total	**-**	**7,238**	**-**

Table 51 Dimensioning or detention ponds

In this way the sediment, but also water with higher content of fertilizers, would be accumulated for deposition: at a later time sediments could be collected, analyzed, treated (if needed) and reuse.

This would minimized also the discharge of fertilizers in the natural environment. However, in order to control water drained from fields and replace natural ponds within the study area, some storm water ponds have been envisaged downstream the detention ponds.

The suggested volume of this structure could be about 20,000 m³ thus, being at the end of the 8 primary drains, their overall volume would be 160,000 m³. This water amount is far from the storm water volume estimated in paragraph 6.3.4 (124 Mm³) because its accumulation would be high costly and probably not necessary. They would reasonable be filled for each rainy event occurring in the study area and generating some runoff.

These structures are mainly a proposal to compensate the natural water ponds that are currently within the study area and would be substitute by the irrigation fields. Rough estimation on the basis on satellite imagery and topographic survey has lead to delimitate an overall extension for existing water pans of about 0.9 km². If it is assumed they have about 30 cm of water depth on average, their total volume is 270,000 Mm³, thus the proposed storm water ponds would recover the 60% of them.

6.7 DRAIN MAINTENANCE

A long-term drainage maintenance plan shall be implemented to ensure that the storm water management system functions as designed. This is mainly to avoid, or at least to minimize, reduction in discharge capacity of channels and, consequently, increase the risk of floods and damages to agricultural practise.

This plan is intended to cover all drainage structures that are represented by primary and secondary drains, but also by sediment traps, detention ponds and storm water ponds. These latter infrastructures, as already explained in paragraph 6.6, have been envisaged exactly to facilitate the maintenance of the drainage system.

The storm water management system protects and enhances the storm water runoff water quality through the removal of sediment and pollutants, and source control significantly reduces the amount of pollutants entering the system. This is particularly important because within the drained area there will be use of fertilized and because the natural environment is particularly sensitive.

The MoA possesses the primary responsibility for overseeing and implementing the maintenance plan and designating a person who will be responsible. This responsible and maintenance staff will conduct the maintenance program and they will ensure that inspections and record keeping are timely and accurate and that cleaning and maintenance are performed at least on a bi-annual basis.

A Log Forms is typically compile including the date and the amount of the last significant storm event in excess of a certain rainfall depth in a 24-hour period (10 mm could be the threshold), physical conditions of channel bottom and slope (erosion), depth of sediment, evidence of overtopping of the field banks or debris blockage at the culverts.

Records of maintenance will be kept on file at the responsable’s office and copies of maintenance log sheets indicating all work and inspections will be available upon request. All storm water management structures will be inspected two times per year, before the beginning of rainy season (November) and a month after the first heavy rain (January), as site conditions warrant.

7 COST ESTIMATION AND WORK PLAN

For the drafting of the bill of quantity, unit prices used for previous projects done in other African countries have been used as reference and, in some cases, surveys have been carried out.

The prices are in Pula, local currency in Botswana. The exchange rate to Euro (at the moment of the document drafting) is: 1 euro = 11.569 Pula. The total amount for the works is 2,298,581,477.14 Pula (equivalent to 198,684,542.93 euro).

The document is divided in chapters that correspond to complete works and chapters contains specific articles about each work. Here below the description of the chapters.

General Provision

This chapter is related to the general and temporary works paid to the Company. Particularly the temporary fence of the area (accommodation, offices, laboratories available for the Construction Supervision) and the temporary signposting of the construction site.

Land grading

The chapter pertains to the first operation of cleaning the area from vegetation and the second operation of set up of the superficial layout of the ground.

Main Drains

This chapter pertains to the operation of leveling the ground according to the elevations defined in the project with the related earthworks and backfill.

Then the main channels will be carried out with the geometric characteristics planned in the specific relations. The section of the channel will be trapezoidal with bank slope of 2/1. Part of the earthworks will need the use of the hydraulic hammer due the presence of rocks.

Secondary Drains

This chapter pertains to the operation of leveling the ground according to the elevations defined in the project with the related earthworks

Then the main channels will be implemented with the geometric characteristics planned in the specific relations. The section of the channel will be trapezoidal with bank slope of 2/1. Part of the earthworks will need the use of the hydraulic hammer do the presence of rocks.

Drain Crossing

In case drains cross the roads, cast-in-place reinforced concrete culverts will be used. For concrete Class C40 will be used as well as reinforced steel bars.

Sediment Trip

Lower areas will be placed along the channels to block sediments. Part of the earthworks will need the use of the hydraulic hammer. A concrete weir will be placed.

Primary irrigation pipe

For the primary irrigation network, cast in place concrete slab will be done and FRB FIBERGLASS PIPE with different diameter will be used, surrounding them with sand and arid material. Network development is 89 km.

Secondary irrigation pipe

Also in this case FRB FIBERGLASS PIPE will be used. Network development is 325 km.

Detention pond

The detentions pond will be carried out with earthwork and rock excavation. An adequate metal gate will be put in place with connected structures in reinforced concrete.

Stormwater pond

The detentions pond will be implemented with earthwork and rock excavation. An adequate metal gate will be put in place with connected structures in reinforced concrete.

Primary e secondary roads

Primary roads will have carriageway of 6 m (3 m for each lane) with two shoulders of 1.5 m. Total 9 m

Tertiary roads will have carriageway of 5 m (2.5 m for each lane) with two shoulders of 1 m. Total 7 m

The road package will be done with starting from the lower layers

- Natural gravel for sub base cm 20
- Crushed rock for base cm 20
- Selected material layer G15 Mpa cm 15
- Total cm 55

On the basis of designed longitudinal profile, excavation and backfill will be defined. In case of excavation it has been envisaged a certain percentage in rock, while for backfill excavation material will be used after being compressed according to a layer of about 50 cm.

Along the road small retaining walls will be probably carried out. An exiguous quantity of gabions and pavements have been considered. Adequate signals will be positioned. It has to be underlined that a top layer with material G15 Mpa will be finally realized. It is also envisaged lateral channel to collect water drained along the road.

Field roads

Tertiary roads will have a carriageway of 4 m (2 m for each lane):

The road package will be done with starting from the lower layers

- Natural gravel for sub base cm 10
- Crushed rock for base cm 10
- Selected material layer G15 Mpa cm 10
- Total cm 30

Main Pumping station

The pumping station will be realized with reinforced concrete with pups that will lift the design discharge. 9 pumps (7 main pumps + 2 secondary pumps), Q = 3 m^3/s and H = 50 m have been envisaged. Total of 12.5 MW.

The work will be completed with mechanical components, pipes, valves, wiring, switchboard and civil works. A transform plant will be built upstream the pumping station.

Electrified fence

An electrified fence 3 m high of galvanized metal that leans to reinforced concrete plinth will be realized. It will be equipped with low voltage cables (24 Volt) each meter along the height to avoid intrusions.

Transportation

4 million cubic meter of earth are foreseen will be displaced and used on the site or moved to another placed suggested by the Administration within the next 20 km. The first meter of earthwork will be reused for the new embankment.

Center pivot sprinkler

The cost for center pivot irrigation system contains mainly the total cost of the unit. There are 275 center pivot units designed for the net irrigation command area of 15,000 ha. Moreover, the cost for 275 three phase generators are also include

Drip irrigation system

The cost for drip irrigation contains for the lateral and manifold pipes with diameter of 25 mm and 110 mm respectively for 10,000 ha. Beside this, the cost for auxiliary components like measurement devices, water control valves, fertilizer applicators and all other fittings

Typical Farmstead infrastructure

This infrastructure is composed by:

- Project housing & services;
- Satellite village
- Office facilities

- Mechanical facilities, workshop
- Storage facilities

Here below a brief description of the mentioned infrastructures is given, while graphical representation of them can be found in the drawings attached to the present report.

Project Housing and Services

The typical housing and services designed at this stage of the feasibility study of this project is a service quarter, which contains living quarter (bedroom), dining room, kitchen, shower, toilet.

Satellite village

The general planning criteria adopted for the village's establishment are as follows:

• Villages should be located to minimize travel distances to work area;

• Villages should be located on non-irrigable land, as much as possible, and where possible in a well drained land;

• Villages should be located away from irrigation infrastructures and open drains where possible to be free from the effects of water born diseases.

Hence, the typical design of these villages contains living quarter (bedroom), dining room, kitchen, shower, toilet.

Office facilities

The typical office complex designed at this stage of the feasibility study of this project contains office, conference room, archives, store, toilet, casher room.

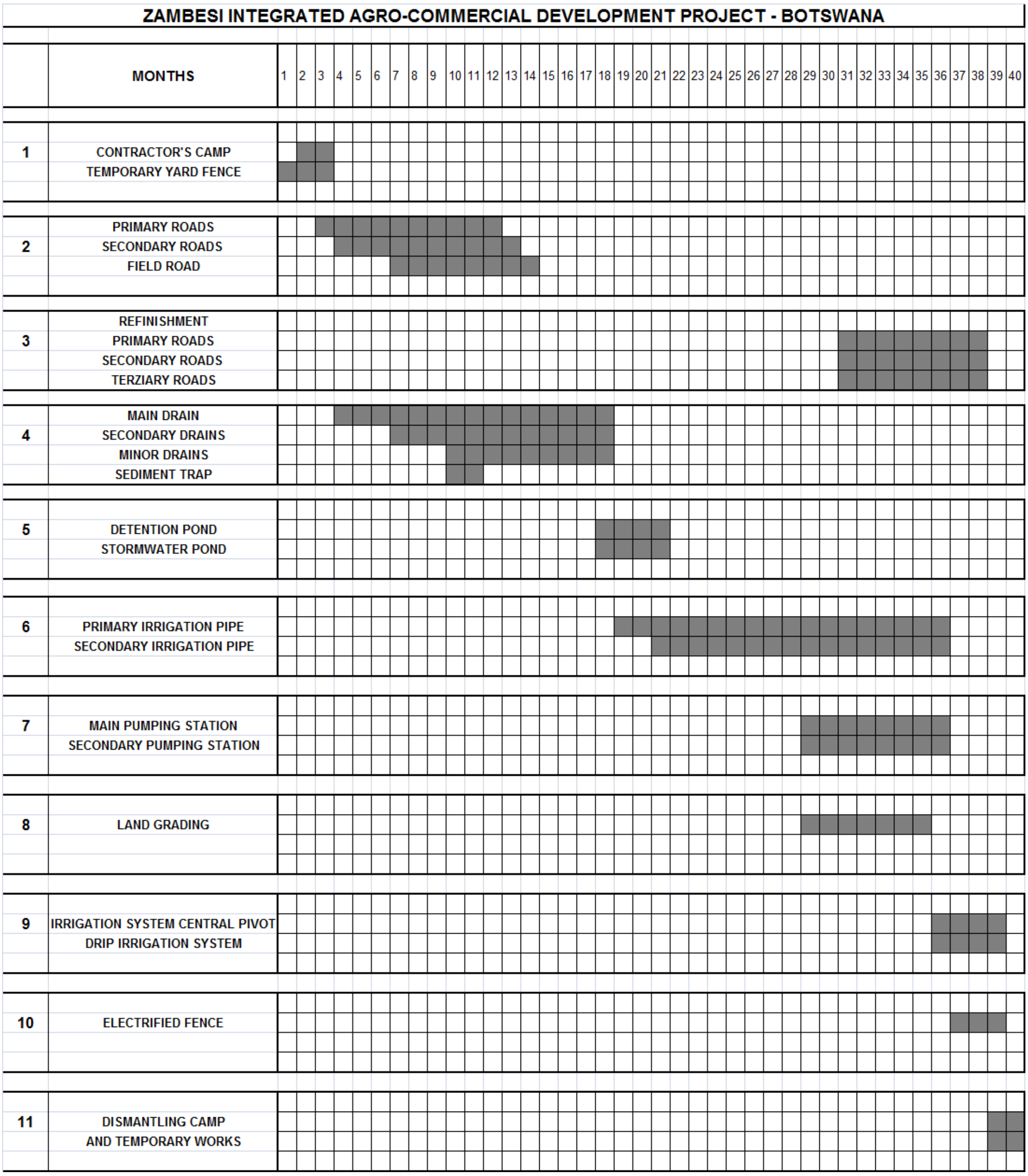
Mechanical facilities (workshop)

The typical farm workshop was designed for mechanical maintenance of farm machineries and others. This workshop contains office, welding bay, inspection pits, oil and lubricant store, sinks, toilet, hand washbasin.

Storage facilities

Typical design of on-farm storage facility was done for this project as one of the farmsteads. The storehouse is mainly for the purpose of storing farm input and produces. It contains office for the staff and the storehouses.

A first hypothesis of project work plan showing the work breakdown phasing is reported in the following table.



8 FINAL CONSIDERATION ABOUT ZIACDP FEASIBILITY

As per the ToR, pressurized irrigation system has been considered for the project area. For this depending on the soil type, both sprinkler and drip irrigation systems have been selected.

Due to several reasons listed in the document, continuous sprinkler irrigation system was taken for design as compared to the conventional sprinkler system. Out of several continuous sprinkler systems, for the same reason, centre pivot sprinkler has been selected and designed on sandy clay loam, sand loam and loamy sand soils at 17,814 ha net and 15,000 cropped area.

For sandy soils of the project area, drip irrigation system has been designed at 10,554 net area and 10,000 ha cropped area. Moreover, 15,000 ha has been allotted for both three wet (Sorghum, Sunflower and Beans) and four dry season crops (Maize, Wheat, Soybean and Alfalfa).

From this, the monthly water demand for both wet and dry season as supplementary and full-fledge irrigation supply ranges from 10.43 Mm^3 to 42.08 Mm^3 during December and September respectively.

In general, the investigation of the current feasibility study shows that the project area can be used for pressurized irrigation development, bearing in mind that application of some of the correction measures recommended are implemented in order to improve the production of crops and sustainability of soil fertility.

Therefore, it could be concluded that pressurized irrigation system is technically feasible for Zambezi Integrated Agro-Commercial Development Project.

9 ANNEXES

9.1 SOIL PHYSICAL AND CHEMICAL PROPERTIES

Within the following tables:

- Infiltration is the Basic Infiltration Rate;
- HC is Hydraulic conductivity;
- AWC is Available Water holding Capacity.

9.1.1 Soil parameters from profile pits on sandy soils

Code	East	North	pH (0-30)	Infiltration (cm/hr)	HC(m/day)	AWC (mm/m)
P2	344150	7928820	5.3			
P3	341540	7928290	5.7			
p4	340140	7928380	5.7			
p6	342523	7929808	6.6	45.4	34.4	
P7	348384	7929442	6.2			
P8	350980	7929718	5.9	22.9	23.3	
P10	347362	7927467	5.0			60.4
P11	342950	7927734	5.2	33.2	10.5	
P12	337415	7931960	6.5			
P13	339744	7929600	5.7			
P14	336975	7929870	4.9			
P15	336758	7927885	6.0			60.6
P16	335609	7928377	5.5	27.9	23	
P18	330000	7932435	6.2	20.2	21.9	69.2
P20	325891	7929286	5.8	19.9	13.2	
P21	328402	7928377	5.3	34.7	13.2	
P22	335609	7931558	5.9	30.6	12.2	
P23	334630	7930000	4.7	16.7	24.1	68.5
P25	330935	7930115	5.6			
P26	330341	7928232	6.1			
P30	337412	7932737	5.5			
P32	336749	7933991	6.8			
P33	337135	7937400	6.0			
P34	335947	7937307	5.0			
P35	334715	7936804	5.1			
P36	332505	7937300	6.0			
P37	331238	7937250	6.8			
P39	326189	7937300	6.2			
P41	320663	7937550	5.8	22.7	31	60.8
P44	325807	7935560	5.5			
P45	328950	7936138	7.2	19.1	9.1	

Code	East	North	pH (0-30)	Infiltration (cm/hr)	HC(m/day)	AWC (mm/m)
P48	330325	7935000	5.7			
P50	319908	7932450	5.3	14.2	6.8	
P51	327000	7932450	6.1	11.6	6.8	
P52	332600	7935975	6.1	19.1	16.2	
P54	327918	7940245	5.6	19.1	6.1	
P57	322834	7941726	5.5			
P58	322571	7939555	5.5			
P59	325463	7939199	6.5			56.0
P60	332777	7938719	5.0	32.7	11.5	
P62	328385	7942750	4.69			65.7
P67	330938	7940853	6.2			
P68	334633	7940220	5.4			
P69	332020	7942400	6.5			
P70	335344	7938364	6.2			
P71	337960	7947950	5.58	32.8	23.6	
P73	334554	7947276	5.71			
P77	331138	7947500	5.1			
P78	334138	7942900	5.5			
P80	324000	7945000	5.5			
P81	327000	79475000		32.5	10.7	
P82	333238	7944049	5.07			
P83	336650	7943963		22.5	21	
P84	337707	7946197	5.9			
P85	338113	7948313	5.0			
P86	339000	7949613	5.4			
P87	336650	7949288	6.5			
P89	333000	7948313	5.1			
P90	329900	7948800	5.9			
P91	328138	7948475	5.8			57.7
P92	325788	7948475	5.9	29.8	37.9	
P93	330000	7950000	6.4			
P94	338804	7939225	6.4	26.8	25.1	
P95	341324	7940304	5.3			58.4
P99	345440	7937628	4.9			
P100	342898	7937763	5.3	50.2	38.9	
P101	341212	7938482	5.1			
P102	339771	7936795	5.3			
		Average	5.71	26.57	19.11	61.92

9.1.2 Soil parameters from profile pits on loamy sand soils

Code	East	North	pH (0-30)	Infiltration (cm/hr)	HC(m/day)	AWC (mm/m)
P1	352335	7927848	5.0	0.8	**2.3**	
P17	334225	7932160	5.5	1.4	0.7	60.3
P38	330474	7936847	7.7			
P40	324963	7937209	6.0			
P46	333000	7933963	6.1			
P49	323997	7934071	6.3	19.9	18.9	87.5
P53	331950	7934288	5.8	11.7	8.8	
P55	326165	7941910		8.5	8.8	
P56	324277	7942783		13.7	9.8	
P61	329295	7937650	5.9	2.5	1.2	59.8
P74	335400	7945803	5.96	15.4	1.3	98.2
P88	334950	7948475	5.4	6.8	1.6	68.0
P96	342696	7939652	5.0			
P97	345755	7939405	5.5			87.2
P98	346767	7938437	5.1	28.3	10.3	
		Average	5.78	10.9	6.37	76.85

9.1.3 Soil parameters from profile pits on sandy loam soils

Code	East	North	pH (0-30)	Infiltration (cm/hr)	HC(m/day)	AWC (mm/m)
P5	338440	7928235	7.4	2.4	2.4	
P9	338225	7931438	7.6	6	2.4	78.5
P19	325304	7931558	7.1	4.6	1.7	
P24	328250	7930400	6.0			
P27	333029	7927590	6.4			
P28	332490	7932403	7.8			
P31	336250	7932800	6.1	4.2	2.3	81.4
P42	335076	7933700	7.6	5.8	4.2	92.5
P43	335180	7934816		4.8	3.4	
P75	328131	7945051	6.43			
P79	327661	7943422	6.1	5.3	1.4	88.1
		Average	6.93	4.73	2.54	85.11

9.1.4 Soil parameters from profile pits on sandy loam soils

Code	East	North	pH (0-30)	Infiltration (cm/hr)	HC(m/day)	AWC (mm/m)
P47	328985	7933649	6.8	6.1	3.7	
P63	334633	7941348		3.3	1.7	
P64	337218	7938859		2.4	3.1	
P65	336313	7942780	6.5	3.7	2.3	
P72	336513	7947900	5.75	1	1.8	
P76	331625	7945000	6.0	0.6	3.3	97.2
		Average	6.3	2.9	2.7	97.2

9.2 Calculation related to crop water requirement

9.2.1 Details of crop water requirement for sorghum as a supplementary irrigation

Crop-SORGHUM		Area	**6,750**	P Date	Nov. 15							
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Crop Factor(Kc)	0.91	0.93	0.56								0.15	0.64
ETo (mm)	144.46	134.68	139.5								157.5	149.11
ETc (mm)	131.88	125.84	78.75								23.63	94.76
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23	65	114
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80	29.00	66.20
Net Irrigation Requirement (mm)	48.08	61.24	49.75	0.00							0.00	28.56
Field Irrigation Requirement (mm)	60.10	76.54	62.19	0.00			77.00				0.00	35.70
Field Irrigation Requirement (mm/day)	1.94	2.73	2.49								0.00	1.15
Crop Water Needs(l/s/ha)	0.22	0.32	0.29								0.00	0.13
Flow (l/s)	1517.9	2140.5	1947.71								0.00	901.62
Mm^3	4.07	5.18	4.21								0.00	2.41

9.2.2 Details of crop water requirement for sunflower as a supplementary irrigation

Crop-SUNFLOWER		Area	**3,750**	P Date	Dec. 1							
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Crop Factor(Kc)	0.76	1.05	0.86	0.19								0.38
ETo (mm)	144.46	134.68	139.5	126								149.11
ETc (mm)	109.28	141.41	119.7	23.52								56.28
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23	65	114
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	o	0.00	0.00	0.00	0.00	3.80	29.00	66.20
Net Irrigation Requirement (mm)	25.48	76.81	90.70	0.00								0.00
Field Irrigation Requirement (mm)	31.85	96.02	113.38	0.00								0.00
Field Irrigation Requirement (mm/day)	1.03	3.43	4.05	0.00								0.00
Crop Water Needs(l/s/ha)	0.12	0.40	0.47	0.00								0.00
Flow (l/s)	446.87	1491.70	1761.36	0.00								
Mm^3	1.20	3.61	4.72	0.00								0.00

9.2.3 Details of crop water requirement for beans as a supplementary irrigation

Crop-BEANS		Area	**3,000**	P Date	Dec. 15							
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Crop Factor(Kc)	0.65	1.01	0.84	0.09								0.15
ETo (mm)	144.46	134.68	139.5	126								149.11
ETc (mm)	93.67	136.3635	117.675	10.92								23.088
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23	65	114
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80	29.00	66.20
Net Irrigation Requirement (mm)	9.87	71.76	88.68								0.00	0.00
Field Irrigation Requirement (mm)	12.33	89.70	110.84								0.00	0.00
Field Irrigation Requirement (mm/day)	0.40	3.20	3.58									0.00
Crop Water Needs(l/s/ha)	0.05	0.37	0.41									0.00
Flow (l/s)	138.44	1114.90	1244.31									
Mm^3	0.37	2.70	3.33									0.00

9.2.4 Details of crop water requirement for maize as a supplementary irrigation

Crop-MAIZE		Area	6,000	P Date	May. 15					
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**
Crop Factor(Kc)					0.21	0.65	0.87	1.05	0.91	0.06
ETo (mm)	144.46	134.68	139.5	126	119.97	94.5	99.2	129.58	160.8	186
ETc (mm)					24.77	60.95	85.92	136.06	146.33	10.80
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80
Net Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	24.77	60.95	85.92	136.06	146.33	7.00
Field Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	30.96	76.19	107.40	170.07	182.91	8.75
Field Irrigation Requirement (mm/day)					1.94	2.54	3.46	5.49	6.10	4.38
Crop Water Needs(l/s/ha)					0.22	0.29	0.40	0.64	0.71	0.51
Flow (l/s)					1346.76	1767.62	2411.30	3818.43	4243.51	3045.00
Mm^3					3.61	4.58	6.46	10.23	11.00	8.16

9.2.5 Details of crop water requirement for wheat as a full-fledged irrigation

Crop-WHEAT		Area	4,500	P Date	May. 15				
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**
Crop Factor(Kc)					0.39	0.71	0.75	1.02	0.68
ETo (mm)					119.97	94.5	99.2	129.58	160.8
ETc (mm)					46.44	67.41	74.40	132.30	108.54
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00
Net Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	46.44	67.41	74.40	132.30	108.54
Field Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	58.05	84.26	93.00	165.37	135.68
Field Irrigation Requirement (mm/day)					3.63	2.81	3.00	5.33	5.03
Crop Water Needs(l/s/ha)					0.42	0.33	0.35	0.62	0.58
Flow (l/s)					1893.88	1466.17	1566.00	2784.64	2623.05
Mm^3					5.07	3.80	4.19	7.46	6.80

9.2.6 Details of crop water requirement for soybean as a full-fledged irrigation

Crop-SOYBEAN		Area	**3,000**	P Date	Jun. 15					
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**
Crop Factor(Kc)						0.18	0.69	1.01	1.05	0.71
ETo (mm)						47.25	99.2	129.58	160.8	168
ETc (mm)						8.27	68.00	131.04	168.84	118.68
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80
Net Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	0.00	8.27	68.00	131.04	168.84	114.88
Field Irrigation Requirement (mm)	0.00	0.00	0.00	0.00	0.00	10.34	85.00	163.80	211.05	143.60
Field Irrigation Requirement (mm/day)						0.69	2.74	5.28	7.04	5.13
Crop Water Needs(l/s/ha)						0.08	0.32	0.61	0.82	0.59
Flow (l/s)						239.79	954.19	1838.83	2448.18	1784.80
Mm^3						0.62	2.56	4.93	6.35	4.78

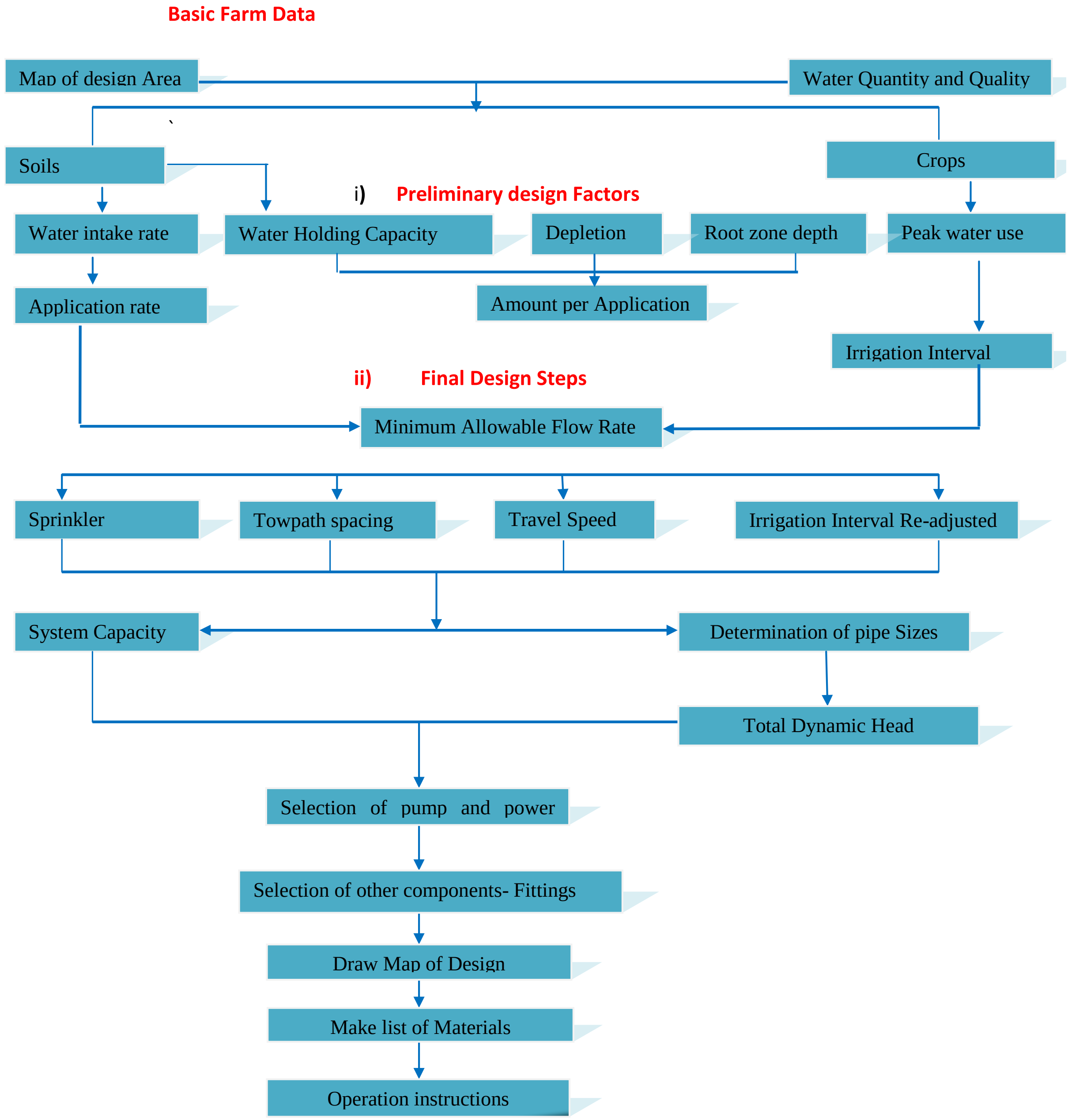
9.2.7 Details of crop water requirement for alfalfa as a full-fledged annual irrigation

Crop-ALFALFA		Area	**1,500**	P Date	Jan. 1							
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Crop Factor(Kc)	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
ETo (mm)	144.46	134.68	139.5	126	119.97	94.5	99.2	129.58	160.8	186	157.5	149.11
ETc (mm)	130.01	121.21	125.55	113.40	107.97	85.05	89.28	116.62	144.72	167.40	141.75	134.20
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23	65	114
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80	29.00	66.20
Net Irrigation Requirement (mm)	46.21	56.61	96.55	110.80	107.97	85.05	89.28	116.62	144.72	163.60	112.75	68.00
Field Irrigation Requirement (mm)	57.77	70.77	120.69	138.50	134.97	106.31	111.60	145.78	180.90	204.50	140.94	85.00
Field Irrigation Requirement (mm/day)	1.86	2.53	3.89	4.62	4.35	3.54	3.60	4.70	6.03	6.60	4.70	2.74
Crop Water Needs(l/s/ha)	0.22	0.29	0.45	0.54	0.51	0.41	0.42	0.55	0.70	0.77	0.54	0.32
Flow (l/s)	324.24	439.75	677.41	803.30	757.55	616.61	626.40	818.24	1049.22	1147.84	817.44	477.09
Mm^3	0.87	1.06	1.81	2.08	2.03	1.60	1.68	2.19	2.72	3.07	2.12	1.28

9.2.8 Details of crop water requirement for mango as a full-fledged annual irrigation

Crop-ASSORTED FRUIT TREES		Area	10,000	P Date	Jan. 1							
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Crop Factor(Kc)	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
ETo (mm)	144.46	134.68	139.50	126.00	119.97	94.50	99.20	129.58	160.80	186.00	157.50	149.11
ETc (mm)	122.79	114.48	118.58	107.10	101.97	80.33	84.32	110.14	136.68	158.10	133.88	126.74
Mean Monthly Rainfall (mm)	136.00	112	65	21	1	0.8	0	0	1.5	23	65	114
Effective Rainfall (ER) (mm)	83.80	64.60	29.00	2.60	0.00	0.00	0.00	0.00	0.00	3.80	29.00	66.20
Net Irrigation Requirement (mm)	38.99	49.88	89.58	104.50	101.97	80.33	84.32	110.14	136.68	154.30	104.88	60.54
Field Irrigation Requirement (mm)	43.32	55.42	99.53	116.11	113.31	89.25	93.69	122.38	151.87	171.44	116.53	67.27
Field Irrigation Requirement (mm/day)	1.40	1.98	3.21	3.87	3.66	2.98	3.02	3.95	5.06	5.53	3.88	2.17
Crop Water Needs(l/s/ha)	0.16	0.23	0.37	0.45	0.42	0.35	0.35	0.46	0.59	0.64	0.45	0.25
Flow (l/s)	1621.13	2295.97	3724.27	4489.63	4239.80	3451.00	3505.78	4579.42	5872.18	6415.34	4505.74	2517.22
Mm^3	4.34	5.55	9.98	11.64	11.36	8.94	9.39	12.27	15.22	17.18	11.68	6.74

9.3 FLOW CHART OF CONTINUOUS MOVE SPRINKLER SYSTEM DESIGN



Flow Chart: Design of Continuous-Move sprinkler system

9.4 DESIGN TABLE OF DRIP IRRIGATION SYSTEM

DU Type **L50/E2**

This will work on max. manifold Elevation difference + or - 1.5 m

Pipe Size Determination 2

1. Lateral

Flow rate,Q_l(m^3/h)	External dia, Do(mm)	Internal dia,Di(mm)	Hydraulic gradient, J ($^0/_{00}$)	Length,l (m)	F	Head loss,h_f(m)	Average Horizontal slope (crop direction) in all BDU's, z (%)	Average operating H of dripper - nominal pressure,□h_s(m)	□z(m)	H_i (lateral inlet H),(m)	H_d(H at the closed end of the lateral),(m)	□h=H_i - H_d (difference b/n first and last dripper),(m)
0.37	25	20.8	0.81	292	.37	0.87	0.05	15	0.15	15.73	14.71	1.02

Check for 20% rule, Δh=	**1.02**	**Ok!**	*For practical limits shall be less than 50% of allowable pressure difference*
Check for 50% of 20% , Δh=	**1.02**	**Ok!**	

2. Manifold

Flow rate,Q_m(m^3/h)	External dia,Do(mm)	Internal dia,Di(mm)	Hydraulic gradient ($^0/_{00}$)	Length , l (m)	F	Head loss,h_f(m)	Average vertical slope in all BDU's, z (%)	Hi (lateral inlet H),(m)	□z(m)	Hm (manifold inlet H)	H_d(H of last dripper),(m)	□h=H_m - H_d (difference b/n manifold inlet and last dripper),(m)
16.685	110	105.6	0.28	447.00	0.37	0.46	0.20	15.73	0.89	16.52	14.71	1.81

3. Down ward riser pipe(connect to manifold)

Flow rate, Q (m^3/h)	External dia, Do (mm)	Internal dia, Di (mm)	Hydraulic gradient ($^0/_{00}$)	Length (m)		Head loss, h_f (m)	Average Horizontal slope along the connecter pipe, z (%)	Hm (manifold inlet H)	□z(m)	Hc (connector pipe inlet H)	H_d (H of last dripper), (m)	□h=H_c -H_d (difference b/n connector pipe inlet and last dripper), (m)
16.685	110	105.6	0.28	1.50		0.00	0	16.52		16.02	14.71	1.32

4. Upward riser

4.1 Head loss

Flow rate, Q (m^3/h)	External dia, Do (mm)	Internal dia, Di(mm)	Hydraulic gradient, J ($^0/_{00}$)	Length, l (m)		Head loss, h_f(m)	Average vertical slope, z (%)	Hc (connector pipe inlet H)	□z(m)	Hr(riser pipe inlet H)
33.37	110	105.6	0.95	1.80		0.02	0	16.02	1.80	16.94

4.2. Operating head pressure	
** Loss in the field head control-valve (m)*	*5*
*** Riser pipe Hr(m)*	*16.94*
Summed to	**21.94**