REPUBLIC OF KENYA

KENYA TOWNS SUSTAINABLE WATER AND SANITATION PROJECT

Consultancy Services for

Feasibility Study, Preliminary Design and Detailed Design for the Proposed Olooloitikosh – Kitengela - Kajiado Water and Sewerage Project in Kajiado County

FEASIBILITY STUDY REPORT

Consultants:

SERING INGEGNERIA In a Joint Venture with FCL FRAME CONSULTANTS LTD

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LIST OF ACRONYMS AND ABBREVIATIONS

ADF	African Development Fund
AHJ	Authority Having Jurisdiction
AWSB	Athi Water Services Board
BWB	Basin Water Boards
BWO	Basin Water Offices
CAG	Controller Audit General
CBD	Central Business District
CBS	Central Bureau of Statistics
CC	City Council
CECM	County Executive Committee Member
Client	Athi Waters Services Board
Client	Athi Water Service Board
Consultant	JV: SERING INGEGNERIA and FRAME Consultant
CWSSP	Community Water Supply and Sanitation Programme
DO's	District Officer's Office
DRH	Direct Runoff Hydrograph
DTM	Digital Terrain Model
ESIA	Environmental and Social Impact assessment
GOK	Government of Kenya
GPS	Global Positioning System
IDF	Intensity Duration Frequency
ISO	International Organization for Standardization
KC	Kajiado County
LG	Local Government
MC	Municipal Council
MDG	Millennium Development Goal
MIC-TAF	Middle Income County Technical Assistance Fund
MODIS	Moderate-Resolution Imaging Spectroradiometer
MTEF	Medium Term Expenditure Framework
MWI	Ministry of Water and Irrigation
N/A	Non Applicable
NEMA	National Environment Management Authority
NP	National Project
NWP	National Water Policy
NWSDS	National Water Sector Development Strategy
PMF	Probable Maximum Flood
RAP	Resettlement Action Plan

RAP	Resettlement Action Plan
RFP	Request for Proposal
RPF	Request for Proposal
RWST	Regional Water and Sanitation Team
STP	Sewerage Treatment Plant
TAWSB	Tanathi Water Services Board
TB	Tender Board
TC	Town Council
ToR	Terms of Reference
TWSB	Tanathi Water Service Board
UH	Unit Hydrograph
UN	United Nations
W.H.O	World Health Organization
WB	World Bank

EXECUTIVE SUMMARY

This Feasibility Design Report presents the concepts to be adopted in the phases concerning Preliminary Engineering Design, Detailed Engineering Design and Tender Documentation.

The present Report will clarify and define the base data that shall be considered for the project development and it will assess which is the best option for the dam location, drinking water treatment plant features and location, potable water and sewerage network, treatment ponds features and locations.

The main goal of the Project is to provide water supply and sewerage infrastructure to Kitengela, Kajiado Towns and their environs. The proposed dam, after its implementation, will harness runoff waters and thereafter make them available for domestic and irrigation use, with the aim of achieving food security and poverty reduction. This will reduce the dependence on relief food as well as contribute towards alleviation of poverty within the project area through production of both subsistence and cash crops.

The project area is defined as the Kitengela Township area, Kajiado Township area and the most suitable location for the proposed dam. The climate of the project area is largely arid and semi-arid, consisting of dry and wet periods. The short rains fall between October and December while the long rains fall between March and May. From analysis of the observed climate data, there is an indication that recent changes in regional temperature as a result of global warming has had impacts on the hydrological cycle as well as changes in the variability of the climate, particularly in the frequency and intensity of some extreme climate parameters. This is attributed to an increase in available energy for evaporation due to warmer temperatureswhich in turn alters infiltration rates and soil moisture content consequently altering the precipitation amount and magnitude of runoff thus leading to droughts.

Hydrological assessments developed in this phase of the study **provide a first indication on average water availability to be compared with the projected water demand**. The outcomes of this comparison are helpful to understand which catchments need to be developed to meet present and future water demand, and the priority of intervention.

An empirical socio-economic survey was carried out within the project area to ascertain information relating to the current water and sanitation situation in the town, including issues such as demands for water and sanitation, gender issues, preferences for type of water delivery, ability and willingness to pay, current problems faced and other social economic information.

Sample size for household surveys was calculated using standard statistical formula to achieve a confidence level of 90% and an error margin of 10%. A total of 100 households were surveyed in the 3 areas covering all the towns. A systematic random sampling was chosen for its ability to produce maximally heterogeneous results.

Data was collected using other complimentary methods such as household surveys, key informant interviews, observations on the ground and study of previous literature on the topic of water and sanitation issues in the study area.

Results were coded and entered in Statistical Package for Social Sciences (SPSS) tool whereby the data was analysed by running frequencies and cross tabulations. 3 enumerators were selected based on their previous experience and a half day training program was given to them on the survey objective and methodology. Community leaders and other key stakeholders were consulted before starting the surveys.

The 2009 Kenya Population and Housing Census placed the population of Kajiado County at 687,312 people with the male and female populations standing at 345,146 and 342,166 respectively.

The estimated current (2018) population in the project areas is as follows;

a.	Kitengela Township Area	91,210
b.	Kajiado Township Area	23,336
c.	Isinya Shopping Center Area	13,469
d.	Kisaju Area	9,644
e.	Olooloitikosh Shopping Center Area	3,581
f.	Bisil Area	8,352
g.	Semi-Urban and Rural areas within the project Area	14,811

The total population within the project area is estimated to be **164,404 persons**. The rapid growth of the project towns in the recent years and lack of water and sewerage infrastructure development has led to the increased intensity of water shortage and water borne diseases. Per capita consumption rates are given in the Practice Manual for Water Supply Services in Kenya, which were applied to the population projections to compute the water demand figures. **The estimated Total Water Demand for the project is estimated to be 23,234 m^3/d, 30,588 m^3/d, 60,504 m^3/d, 101,117 m^3/d for the Present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.**

Table 0-1: Water Demand Estimates for the project sub-area and sub-components (m3/d)

Project Sub-Areas	Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	9,364	12,630	25,984	45,529
Kajiado Township	2,521	3,638	8,844	18,427
Isinya Area	1,359	1,765	3,308	5,268
Kisaju Area	975	1,265	2,370	3,774
Olooloitikosh Area	365	473	883	1,404
Bisil Area	845	1,096	2,053	3,268
Rural Areas	339	440	884	1,634
Livestock Water Demand	691	934	2,073	3,313
Crop Irrigation (Raw)	5,951	7,234	11,783	14,364
Other Water Demand	823	1,112	2,320	4,131
	23,234	**30,588**	**60,504**	**101,117**

Below are the approximate values of Water Demand in cubic meters per year (m^3/y) for each design year:

- Present (2018) 8,490,000 m^3/y;
- Initial (2022) 11,165,000 m^3/y;
- Future (2032) 22,084,000 m^3/y;
- Ultimate (2042) 36,908,000 m^3/y.

The volume of wastewater generated from Kajiado and Kitengela was initially estimated as a percentage of the water demand for the various components as 85% less 10% to account for minor leaks as per the guidelines given by the Practice Manual for Sewerage and Sanitation Services in Kenya (2008). A conversion factor of 80% was eventually adopted however as a conservative estimate. The estimated Wastewater flow rates for the main urban areas within the project area are as shown below.

Table 0-2: Estimated Wastewater Generation Rates for the main urban areas within the project (m3/d)

Project Sub-Areas	Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	7,491	10,104	20,787	36,423
Kajiado Township	2,016	2,910	7,075	14,742
	9,507	**13,014**	**27,865**	**51,165**

Several dam sites have been taken into consideration, surveyed and evaluated during these four months. Five dam sites have been identified in which it could be possible to build a dam.

These sites are known as the Ereteti, Airfield, Kitengela, Noopopong and Oloyankilani sites and they are along the main Kiserian – Isinya Road. The locations and main features of the five dam sites have been studied and detailed.

From the information generated on the water demands and the storage capacities of the five reservoirs as mentioned above deductions have been made. The storage capacities of the 5 dams mentioned above have been initially estimated as follow:

- Ereteti 4,800,000.00 m3;
- Airfield Dam 7,300,000.00 m3;
- Kitengela 34,200,000.00 m3;
- Noopopong 1,800,000.00 m3;
- Oloyankilani 4,400,000.00 m3.

The five sites have been evaluated and studied from a Geological point of view. Groundwater levels are present in the area, wells are situated in natural drainage basins, obtaining water from permeable bands such as biotite gneisses, within the Basement System. In volcanic areas the best supply comes from the junction of volcanic rocks and the Basement System, the weathered surface of which acts as an aquifer.

Preliminary survey confirmed the presence on site of material with proper general characteristics (which will be tested) for the dam construction. There is availability of material, which can be recycled as aggregate for concrete for the construction of structural works and for the body of earthfill/rockfill dams. The rock outcrops in the dam sites seem to fit the purpose of a dam construction from geo-mechanical and geotechnical point of view. The permeability of this type of rocks is very low in the intact portion, but it must be considered that the permeability is greatly increased by the presence of fracture systems and weathered layers.

A first selection of the best type of dam for the particular sites has been carried out. The said first selection calls for thorough consideration of the characteristics of each type, as related to physical features of the site and the adaptation to the purposes the dam is supposed to serve, as well as safety, economy, and other pertinent limitations.

The geological, hydrogeological and geotechnical studies, together with the results of the site investigations, will allow to choose the most appropriate location of the dam and of the section type and to define the nature, the geometry and the consistency of the available materials. The design criteria of dam structures using locally available materials and adaptable to the

socio-economic conditions should consider two types of cost-effective structures, which include **earth fill dams and rock fill dams**.

The option to use a large quantity of the same material indicates the possibility of proposing a homogeneous rock fill dam. Sufficient quantities of both pervious and impervious materials lead to the use of zoned earth fill dams. For each structure, the final specifications concerning height, foundations and waterproofing vary widely, depending on the numerous factors listed previously.

Based on the geomorphological conditions of the dam sites and our experience, various watertight options may be related to earth fill and rock fill dams:

- an impervious central silty clayey core;
- a plastic concrete core or a bituminous core;
- an impervious lining upstream.

Depending on the geological and geotechnical features of the dam foundation soils (nature, grainsize composition, permeability, fracturing, etc.) a curtain grouting or a foundation diaphragm can be realized to achieve the water tightness of the dam foundation and the safety against piping. Geotechnical investigations and laboratory tests will be carried out to develop the next design phases for the selected dam. It has been described in the list of investigations to be performed. The investigations that will be carried out in the next phase (the Draft Preliminary Design phase) aim to:

- recognize in detail the geological, hydrogeological and geotechnical features of the foundation soils;
- define the physical/mechanical properties of soils and rocks;
- define the soil-structure interaction;
- identify depth, type and strength of bedrock and other pertinent sub-surface data;
- set down the extent and the quantities of suitable borrow materials.

In this phase **we developed at an advanced stage the drinking water treatment plant** in order to have an idea of the land needed to build it. The report proposes a DWTP with two different parallel water lines in order to allow the client to phase the implementation of the project. In the first phase, after the total acquisition of the land for the entire treatment plant, the client can limit the construction to a single water and sludge line. This will satisfy the actual demand. Next phase is to duplicate the existing lines to complete the treatment plant and meet the future demand.

The treatment cycle consists of the following phases:

Water line

i. Duct and fine Grilling
ii. Equalization Chamber and Primary roughing
iii. Clariflocculation with single basin system "Accelerator"
iv. Quick filtration on sand beds
v. Final disinfection by sodium hypochlorite
vi. Treated water storage
vii. Backwashing of filters
viii. Accumulation of backwash water

Sludge Line

i. Thickening Sludge Treatment
ii. Chemical conditioning
iii. Mechanized dehydration
iv. Drying beds

With regards to the method of wastewater treatment, a **waste stabilization ponds (WSP) system is recommended for Kitengela due to its low operation and maintenance, and the proven effectiveness in the local climate**. The capital cost of buying land required for WSPs by far outweighs the operational and maintenance cost of the other alternatives. Additionally, the Consultant proposes a centralized sewerage system as opposed to a decentralized one as discussed within this feasibility report.

The Consultants further proposes phasing of the sewerage project as follows;

Phase I – This will entail the following:

- Acquisition of all 75 Hectares required for the present and future construction of the waste water treatment plants.
- Construction of an 18,250m^3/d waste stabilization ponds system.
- Construction of Trunk Sewers up to the currently settled areas.
- Construction of Reticulation Sewers in the already densely settled areas of the project area. These areas include the main Kitengela Business Center, Milimani area, Noonkopir, residential area bordering the EPZA and the Yukos area.

Phase II – This will entail construction of the following;

- An additional 18,250 m^3/d waste stabilization ponds system.
- Extension of the Imambaraswani River Trunk Sewer and Kitengela River Trunk Sewer to cover the additional areas currently not settled but predicted to be settled in the future.

Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions. These areas include the area south west of the project area along the Imambaraswani river and the area south (upstream) of Kitengela River. As with Kitengela, a centralized sewerage system is proposed for Kajiado, with a waste stabilization ponds (WSP) system being recommended as the cost of land is relatively low and the energy cost is very high.

The Consultant proposes that the sewerage project in Kajiado be implemented into two (2) phases as follows;

Phase I – This will entail the following;

- Acquisition of all Thirty (30) Hectares required for the present and future construction of the waste water treatment plants.
- Construction of a 7,350m^3/d waste stabilization ponds system.
- Construction of the 11.9km Olkejuado Trunk Sewers.
- Construction of the sub-trunk sewers and Reticulation Sewers in the already densely settled areas of the project area.

Phase II – This will entail construction of the following;

- An additional 7,350m^3/d waste stabilization ponds system.
- Extension of the Olkejuado River Trunk Sewer upstream to cover the additional areas next to the airstrip that is currently not settled but predicted to be settled in the future.

Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions.

Together with the recommended WSP system, comprehensive inlet works are recommended for both townships. The inlet works will comprise coarse and fine screens, grit removal mechanisms and flow measuring mechanisms as well as emergency by-passes.

Additionally, in order to supplement the available water for irrigation from the proposed dam, the Consultant recommends the use of treated wastewater for unrestricted irrigation.

It is proposed to use on-site sanitation facilities in the peri-urban and rural areas within the project that are not covered by the sewerage networks as designed. These facilities include:

i. Pit Latrines in Rural Areas,
ii. Ventilated Pit Latrine in Rural Areas with higher Population,
iii. Pour Flush Latrines in Institutions in Rural Areas,
iv. Septic Tanks and Package Waste Water Treatment Plants in Peri-Urban Areas.

Where soil drainage capability is poor, use of advanced package plants is recommended.

Like any other development, the proposed Dam, Wastewater and Water Treatment plant projects will bring, in their implementation, certain impacts. These projects are listed in the Second Schedule of Environment Management and Coordination Act (EMCA, Revised 2015) among projects that must undergo Environmental Impact Assessment (EIA) before they can commence. Impacts identification is followed by impact prediction in which case the significance of the potential impacts is determined. Predictions are normally based on explicit assumptions about environmental processes, professional judgment and different value judgments expressed by various stakeholders during consultations.

Environmental impacts manifest at all stages of a project. That is Preconstruction, Construction, Operation and Decommissioning Phases. This is because of the phase specific project activities that inform particular actions which in turn act on environmental factors in different ways generating the impacts. While some impacts are phase specific, others persist throughout the entire lifespan of the project. Conversely, some impacts are project specific mainly due to inputs and outputs. The significance of these impacts is also varied. The impacts vary in severity, duration, spatial extent, and reversibility among others. The potential impacts are presented and discussed in the main report.

A RAP report will be prepared in line with the Kenyan guidelines and international best practices including the AfDB's safeguards that have been designed to mitigate against impoverishment risks associated with Involuntary Resettlement and the restoration or improvement of income-earning capacity of the Project Affected Persons (PAP). The policy requires full public participation in resettlement planning and implementation and describes the conditions that borrowers are obliged to meet in operations involving involuntary resettlement.

The Financial and Economic analysis carried out follows the incremental method. The project is analysed based on data locally collected and on the difference of the costs and revenues between the baseline situation (without the project) and the investment situation (with the

project). Therefore, all incremental costs and benefits following implementation of the project will be quantified. Regarding the methodology, the analysis focuses on three main project components and its alternatives for Kitengela and Kajiado townships, such as Dams + Water supply network +Sewerage system.

Analysis gives evidence on a system composed of dam, water supply network and sewerage system. In order to select the most appropriate configuration, using both financial criteria and socioeconomic indicators. The following have been calculated for each of the project options: **the project's financial profitability based on the investment costs, the operating and maintenance costs and the revenue over the project horizon**, i.e. the project's cash flow. The financial profitability is assessed in terms of the project's net present value (NPV). The NPV is the difference between the present value of cash inflows and the present value of cash outflows and is used in capital budgeting to analyse the profitability of a projected investment or project. When the cash flow is discounted (with the weighted average cost of capital) back to a net present value, we obtain the project's NPV[1]. The main results of the financial analysis can be summarised as it follows:

- **Kitengela dam** is the only option able to cover the higher percentage of water demand expected by the forecast. In these terms, it will be cover 100% (2018); 100% (2022); 75% (2032); 45% (2042).
- If we give the priority to satisfy the demand for domestic, school and health as project priority Kitengela dam will be able to cover 100% (2018); 100%(2022); 100% (2032); 59% (2042).
- In terms of operating and maintenance costs, the most suitable options are Kitengela, Ereteti and Oloyankilani. For all these options, the net cash flow is positive according to the demand and tariff considered as assumptions.
- The Net Present Value for Kitengela, Ereteti and Oloyankilani net cash flow is positive considering two-discounted rate as parameters (8%; 13%).

According to international benchmarks and recommendations from World Health Organisation, it is possible to resume the main project externalities as follows:

- Direct Expenditures avoided, due to illness diarrhoeal disease reduction;
- Income gained, due to the reduction of day-off due to disease;

[1] The discounted rate calculated for our analysis is based on the interest rate used for twenty five year fixed coupon Treasury bond issue from Central Bank of Kenya.

- Days of school absenteeism avoided;
- Productive parent days lost avoided, due to less child illness;
- Time spent every day to collect water.

A project team composed of dam engineers, hydraulic engineers, socioeconomic and financial experts, environmental experts and sociologists evaluated each dam site option, referring to seven indicators. The indicators are:

1) Capital costs;
2) Operation & Maintenance costs;
3) Resettlement cost;
4) Community Benefits;
5) Financial Sustainability;
6) Water Demand satisfied;
7) Site Suitability for Dam Construction;

The outcomes of such evaluation and analysis show that **Kintegela and Ereteti** site options are the most suitable and present the major benefits for the involved population. They present the best indicators from social, engineering and financial perspectives. In these terms, the ranking is (from the higher score to the lower): Kitengela (73%) - Ereteti (65%) - Noopopong (35%) - Oloyankilani (20%) - Airfield (18%).

Considering the foreseen water demand and the reservoir capacity of the two high ranked sites (Kitengela and Ereteti), only Kitengela site option should be considered as the most recommended option for the dam construction.

At the end of the document in the **Project Cost Estimates** paragraph it is reported the total construction costs and total operational and maintenance costs for two different option and for each dam site. The ones that have the total maximum costs (considering dam type and one different WWTP option) and the ones that have total minimum costs both construction and operation costs.

1 INTRODUCTION

1.1 OBJECTIVES AND SCOPE OF THE ASSIGNMENT

1.1.1 Premises

The main objective of the assignment is to provide consultancy services for the development of detailed studies for the Olooloitikosh-Kitengela-Kajiado Water and Sewerage Project. These detailed studies include the provision of detailed designs, ESIA and RAP studies for water supply and sewerage infrastructure within Kitengela, Kajiado, Olooloitikosh and its environs. The proposed project is aimed at enhancing hygiene and sanitation, while also setting up the basic infrastructure to allow for the development of irrigation capabilities for purposes of achieving food security and poverty reduction within the region. This will reduce the dependence on relief food as at present, farming within the project area is mainly rain-fed and hence very unreliable because of the erratic nature of rainfall.

1.1.2 Objectives of the Consultancy Service

The key objective of the consultancy assignment is to develop Detailed Studies for the Project. It also includes project planning where the consultant will clarify the timing, sequencing and coordination of studies in the implementation phase, as well as finalizing the TOR for a Consultancy Services for Review and Supervision of the Works.

Additionally, the assignment aims to assist Athi Water Service Board and the stakeholders in adjusting the Environmental and Social Management Framework to specify sufficient safeguard policy measures for effective mechanisms to ensure its implementation.

1.1.3 Scope of the Assignment

The primary task of the consultant/s is to support the activities of Athi Water Services Board and other stakeholders in the preparation of Detailed Studies for the Oloolotikosh-Kitengela-Kajiado Water and Sewerage Project. It is assumed that the Consultancy Team will provide professional guidance with high quality services within a timeframe of 12 months.

The scope is to build from any available desk materials, field data and carry out studies set out in this document. The consultant/s will be accountable for ensuring that the Designs are in line with international standards and the Ministry of Water and Irrigation design manual among other accepted reference manuals for water projects.

The consultant is also required to identify any other alternatives, if any, and assess the alternatives in terms of geotechnical condition, technical possibilities, generation efficiency, preliminary environmental impacts, economic/financial viability and other related costs.

Additionally, the Consultant is to propose the best site for the dam based on the assessment, consider alternative dam sites and whether there will likely be more dams in the future upstream or downstream and consider possible stepwise development, if applicable.

The Consultant shall recommend the types of further studies and assessments needed based on current internationally accepted practices applied when preparing similar projects.

The Consultant will be expected to undertake the following tasks in order to prepare the detailed Design Study of Oloolotikosh-Kitengela-Kajiado Water and Sewerage Project;

a) Carry out preparatory studies including geotechnical, hydrological, and topological studies;
b) Perform an analysis of the alternatives,
c) Perform additional preparatory studies for the chosen site,
d) Develop a preliminary design of the dam, water supply and sewerage infrastructure;
e) Conduct preliminary economic and financial study,
f) Develop final studies for:
   I. Detailed Economics and Financial Study; and
   II. Technical Detailed Design and interrelating those parts in one process;
g) Finalize the TOR for consultancy services in Design Review and Supervision of Works in consultation with AWSB, and Stakeholders, including holding public consultations/hearings.

The detailed scope of works shall include the following;

a) Conceptual Design and Detailed Feasibility Study,
b) Preliminary Design,
c) Social and Environmental Impacts Assessment and Resettlement Action Plan for the selected dam site,
d) Detailed Design.

1.2 SCOPE OF THE FEASIBILITY STUDY REPORT

The Feasibility Study Report has been prepared in accordance with the Reporting Requirements set out in the signed Contract No. AWSB/KTSWSSP/CS/06/2016 and the Contract Addendum.

It presents the concepts to be adopted in the following studies. It presents the following;

a) Brief Introduction Information of the Project,

b) Brief Background information of the project area,

c) Current and Future Development Planning within the project area,

d) Population Estimates,

e) Water Demand and Sewage flows estimates for the various areas within the project area,

f) Water quality assessment,

g) Hydrological study outcomes,

h) Feasibility evaluation of the dam sites, and dams type selection;

i) Economic and financial feasibility studies.

2 BACKGROUND INFORMATION

2.1 PROJECT LOCATION

Kajiado County is located in the southern part of Kenya between longitudes 36º 5' and 37º 55' East and latitudes 1º 10' and 3º 10' South bordering Nakuru and Nairobi City to the north, Narok to the west, Makueni and Machakos to the east and Taita-Taveta and the United Republic of Tanzania to the south. The county covers an approximate area of 21,901km2.

The county has five sub-counties: Kajiado Central, Kajiado North, Loitokitok, Isinya and Mashuuru with the total number of administrative divisions coming to seventeen (17No.).

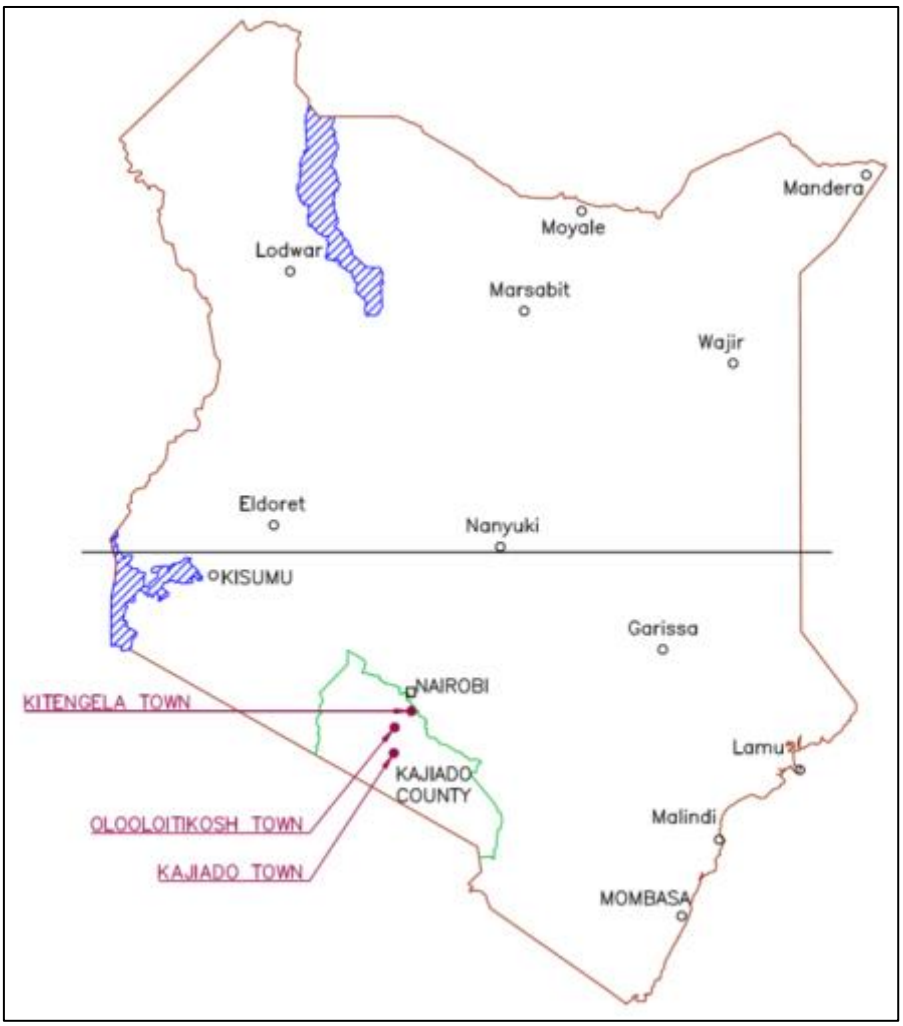


Figure 2.1: Location of the Project Area within Kajiado County and Republic Kenya

2.2 TOPOGRAPHY

Physical features in the project area range from plains to hills with altitude ranging from 1500 meters in the Kitengela area and as high as 1700 meters above sea level in the Olooloitikosh

and Kajiado Township areas. The topography of the project area can be defined as the central broken ground between the Ngong Hills and the Athi Kapiti plains.

The Athi Kapiti Plains are made up of gently winding slopes which begin to roll as they approach the Ngong hills. These hills are a catchment for Athi River, which is fed by Mbagathi and Kiserian tributaries.

The Central Broken Ground area stretches from the north to the eastern boarder across the project area to the southwest spanning about 70 kilometers.

2.3 CLIMATE

The climate of project area is largely of the arid and semi-arid type consisting of dry and wet periods. The short rains fall between October and December while the long rains fall between March and May. There is a general rainfall gradient that increases with altitude.

Temperatures vary both with altitude and season. The highest temperatures of about 34°C are recorded at Lake Magadi with the lowest temperatures of 10ºC recorded at Loitokitok on the eastern slopes of Mt. Kilimanjaro. The coolest period is between July and August, while the hottest months are from November to April.

Discussion of the various climatic parameters of the project area in presented in a separate topic.

2.4 GEOLOGY AND SOILS

The Geology varies from one area to the next. The basement complex of Kajiado Township area was formed during the Precambrian Era. Gneiss, schist and limestone were severely weathered leading to the exposure of some of these rock formations. The approximate areas in the Ololooitikosh consist of tertiary formation. Quaternary volcanic soils are also found in the project area.

Soils vary based on parent material relief, vegetation and human and animal activity. Most of the soils are poorly developed prohibiting large scale farming. The soil depth in the Athi Kapiti Plains ranges from 0.5 to 1.0 meters.

The texture varies from clayey soils of the Athi plains to sandy soils of south of the Kajiado Township area.

2.5 DEMOGRAPHICS

The 2009 Kenya Population and Housing Census placed the population of Kajiado County at 687,312 people with the male and female populations standing at 345,146 and 342,166 respectively. The County Government of Kajiado projects the population of the County in 2017 to be 999,819 people of which the male and female populations are 502,077 and 497,742 respectively.

The estimated current (2018) population in the areas to be considered under the project are as follows;

a) Kitengela Township Area – 91,210
b) Kajiado Township Area – 23,336
c) Isinya Shopping Center Area – 13,469
d) Kisaju Area – 9,644
e) Ololoitikosh Shopping Center Area – 3,581
f) Bisil Area – 8,352
g) Semi-Urban and Rural areas within the project Area – 14,811

The total population is estimates to be 164,404 persons.

2.6 ECONOMIC ACTIVITIES AND LIVELIHOOD

Livestock rearing as the main economic activity can be attributed to the arid and semi-arid nature of the county where sheep, goats and cattle are kept for both commercial and domestic purposes. These animals are kept by both large and small scale farmers.

A growing number of farmers have taken to drip irrigation farming in some plots in Kajiado and Kitengela where crops such as kale, cabbages, onions and tomatoes are grown.

Tourism is a major economic activity attracting a significant number of domestic and foreign tourists.

2.7 INFRASTRUCTURAL DEVELOPMENT

There are only two (2) major tarmacked roads in the project area; the Namanga-Kajiado-Athi River Road and Isinya-Kiserian Road.

The railway line in Kajiado is mostly meant to connect Tata Chemicals Ltd to the Nairobi-Mombasa railway line. The railway line is meant to transport soda ash and its by-products to Mombasa.

There are several airstrips within the project area. They are found in Kajiado Town, Olooloitikosh, Ngong, Magadi, Daraja and Amboseli National Park.

2.8 HEALTH CARE

Within the county, there are 4 sub-county hospitals, 16 health centres and 60 dispensaries, all of which are run by the county government. The distribution of privately owned health facilities is as follows; there are 6 hospitals, 13 nursing homes, 7 health centres, 27 dispensaries and 101 clinics. These are mainly found in the urban areas.

The doctor to patient ratio stands at 1:26,000 whereas the nurse population ration is 1:1,068.

Non-governmental organizations are working to better access to healthcare in Kajiado County.

2.9 EDUCATION

There are a number of public and private schools in the area from ECDE schools, primary school, Secondary Schools, Colleges, Polytechnics and Private Universities.

Enrolment to school remains low in the rural areas. This has contributed to the low literacy rate of 65.2% in the Kajiado County compared to the national average of 71.4%.

2.10 HOUSING

There is a marked variation in housing in the urban, semi-urban and rural settlements. In urban centres there are both high-end homes and low-income homes. The semi-urban settlements have mainly permanent and semi-permanent houses. While in the rural areas there are semi-permanent houses and manyattas.

2.11 WATER RESOURCES AND WATER SUPPLY

The area boasts of accessible underground and surface water sources. Ground water yields vary throughout the area from 0.01 to 35.77 cubic meters per hour and this water is reported to be of good quality as it is used for domestic, livestock and irrigation purposes.

There are high yielding springs in the region bordering Mt. Kilimanjaro with average yields of $20m^3$/hr to $510m^3$/hr. Other water sources for domestic purposes and for livestock are found in the sub-surface region where they consist of water pans, dams and shallow wells.

Due to the growing population, the areas of Kajiado and Kitengela towns and parts of lower Athi, located to the north of Kajiado County have been faced with persistent water consumption shortages.

The area is facing a serious water shortfall in the near future, at which point the existing water sources which are already outstretched may be obsolete. Water vending has become a booming business as the precious commodity gets scarce by the day. This water shortage will continue increasing in respect to the rapid expansion of these towns and their populace.

2.12 SANITATION AND SEWERAGE FACILITIES

The growth of the towns and lack of water and sewerage infrastructure development has led to the increased intensity of water shortage and water borne diseases.

Sanitation in the project area is generally inadequate with no reticulated sewerage system. Sewerage service is mostly by use of septic tanks and cesspools in the Kitengela and Kajiado Townships. Landlords are known to empty their sewerage into the streets at night while some have built pipelines that run straight into Rivers.

In the rural areas, 50% of the households have been reported to practice open defecation. Pit latrines are also common in the area.

3 POPULATION ESTIMATES

3.1 DEVELOPMENT OF THE PROJECT AREA

3.1.1 General

In the 1980s the land tenure system consisted of alienated land registered as government land. Registered trust land is land registered in the name of individual owners and unregistered trust land is held communally and allocated by customary laws (no title deeds are held on this type of land). Forest reserves are under the control of the chief conservator of forests. The dominant tenure system is the unregistered trust land. Subsequent years saw the land that was held communally dissolved and the clamor for privatization ensued which led to a shift in land use.

The high population growth rate has led to severe encroachment of reservations. This has been enhanced by the practice of sub-division where an individual buys land meant for a single purpose, sub-divides it and sells it to other individuals who use it for differing purposes. Encroachment of land in fragile areas such as water catchments and wetlands has led to desertification in the county and breaking down of ecological cycles. This has also led to human-wildlife contact leading to cases of death and injury.

3.1.2 Future Development Patterns

There are a large number of residential and commercial construction activities currently ongoing in the project area with most of the land been converted from pastoral/livestock use to residential estates or educational institutions and industrial areas especially along the tarmacked roads.

These development patterns are envisaged to continue at least up to the future design horizon (as defined in POPULATION PROJECTIONS AND DISTRIBUTION) at the current pace. The rate of these developments is expected to increase rapidly with this Water Supply and Sewerage Project implementation.

The Kajiado County government has put in place a Spatial Plan for land use in the county. This will ensure that legal land owners use it according to the prescribed zoning laws. One aim of this plan is to counter the rampant practice of buying agricultural land and selling it for residential purposes.

3.2 POPULATION PROJECTIONS AND DISTRIBUTION

3.2.1 Population Estimates and Projections

3.2.1.1 Introduction and Planning Horizons

In order to produce more accurate estimates of the water demand, the Practice Manual for Water Supply Services in Kenya (2005) recommends making projections for the "initial", the "future" and the ultimate" year. The "initial" year is the year when the supply is expected to begin operating and may be assumed to be 0-5 years from the date of the commencement of the preliminary design. For the Olooloitikosh – Kitengela – Kajiado Water and Sewerage Project, the "initial" year will be taken to be 4 years from the beginning of the design stage, that is, 2022. The "future" year is 10 years and the "ultimate" year is 20 years from the "initial" year, that is, 2032 and 2042 respectively.

Estimates have been done by dividing the entire project area into 3 different areas: Townships, Urban Areas and Rural Areas; where Townships have populations greater than 10,000 people and Urban Areas have populations between 2,000 and 10,000 people. Rural areas are all other areas that are not Townships or Urban Areas.

The population estimates and forecasts were done for the entire project area as described in the Inception Report in addition to Bisil Urban Area and the Rural Areas between Kajiado Township and Bisil Urban Area. As such the population projections are done for the following areas:

1) Kitengela Township
2) Kajiado Township
3) Isinya Urban Area
4) Kisaju Urban Area
5) Olooloitikosh Urban Area
6) Bisil Urban Area
7) Peri-Urban and Rural Areas

3.2.1.2 Population Growth Rates

As all the urban areas are along the Namanga-Nairobi Road, the population growth rates are quite high, being 2 – 3 times the national growth rate and higher than the Kajiado County growth rate which is given as 5.5% by the Kajiado County Integrated Development Plan. The population growth rates for Kitengela and Kajiado Townships had been estimated to be 7.8% and 9.6% respectively between 2009 and 2030 by the former Ministry of Metropolitan

Development in 2008 (Feasibility Study and Master Plan for Developing New Water Sources for Nairobi and Satellite Towns, pp. 3-24). Hence, these population growth rates were adopted up to 2022 and thereafter the growth rate was reduced by 1% per decade for the subsequent design horizons. The growth rates of 7.8% and 9.6% for Kitengela and Kajiado Townships, respectively, were not used until 2032 as they are higher than the growth rates used in similar studies. Additionally, the growth rates for Kitengela were adopted for Isinya, Kisaju, Olooloitikosh and Bisil Urban Areas, less one percent due to their proximity to the Nairobi – Namanga Road which is a crucial reason why the Kitengela and Kajiado growth rates are high. The growth rates for these urban areas are less than those for Kitengela by one percent due to Kitengela attractive more settlers than the other areas.

The Kajiado Township growth rates were not adopted for the urban areas as the township is Kajiado County capital and as such is expected to have a higher growth rate than other urban areas within the county.

The population growth rate for the rural areas were pegged to the national average, as a national growth rate would be more reflective of the rural conditions than the urban or county-wide population growth rates.

Table 3-1:- Population growth rates for the various design horizons

Area	Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	7.8%	7.8%	6.8%	5.8%
Kajiado Township	9.6%	9.6%	8.6%	7.6%
Isinya Urban Area	6.8%	6.8%	5.8%	4.8%
Kisaju Urban Area	6.8%	6.8%	5.8%	4.8%
Olooloitikosh Semi-Urban Area	6.8%	6.8%	5.8%	4.8%
Bisil Semi-Urban Area	6.8%	6.8%	5.8%	4.8%
Rural Areas	2.6%	2.6%	2.5%	2.0%

Table 3-2:- Comparison of population growth rates from different studies

	Source	Period	Annual Population Growth Rate
1	Nairobi Metropolitan Spatial Planning Concept (2012)	Kitengela Township: 2009-2010 2010-2030 2030-2040	 19.6% 7.3% 4.2%
2	Ministry of Nairobi Metropolitan Development (2008)	Kitengela Township: 2009-2030 Kajiado Township: 2009-2030	 7.8% 9.6%
3	The Project on Integrated Urban Development Master Plan for the City of Nairobi in the Republic of Kenya - Nippon Koei Co. (2014)	Kitengela Township: 2009-2030 Kajiado Township: 2009-2015 2015-2020 2020-2025 2025-2030	 9.8% 5.7% 6.8% 6.8% 6.9%
4	Prefeasibility, Feasibility Studies and Conceptual Designs for Kajiado Water and Sanitation Project - Kise Consult (2016)	Kitengela Township: 2009-2040 Kajiado Township: 2009-2020 2020-2040 Kisaju Urban Area: 2009-2020 2020-2040	 6.6% 6.8% 6.6% 9.3% 6.6%

	Source	Period	Annual Population Growth Rate
5	Feasibility Study and Master Plan for Developing New Water Sources for Nairobi and Satellite Towns - Egis Bceom and Mangat, I.B Patel & Partners (2011)	Kajiado Township: 2009-2035	6.4%
6	Kajiado County Statistical Abstract - KNBS (2015)	Kajiado County: 2009-2010 2010-2014	 2.1% 3.7%
7	Kajiado County Integrated Development Plan - Kajiado County Government (2013)	Kajiado County: 2009-2012 2012-2015 2015-2017 Kitengela Township: 2009-2012 2012-2015 2015-2017 Kajiado Township: 2009-2012 2012-2015 2015-2017 Isinya Urban Area: 2009-2012 2012-2015 2015-2017	 5.5% 3.6% 5.5% 5.5% 3.6% 5.5% 5.5% 3.6% 5.5% 5.5% 3.6% 5.5%
8	2009 Kenya Population and Housing Census: Analytical Report on Population Projections - KNBS (2012)	Kajiado County: 2010-2015 2015-2020 2020-2025 2025-2030	 3.5% 3.3% 3.0% 2.7%

	Source	Period	Annual Population Growth Rate
9	National Statistical Abstract - KNBS (2017)	Kajiado County: 2009-2016	3.4%

3.2.2 Domestic Population

The base population used to calculate the domestic population forecasts are based on 2017 population projections computed by the Kenya National Bureau of Statistics (KNBS).

Table below shows the different populations for the Townships, Urban Areas and Rural Areas within the project area.

A 2017 base population was chosen over the 2009 census because:

- The projections had already been done by KNBS;
- The growth rate chosen to apply on the 2009 population would have to be very accurate due to the amount of time that has passed.

Table 3-3: 2017 population figures within the project area

Project Areas	Male	Female	Total	Households	Area (km^2)	Population Density ($persons/km^2$)
Kitengela Township	42,485	42,148	84,633	23,644	67.6	1,252
Kajiado Township	10,686	10,602	21,288	5,952	16.7	1,275
Isinya Urban Area	6,333	6,282	12,615	3,527	22.0	573
Kisaju Urban Area	4,534	4,499	9,033	2,526	2.0	4,516
Olooloitikosh Urban Area	1,684	1,670	3,354	938	4.6	729
Bisil Urban Area	3,927	3,896	7,822	2,187	4.3	1,823
Rural Areas	7,246	7,189	14,435	4,036	313.8	46
TOTAL	**76,894**	**76,286**	**153,180**	**42,830**	**431.0**	**355**

The base populations for Kitengela Township, Kajaido Township, Isinya Urban Area and Bisil Urban Area are from the Kenya National Bureau of Statistics, accessed through the Kajiado County Integrated Plan.

The base population for the rural areas was calculated by applying the 2017 Kajiado County population density of 46 people/km^2 to the rest of the project area other than the townships and urban areas, that is 313.8 km^2.

Table 3-4: Domestic population forecasts for the various design horizons

Specific Project Areas	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	84,633	91,210	123,045	255,271	447,328
Kajiado Township	21,288	23,336	33,700	82,914	172,805
Isinya Urban Area	12,615	13,469	17,504	33,076	52,708
Kisaju Urban Area	9,033	9,644	12,534	23,684	37,741
Olooloitikosh Urban Area	3,354	3,581	4,654	8,794	14,013
Bisil Urban Area	7,822	8,352	10,854	20,510	32,683
Rural Areas	14,435	14,811	16,412	21,173	25,810
TOTAL	**153,180**	**164,404**	**218,701**	**445,420**	**783,089**

3.2.3 Institutional Population

Institutional population forecasting has to do with projecting the growth of educational and healthcare establishments within the project area. The population projections can be found in Table 3-5 and Table 3-6 for student populations and number of beds in medical centres respectively.

Table 3-5: Population of students within the project area

Specific Project Areas	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	26,954	29,048	39,184	75,441	132,200

Specific Project Areas	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kajiado Township	9,808	10,751	15,525	35,491	73,969
Isinya Urban Area	3,879	4,142	5,383	9,432	15,031
Kisaju Urban Area	2,778	2,966	3,854	6,754	10,763
Olooloitikosh Urban Area	1,031	1,101	1,431	2,508	3,996
Bisil Urban Area	2,405	2,568	3,338	5,849	9,321
Rural Areas	5,451	5,593	6,197	7,933	9,670
TOTAL	**52,307**	**56,170**	**74,912**	**143,408**	**254,950**

Data on student populations was pulled from the Kajiado County Statistical Abstract (2014) and the growth rates applied to the base population were the same as the population growth rates as it is assumed the student population will grow similarly to the domestic population. The following types of educational establishments/centres were considered when it came to estimating and forecasting the student population in Kajiado County:

- ECDE Centres
- Primary Schools
- Secondary Schools
- Technical Institutions
- Youth Polytechnics
- Adult Education Centres
- Universities

Table 3-6: Population projections for number of beds in health facilities within the project area

Specific Project Areas	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	119	164	221	459	805

Specific Project Areas	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kajiado Township	30	42	61	149	311
Isinya Urban Area	18	24	32	60	95
Kisaju Urban Area	13	17	23	43	68
Olooloitikosh Urban Area	5	6	8	16	25
Bisil Urban Area	11	15	20	37	59
Rural Areas	20	27	30	38	46
TOTAL	**216**	**296**	**394**	**802**	**1,410**

The data and information used to project the number of beds in health facilities is from the Kenya Health Sector Strategic Plan (2016) and takes into account the following establishments:

- Dispensaries
- Health Centres
- Primary Hospitals
- Tertiary Hospitals

The KHSSP gives the bed density per 10,000 people as 14.1 beds and a target bed density per 10,000 people of 18 beds for Kajiado County. The figure of 14.1 beds per 10,000 people was adopted for the base year and 18 beds per 10,000 people was adopted for the rest of the planning horizons. These bed densities were applied to the domestic population projections and hence the number of beds increases as the domestic population grows.

3.2.4 Commercial Population

Commercial population projections took into consideration the following sectors:

- Hotels (one-star to five-star hotels and unclassified hotels)
- Bars and Restaurants
- Shops (Trading Centres, Retail Traders, Wholesale Traders and Megastores)

The population projections can be found in Table 3-7 below. Data and information on hotels was referenced from the Kajiado County Statistical Abstract (2014) and the Hotel Outlook Report by PWC (2017). Data on bars, restaurants and shops was acquired from the Kajiado County Statistical Abstract (2014).

Table 3-7: Commercial population projections for various sectors

Specific Project Areas	**Sector**	**Details**	**Unit**	**Base Year** (2017)	**Current Year** (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
Kitengela Township	Hotels	Five Star	Beds	26	28	37	77	135
		Four Star	Beds	34	36	49	102	179
		Three Star	Beds	34	36	49	102	179
		Two Star	Beds	26	28	37	77	135
		One Star	Beds	26	28	37	77	135
		Unclassified Hotels	Beds	1269	1368	1846	3829	6710
	Bars and Restaurants		Number	84	91	122	254	445
	Shops	Trading Centre	Number	5	5	7	14	25
		Retail Traders	Number	490	528	712	1478	2590
		Wholesale Traders	Number	26	28	37	78	136
		Megastore	Number	1	1	2	4	7
Kajiado Township	Hotels	Five Star	Beds	6	7	10	25	52
		Four Star	Beds	9	9	13	33	69
		Three Star	Beds	9	9	13	33	69
		Two Star	Beds	6	7	10	25	52
		One Star	Beds	6	7	10	25	52
		Unclassified Hotels	Beds	319	350	505	1244	2592
	Bars and Restaurants		Number	21	23	33	82	172
	Shops	Trading Centre	Number	1	1	2	5	10
		Retail Traders	Number	123	135	195	480	1001
		Wholesale Traders	Number	6	7	10	25	53
		Megastore	Number	0	0	0	1	3

Specific Project Areas	Sector	Details	Unit	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Isinya Urban Area	Hotels	Five Star	Beds	4	4	5	10	16
		Four Star	Beds	5	5	7	13	21
		Three Star	Beds	5	5	7	13	21
		Two Star	Beds	4	4	5	10	16
		One Star	Beds	4	4	5	10	16
		Unclassified Hotels	Beds	189	202	263	496	791
	Bars and Restaurants		Number	13	13	17	33	52
	Shops	Trading Centre	Number	1	1	1	2	3
		Retail Traders	Number	73	78	101	192	305
		Wholesale Traders	Number	4	4	5	10	16
		Megastore	Number	0	0	0	0	1
Kisaju Urban Area	Hotels	Five Star	Beds	3	3	4	7	11
		Four Star	Beds	4	4	5	9	15
		Three Star	Beds	4	4	5	9	15
		Two Star	Beds	3	3	4	7	11
		One Star	Beds	3	3	4	7	11
		Unclassified Hotels	Beds	135	145	188	355	566
	Bars and Restaurants		Number	9	10	12	24	38
	Shops	Trading Centre	Number	1	1	1	1	2
		Retail Traders	Number	52	56	73	137	219
		Wholesale Traders	Number	3	3	4	7	12
		Megastore	Number	0	0	0	0	1
Olooloitikosh Urban Area	Hotels	Five Star	Beds	1	1	1	3	4
		Four Star	Beds	1	1	2	4	6
		Three Star	Beds	1	1	2	4	6
		Two Star	Beds	1	1	1	3	4
		One Star	Beds	1	1	1	3	4
		Unclassified Hotels	Beds	50	54	70	132	210
	Bars and Restaurants		Number	3	4	5	9	14
	Shops	Trading Centre	Number	0	0	0	0	1

Specific Project Areas	Sector	Details	Unit	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
		Retail Traders	Number	19	21	27	51	81
		Wholesale Traders	Number	1	1	1	3	4
		Megastore	Number	0	0	0	0	0
Bisil Urban Area	Hotels	Five Star	Beds	2	3	3	6	10
		Four Star	Beds	3	3	4	8	13
		Three Star	Beds	3	3	4	8	13
		Two Star	Beds	2	3	3	6	10
		One Star	Beds	2	3	3	6	10
		Unclassified Hotels	Beds	117	125	163	308	490
	Bars and Restaurants		Number	8	8	11	20	32
	Shops	Trading Centre	Number	0	0	1	1	2
		Retail Traders	Number	45	48	63	119	189
		Wholesale Traders	Number	2	3	3	6	10
		Megastore	Number	0	0	0	0	0
Rural Areas	Hotels	One Star	Beds	4	4	5	6	8
		Unclassified Hotels	Beds	217	222	246	318	387
	Bars and Restaurants		Number	14	15	16	21	26
	Shops	Trading Centre	Number	1	1	1	1	1
		Retail Traders	Number	84	86	95	123	149
		Wholesale Traders	Number	4	5	5	6	8
TOTALS	**Hotels**	**Five Star**	**Beds**	**42**	**45**	**61**	**128**	**229**
		Four Star	**Beds**	**55**	**60**	**81**	**170**	**303**
		Three Star	**Beds**	**55**	**60**	**81**	**170**	**303**
		Two Star	**Beds**	**42**	**45**	**61**	**128**	**229**
		One Star	**Beds**	**46**	**51**	**67**	**136**	**238**
		Unclassified Hotels	**Beds**	**2298**	**2517**	**3337**	**6753**	**11834**
	Bars and Restaurants		**Number**	**152**	**167**	**221**	**447**	**784**
	Shops	**Trading Centre**	**Number**	**9**	**9**	**12**	**25**	**44**

Specific Project Areas	Sector	Details	Unit	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
		Retail Traders	**Number**	**887**	**972**	**1288**	**2607**	**4568**
		Wholesale Traders	**Number**	**47**	**51**	**68**	**137**	**240**
		Megastore	**Number**	**2**	**2**	**3**	**6**	**11**

3.2.5 Administrative Population

The administrative population forecasts can be found in Table 3-8. The following sectors were considered in this section:

- Financial Services
- Professional and technical services
- The public sector

The data required to establish the population projections for the different design horizons was obtained from the Kajiado County Integrated Plan (2013) and the Kajiado County Statistical Abstract (2014).

Table 3-8: Population projections for administrative offices

Specific Project Areas	Sector	Unit	Base Year (2017)	Current Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Kitengela Township	Financial Services	Head	204	320	432	896	1571
	Profession and Technical Services	Head	215	337	455	944	1654
	Public Sector	Head	508	796	1074	2229	3906
Kajiado Township	Financial Services	Head	53	82	118	291	607
	Profession and Technical Services	Head	55	86	125	307	639
	Public Sector	Head	131	204	294	724	1509
Isinya Urban Area	Financial Services	Head	30	47	61	116	185
	Profession and Technical Services	Head	32	50	65	122	195
	Public Sector	Head	76	118	153	289	460

Specific Project Areas	**Sector**	**Unit**	**Base Year** (2017)	**Current Year** (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
Kisaju Urban Area	Financial Services	Head	22	34	44	83	133
	Profession and Technical Services	Head	23	36	46	88	140
	Public Sector	Head	54	84	109	207	330
Olooloitikosh Urban Area	Financial Services	Head	8	13	16	31	49
	Profession and Technical Services	Head	9	13	17	33	52
	Public Sector	Head	20	31	41	77	122
Bisil Urban Area	Financial Services	Head	19	29	38	72	115
	Profession and Technical Services	Head	20	31	40	76	121
	Public Sector	Head	47	73	95	179	285
Rural Areas	Financial Services	Head	51	64	71	91	111
	Profession and Technical Services	Head	53	67	75	96	117
	Public Sector	Head	126	159	176	227	277
SUB TOTALS	**Financial Services**	**Head**	**387**	**589**	**781**	**1,581**	**2,770**
	Profession and Technical Services	**Head**	**407**	**620**	**822**	**1,665**	**2,917**
	Public Sector	**Head**	**962**	**1,465**	**1,942**	**3,931**	**6,888**
TOTAL		**Head**	**1,756**	**2,674**	**3,546**	**7,177**	**12,575**

3.2.6 Industrial Population

The make-up of the industrial population is as follows:

- Manufacturing Industries,
- Bakeries,
- Jua Kali Artisans.

Projections for the industrial population can be found in Table 3-9 below. The information used to establish the forecasts as in Table 3-9 is from the Kajiado County Statistical Abstract (2014) and the Kajiado County Integrated Development Plan (2013).

Table 3-9: Industrial population projections for various sectors

Specific Project Areas	**Sector**	**Unit**	**Base Year** (2017)	**Current Year** (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
Kitengela Townships	Manufacturing	Number	3	3	4	9	16
	Bakeries	Number	0	0	0	0	1
	Jua Kali Artisans	Number	20	22	29	60	106
Kajiado Townships	Manufacturing	Number	1	1	1	3	6
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	5	6	8	20	41
Isinya Urban Area	Manufacturing	Number	0	0	1	1	2
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	3	3	4	8	12
Kisaju Urban Area	Manufacturing	Number	0	0	0	1	1
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	2	2	3	6	9
Olooloitikosh Urban Area	Manufacturing	Number	0	0	0	0	1
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	1	1	1	2	3
Bisil Urban Area	Manufacturing	Number	0	0	0	1	1
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	2	2	3	5	8
Rural Areas	Manufacturing	Number	1	1	1	1	1
	Bakeries	Number	0	0	0	0	0
	No. of Jua Kali Artisans	Number	3	3	4	5	6

Specific Project Areas	**Sector**	**Unit**	**Base Year** (2017)	**Current Year** (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
TOTALS	**Manufacturing**	**Number**	**5**	**6**	**8**	**16**	**28**
	Bakeries	**Number**	**0**	**0**	**0**	**0**	**1**
	No. of Jua Kali Artisans	**Number**	**36**	**39**	**52**	**105**	**185**

3.2.7 Livestock Population

The main livestock breeds considered in this section are:

- Dairy and beef cattle,
- Sheep,
- Dairy and meat goats,
- Pigs,
- Poultry (broilers, layers, indigenous, quails, ducks and others);
- Donkeys,
- Camels,
- Beehives,
- Ostriches.

The livestock population projections were estimated from data taken from the Kajiado County Statistical Abstract (2014), which gave the total population figures for the different livestock breeds over the whole county. As such, it was assumed that livestock were equally spread over the entire region and therefore apportioned over the specific project area.

Table 3-10: Livestock Population Projections

Category	**Sub Category**	**Units**	**Base Year (2014)**	**Current Year (2018)**	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
Cattle	Dairy	Livestock Units	1,548	2,185	3,084	7,301	11,893
	Beef	Livestock Units	3,446	4,864	6,866	16,254	26,477
Sheep		Livestock Units	1,287	1,817	2,565	6,073	9,892
Goats	Dairy	Livestock Units	2	3	4	10	16
	Meat	Livestock Units	1,417	2,001	2,824	6,686	10,890
Pigs		Livestock Units	87	122	173	408	665

Rabbit		Livestock Units	4	6	9	20	33
Poultry	Broilers	Livestock Units	67	95	133	316	515
	Layers	Livestock Units	10	15	21	49	80
	Indigenous	Livestock Units	42	59	83	196	320
	Others	Livestock Units	0	0	1	2	3
	Quails	Livestock Units	0	0	1	2	2
	Ducks	Livestock Units	0	0	0	0	0
Donkeys		Livestock Units	405	572	808	1,913	3,116
Camels		Livestock Units	18	26	36	86	140
Hives	Log	Livestock Units	3	5	7	16	26
	KTBH	Livestock Units	5	7	10	23	38
	Lang	Livestock Units	3	4	5	13	21
Ostrich		Livestock Units	12	17	24	56	91
TOTAL			**10,372**	**13,816**	**18,675**	**41,456**	**66,260**

3.2.8 Area under irrigation

The area under irrigation and the projections for the increase of the size of this area within the project area are as shown in Table 3-11. Information was obtained from the Kajiado County Statistical Abstract (2014).

Table 3-11:- Acreage under irrigation within the project area

	Units	**Current Year (2018)**	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
Crop Irrigation	Ha	55	67	109	133

The area under irrigation is considered to be in close proximity to the chosen dam location and wastewater treatment works for both Kitengela and Kajiado Townships. Additionally, the area under irrigation would also depend on the types of crops to be planted, soil type,

evaporation rates, amongst other factors. However, the following crops are currently grown within the project area:

- Tomatoes,
- Onions,
- Cabbages,
- Kales,
- Spinach,
- Carrots,
- Bananas,
- Paw Paws,
- Melons.

These same crops are what are envisioned to be grown around the proposed dam and irrigated with water from it.

4 WATER DEMAND ESTIMATES

4.1 GENERAL

Per capita consumption rates are given in the Practice Manual for Water Supply Services in Kenya (2005), which were applied to the population projections to compute the water demand figures presented in the sections below.

The unit consumption rates given by the Practice Manual for Water Supply Services in Kenya includes 20% allowance for unaccounted for water. The unit consumption rates from the practice manual are shown in Figure 4.1. The outpatient consumption rate has been taken as 5,000l per day, the described minimum as in Figure 4.1.

CONSUMER	UNIT	RURAL AREAS			URBAN AREAS		
		High potential	Medium potential	Low potential	High Class Housing	Medium Class Housing	Low Class Housing
People with individual connections	1/head/ day	60	50	40	250	150	75
People without connections	1/head/ day	20	15	10	-	-	20
Livestock unit	1/head/ day	50			-		
Boarding schools	1/head/ day	50					
Day schools with WC without WC	1/head/ day	25 5					
Hospitals Regional District other	1/bed/ day	400 200 100 } + 20 1 per outpatient and day (minimum 5000 1/day)					
Dispensary and Health Centre	1/day	5000					
Hotels High Class Medium Class Low Class	1/bed/ day	600 300 50					
Administrative offices	1/head/ day	25					
Bars	1/day	500					
Shops	1/day	100					
Unspecified industry	1/ha/day				20,000		
Coffee pulping factories	1/kg coffee	25 (when re-circulation of water is used).					

Figure 4.1: Consumption Rates

4.2 KITENGELA TOWNSHIP

The estimated water demand for the Kitengela Township area is estimated to be 9,364m^3/d, 12,630m^3/d, 25,984m^3/d, 45,529m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-1: Water demand for Kitengela Township

Consumer	Component/Details	Consumer Unit	Rate	Water Demand (m^3/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	High Class Housing	l/h/d	250	969	1307	2712	4753
	Medium Class Housing	l/h/d	150	5815	7844	16274	28517
	Low Class Housing (IC)	l/h/d	75	262	353	732	1283
	Low Class Housing (NC)	l/h/d	20	628	847	1758	3080
Schools	Boarding Schools	l/h/d	50	726	980	1886	3305
	Day Schools with WC	l/h/d	25	327	441	849	1487
	Day Schools without WC	l/h/d	5	7	10	19	33
Health Facility		l/b/d	200	38	49	97	166
Hotels	High Class	l/b/d	600	38	52	107	188
	Medium Class	l/b/d	300	19	26	54	94
	Low Class	l/b/d	50	1	2	4	7
	Unclassed	l/b/d	50	68	92	191	335
Bars and		l/d	500	45	61	127	222
Restaurants							
Shops	Trading Centres	l/d	500	3	3	7	12
	Retail Traders	l/d	100	53	71	148	259
	Wholesale Traders	l/d	500	14	19	39	68
	Megastores	l/d	10000	13	18	37	65
Administrative		l/h/d	25	36	49	102	178
Offices							
Industries	Manufacturing	l/d	90,000	294	396	822	1,440
	Bakeries	l/d	40,000	5	7	14	24
	Jua Kali Artisans	l/d	100	2	3	6	11
TOTAL				**9,364**	**12,630**	**25,984**	**45,529**

4.3 KAJIADO TOWNSHIP

The estimated water demand for the Kajiado Township area is estimated to be 2,21m^3/d, 3,638m^3/d, 8,844m^3/d, 18,427m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-2: Water demand for Kajiado Township

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year	Initial Year	Future Year	Ultimate Year
				(2018)	(2022)	(2032)	(2042)
Domestic	High Class Housing	l/h/d	250	248	358	881	1836
	Medium Class Housing	l/h/d	150	1488	2148	5286	11016
	Low Class Housing (IC)	l/h/d	75	67	97	238	496
	Low Class Housing (NC)	l/h/d	20	161	232	571	1190
Schools	Boarding Schools	l/h/d	50	269	388	887	1849
	Day Schools with WC	l/h/d	25	121	175	399	832
	Day Schools without WC	l/h/d	5	3	4	9	18
Health Facility		l/bed/d	200	13	17	35	67
Hotels	High Class	l/bed/d	600	10	14	35	73
	Medium Class	l/bed/d	300	5	7	17	36
	Low Class	l/bed/d	50	0	1	1	3
	Unclassed	l/bed/d	50	18	25	62	130
Bars and		l/d	500	12	17	41	86
Restaurants							
Shops	Trading Centres	l/d	500	1	1	2	5
	Retail Traders	l/d	100	14	20	48	100
	Wholesale Traders	l/d	500	4	5	13	26
	Megastores	l/d	10000	3	5	12	25
Administrative		l/h/d	25	9	13	33	69
Offices							
Industries	Manufacturing	l/d	90,000	75	109	267	556
	Bakeries	l/d	40,000	1	2	4	9
	Jua Kali Artisans	l/d	100	1	1	2	4
TOTAL				**2,521**	**3,638**	**8,844**	**18,427**

4.4 ISINYA URBAN AREA

The estimated water demand for the Isinya Urban area is estimated to be 1,359m^3/d, 1,765m^3/d, 3,308m^3/d, 5,268m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-3: Water demand for Isinya Urban Area

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	High Class Housing	l/head/day	250	143	186	351	560
	Medium Class Housing	l/head/day	150	859	1116	2109	3360
	Low Class Housing (IC)	l/head/day	75	39	50	95	151
	Low Class Housing (NC)	l/head/day	20	93	121	228	363
Schools	Boarding Schools	l/head/day	50	62	81	141	225
	Day Schools with WC	l/head/day	25	65	85	149	237
	Day Schools without WC	l/head/day	5	1	2	3	5
Health Facility		l/bed/day	200	10	11	17	24
Hotels	High Class	l/bed/day	600	6	7	14	22
	Medium Class	l/bed/day	300	3	4	7	11
	Low Class	l/bed/day	50	0	0	0	1
	Unclassed	l/bed/day	50	10	13	25	40
Bars and		l/day	500	7	9	16	26
Restaurants							
Shops	Trading Centres	l/day	500	0	0	1	1
	Retail Traders	l/day	100	8	10	19	31
	Wholesale Traders	l/day	500	2	3	5	8
	Megastores	l/day	10000	2	3	5	8
Administrative		l/head/day	25	5	7	13	21
Offices							
Industries	Manufacturing	l/day	90,000	43	56	106	170
	Bakeries	l/day	40,000	1	1	2	3
	Jua Kali Artisans	l/day	100	0	0	1	1
Total				**1,359**	**1,765**	**3,308**	**5,268**

4.5 KISAJU URBAN AREA

The estimated water demand for the Kisaju Urban area is estimated to be 975m^3/d, 1,265m^3/d, 2,370m^3/d and 3,774m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-4: Water demand for Kisaju Urban Area

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	High Class Housing	l/head/day	250	102	133	252	401
	Medium Class Housing	l/head/day	150	615	799	1510	2406
	Low Class Housing (IC)	l/head/day	75	28	36	68	108
	Low Class Housing (NC)	l/head/day	20	66	86	163	260
Schools	Boarding Schools	l/head/day	50	44	58	101	161
	Day Schools with WC	l/head/day	25	47	61	106	170
	Day Schools without WC	l/head/day	5	1	1	2	4
Health Facility		l/bed/day	200	8	10	14	19
Hotels	High Class	l/bed/day	600	4	5	10	16
	Medium Class	l/bed/day	300	2	3	5	8
	Low Class	l/bed/day	50	0	0	0	1
	Unclassed	l/bed/day	50	7	9	18	28
Bars and		l/day	500	5	6	12	19
Restaurants							
Shops	Trading Centres	l/day	500	0	0	1	1
	Retail Traders	l/day	100	6	7	14	22
	Wholesale Traders	l/day	500	1	2	4	6
	Megastores	l/day	10000	1	2	3	6
Administrative		l/head/day	25	4	5	9	15
Offices							
Industries	Manufacturing	l/day	90,000	31	40	76	122
	Bakeries	l/day	40,000	1	1	1	2
	Jua Kali Artisans	l/day	100	0	0	1	1
Total				**975**	**1,265**	**2,370**	**3,774**

4.6 OLOOLOITIKOSH URBAN AREA

The estimated water demand for the Olooloitikosh urban area is estimated to be 365m^3/d, 473m^3/d, 883m^3/d, 1,404m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-5: Water demand for Olooloitikosh Urban Area

				Water Demand (m³/day)			
Consumer	**Component**	**Consumer Unit**	**Rate**	**Present Year**	**Initial Year**	**Future Year**	**Ultimate Year**
				(2018)	(2022)	(2032)	(2042)
Domestic	High Class Housing	l/head/day	250	38	49	93	149
	Medium Class Housing	l/head/day	150	228	297	561	893
	Low Class Housing (IC)	l/head/day	75	10	13	25	40
	Low Class Housing (NC)	l/head/day	20	25	32	61	96
Schools	Boarding Schools	l/head/day	50	17	21	38	60
	Day Schools with WC	l/head/day	25	17	23	39	63
	Day Schools without WC	l/head/day	5	0	1	1	1
Health Facility		l/bed/day	200	6	7	8	10
Hotels	High Class	l/bed/day	600	2	2	4	6
	Medium Class	l/bed/day	300	1	1	2	3
	Low Class	l/bed/day	50	0	0	0	0
	Unclassed	l/bed/day	50	3	3	7	11
Bars and		l/day	500	2	2	4	7
Restaurants							
Shops	Trading Centres	l/day	500	0	0	0	0
	Retail Traders	l/day	100	2	3	5	8
	Wholesale Traders	l/day	500	1	1	1	2
	Megastores	l/day	10000	1	1	1	2
Administrative		l/head/day	25	1	2	4	6
Offices							
Industries	Manufacturing	l/day	90,000	12	15	28	45
	Bakeries	l/day	40,000	0	0	0	1
	Jua Kali Artisans	l/day	100	0	0	0	0
TOTAL				**365**	**473**	**883**	**1,404**

4.7 BISIL URBAN AREA

The estimated water demand for the Bisil Urban area is estimated to be 845m^3/d, 1,096m^3/d, 2,053m^3/d, 3,268m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-6: Water demand for Bisil Urban Area

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	High Class Housing	l/head/day	250	89	115	218	347
	Medium Class Housing	l/head/day	150	532	692	1307	2084
	Low Class Housing (IC)	l/head/day	75	24	31	59	94
	Low Class Housing (NC)	l/head/day	20	58	75	141	225
Schools	Boarding Schools	l/head/day	50	39	50	88	140
	Day Schools with WC	l/head/day	25	40	53	92	147
	Day Schools without WC	l/head/day	5	1	1	2	3
Health Facility		l/bed/day	200	8	9	12	17
Hotels	High Class	l/bed/day	600	4	5	9	14
	Medium Class	l/bed/day	300	2	2	4	7
	Low Class	l/bed/day	50	0	0	0	0
	Unclassed	l/bed/day	50	6	8	15	25
Bars and		l/day	500	4	5	10	16
Restaurants							
Shops	Trading Centres	l/day	500	0	0	1	1
	Retail Traders	l/day	100	5	6	12	19
	Wholesale Traders	l/day	500	1	2	3	5
	Megastores	l/day	10000	1	2	3	5
Administrative		l/head/day	25	3	4	8	13
Offices							
Industries	Manufacturing	l/day	90,000	27	35	66	105
	Bakeries	l/day	40,000	0	1	1	2
	Jua Kali Artisans	l/day	100	0	0	0	1
TOTAL				**845**	**1,096**	**2,053**	**3,268**

4.8 RURAL AREAS

The estimated water demand for the rural areas, defined as all those areas within the project area that are not Townships or Urban Areas, is estimated to be 339m^3/d, 440m^3/d, 884m^3/d, 1,634m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-7: Water demand for Rural Areas

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	Medium Potential (IC)	l/head/day	50	42	54	205	654
	Medium Potential (NC)	l/head/day	15	113	147	246	294
Schools	Boarding Schools	l/head/day	50	39	50	88	140
	Day Schools with WC	l/head/day	25	40	53	92	147
	Day Schools without WC	l/head/day	5	1	1	2	3
Health Facility		l/bed/day	200	8	9	12	17
Hotels	Low Class	l/bed/day	50	0	0	0	0
	Unclassed	l/bed/day	50	0	0	0	1
Bars and		l/day	500	2	2	4	7
Restaurants							
Shops	Trading Centres	l/day	500	1	2	3	5
	Retail Traders	l/day	100	0	0	1	1
	Wholesale Traders	l/day	500	63	81	154	245
Administrative		l/head/day	25	3	4	8	13
Offices							
Industries	Manufacturing	l/day	90,000	27	35	66	105
	Bakeries	l/day	40,000	0	1	1	2
	Jua Kali Artisans	l/day	100	0	0	0	1
TOTAL				**339**	**440**	**884**	**1,634**

4.9 TOTAL WATER DEMAND

The estimated Total Water Demand for the project is estimated to be 23,234m^3/d, 30,588m^3/d, 60,504m^3/d, 101,117m^3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 4-8: Total Water Demand for the Project Area

Consumer	Component	Consumer Unit	Rate	Water Demand (m³/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic (Urban)	High Class Housing	l/h/d	250	1,589	2,149	4,508	8,046
	Medium Class Housing	l/h/d	150	9,537	12,896	27,046	48,277
	Low Class Housing (IC)	l/h/d	75	429	580	1,217	2,172
	Low Class Housing (NC)	l/h/d	20	1,030	1,393	2,921	5,214
Domestic (Rural)	Medium Potential (IC)	l/h/d	50	42	54	205	654
	Medium Potential (NC)	l/h/d	15	113	147	246	294
Schools	Boarding Schools	l/h/d	50	1,195	1,628	3,229	5,881
	Day Schools with WC	l/h/d	25	658	889	1,727	3,082
	Day Schools without WC	l/h/d	5	15	20	38	68
Health Facility		l/bed/d	200	92	112	195	319
Hotels	High Class	l/bed/d	600	63	85	179	319
	Medium Class	l/bed/d	300	32	43	90	160
	Low Class	l/bed/d	50	4	5	11	18
	Unclassed	l/bed/d	50	112	152	318	568
Bars and Restaurants		l/d	500	76	102	214	381
Shops	Trading Centres	l/d	500	5	7	15	26
	Retail Traders	l/d	100	87	117	246	439
	Wholesale Traders	l/d	500	85	112	218	361
	Megastores	l/day	10000	25	34	70	123
Administrative Offices		l/h/d	25	60	81	169	302
Industries	Manufacturing	l/d	90,000	509	686	1,432	2,544
	Bakeries	l/d	40,000	8	11	24	42
	Jua Kali Artisans	l/d	100	4	5	10	19
Livestock Unit		l/h/d	50	691	934	2,073	3,313
Crop Irrigation				5,951	7,234	11,783	14,364
Internal Demand in the Water Works (Backwashing of rapid sand filters)				823	1,112	2,320	4,131
TOTAL				**23,234**	**30,588**	**60,504**	**101,117**

5 WASTEWATER GENERATION RATES ESTIMATES

5.1 GENERAL

Waste water estimates has been limited to the townships within the project area. The water demand estimates have been used to estimate the waste water generated this areas.

5.2 KITENGELA TOWNSHIP

The volume of waste water generated from Kitengela was initially estimated as a percentage of the water demand for the various components as 85% less 10% to account for minor leaks as per the guidelines given by the Practice Manual for Sewerage and Sanitation Services in Kenya (2008). A conversion factor of 80% was eventually adopted however as a conservative estimate. The estimated wastewater flow rates for the Kitengela Township area is estimated to be 7,491m3/d, 10,104m3/d, 20,787m3/d, 36,423m3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 5-1: Wastewater Flows for Kitengela Township

Consumer	Component	Wastewater Flows (m^3/day)			
		Present Year (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
Domestic	High Class Housing	775	1,046	2,170	3,802
	Medium Class Housing	4,652	6,275	13,019	22,814
	Low Class Housing (IC)	209	282	586	1,027
	Low Class Housing (NC)	502	678	1,406	2,464
Schools	Boarding Schools	581	784	1,509	2,644
	Day Schools with WC	261	353	679	1,190
	Day Schools without WC	6	8	15	26
Health Facility		30	39	78	133
Hotels	High Class	31	41	86	151
	Medium Class	15	21	43	75
	Low Class	1	1	3	5
	Unclassed	55	74	153	268
Bars and Restaurants		36	49	101	178
Shops	Trading Centres	2	3	6	10
	Retail Traders	42	57	118	207
	Wholesale Traders	11	15	31	55
	Megastores	11	14	30	52
Administrative Offices		29	39	81	143
Industries	Manufacturing	235	317	658	1,152
	Bakeries	4	5	11	19

Consumer	Component	Wastewater Flows (m³/day)			
		Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
	Jua Kali Artisans	2	2	5	8
TOTAL		**7,491**	**10,104**	**20,787**	**36,423**

5.3 KAJIADO TOWNSHIP

The volume of waste water generated from Kajiado was initially estimated as a percentage of the water demand for the various components as 85% less 10% to account for minor leaks as per the guidelines given by the Practice Manual for Sewerage and Sanitation Services in Kenya (2008). A conversion factor of 80% was eventually adopted however as a conservative estimate. The estimated Wastewater flow rates for the Kajiado Township area is estimated to be 2,016m3/d, 2,910m3/d, 7,075m3/d, 14,742m3/d in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Table 5-2: Wastewater flows for Kajiado Township

Consumer	Component	Wastewater Flows (m³/day)			
		Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic	High Class Housing	198	286	705	1469
	Medium Class Housing	1190	1719	4229	8813
	Low Class Housing (IC)	54	77	190	397
	Low Class Housing (NC)	129	186	457	952
Schools	Boarding Schools	215	310	710	1479
	Day Schools with WC	97	140	319	666
	Day Schools without WC	2	3	7	15
Health Facility		11	14	28	54
Hotels	High Class	8	11	28	58
	Medium Class	4	6	14	29
	Low Class	0	0	1	2
	Unclassed	14	20	50	104
Bars and Restaurants		9	13	33	69
Shops	Trading Centres	1	1	2	4
	Retail Traders	11	16	38	80
	Wholesale Traders	3	4	10	21

Consumer	Component	Wastewater Flows (m^3/day)			
		Present Year (2018)	**Initial Year** (2022)	**Future Year** (2032)	**Ultimate Year** (2042)
	Megastores	3	4	10	20
Administrative Offices		7	11	26	55
Industries	Manufacturing	60	87	214	445
	Bakeries	1	1	4	7
	Jua Kali Artisans	0	1	2	3
TOTAL		**2,016**	**2,910**	**7,075**	**14,742**

6 HYDROLOGIC ASSESSMENTS

6.1 PREMISES

In a feasibility study for the development of new supply sources through a dam system, hydrological assessments system is mainly aimed at evaluating the quantity of water that can be stored in the system and that can be hence allocated to the different users. These assessments are crucial to both:

1) understanding whether the projected new supply source can, and to what extent, meet the expected water demand and;
2) to supporting optimal system design in terms of reservoir's storage capacity.

While for objective:

1) an estimation of average water availability may suffice, objective;
2) requires a full characterization of runoff variability, both at a seasonal and at an annual scale.

Hydrological assessments in this phase of the study are hence expected to provide a first indication on average water availability to be compared with the projected water demand. The outcomes of this comparison may be helpful to understand which catchments need to be developed to meet present and future water demand, and the priority of intervention.

As reservoirs are meant to regulate runoff, this section will focus on how much runoff is likely to take place in the catchments considered in this study. The section will be articulated as follows:

- Review and analysis of the available data
- Drought analysis
- Methodology for rainfall – runoff modeling
- A first average assessment of yearly runoff

6.2 REVIEW AND ANALYSIS OF THE AVAILABLE DATA

Available data consist of rainfall and temperature data recorded from January 1985 to December 2016 at three meteorological stations (Figure 6.1):

- 9136130 Wilson Airport
- 9136168 J.K.I.A.
- Machakos

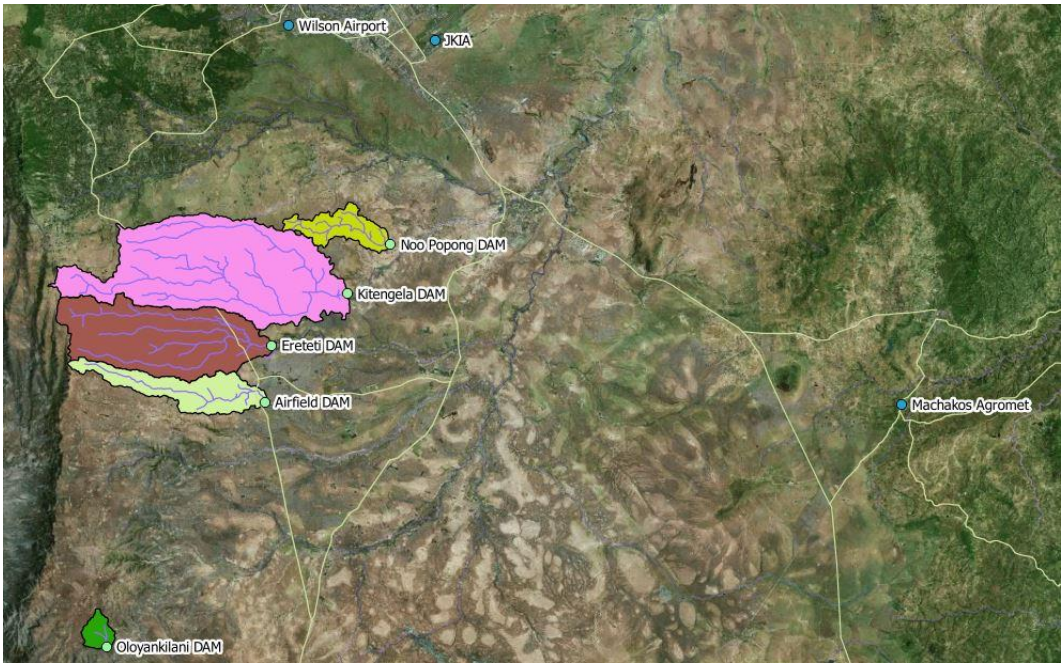


Figure 6.1: Location of meteorological stations (blue dots) in the studied area

The available data-set consists of these types of data:

1) Daily rainfall totals
2) Daily maximum temperature at monthly scale (one value per month)
3) Daily minimum temperature at monthly scale (one value per month)
4) Pan evaporation data (average daily data per month)

Table 6-1: Summary of the available rainfall and temperature data used for the hydrologic assessments.

Meteorological Station	RAINFALL		
	Mean annual rainfall 1985-2016 [mm]	Missing data	% of missing data (on monthly totals)
Wilson Airport	977.7	1994 Feb; 2014 Jan, May, Oct to Dec; 2016 Jan to Dec	4.8%
J.K.I.A.	786.3	1988 Aug; 1994 Feb; 1997 Aug; 2004 May, Dec; 2005 Oct to Dec; 2006 Jan to Dec; 2011 Aug; 2012 Apr, Dec; 2013 Nov, Dec; 2014 Jan to May, Oct to Dec; 2016 Apr, June to Dec	12.1%
Machakos	769.6	1989 Aug; 1994 Feb; 1997 Aug; 2004 May, Dec; 2005 June to Dec; 2011 Aug, Oct, Dec; 2012 Apr, Dec; 2013 Jan; 2014 Jan, Oct, Dec; 2016 Apr, June to Oct	7.8%

Meteorological Station	TEMPERATURE MAX		
	Mean annual rainfall 1985-2015 [°C]	Missing data	Mean annual rainfall 1985-2015 [°C]
Wilson Airport	28.1	2001 Aug; 2006 Mar to Jul; 2007 Aug; 2011 Mar, Jul; 2012 Jul, Oct, Dec; 2013 May, Jul to Nov; 2014 Jan to May, Oct to Dec; 2015 Sept to Dec	7.8%
J.K.I.A.	28.7	1993 Dec; 1994 Whole year; 1995 May; 1997 Aug, Sept; 1998 Nov, Dec; 1999 Whole year; 2000 Mar-Apr, Oct; 2001 June to Dec; 2002 Whole, year; 2003 Jan, Feb; 2005 Sept to Dec; 2006 whole year; 2007 May to Aug; 2012 Apr, Dec; 2013 Jan to Apr, Nov, Dec; 2014, 2015 whole year	29.7%
Machakos	28.1	1986 Jan to May, Aug; 2005 June to Dec; 2013 Jan; 2014 Dec; 2015 Feb-Dec	3.0%

Meteorological Station	TEMPERATURE MIN		
	Mean annual 1985-2016 [mm]	Missing data	% of missing data (on monthly totals)
Wilson Airport	11.1	1994 Nov-Dec; 1995 Jan-Feb, Apr to July; 1996 Feb to June, Sept to Dec; 1997 Jan, Jul; 2012 Jul, Oct, Dec; 2013 May, May to July; 2001 Aug; 2006 March to Jul; 2011 Feb, July; 2012 Sept to Nov; 2013 Apr, Jun to Oct; 2014 Jan to Apr, Sept to Nov; 2015 Aug, Nov to Dec	13.6%
J.K.I.A.	9.6	1993 Dec; 1994 Whole year; 1995 May; 1997 Aug, Sept; 1998 Nov, Dec; 1999 Whole year; 2000 Mar-Apr, Oct; 2001 June to Dec; 2002 Whole, year; 2003 Jan, Feb; 2005 Sept to Dec; 2006 whole year; 2007 May to Aug; 2012 Apr, Dec; 2013 Jan to Apr, Nov, Dec; 2014, 2015 whole year	29.7%
Machakos	28.1	1986 Jan to May, Aug; 2005 June to Dec; 2012 Oct; 2014 Dec; 2015 Feb-Dec	3.0%

6.2.1 Precipitation

Figures below show box plots of monthly precipitation totals for the three stations. The figures show in the first place the well-known concentration of rains in two periods: the so called “short rain” period between November and December and the “long rain” period (March to May). This is clearly distinguishable in each of the three stations. However, the figures also show the very large spread between maximum and minimum precipitation in the rainy months.

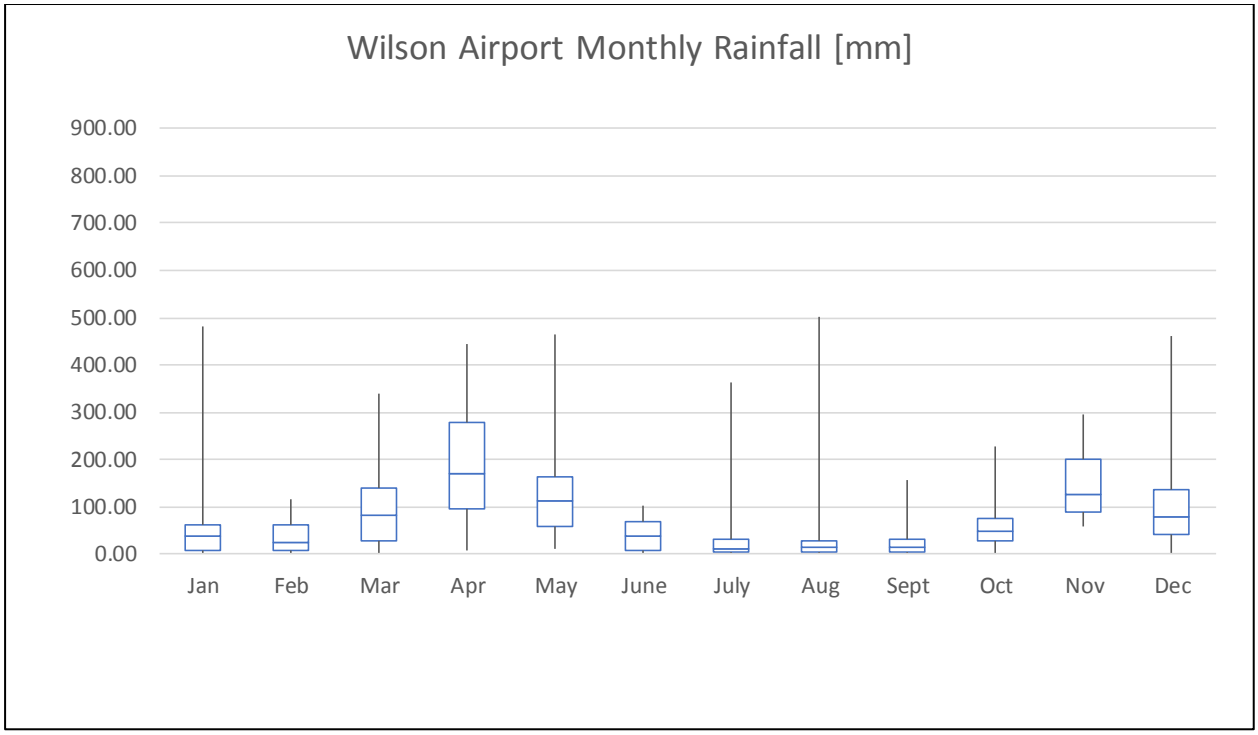


Figure 6.2: Box plots of monthly precipitation totals recorded at Wilson Airport meteorological station

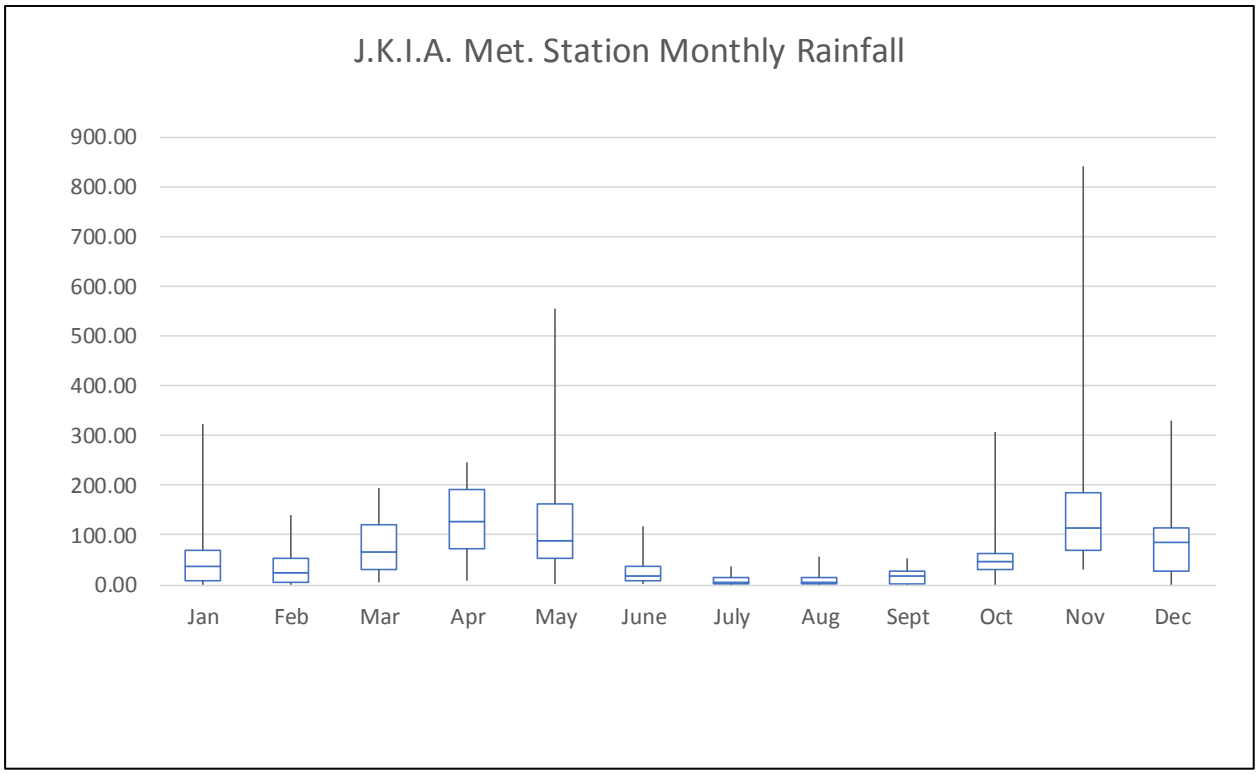


Figure 6.3: Box plots of monthly precipitation totals recorded at J.K.I.A. meteorological station

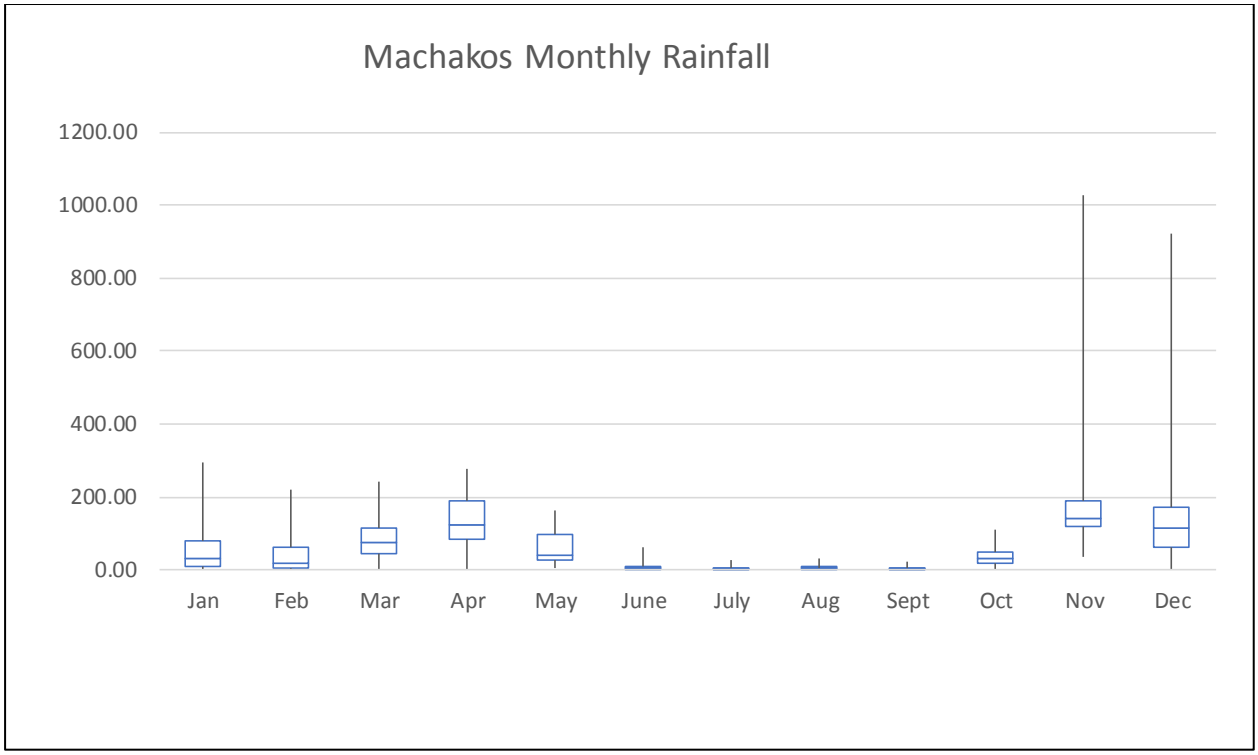


Figure 6.4: Box plots of monthly precipitation totals recorded at Machakos meteorological station

Data from the three stations were also inspected with regard to the correlation between aggregated values such as yearly rainfall totals or seasonal totals. This analysis is useful to gain some understanding on rainfall variability at spatial scale. Overall, poor correlation was observed between observed yearly totals recorded at the different stations (correlation coefficients are lower than 0.3 for all combinations of pairs of stations) even if stations are quite close to each other (distance between Wilson Airport and JKIA is around 18 km). Higher correlations are observed for aggregated rainfall totals in the long rain seasons and in the short rain season, as shown in figures below.

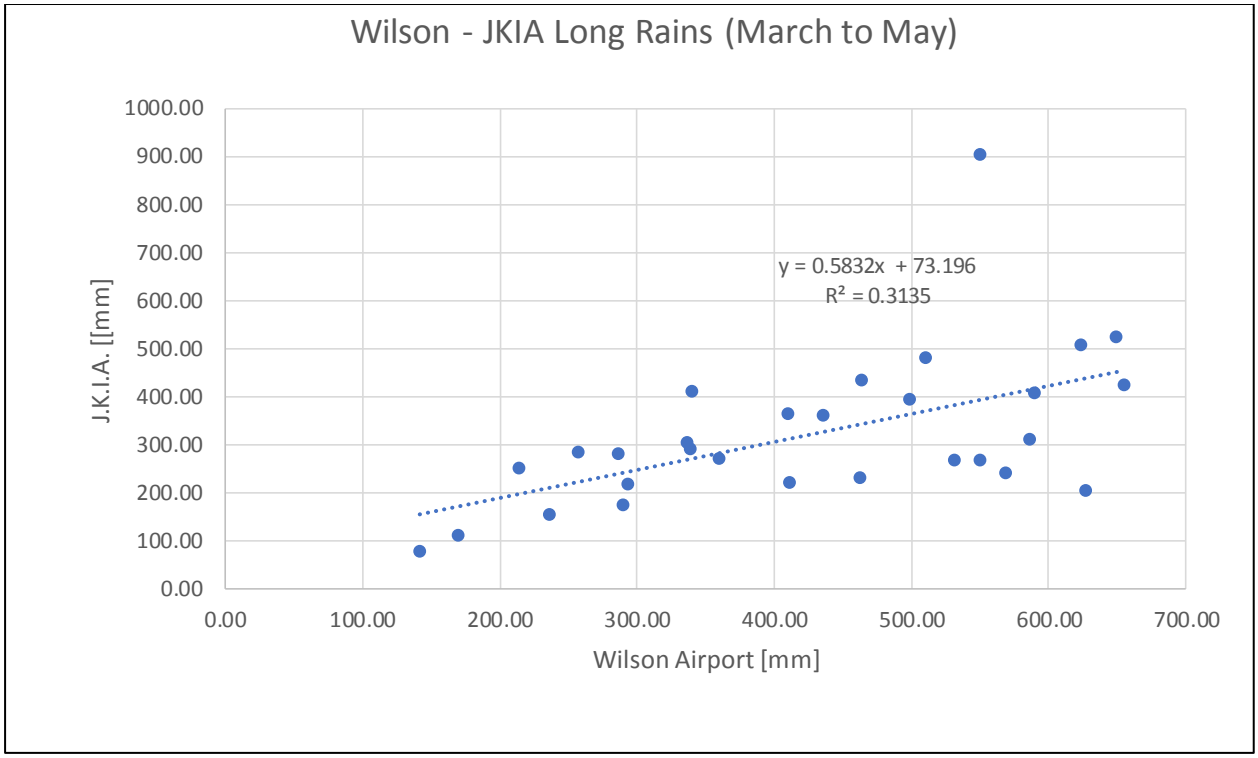


Figure 6.5: Scatterplot of rainfall totals in months March-May 1985 – 2016 observed at Wilson Airport and those observed at JKIA meteorological station

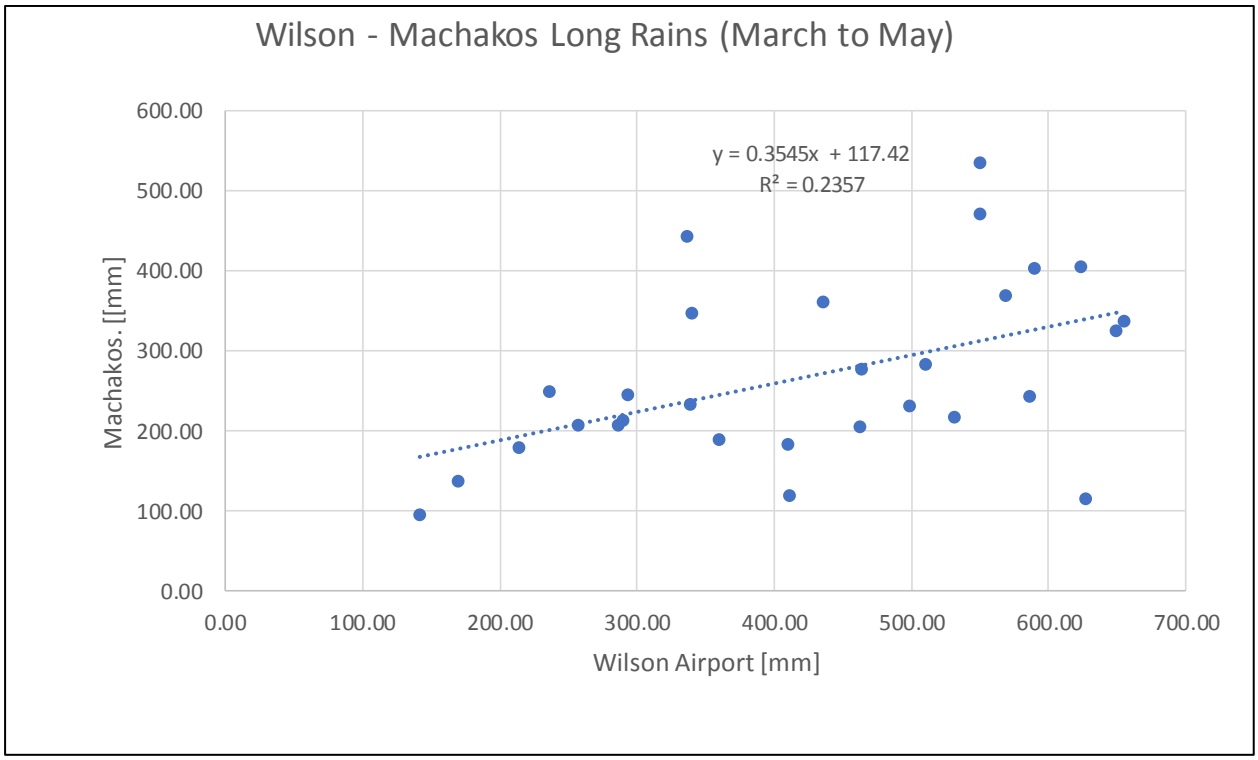


Figure 6.6: Scatterplot of rainfall totals in months March-May 1985 – 2016 observed at Wilson Airport and those observed at Machakos meteorological station

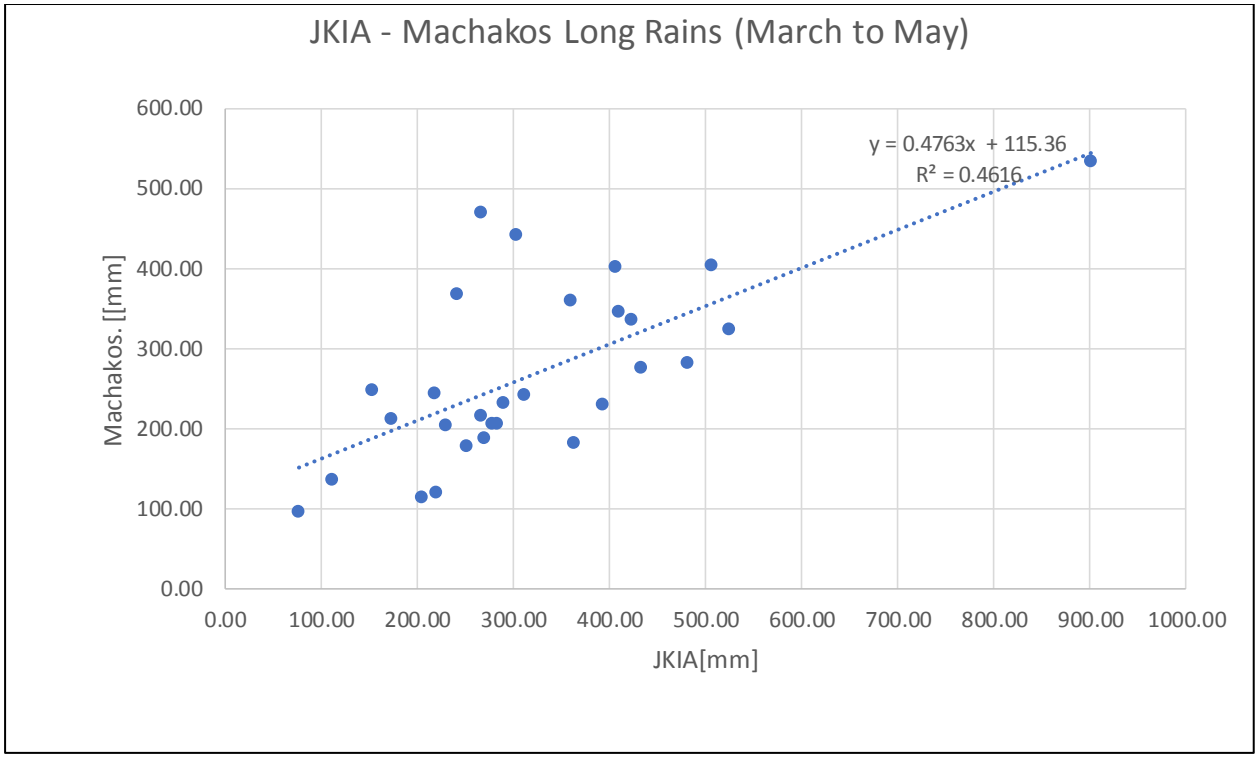
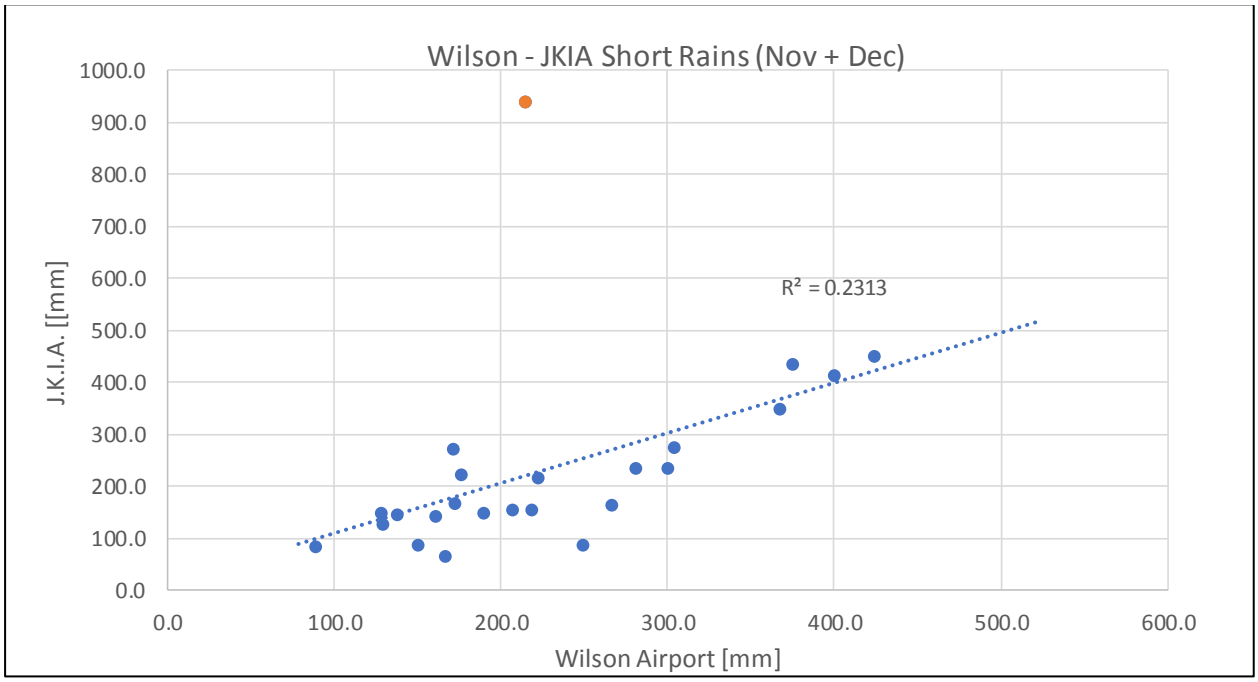


Figure 6.7: Scatterplot of rainfall totals in months March-May 1985 – 2016 observed at JKIA meteorological station and those observed at Machakos

Correlation values during the short rain seasons are similar to those observed to the long rain seasons, except that in the short rain seasons a large outlying value at JKIA station (year 1991) is responsible for lowering correlation coefficients. Omitting these values results in higher correlation between observations, as shown in figures below.



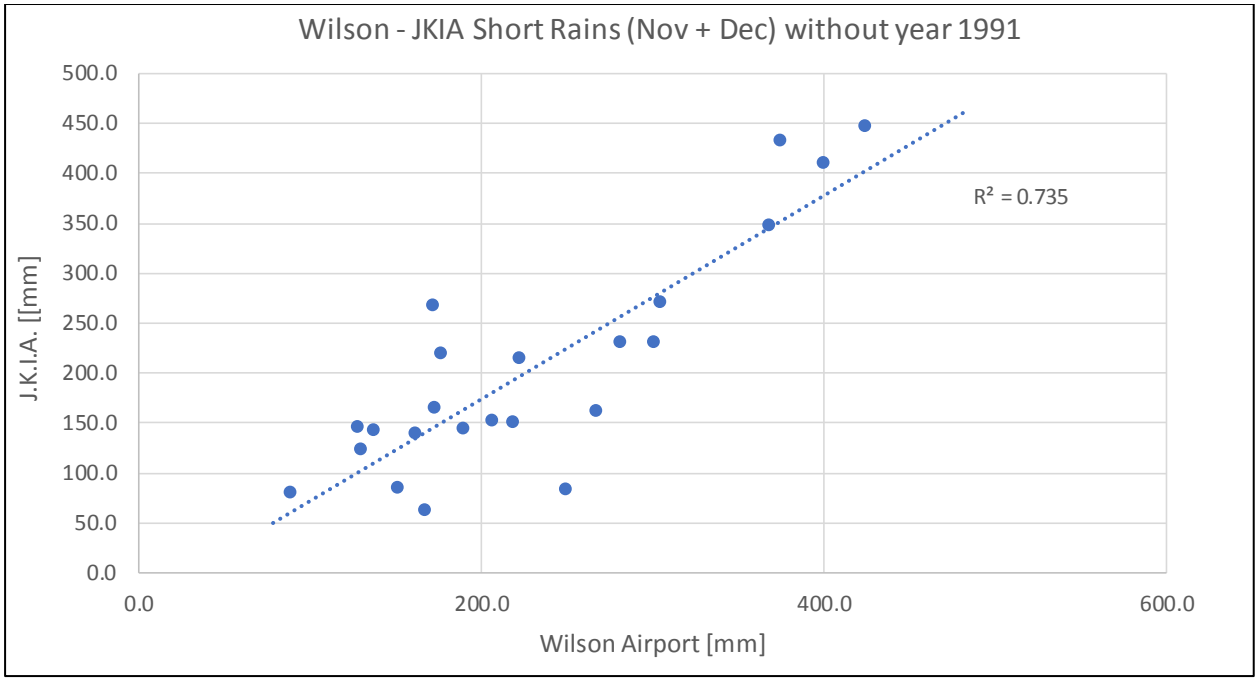
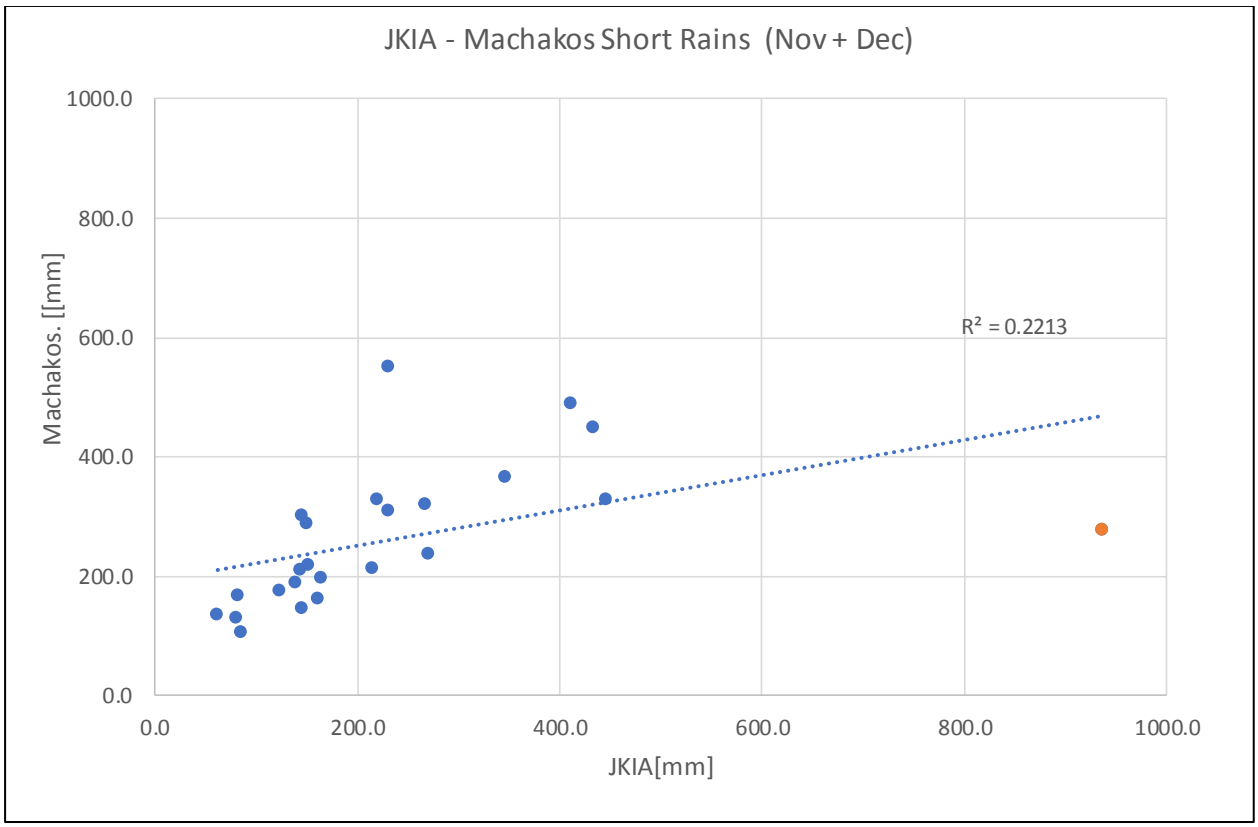


Figure 6.8 a-b: Scatterplot of rainfall totals in months November-December 1985 – 2016 observed at Wilson Airport and those observed at JKIA meteorological station with (up) and without (down) year 1991 (orange dot)



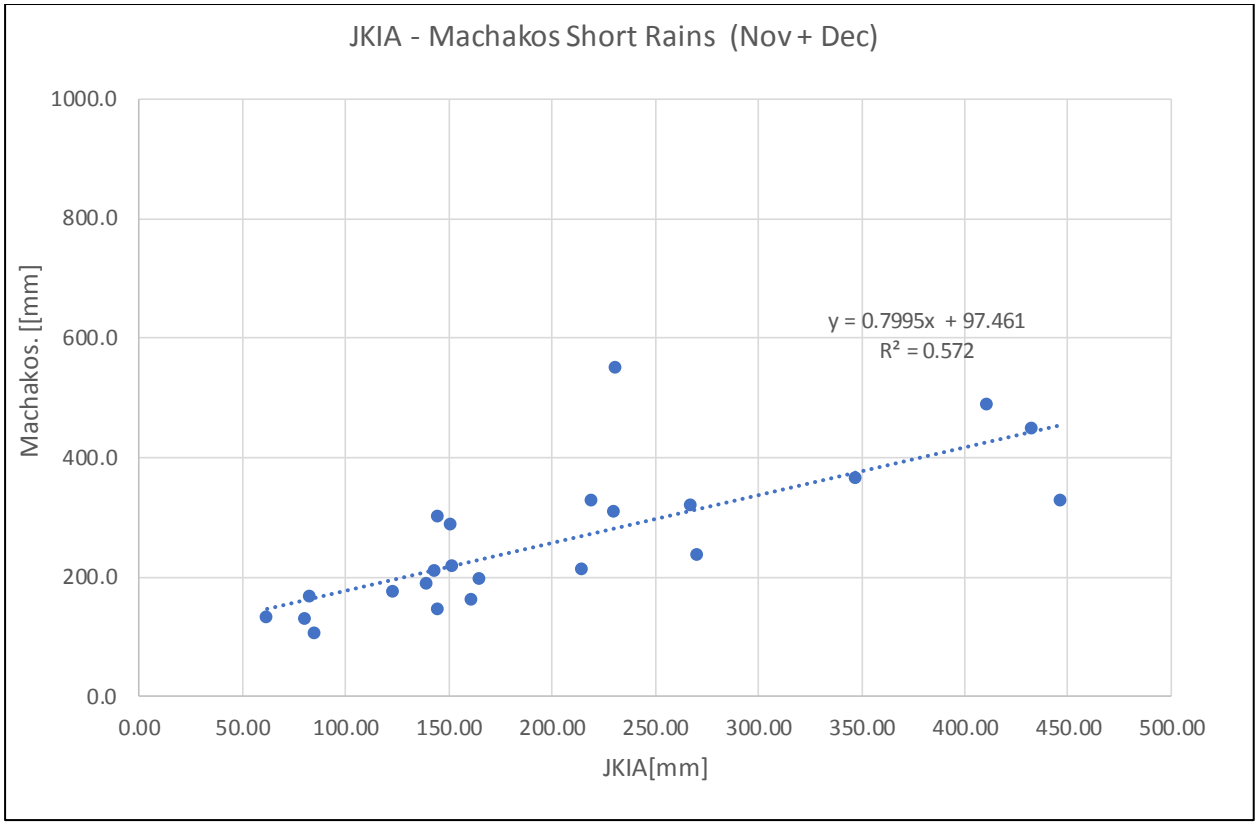


Figure 6.9 a-b: Scatterplot of rainfall totals in months November-December 1985 – 2016 observed at JKIA and and those observed at Machakos meteorological station with (up) and without (down) year 1991 (orange dot)

These figures show that, besides temporal variability, also high spatial rainfall variability is observed in the area and, quite likely, also in the catchments studied as potential sites for a reservoir. This may have implications when choosing a method to turn point to area rainfall observations.

Basic monthly and yearly statistics for observed rainfall at the three stations are summarized in table below.

Table 6-2: Basic statistic (mean and standard deviation) of monthly rainfall totals recorded at three meteorological stations

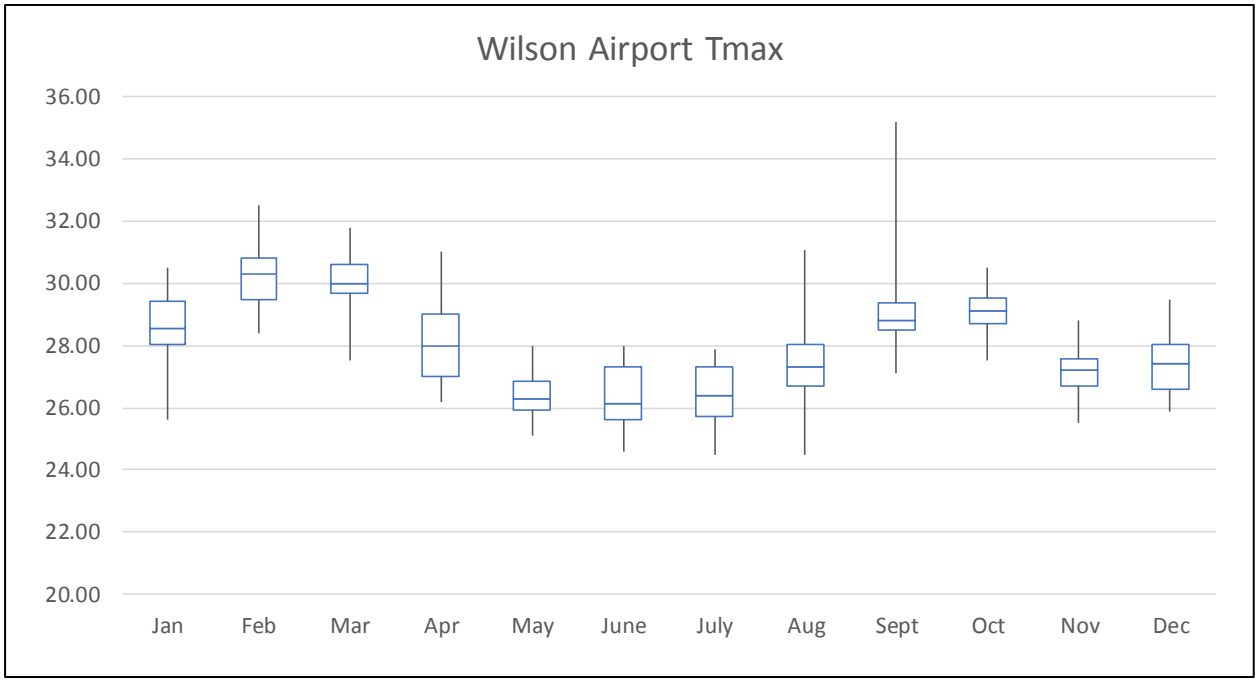
Station	Wilson Airport		JKIA		Machakos	
	Mean	St. Dev.	Mean	St. Dev.	Mean	St. Dev.
Jan	72.3	117.2	60.01	84.84	61.2	79.15
Feb	37.3	38.1	35.41	39.78	37.73	47.43
Mar	99.9	82.2	77.19	55.96	89.98	63.81
Apr	185.5	109.5	132.39	69.8	130.46	67.5
May	135.2	105.7	121.75	115.85	58.39	44.97

June	41.2	32.5	26.39	27.08	9.45	15.51
July	33	67.4	10.65	11.13	3.55	5.69
Aug	35.3	88.6	11.98	14.79	5.79	8.12
Sept	26.6	32.7	16.21	14.97	3.53	5.27
Oct	59.1	50.7	51.02	56.35	36.97	30.97
Nov	146.1	72	151.43	155.81	189.58	176.91
Dec	106.1	100.9	83.04	73.31	143.73	173.7
Year	**959.8**	**227.6**	**752.22**	**276.41**	**735.8**	**366.54**

Table 6-2 shows that average yearly rainfall at Wilson Airport is quite higher than that recorded at JKIA and Machakos meteorological stations. However, due to high seasonal and yearly variability, the three stations may have the same underlying probability distribution. To test this hypothesis, both a parametric test (paired t test) and a non-parametric test (the Rank-Sum test) were applied to yearly totals. The paired t-test is applied to the sequence of the differences between observed values in the same year at two different stations (Wilson-JKIA, Wilson-Machacos, JKIA-Machakos), while the Rank-Sum test considers the two samples separately. Results coincide for the two type of tests and confirm that yearly rainfall totals at JKIA and Machakos, regardless their differences, can be considered as identically distributed, while yearly totals at Wilson cannot be assumed to come from the same rainfall distribution. Probability distributions were also fitted to data and it was observed that yearly rainfall totals at Wilson and Machakos can be assumed to be normally distributed according to the Probability Plot Correlation Coefficient Test (PPCC), while a lognormal distribution is preferable for annual rainfall totals recorded at JKIA.

6.2.2 Temperatures

Figures below show box plots of monthly maximum and minimum temperatures recorded at the three stations.



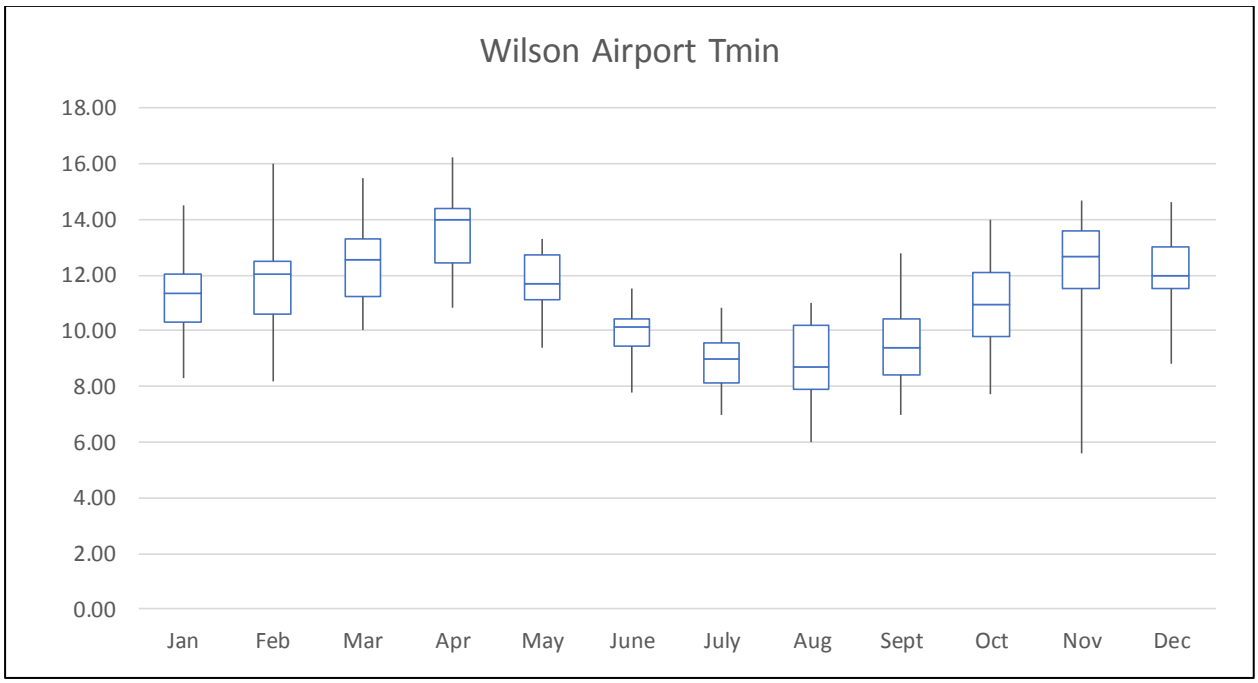
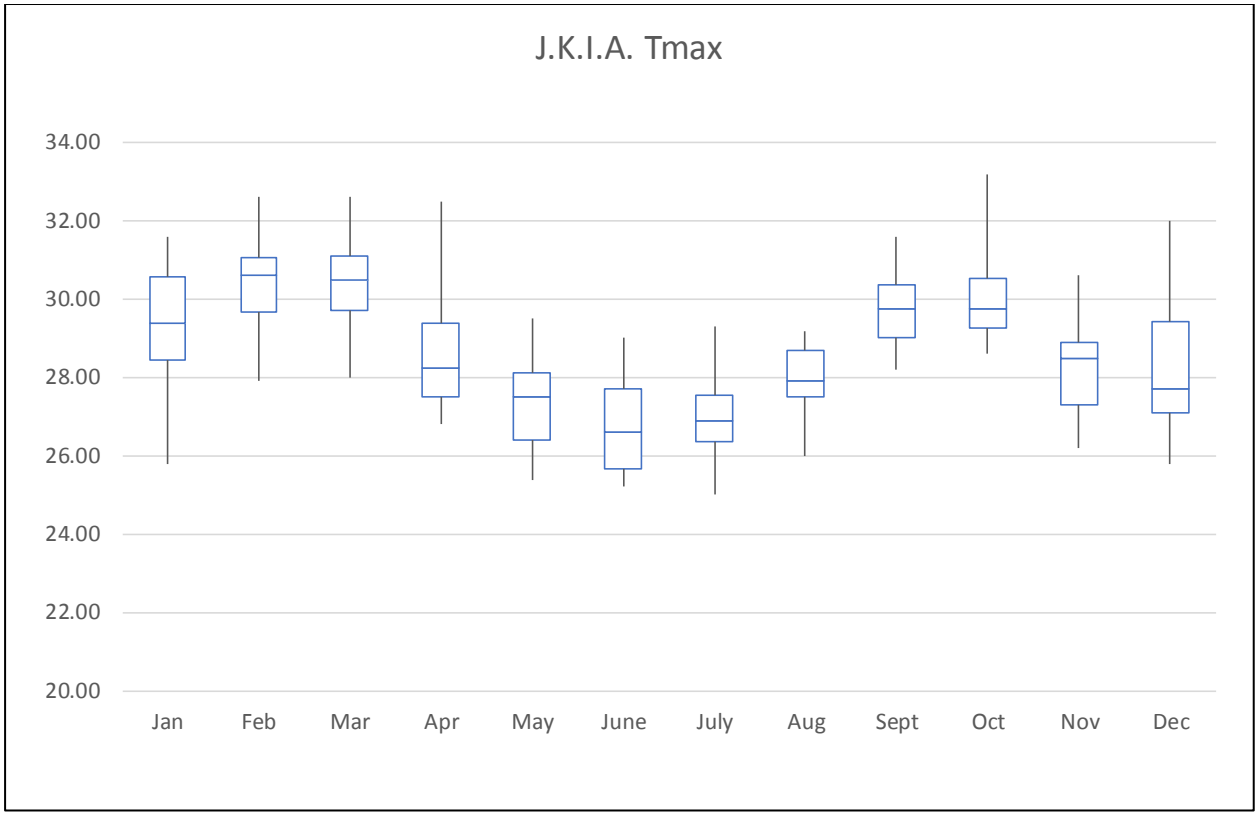


Figure 6.10 a-b: Box plots of monthly maximum and minimum temperatures recorded at Wilson Airport Meteorological Station



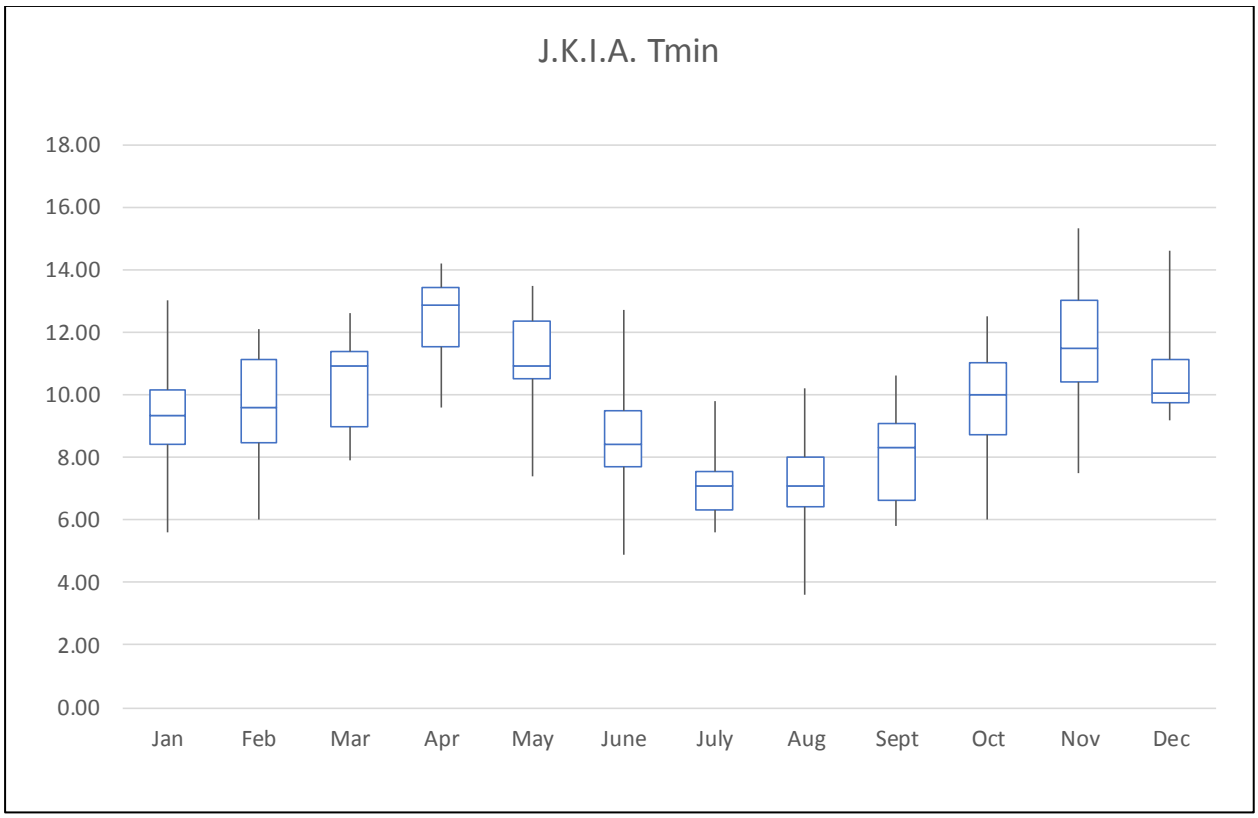
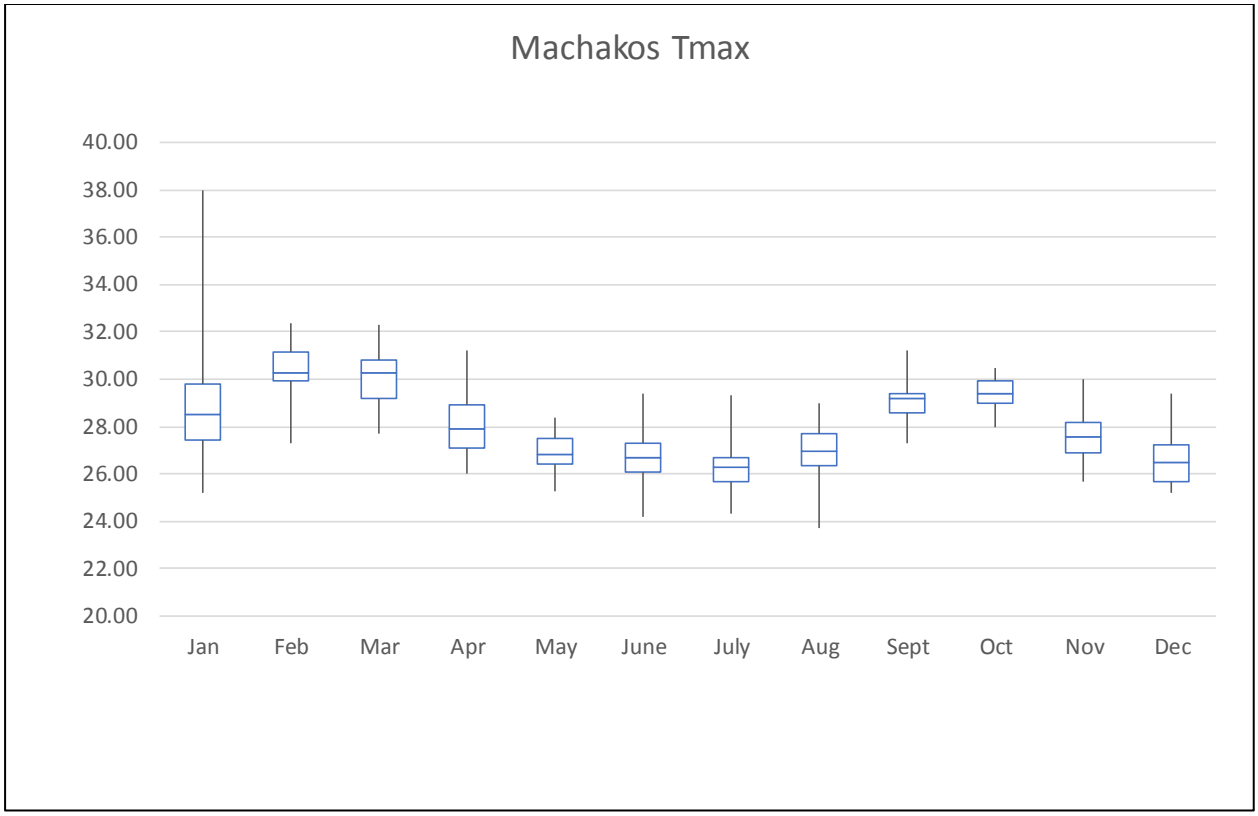


Figure 6.11 a-b: Box plots of monthly maximum and minimum temperatures recorded at JKIA Meteorological Station.



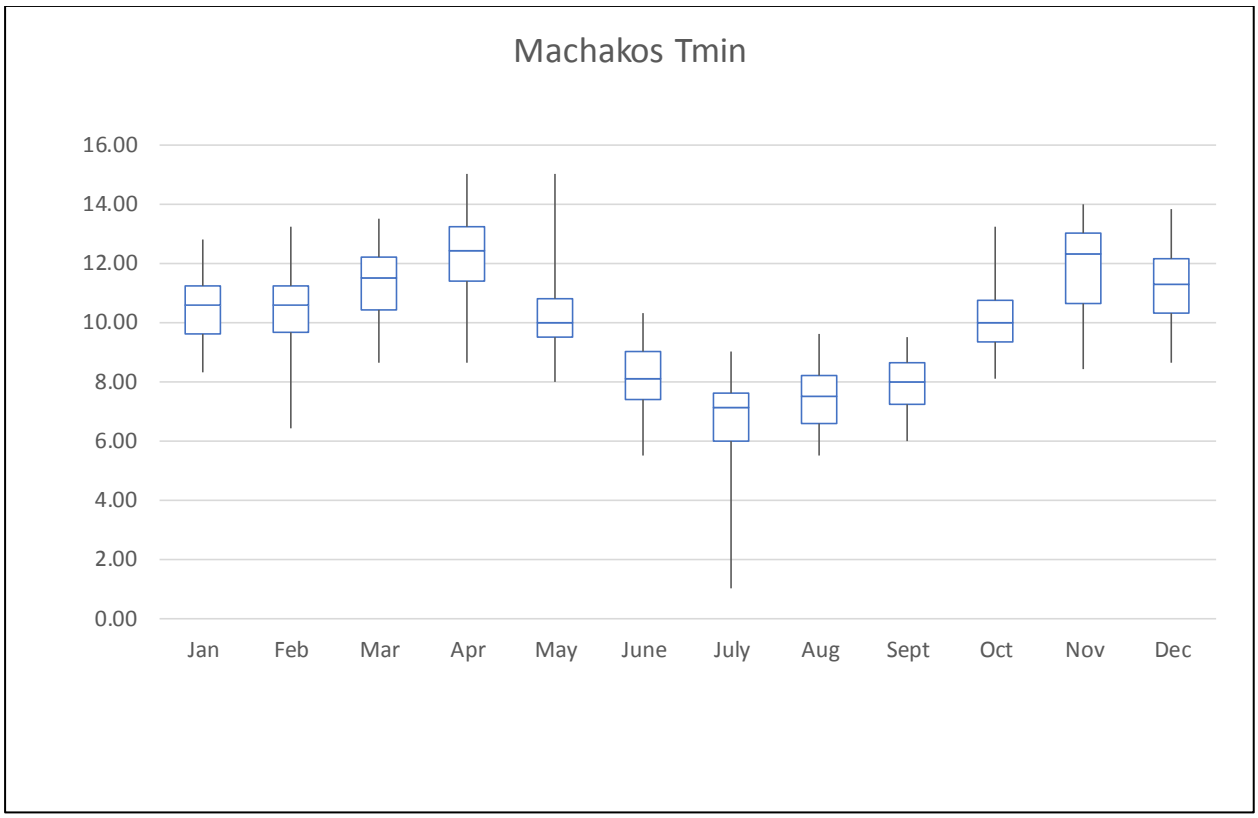


Figure 6.12 a-b: Box plots of monthly maximum and minimum temperatures recorded at Machakos Meteorological Station

The figures show that, unlikely precipitation, the temperature pattern is quite similar in the three records and that temperature variability is quite small. Tables below report respectively average maximum and minimum temperatures recorded at the three stations, and their standard deviations. It is worth highlighting that these average values represent the average over the observation period (1985 – 2016) of daily maxima and minima recorded in each month-type.

Table 6-3: Basic statistic (mean and standard deviation) of monthly maximum temperatures recorded at three meteorological stations

Station		Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Wilson Airport	Mean	28.6	30.2	30.0	28.1	26.4	26.3	26.4	27.4	29.1	29.1	27.2	27.4
	St. Dev.	1.1	1.0	0.9	1.3	0.8	1.0	1.0	1.3	1.4	0.7	0.9	1.0
JKIA	Mean	29.4	30.4	30.4	28.7	27.3	26.8	27.0	28.0	29.7	30.0	28.3	28.2
	St. Dev.	1.5	1.2	1.3	1.6	1.2	1.2	1.1	0.8	0.9	1.0	1.2	1.6
Machakos	Mean	28.6	30.3	30.0	28.2	26.9	26.7	26.4	26.9	29.1	29.4	27.6	26.7
	St. Dev.	2.3	1.3	1.2	1.4	0.8	1.2	1.3	1.2	0.9	0.6	1.1	1.1

Table 6-4: Basic statistic (mean and standard deviation) of monthly minimum temperatures recorded at three meteorological stations

Station		Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Wilson Airport	Mean	11.2	11.8	12.3	13.5	11.8	10.0	8.8	8.9	9.4	10.9	12.3	12.2
	St. Dev.	1.5	1.7	1.4	1.4	1.1	0.9	1.0	1.4	1.4	1.7	2.1	1.2
JKIA	Mean	9.3	9.5	10.2	12.5	11.1	8.6	7.1	7.2	8.0	9.9	11.5	10.7
	St. Dev.	1.7	1.7	1.5	1.3	1.4	1.7	1.0	1.7	1.5	1.7	1.8	1.4
Machakos	Mean	10.4	10.5	11.3	12.3	10.3	8.1	6.7	7.4	7.9	10.0	11.9	11.3
	St. Dev.	1.1	1.4	1.4	1.4	1.4	1.2	1.6	1.1	1.0	1.1	1.5	1.4

6.3 DROUGHT ANALYSIS

Drought is the result of climatic variability and is hence a natural phenomenon whose main characteristics are length, magnitude and areal extent. In addition, drought is a multi-faceted phenomenon, so that the variables, as well as the appropriate spatial and temporal unit to consider for drought analysis and characterization, depend on the specific aspect we are looking at and on the climatic context. For instance, while for rainfed crops in humid climates the relevant drought period is the day and the key variable is precipitation, in semiarid and arid climates drought analysis for reservoir design/management makes sense at a larger time scale, monthly or even yearly, and the key variable is streamflow.

In this study, the focus is to understand the impact of drought on water supply. One of the main goals of the dam system that is the object of this study is to increase the reliability of water supply and hence reduce the risk related to droughts. Design of the dam system will hence have to include explicitly the occurrence of critical dry periods.

It has been mentioned above that in arid and semiarid climates the pattern of rainfall at short time scale can significantly impact the rainfall-runoff process. This implies that precipitation totals itself provide poor approximations of streamflow variability. As a consequence, given the aims of this study, a drought analysis merely based on aggregated rainfall does not appear meaningful, and may be bring to misleading results. Drought analysis will be performed in the second part of the study and integrated with dam design and will be based on streamflow data.

6.4 METHODOLOGY FOR RAINFALL-RUNOFF MODELLING

Unless streamflow measurements are available at the sites under study, runoff assessment is generally performed through rainfall-runoff modelling. The study of the processes that turn precipitation into streamflow is central in the hydrological sciences and countless mathematical models of the most diverse form and complexity have been proposed, aimed at describing the process at all the different, relevant spatial and temporal scales and in the most diverse climatic contexts.

6.4.1 Available streamflow data

For such analyses it would be very desirable that streamflow measurements, either direct or indirect, be available, in order to calibrate the rainfall-runoff model. Even though there is no streamflow measurement in the catchments considered as potential reservoir sites, there are indeed some streamflow gauge stations in the Athi river basin that are relatively near to the study area (see Figure below).

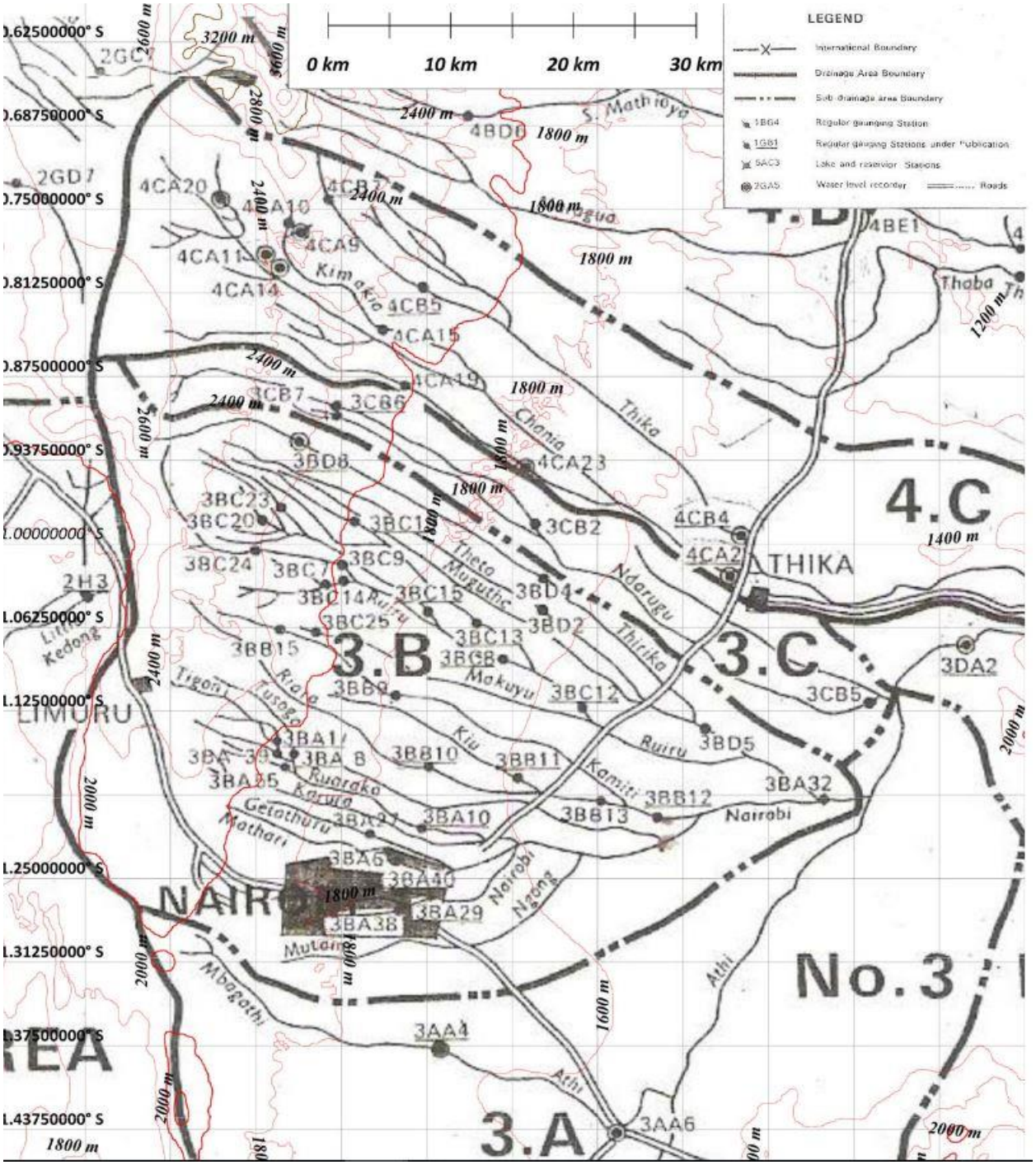


Figure 6.13: Location of streamflow gauge stations in the upper Athi river basin (source: WANJOHI KANYI, 2009)

As well known, the Athi river basin is one of the six sub-basins in which the whole national area is subdivided. The important amount of information concerning the Athi River Basin contained in the National Water Master Plan 2030 is generally available at an aggregate scale. This implies that considerable care must be exercised when transferring results for the whole river sub-basin (covering around 60,000 km^2, 37,750 km^2 of which pertain to the actual basin of river Athi) to a small area as the one considered in this study. More information on detailed

modelling at sub-basin level in the area of the Athi river basin contained in the National Master Plan and relevant for this study is reported in the following section.

6.4.2 Streamflow assessments in the National Water Masterplan 2030

The catchments considered in this study as potential dam sites fall within sub-basin of the Athi river basin denoted as 3AA (Figure 6.14). Sub-basin 3AA has an area of 724 km^2. In the National Water Masterplan, rainfall-runoff modelling has been performed for this sub-basin, as for all other sub-basins of the Athi river, using rainfall and temperature data from 1991 to 2010. Results are available in volume VII of the Plan (Databook – Part A - Hydrological and Meteorological Data) as average monthly flow (in m^3/s) at the sub-basin's closing section.

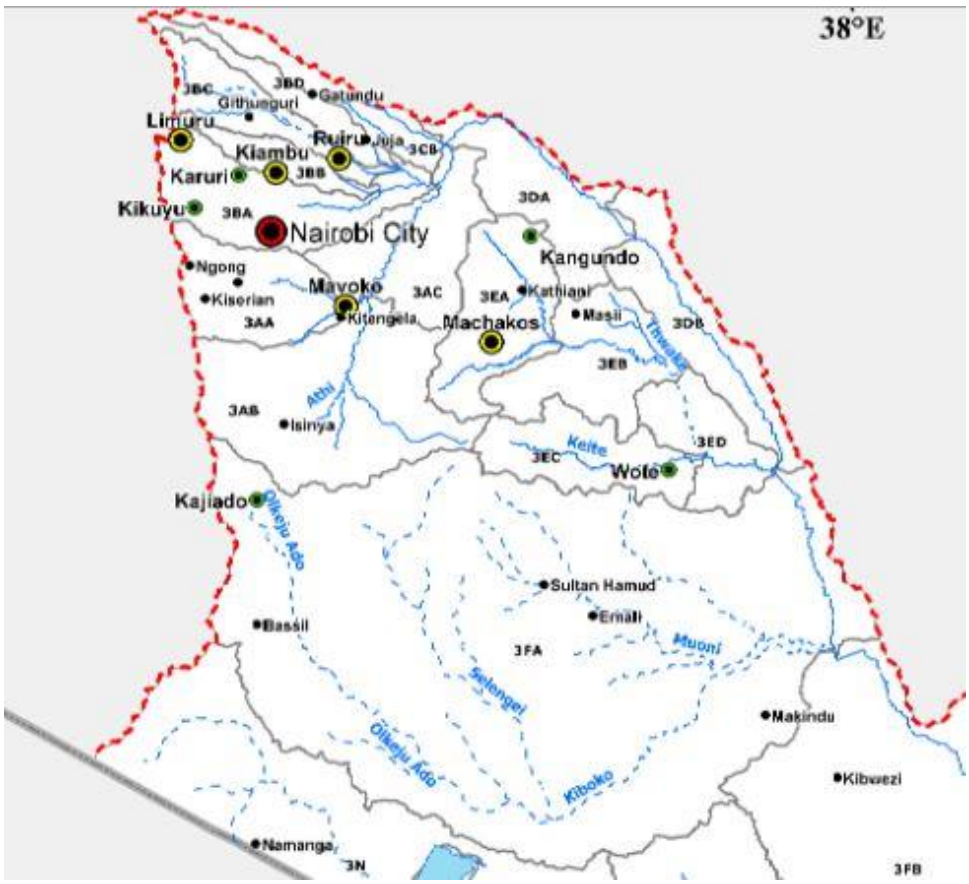


Figure 6.14: Subdivision of the river Athi in sub-basins (Source: Fig. 4.3.4 of Volume 5, sectoral 05, of National Water masterplan 2030)

Table 6-5 reports these data, together with a yearly total, expressed in 10^6 m^3. Dividing mean annual streamflow by the basin area (724 km^2), one obtains the streamflow per unit area (in mm), that is hence equal to 81.4x1000/724 = 112.5 mm.

These results are in good agreement with streamflow data from a gauging station on Mbagathi river (station 3AA04), with a catchment area of 257 km^2.Table 6-6 contains average monthly

discharges from this station recorder from 1956 to 1970, with missing values, and yearly streamflow totals in 10^6 m^3. The average yearly total of 28.35 Mm^3 corresponds to an average unit streamflow value of 28.35*1000/257 = 110 mm. While in average terms this value is in good agreement with average estimations from rainfall-runoff modelling from the National Water Masterplan, albeit for a different time window (1956-1970 instead of 1991-2010 of the National Master Plan), it should be observed that streamflow variability, expressed by monthly coefficients of variation, is always quite higher in the observed data that in the modelled ones.

Table 6-5: Average monthly streamflow (in m3/s) at the closing section of sub-basin 3AA reconstructed through rainfall-runoff modelling (Source: Volume VII – Databook, National Water Master Plan 2030) and yearly streamflow totals (in 106 m3)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year [Mm^3]
1991	2.40	1.50	1.30	1.70	5.90	3.70	1.60	1.30	1.20	1.30	1.40	2.00	66.5
1992	1.80	1.40	1.30	4.30	9.50	2.20	1.60	1.30	1.20	1.20	1.50	1.80	76.5
1993	6.90	10.30	2.60	1.60	1.60	1.40	1.30	1.20	1.20	1.20	1.40	1.90	85.7
1994	1.40	1.50	1.40	2.10	3.70	2.10	1.40	1.30	1.20	1.40	5.60	4.90	73.6
1995	2.20	1.80	4.60	3.50	6.70	2.40	1.40	1.30	1.20	1.30	1.60	1.50	77.5
1996	1.40	1.30	1.30	2.30	1.80	2.00	1.50	1.30	1.20	1.20	1.60	1.90	49.4
1997	1.30	1.20	1.20	7.00	9.80	2.40	1.40	1.30	1.20	1.40	6.70	12.60	124.8
1998	12.70	13.00	6.40	6.10	12.90	7.40	2.70	2.00	1.50	1.40	1.40	1.00	180.0
1999	1.40	1.40	1.70	3.50	2.70	1.60	1.40	1.30	1.40	1.30	1.80	8.50	73.6
2000	2.70	1.50	1.40	1.50	1.50	1.50	1.40	1.30	1.30	1.30	1.50	1.80	49.1
2001	11.30	4.80	4.20	9.30	4.90	2.90	1.80	1.40	1.40	1.40	2.00	1.90	124.3
2002	1.60	1.50	1.80	1.70	10.50	2.50	1.50	1.40	1.40	1.40	2.00	4.50	83.6
2003	4.70	1.90	1.50	1.80	12.70	4.80	1.90	1.50	1.40	1.40	1.80	1.70	97.5
2004	1.60	1.60	1.60	3.80	9.50	2.20	1.50	1.40	1.40	1.40	1.70	1.00	75.4
2005	1.60	1.70	1.70	2.10	6.70	5.10	2.00	1.50	1.40	1.40	1.60	1.50	74.4
2006	1.40	1.40	1.60	1.90	2.30	1.60	1.40	1.30	1.40	1.30	1.80	2.90	53.3
2007	3.90	2.40	1.60	1.60	1.70	1.50	1.30	1.30	1.30	1.30	1.30	1.30	53.9
2008	1.40	1.40	1.40	2.00	2.10	1.40	1.30	1.30	1.30	1.50	2.20	1.60	49.7
2009	1.30	1.40	1.30	1.40	1.50	1.50	1.40	1.30	1.20	1.30	1.50	1.60	43.9
2010	3.70	2.00	4.10	9.00	12.80	3.50	1.60	1.30	1.30	1.40	1.70	1.70	115.9
Mean	3.3	2.8	2.2	3.4	6.0	2.7	1.6	1.4	1.3	1.3	2.1	2.9	81.4
Standard Deviation	3.3	3.2	1.4	2.5	4.2	1.5	0.3	0.2	0.1	0.1	1.4	2.9	33.6
CV	1.0	1.2	0.7	0.7	0.7	0.6	0.2	0.1	0.1	0.1	0.7	1.0	0.4

Unavailability of rainfall records for the same period of observation of 3AA03 does not allow calibrating a rainfall-runoff model for this catchment, an attractive choice given that observations concern a catchment of dimensions similar to those under study as potential dam

sites. This lack of data leads to opting for a simple, widely applied, rainfall – runoff model: the CN methodology that will be illustrated in the next section.

Table 6-6: Average monthly streamflow (in m3/s) recorded at gauge station 3AA04 (Mbagathi river) and yearly streamflow totals (in Mm3) (Source: WRMA via Kwamboka, 2014)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year [Mm^3]
1956	0.259	0.265	0.077	0.705	0.891	0.06	0.049	0.046	0.039	0.029	0.061	0.045	6.6
1957	0.171	0.505	0.107	0.675	5.27	-	-	-	-	-	0.259	0.506	33.8
1958	0.06	0.067	0.075	0.105	4.31	0.318	0.3	0.21	0.114	0.079	0.075	0.007	15.0
1959	0.056	0.051	0.156	0.104	0.355	0.032	0.05	0.032	0.051	0.052	0.099	0.102	3.0
1960	0.026	0.024	0.094	0.263	0.15	0.06	0.06	0.047	0.034	0.048	0.05	0.018	2.3
1961	0	0.005	0.01	0.113	0.169	0.058	0.035	0.04	0.044	0.194	45.9	11	151.3
1962	6.41	0.855	0.507	0.572	1.57	0.942	0.529	0.342	0.106	-	-	-	41.5
1963	-	-	-	-	-	1.78	0.593	0.507	0.26	0.201	0.351	1.69	-
1964	1.94	0.356	0.572	10.4	1.46	0.642	0.338	0.486	0.352	0.269	0.207	0.227	45.3
1965	0.469	0.133	0.119	2.01	2.4	0.216	0.196	0.166	0.126	0.219	0.232	0.389	17.5
1966	0.146	0.144	0.289	2.93	0.353	0.284	0.177	0.156	0.162	0.124	0.263	0.037	13.3
1967	0.074	0.088	0.066	0.292	7.09	0.687	0.35	0.268	0.223	0.14	0.158	0.197	25.3
1968	0.084	0.118	0.3	0.765	0.803	0.542	0.263	0.2	0.152	0.142	0.595	1.93	15.5
1969	0.246	0.259	0.301	0.185	0.298	0.199	0.159	0.159	0.141	0.132	0.19	0.118	6.3
1970	0.174	0.116	0.11	3.5	2.14	0.675	0.243	0.18	0.144	0.137	0.158	0.108	20.2
Mean	0.72	0.21	0.20	1.62	1.95	0.46	0.24	0.20	0.14	0.14	3.47	1.17	28.35
St. Dev	1.71	0.23	0.17	2.75	2.15	0.48	0.17	0.15	0.09	0.07	12.21	2.90	37.87
CV	2.37	1.08	0.86	1.70	1.11	1.03	0.73	0.76	0.65	0.53	3.52	2.48	1.34

6.4.3 Rainfall-runoff model: The CN method

The traditional CN method is an event-based, lumped rainfall–runoff model. It is based on the combination of the water balance equation:

$$P = Ia + F + Q \qquad (1)$$

and the following two proportional equality hypotheses, expressed as:

$$Q/(P - Ia) = F/S \qquad (2)$$

and

$$Ia = \lambda S \qquad (3)$$

where P is total rainfall, Ia is initial abstraction, F is cumulative infiltration excluding Ia, Q is direct surface runoff, S is potential maximum retention and λ is the initial abstraction

coefficient. A combination of Equations (1) and (2) yields the popular form of the NRCS-CN method:

$Q = (P - Ia)^2 / (P - Ia - S)$

for $P > Ia$

$Q = 0$ for $P < Ia$ (4)

Parameter S is mapped to the CN using:

$S = 25.4*(1000/CN - 10)$ (5)

where S is expressed in mm and CN is a dimensionless runoff index varying in the range 0–100 and also varying with AMC, generally described using a 5-day antecedent rainfall.

Parameter Ia represents a specific rain volume subtracted preliminarily from the water budget and describes comprehensively different processes - interception by vegetation, accumulation in superficial depressions and initial soil imbibition. It is expressed as a fraction of potential maximum retention, according to a coefficient λ.

In the US literature λ is generally variable from 0.1 to 0.2. Applications in other contexts have shown that c is variable from 0.0 to 0.3.

The CN itself, on the other hand, varies according to 4 different classifications:

1) classification of the possibility of soil infiltration ('hydrologic soil group') A, B, C and D (where A indicates the most permeable soils and D those with the lowest permeability) ;
2) use of the soil (crops arranged in rows, pastures, woods, etc.), surface treatment (straight furrows, rollable furrows, support and terrace furrows);
3) drainage condition (bad, fair, good) ;
4) initial saturation conditions of soils.

Using tabulated values, the CN parameter can be quantified for the different conditions specified above in classifications 1 to 3 and for an average initial saturation value of soils (defined as AMC-II, where AMC indicates Antecedent Moisture Condition, initial saturation condition).

These three corresponding different AMC conditions (AMC-I, AMC-II and AMC-III, from maximum infiltration potential to minimum) are defined according to the rainfall depth occurred

in the five days preceding the event, on the basis of the rainfall thresholds reported in the following table.

Table 6-7: Thresholds of precipitation depths in the 5 days preceding the current one defining the three different antecedent soil moisture conditions

AMC	Precipitation depth in 5 antecedent days [mm]	
	Rest phase	Growing phase
AMC - I	< 12.7	< 35.6
AMC - II	12.7 ÷ 27.9	35.6 ÷ 53.3
AMC - III	> 27.9	> 53.3

CN_1 and CN_3 are defined as follows:

$$CN_1 = CN_2 / (2.3 - 0.013*CN_2) \quad (6)$$

$$CN_3 = CN_2 / (0.43 + 0.0057*CN_2) \quad (7)$$

6.4.4 First results from model application

The objective of this first model application is to obtain estimates of the average yearly runoff (in mm) flowing in the catchments of the study area and of its annual variability. These estimates are key decision variable to understand the adequacy of the projected development water resources plan to meet present and future demand.

When applying the procedure, some type of transformation from point rainfall to areal values must be assumed. In this specific case, the greatest difficulty stems from the lack of precipitation measurements in the catchments considered as potential dam sites. A traditional point-to-area transformation method such as that of Thiessen polygons, would lead to recognize Wilson Airport rainfall station as the only representative station for all the catchments. However, this station, albeit actually the closest to the catchments considered, features average yearly rainfall values that are significantly higher than those of the other two stations. On the other hand, the analyses of rainfall data from the three nearest available stations with an adequate record length (section 6.2.1) have shown that precipitation data are highly variable from station to station.

Hence, given the lack of precipitation data in the catchments and the above consideration, it was decided to apply the CN methodology (eqns. (3) to (7)) on a daily basis using separately the three rainfall series available (Wilson Airport, JKIA and

Machakos agrometeorological station) and to provide results both separately for each station and pooling data together. Uncertainty in the two key parameters λ and CN was accounted for by choosing a nominal value $\lambda_0 = 0.1$ and $CN_0 = 0.60$ and using $\lambda_H = 0.2$ and $\lambda_L = 0.05$ as uncertainty range for λ and $CN_H = 0.80$ and $CN_L = 0.50$ as uncertainty range for CN. A preliminary uncertainty analysis also included uncertainty ranges for the thresholds of 5-days (Table 6-7), but these parameters were excluded from further uncertainty modelling as yearly streamflow totals resulted quite insensitive to uncertainty in this parameter.

In the spirit of a First Order Uncertainty Analysis (Loucks and Van Beek, 2005) for each year from 1985 to 2015, the model was run first using nominal values and was then run again switching, one by one, each of the two parameters to their high or low estimate. Simple assumptions on parameters' probability distribution allow obtaining upper and lower limit of a confidence interval for yearly streamflow totals each year.

The First Order Uncertainty Analysis (FOUA) can be summarized as follows: let I be an indicator that can be extracted from the model. In this application I is the annual runoff total (in mm) in year t (1985, 1986…2015). Given our rainfall-runoff model, indicator I is a function of model's parameters Pi, i = 1…n. In this case the relevant parameters considered are only λ and CN. Denote with P_{0i} the value of the i-th model's parameter at its nominal (best estimate) value. Parameters are characterized by a certain uncertainty. Given a high (P_{iH}) and a low (P_{iL}) estimate of the i-th parameter, the simple assumption is that these parameters correspond respectively to the 5% and to the 95% of the probability distribution of parameter P, assumed as normal. Given the assumption of normally distributed parameters, the distance between nominal (P0) value and high and low estimates is hence 1.645 parameter's standard deviations, so that one can estimate parameter's variance by:

$$VAR[P_i] = \{[P_{iH} - P_{iL}]/(2*1.645)\}^2 \qquad (8)$$

According to FOUA, the expected value of indicator I is the value of the indicator at parameters' nominal values

$$E[I] = F(P0_1, P0_2, \ldots P0_n) \qquad (9)$$

And its variability is given by:

$$Var[I] = \Sigma_i [\partial F/\partial P_i]^2 \, Var[P_i] \qquad (10)$$

where

$$\partial F/\partial P_i = I(P_{iH}) - I(P_{iL})/(P_{iH} - P_{iL}) \qquad (11)$$

In (11) $\partial F/\partial P_i$ is the so-called sensitivity coefficient of parameter i: $I(P_{iH})$ and $I(P_{iL})$ are the value or runoff total for year t calculated using nominal values for all parameters, but for the i-th parameter, for which its high value is used to obtain $I(P_{iH})$ and its low value is used to obtain $I(P_{iL})$.

Denoting with It the annual runoff total in year t (1985, 1986….2015), uncertainty ranges for each year's streamflow may be obtained using the following:

$$It(5\%) = E[It] - 1{,}645*VAR[It]^{0.5} \qquad (12)$$

$$It(95\%) = E[It] + 1{,}645*VAR[It]^{0.5} \qquad (13)$$

Table 6-8 shows an example of the application of the FOUA with data of Wilson Airport station.

Repeating the same procedure with the rainfall data of three stations, on obtains results contained in Table 6-9 and Table 6-10. By doing so, considering years 1985 – 2015 and pooling together the results from the three rainfall stations, an average runoff value of 156 mm/year was obtained, with a standard deviation of 202 mm (CV = 1.30). This value is higher than the 112 mm value obtained through rainfall-runoff modelling in the National Water Masterplan for the whole 3AA sub-basin. However, considering the same time window (1990 – 2010) as the one used in the Master Plan, the average yearly streamflow is 127.7 mm (all stations pooled together), quite close to Master Plan estimations.

In addition, the coefficient of variation of annual streamflow obtained through the CN methodology is around 1.0, and hence much closer to the CV observed from gauged streamflow, against the considerably lower 0.4 estimated in the Master Plan. The comparison of results from the methodology used in this study and that adopted in the National Masterplan is documented in Table 6-11.

In summarizing, the CN methodology applied to daily rainfall data seem to yield results consistent with the average characteristics of the sub-basin of Athi river where the studied catchments are located. **Further refinement of calibration to find nominal, most reliable values of CN accounting for river basin's characteristics is in order for the development of the hydrologic assessments in the stage of preliminary dam design**.

Given the choice of averaging the outcomes of model outputs using the three different rainfall records, from table 6-9 (last row) it can be seen that the estimated average yearly runoff value is 156.3 mm.

Table 6-8: Example of application of the FOUA with data from Wilson Airport meteorological station. The red box highlights the years used for comparison with the rainfall-runoff modelling results of the National Water Master Plan 2030.

Year	Yearly		WILSON AIRPORT			$[\partial F/\partial P_I]^2$	$[\partial F/\partial P_{CN}]^2$	VAR[I]	Range [5% - 95%]	
	P_0	P_{I_L}	P_{I_H}	P_{CN_L}	P_{CN_H}	Var[P]	Var[P]		[5%]	[95%]
1985	153.4	222.9	95.3	116.7	205.5	1466.8	711.1	2178	76.7	230.2
1986	465.5	608.7	322.4	374.6	576.1	7392.3	3658.7	11051	292.6	638.5
1987	87.9	164.7	37.7	51.5	129.9	1453.8	553.2	2007	14.2	161.6
1988	266.6	433.0	130.8	176.0	395.2	8235.8	4335.1	12571	82.2	451.1
1989	147.3	281.6	49.4	78.8	249.6	4863.7	2630.4	7494	4.9	289.7
1990	156.2	246.5	98.5	116.6	224.2	1975.4	1043.3	3019	65.9	246.6
1991	75.5	159.0	28.7	42.0	137.2	1531.0	818.1	2349	0.0	155.2
1992	239.8	327.3	151.2	183.2	307.4	2796.9	1391.5	4188	133.3	346.2
1993	129.2	269.2	30.7	57.1	237.3	5132.1	2927.6	8060	0.0	276.9
1994	79.5	194.0	19.3	35.9	164.8	2751.6	1496.5	4248	0.0	186.7
1995	70.4	187.0	14.2	26.6	157.2	2692.7	1539.5	4232	0.0	177.5
1996	42.1	95.0	14.2	22.6	79.1	588.6	287.7	876	0.0	90.8
1997	180.4	336.7	80.3	111.1	298.9	5929.2	3181.6	9111	23.3	337.4
1998	549.3	720.4	387.9	444.1	682.3	9973.2	5116.4	15090	347.2	751.4
1999	65.1	142.3	26.8	37.4	121.4	1203.6	635.5	1839	0.0	135.6
2000	2.6	37.1	0.0	0.0	24.4	124.0	53.6	178	0.0	24.5
2001	478.3	708.1	287.2	349.4	639.4	15977.6	7585.8	23563	225.7	730.8
2002	131.8	235.1	72.1	89.2	206.5	2396.1	1240.8	3637	32.6	231.0
2003	240.7	374.6	127.7	166.3	343.9	5497.8	2842.8	8341	90.4	390.9
2004	148.7	267.3	76.4	99.4	236.9	3285.0	1706.7	4992	32.5	264.9
2005	152.8	258.0	77.4	100.4	233.4				152.8	
2006	170.8	298.9	81.4	110.4	266.5	4265.3	2198.2	6464	38.6	303.1
2007	37.9	135.4	1.6	7.9	109.0	1613.4	922.2	2536	0.0	120.7
2008	148.8	268.4	73.8	97.4	237.5	3417.1	1769.6	5187	30.3	267.2
2009	90.3	184.0	31.3	51.3	160.1	2103.5	1067.4	3171	0.0	182.9
2010	157.7	299.9	96.1	115.3	259.3	3743.9	1869.1	5613	34.5	281.0
2011	168.2	331.2	53.1	88.4	292.5	6975.6	3758.0	10734	0.0	338.6
2012	389.0	577.6	212.8	277.2	535.5	11998.8	6017.4	18016	168.2	609.8
2013	438.4	604.8	304.4	349.0	567.2	8138.0	4296.4	12434	254.9	621.8
2014										
2015	266.9	468.1	131.1	170.2	420.1	10239.6	5632.5	15872	59.6	474.1

6.4.5 Assessment of available water volumes from the candidate reservoir sites

The average yearly runoff estimated value of 156 mm may be used for all the conterminous catchments considered as potential dam sites to assess the average yearly inflow to reservoir (in Mm^3). As well known, given area Ai of the i-th catchment, average annual inflow Ii is calculated as Ii = 156*Ai/1000. This inflow, however, cannot be considered coincident with the actual water availability because losses will unavoidably take place. The largest of these losses include evaporation, seepage through the dam and spills through the spillways that occur when inflows exceed active storage capacity. These losses will be evaluated individually

in the preliminary design phase. Here, based on previous experience, they are quantified in the 15% of average inflow. Table 6-12 contains these assessments.

Table 6-9: Expected mean annual runoff, standard deviation and coefficient of variation /standard deviation / mean annual runoff) and its uncertainty range

Meteorological Station	RAINFALL	MEAN ANNUAL RUNOFF [mm]			STANDARD DEVIATION OF ANNUAL RUNOFF [mm]			Coefficient of variation		
	Mean annual rainfall 1985-2015 [mm]	5% percentile	Expected	95% percentile	5% percentile	Expected	95% percentile	5% percentile	Expected	95% percentile
Wilson Airport	977.7	72.0	191.0	321.3	96.7	141.4	192.1	1.3	0.7	0.6
J.K.I.A.	786.3	46.5	131.8	226.5	102.3	148.4	194.9	2.2	1.1	0.9
Machakos	769.6	60.9	146.0	244.8	229.8	282.1	334.0	3.8	1.9	1.4
Three station average	844.6	59.8	156.3	264.2	142.9	190.7	240.3	2.4	1.3	0.9

Table 6-10: Mean yearly runoff coefficient (runoff/rainfall) for the three stations considered and three stations average

Meteorological Station	Mean Runoff Coefficient 1985-2015		
	5% percentile	**Expected**	**95% percentile**
Wilson Airport	0.07	0.20	0.33
J.K.I.A.	0.06	0.17	0.29
Machakos	0.08	0.19	0.32
Three station average	0.07	0.18	0.31

Table 6-11: Comparison between rainfall and runoff data from the National Water Master Plan and from this study

Meteorological Station	Mean annual rainfall 1991-2010 [mm]		MEAN ANNUAL RUNOFF 1991-2010 [mm]		STANDARD DEVIATION OF ANNUAL RUNOFF 1991 - 2010 [mm]		Coefficient of variation of annual runoff		Mean runoff coefficient 1991-2010	
	This study	National Water Master Plan Vol VII Databook	This study	National Water Master Plan Vol VII Databook	This study	National Water Master Plan Vol VII Databook	This study	National Water Master Plan Vol VII Databook	This study	National Water Master Plan Vol VII Databook
Wilson Airport	902.9	868.7	159.6	112.5	137.2	46.4	0.9	0.41	0.18	0.13
J.K.I.A.	769.5		130.6		156.4		1.2		0.17	
Machakos	702.4		92.9		91.2		1.0		0.13	
Three station average	791.6		127.7		128.3		1.0		0.16	

Table 6-12: Estimated average yearly inflow into candidate dams and estimated average annual water availability

Dam site	Catchment Area (km^2)	Estimated average yearly inflow [10^6 m^3]	Estimated average yearly water availability [10^6 m^3]
Ereteti	73	11.4	9.7
Airfeld	27.5	4.3	3.6
Kitengela	121	18.9	16.0
Oloyankilani	5	0.8	0.7
NooPopong	15	2.3	2.0

7 DAM SITES FEASIBILITY STUDIES

7.1 PREMISES ABOUT DAM SITES

The project area is not endowed with surface water resources but with seasonal rivers which carry huge runoffs during the rainy seasons. The runoffs from the catchment areas are sufficient to replenish the reservoir during the two Kenyan rainy seasons of March-May and October-December. The selected dams should have the following features:

- The Geology of the dam site and the foundation soil properties should be such that it is capable of withstanding the stresses that will be exerted on it by both the embankment fill material and the water column in the reservoir;
- The reservoir should be watertight to avoid severe water losses through the soils and to save the proper water volume necessary to supply potable water to Kitengela and Kajiado towns;
- The dam should be equipped by an impervious element (core or upstream lining) connected to a foundation diaphragm (if necessary) in order to achieve the water tightness of the dam foundation and the safety against piping.

7.2 ANALYSIS OF THE POTENTIAL DAM SITES

During the initial stages of the project, several survey and studies have been carried out in order to select the best dam site. We have assessed the study of possible dam sites and catchment areas in all the region.

The following five dam sites have been identified in the project area, in which it could be possible to build a dam with a reasonable catchment area:

1. Ereteti.
2. Airfield.
3. Kitengela.
4. Noopopong .
5. Oloyankilani.

The sites are mainly along the main Kiserian – Isinya Road, except Oloyankilani site which is close to Kajiado Town.

The locations and main features of the five dam sites are described below.

7.3 LOCATION OF THE DAM WITH RESPECT TO THE SUPPLY AREAS

The elevation of the proposed dams should be such that water can flow by gravity to the supply areas since this would eliminate pumping and thus reduce operation and maintenance costs of the project.

The following elevations have been considered for the main towns to be included as scope of this project:

- **Kitengela Town** elevation 1,570.00 m.a.s.l.
- **Isinya Town** elevation 1,650.00 m.a.s.l.
- **Kajiado Town** elevation 1,730.00 m.a.s.l.

The following elevations have been considered for each dam site:

- Ereteti Dam, elevation 1,682.00 m.a.s.l.
- Ololotokish Airfield Dam, elevation 1,696.00 m.a.s.l.
- Kitengela Dam, elevation 1,630.00 m.a.s.l.
- Noopopong River, elevation 1,570.00 m.a.s.l.
- Oloyonkilani Dam, elevation 1,800.00 m.a.s.l.

Considering the above listed altitude assessment, the pumping is mandatory in particular to supply Kajiado Town and Isinya. Pumping should be maintained at minimum heads so as to use low rating pumping sets and hence reduce the operation and maintenance costs of the project.

7.3.1 Dam Site N°1: Ereteti

Ereteti Dam site is on Olooloitikosh River, near Grace Church, which lies between Ereteti Primary School and Olooloitikosh Market Centre along the main Isinya Kiserian Road.

The dam lays in an area which is a bit inhabited upstream and with some livestock.

Follow dam site coordinates.

- Ereteti Dam Location

37M UTM	255721E	9829529S	1,680 m a.s.l.

Considering the outcomes of our morphological analysis, the estimated storage capacity of the reservoir is about 4,900,000 m^3. The dam has a total catchment area of about 73 km^2.

The following pictures show Ereteti Dam site on the Olooloitikosh River.

Figure 7.1: The proposed dam location in Ereteti

Figure 7.2: The Olooloitikosh River bed – The proposed dam location in Ereteti

The main features of proposed dam are shown in the following table and figure:

Table 7-1: Main features of Ereteti Dam

Length	485.00 m
Height	22.80 m
Top of the Dam Level	1,692 m a.s.l.
Maximum Water Level	1,690 m a.s.l.
Reservoir Area	0.56 km^2

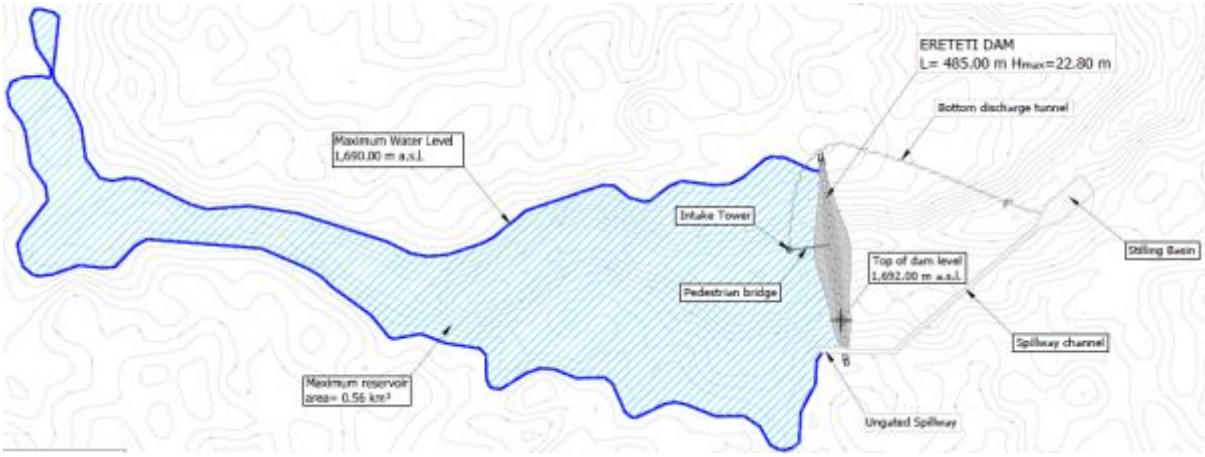


Figure 7.3: Reservoir dam layout of Ereteti Dam

The feasible type of dam should be an earthfill type with a central clay core or a rockfill type with a concrete core. The selection of the former type of dam will depend on the availability of proper material in sufficient quantities and qualities in the reservoir area (rock material for the embankment is abundant within the reservoir area).

7.3.2 Dam Site N°2: Airfield (Olooloitikosh Location)

Airfield Dam is located near the airstrip in the proximity of Ngenjuruarak River between Syathani Secondary School and Olooloitikoshi Market Centre along Isinya - Kiserian Road.

Follow dam site coordinates.

- Olooloitikosh –Airstrip Dam Location

37M UTM	256949E	9824888 S	1,696 m a.s.l.

From the outcomes of our morphological analysis, the estimated storage capacity of the reservoir is about 7,400,000 m^3. The dam has a total catchment area of 27.0 km^2.

The following pictures show Olooloitikosh – Airstrip Dam site on the Enkejo Oolowurak River.

Figure 7.4: The proposed dam location in Olooloitikosh

Figure 7.5: The Enkejo Oolowurak River bed – The proposed dam location in Oloolotikosh

The main features of proposed dam are shown in the following table and figure:

Table 7-2: Main features of Airfield Dam

Length	648.28 m
Height	40.10 m
Top of the Dam Level	1,702 m a.s.l.
Maximum Water Level	1,700 m a.s.l.
Reservoir Area	0.63 km^2

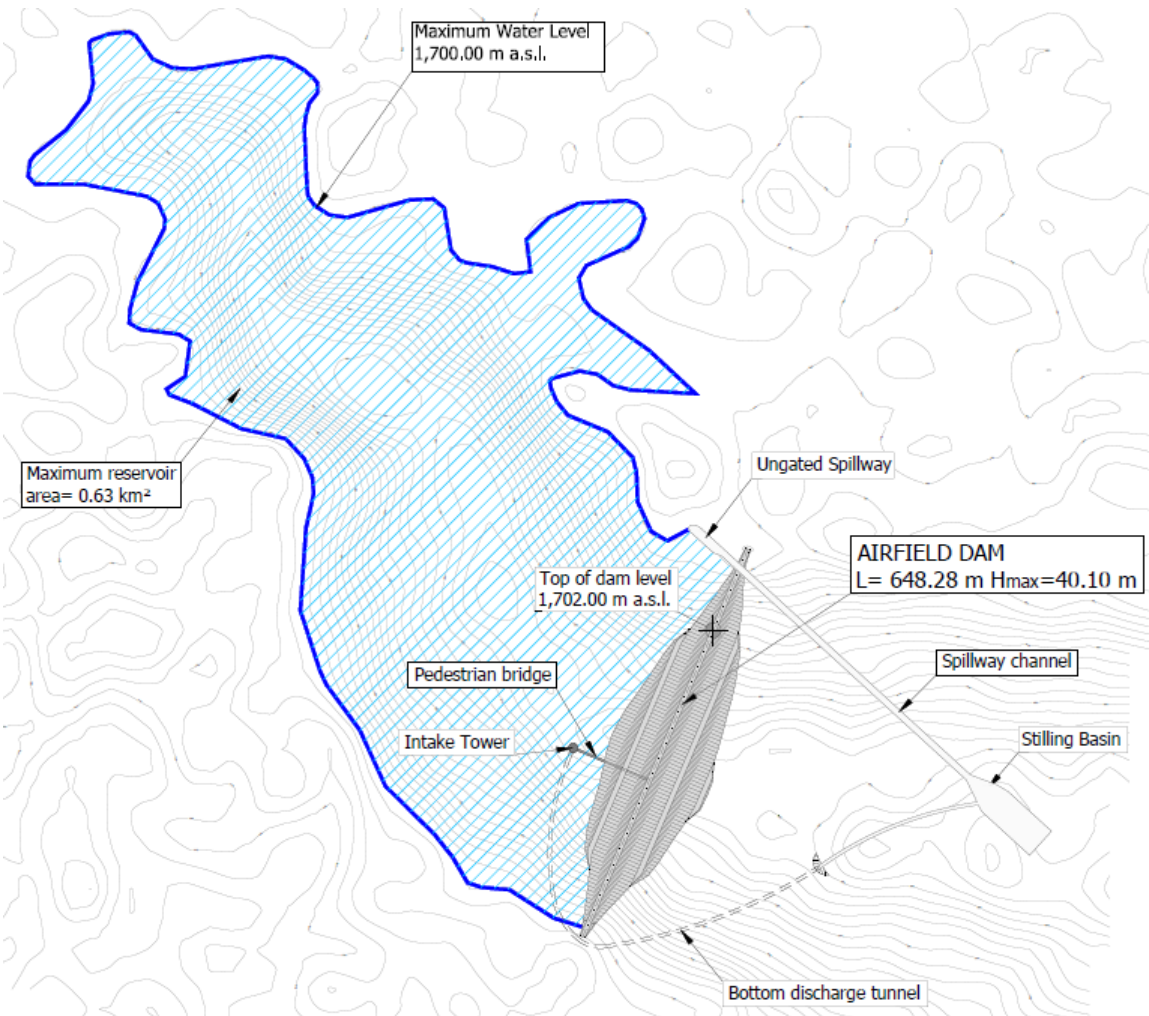


Figure 7.6: Reservoir dam layout of Airfield Dam

The feasible type of dam should be an earthfill type with a central clay core or a rockfill type with a concrete core. The selection of the former type of dam will depend on the availability of proper material in sufficient quantities and qualities in the reservoir area (rock material for the embankment is abundant within the reservoir area).

7.3.3 Dam Site N°3: Kitengela Dam

Kitengela Dam site is located along the Noopopong River, at a distance of approximately 15 km West of Kitengela Township in an area not heavily inhabited. There are no extensive agricultural activities within the dam site and in the catchment area. This is a positive attribute in the reduction of sediment loads, which would ultimately be deposited in the reservoir.

The dam site coordinates are as follows:

- Kitengela Dam Location

37M UTM	262503E	9838342S	1,630 m a.s.l.

Considering the outcomes of our morphological analysis, the estimated storage capacity of the reservoir is about 34,000,000 m^3. .

The dam has a total catchment area of about 124 km^2.

The following pictures show the proposed Kitengela Dam location on the Noopopong River.

Figure 7.7: The proposed dam axis location in Kitengela

Figure 7.8: The Noopopong River bed at proposed dam location in Kitengela

The main features of the proposed dam are shown in following table and figures:

Table 7-3: Main features of Kitengela Dam

Length	570.15 m
Height	33.10 m
Top of the Dam Level	1,627 m a.s.l.
Maximum Water Level	1,624 m a.s.l.
Reservoir Area	2.95 km^2

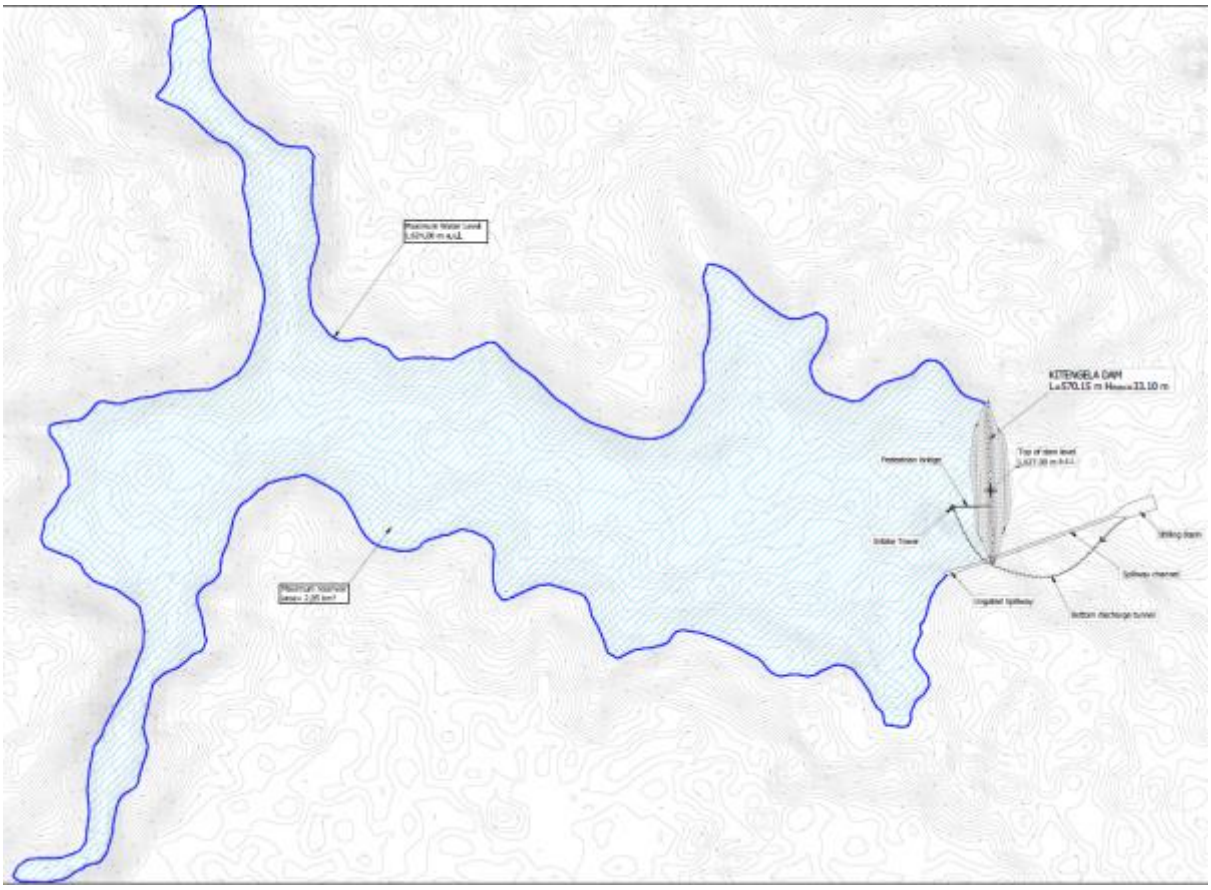


Figure 7.9: Reservoir dam layout of Kitengela Dam

The feasible type of dam should be an earthfill type with a central clay core or a rockfill type with a concrete core. The selection of the former type of dam will depend on the availability of proper material in sufficient quantities and qualities in the reservoir area (rock material for the embankment is abundant within the reservoir area).

7.3.4 Dam Site N°4: Noopopong

Noopopong Dam site is on Olooloitikosh River, which lies close to the Nairobi National Park. The dam lays in an area which is a bit inhabited but close to Kitengela town.

The dam site coordinates are as follows:

- Noopopong Dam Location

UTM 37M 265293.539 E 9837119.670 S 1560 m a.s.l.

Considering the outcomes of our morphological analysis, the estimated storage capacity of the reservoir is about 1,800,000 m^3. The dam has a very tight catchment area that can be estimated about 15.0 km^2.

The proposed dam location in Noopopong has a limited catchment area. Rock material is not available in the surroundings for the dam embankment.

The main features of proposed dam are shown in the following table and figure:

Table 7-4: Main features of Noopopong Dam

Length	500.30 m
Height	15.90 m
Top of the Dam Level	1,582 m a.s.l.
Maximum Water Level	1,580 m a.s.l.
Reservoir Area	0.31 km^2

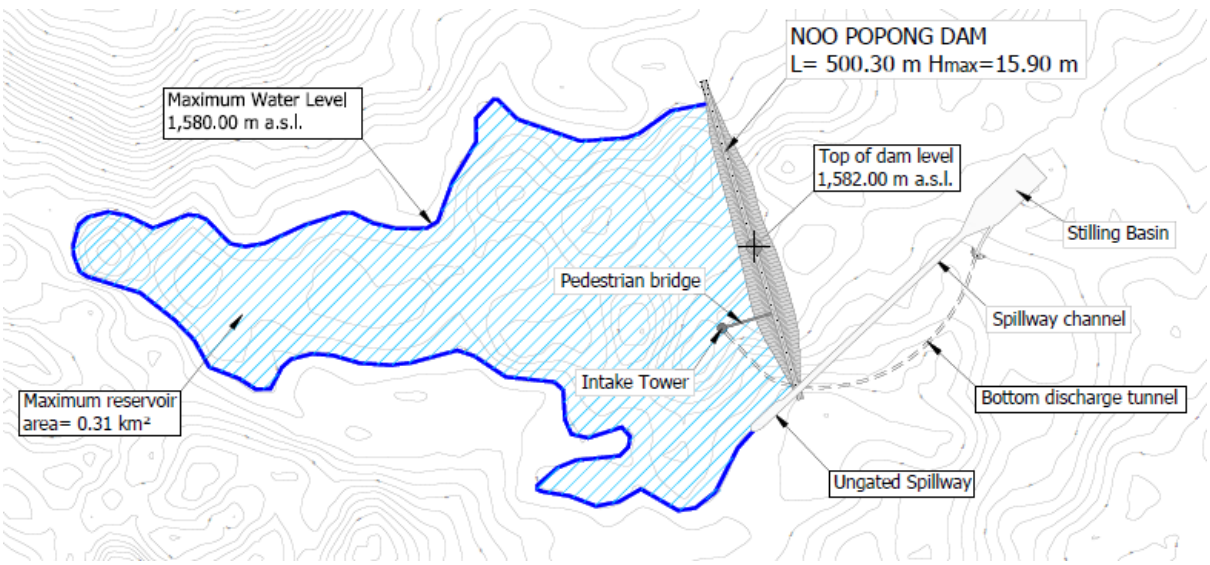


Figure 7.10 – Reservoir dam layout of Noopopong Dam

The following pictures show Noopopong Dam site on the Ololoitikosh River.

Figure 7.11: The proposed dam location in Noopopong

Figure 7.12: The Dam site in Noopopong

7.3.5 Dam Site N°5: Oloyankilani

Oloyankilani site is close to Kajiado town, close to the main Isinya Kiserian Road. The dam is located in an area which is a sparsely populated and with some livestock close by the Ilyagaleni Farm.

UTM 37M 243794.82 E 9806150.30 S 1,800 m a.s.l.

Considering the outcomes of our morphological analysis, the estimated storage capacity of the reservoir is about 4,400,000 m^3. The dam has a small catchment area of about 5.0 km^2.

The following pictures show the Dam site.

Figure 7.13: The proposed dam location in Oloyankilani

Figure 7.14: The proposed dam location in Oloyankilani

Figure 7.15: The proposed dam location in Oloyankilani

Few rock materials are available for the embankment in the surroundings, other materials are abundant within the reservoir area.

The main features of proposed dam are shown in the following table and figure:

Table 7-5: Main features of Oloyankilani Dam

Length	713.39 m
Height	36.00 m
Top of the Dam Level	1,822 m a.s.l.
Maximum Water Level	1,820 m a.s.l.
Reservoir Area	0.62 km^2

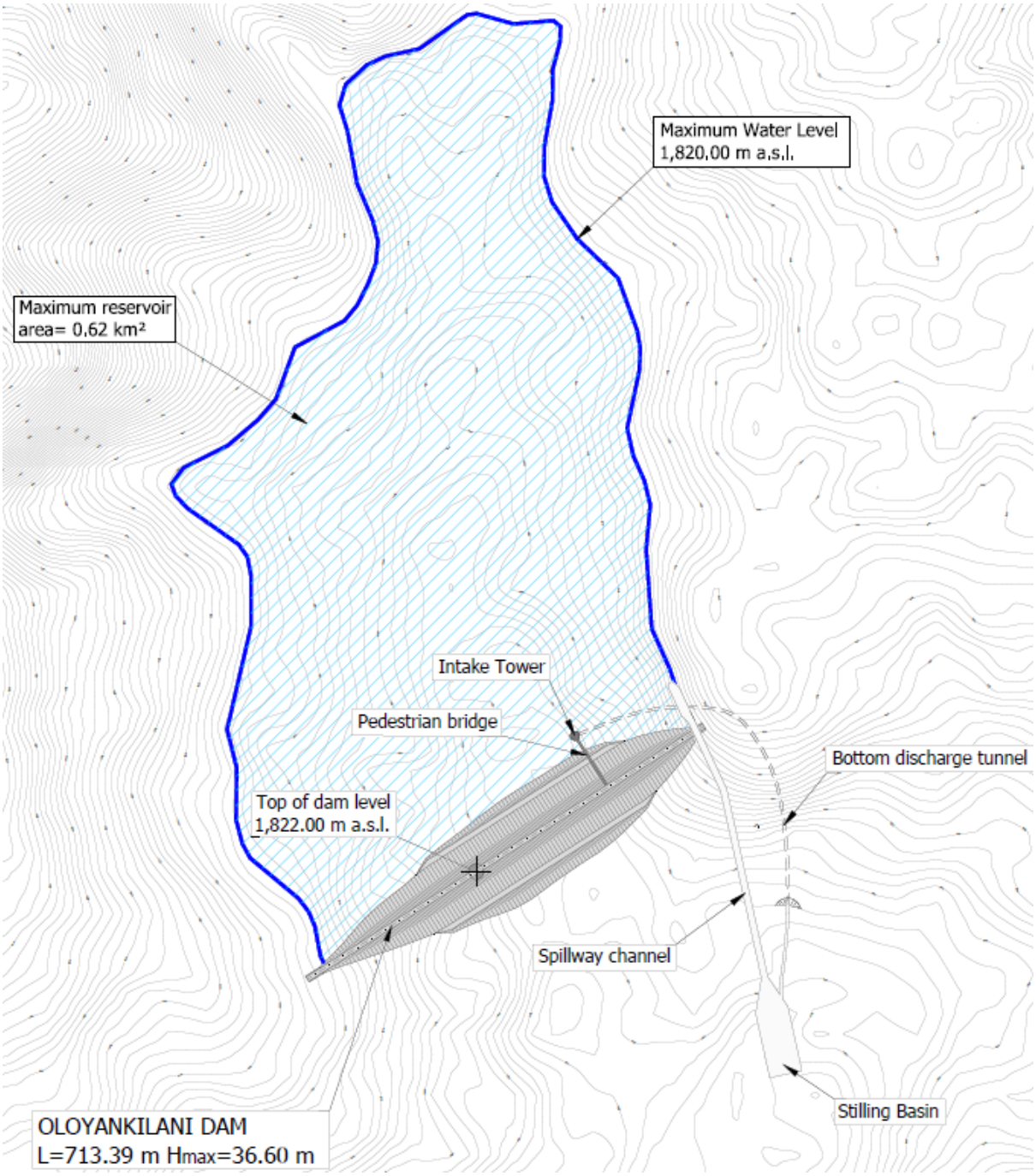


Figure 7.16: Reservoir dam layout of Oloyankilani Dam

7.4 AVAILABILITY OF CONSTRUCTION MATERIALS

Construction materials for the dam should be available in sufficient quantities and qualities and within reasonable distances from the dam site since this will reduce distances and ultimately reduce the construction costs of the project. However, through proper zoning of the embankment, cost of the materials available within the dam site may be readily used with very little treatment if any.

7.5 SOURCE OF POLLUTION

The quality of the raw water should be of acceptable levels so as to reduce water treatment costs. In the case of this project, this is a non-issue, because there are no agro-based industries within the catchment areas. The only source of pollution may be pesticides, which will be diluted in the reservoir and hence not causing any serious problems in terms of treatment. The other possible source of pollution is from the homesteads when toilets overflow during the rainy seasons with them ultimately discharging into the rivers and finally into the reservoir.

7.6 COMPARISON OF THE WATER DEMANDS AND THE STORAGE CAPACITY OF THE RESERVOIRS

From the information generated on the water demands and the storage capacities of the reservoirs, the following deductions are made.

I. The estimated Total Water Demand for the project is estimated to be:
   - 23,234 m^3/d in the present (2018),
   - 30,588 m^3/d Initial (2022),
   - 60,504 m^3/d Future (2032)
   - 101,117 m^3/d Ultimate (2042) design years respectively.

II. The total annual water demand for the initial year i.e. 2018 is about 8,480,000 m^3 and about 22,115,000 m^3 and about 36,900,000 m^3 for the future and ultimate stages of 2032 and 2042 years respectively.

III. The Five storage capacities of the dams, considering a realistic dam height, reported in the attached drawings, could be roughly estimate as follow:

- Ereteti 4,800,000.00 m^3,

- Airfield Dam 7,300,000.00 m^3,
- Kitengela 34,200,000.00 m^3,
- Noopopong 1,800,000.00 m^3,
- Oloyankilani 4,400,000.00 m^3.

8 DAM TYPE EVALUATION

8.1 GENERAL

Dam type selection and dam design are strictly dictated by several factors among which: the scope of the dam, the hydrological characteristics of the catchments; the morphology of the selected site and the regime of the river; the mechanical properties of foundation soils and the construction materials prevailing at the site; the climate; and, finally, the time schedule given for completion.

The selection of the best type of dam for a particular site calls also for thorough consideration of the characteristics of each type, as related to physical features of the site and the adaptation to the purposes the dam is supposed to serve, as well as safety, economy, and other pertinent limitations.

The geological, hydrogeological and geotechnical studies, together with the results of the site investigations, will allow to choose the most appropriate location of the dam axis and the section type and to define the nature, the geometry and the consistency of the available materials.

8.2 SELECTION OF THE DAM TYPE AND GEOMETRY

As before outlined, the selection of the dam type and geometry depends on several technical factors such as:

- Nature, morphology, hydrological and hydraulic characteristics of the catchment (catchment size, slope and shape, drainage pattern, storage in the catchment, land use/vegetative cover complex, etc.)
- Topography and conditions of the dam site with particular reference to accessibility;
- Geology of the area. Competent rock foundations with relatively high shear strength and resistance to erosion and percolation offer few restrictions as to the type of dam. Gravel foundations, if well compacted, are suitable for earthfill or rockfill dams.
- Seismicity of the area.
- Physical-mechanical properties of foundation soils. The embankment foundation must be stable under all conditions of construction and reservoir operation including seismic activities
- Availability and geotechnical properties of construction materials. The most economical type of dam will often be one for which materials can be found within a reasonable haul distance

from the site, including material which must be excavated for the dam foundation, spillway, outlet works, etc

- Height of the dam with respect to the reservoir area and freeboard.
- Location and dimensions of the spillway and of the bottom outlet. Their capacity must be sufficient to prevent overtopping of the embankment.

The design criteria of dam structures using locally available materials and adaptable to the socio-economic conditions should consider, for example, two types of cost-effective structures, which include earthfill dams and rockfill dams.

Both earthfill and rockfill dams have been constructed in almost every conceivable type of topography and allow the designer great flexibility in positioning the artefact in relation to foundation soils and in use and distribution of the available materials.

The possibility to use a large quantity of the same material indicate the choice of a homogeneous rockfill dam. Sufficient quantities of both pervious and impervious materials lead to the use of zoned earthfill dams.

For each structure, the final specifications concerning height, foundations and waterproofing are quite various, depending on the numerous factors before listed.

Based on the geomorphological conditions of the dam site and our past experience various possible watertight options may be related to earthfill and rockfill dam:

- an impervious central silty clayey core;
- a plastic concrete core or a bituminous core;

Depending on the geological and geotechnical features of the dam foundation soils (nature, grainsize composition, permeability, fracturing, etc.) a curtain grouting or a foundation diaphragm can be realized in order to achieve the water tightness of the dam foundation and the safety against piping.

As previously mentioned, the topography of the area is an important factor for the configuration of the dam site. The curve of the river in plan, and changes in valley cross section, may favour the adoption of an upstream facing core rather than central core to reduce the quantities of earthfill. Local changes in slope of the abutments may lead to differential settlement and cracking, needing more extensive filter drains. Narrow valleys, good foundations, and large stream flows

tend to indicate the selection of concrete dams, whereas wide valleys, poor foundations, and small stream flows tend to indicate the choice of embankment dams. The low, rolling plains field would, with equal suitability, suggest an earthfill/rockfill dam with a separate spillway.

As a matter of fact, the knowledge of soil conditions, the foundation bearing capacity, the thickness of the soil strata, the permeability, the existing faults, and fissures will lead to the final selection of the most suitable dam type.

8.2.1 Zoned earthfill dam with impervious core

An earthfill dam is composed of suitable fine grained materials obtained from borrow areas or required excavation and compacted in layers by mechanical means such as tamping rollers, sheepsfoot rollers, heavy pneumatic tired rollers, vibratory rollers etc..

One advantage of an earthfill dam is that it can be adapted to a weak foundation.

This type of dam can be either zoned or homogeneous, the latter requires large quantities of the same material and an impervious lining.

It is preferred from experience to have a zoned earthfill dam with a central core made of compacted silty clayey material (see the following figure).

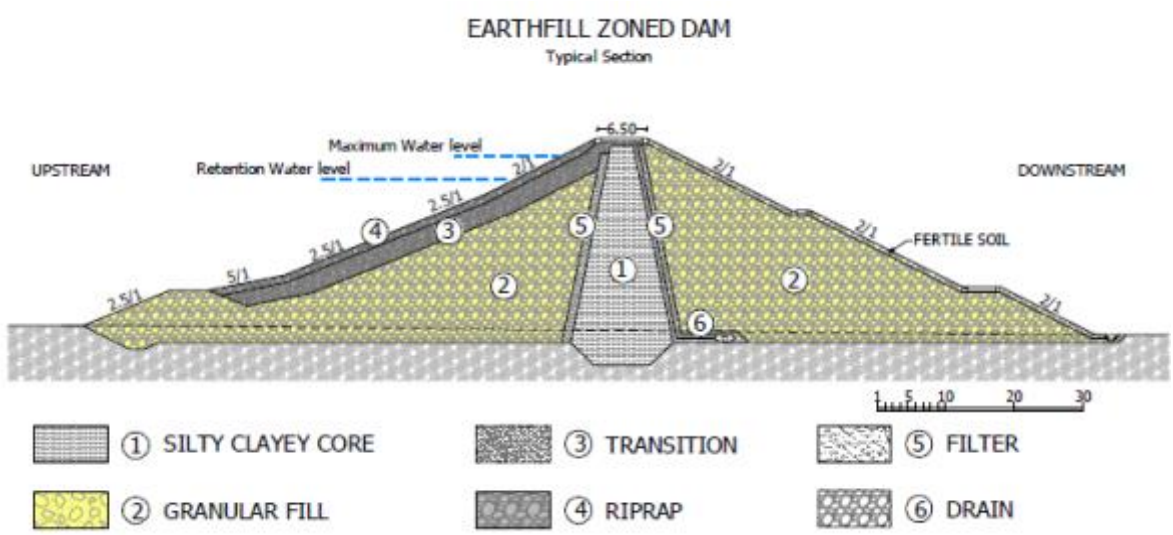


Figure 8.1: Zoned Earthfill Dam

Zoning is performed to ensure safety in terms of adequate strength and control of seepage and cracking. The final selection of the zones, therefore, is a matter of designing the cross section

that results in an ideal balance between safety and economical use of materials from excavation and available borrow materials.

As a defence against potential for cracking, the downstream portion of the dam is drained by a horizontal drainage layer combined with a vertical or inclined filter drainage layer.

A drained downstream abutment (such as that in the figure below) permits use of lower quality materials, a random zone, in a section downstream of the internal drain.

Water tightness of the dam foundation may be achieved, for example, by means of a grouting curtain, as shown in the figure below.

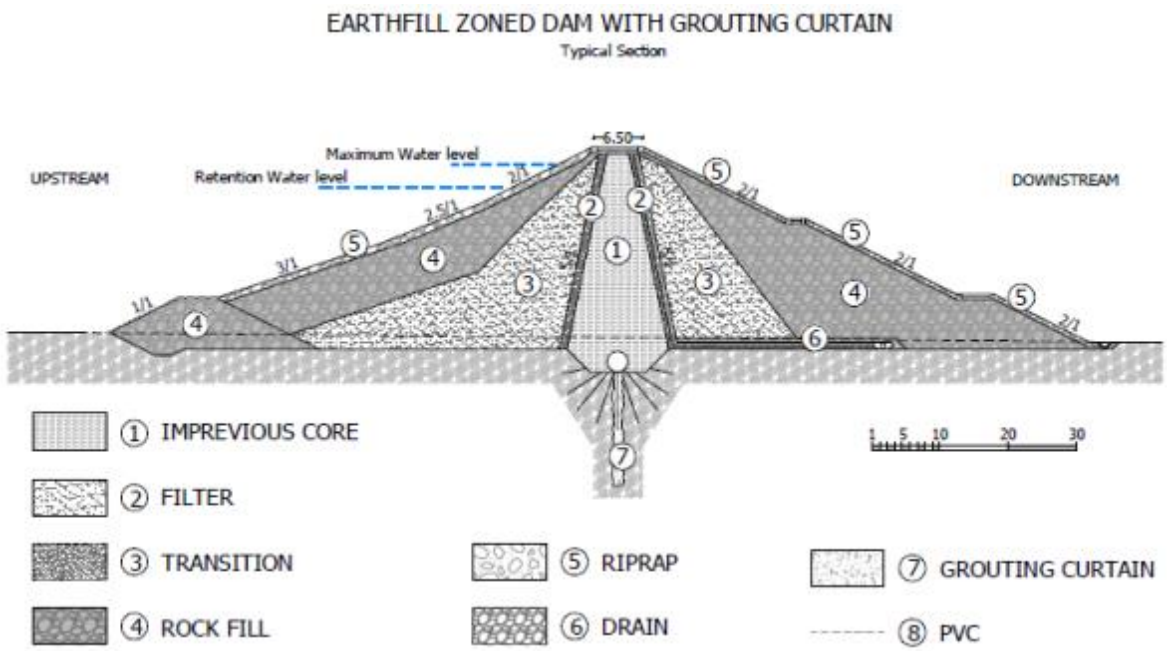


Figure 8.2: Zoned Earthfill Dam with grouting curtain

8.2.2 Rockfill dam with plastic core or bituminous core

A rockfill dam is composed of largely of fragmented rock with an impervious core or an impervious lining upstream. “Rockfill” shall include angular fragments such as those produced by quarrying or occurring naturally as talus and sub-angular or rounded fragments such as coarse gravel, cobbles, and boulders

The following figure shows a rockfill dam with a specific water tightness device (an impervious core made of plastic concrete). The core is separated from the shoulders by a series of transition zones (filters, drains) built of properly graded material.

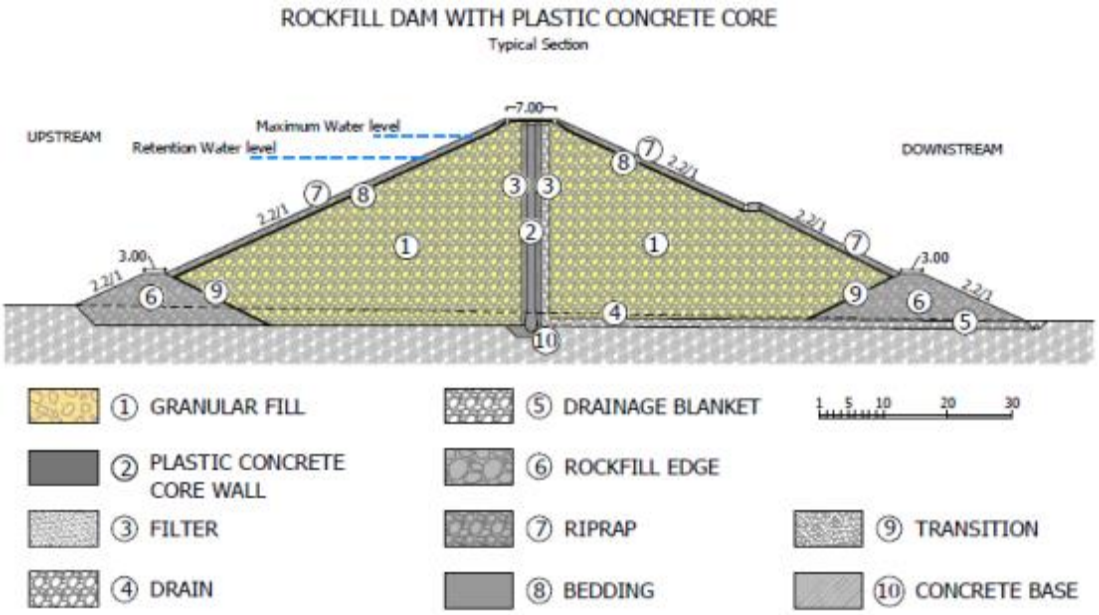


Figure 8.3: Rockfill Dam with plastic core

The adoption of a central bituminous core allows to follow the differential settlements that could develop, without any damage of the watertight element. As known, the rheological properties of bituminous conglomerate are suitable with climatic features and earthfill/rockfill embankment on weak foundation.

The selection of a central watertightness element made of plastic concrete present the advantage of a faster construction than the asphaltic core, processing techniques very simple and not impacting on the environment.

If necessary, the impervious core may be connected to a foundation diaphragm through a concrete basement, in order to achieve the watertightness of the dam foundation and the safety against piping.

The rockfill type is adapted to locations where the supply of good rock is ample, foundation rock is at or near the ground surface, suitable soil for an earthfill dam may not be readily available or long periods of high rainfall may make construction of an earthfill dam impracticable, and the construction of a concrete dam is less economical.

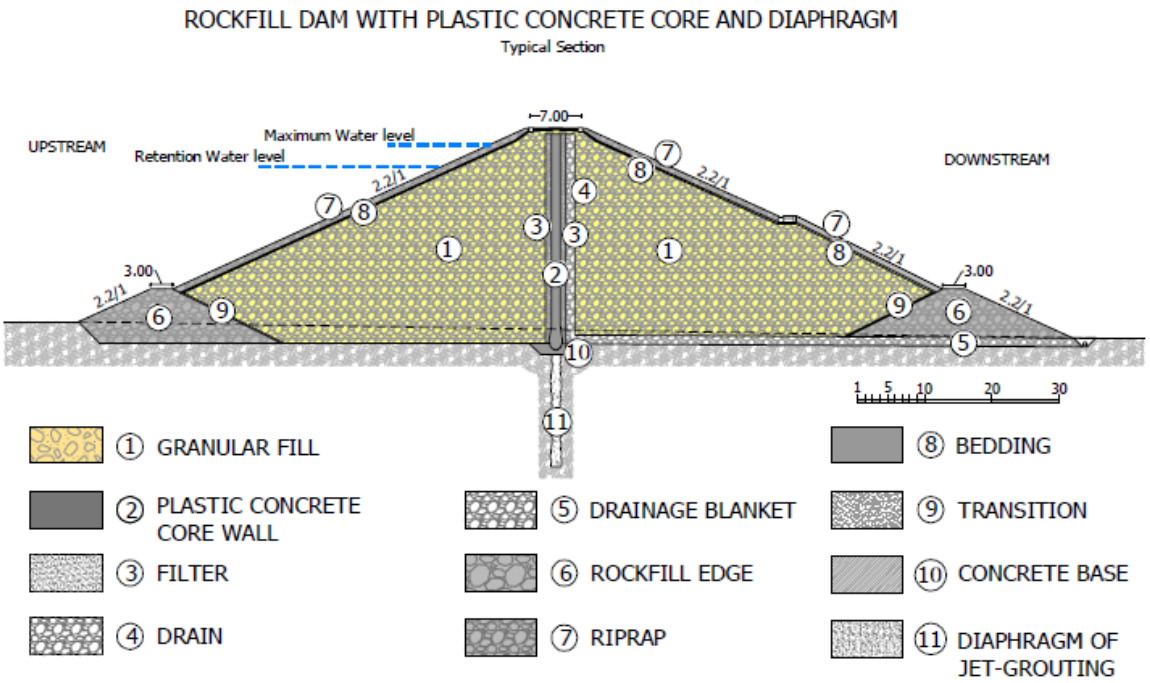


Figure 8.4: Rockfill Dam with plastic core and foundation diaphragm

The best "Dam type" for each of the selected sites will be defined on the basis of quality and quantities of construction materials available on site, and on the results of geotechnical investigations and laboratory tests that shall be carried out once the dam site will be defined.

8.2.3 Main design criteria of the dam embankment

The following criteria must be met to ensure satisfactory earth and rockfill structures:

- The embankment, foundation, abutments, and reservoir rims must be stable and must not develop unacceptable deformation under all loading conditions brought about by construction of the embankment, reservoir operation, and earthquake.
- The reservoir rim must be stable under the most severe operating conditions to eliminate the possibility of an unstable thin rim or the possibility that large existing landslides, when inundated by the reservoir or during rapid drawdown, would be triggered and cause a large volume of slide material to fill the reservoir and create a large wave that would overtop the dam.
- Seepage through the embankment, foundation, abutments, and reservoir rim must be controlled to prevent excessive uplift pressures, internal erosion and piping, instability,

sloughing, removal of material by solutioning, or erosion of material by loss into cracks, joints, and cavities. In addition, the purpose of the project may impose a limitation on the volume of seepage.

- Filters must be required for any high or moderate hazard embankment dam, not just something to consider. •
- Freeboard must be sufficient to prevent overtopping by waves.
- Spillway and outlet capacity must be sufficient to prevent overtopping of the embankment and encroachment on freeboard.

The design should consider seepage control measures such as:

- Foundation cut-offs
- A core of appropriate width of nonbrittle impervious materials
- Upstream impervious blankets
- Transition zones and filters; internal drainage such as inclined or vertical drains, drainage blankets, or pervious zones
- Toe drains
- Relief wells
- Defensive measures against seepage and piping at all embankment/structure interfaces.

In addition, close construction control is required to ensure proper foundation treatment, use of fill materials, gradation control, water content control, compaction control and installation of cut-offs and/or drainage facilities.

8.3 OUTLET WORKS

An important operation, at least equal to that of the dam itself, is assumed by the outlet works (spillways, bottom outlets), through which the water is conveyed either in the open air or underground.

The hydraulic design of such structures (spillway, bottom outlets and dissipation structures) will follow the USBR standard approaches and other relevant standards and recommendations such as those of ICOLD (International Commission on Large Dams).

8.3.1 Spillway

The spillway is a critical part of dam construction. An under-designed spillway will result in the dam overtopping or serious spillway erosion during peak runoff. These situations can cause major water losses, potential flooding and damage downstream, in addition to the costs to repair the dam.

Spillway requirements are dictated primarily by the runoff and streamflow characteristics as well as elevation and water retention capabilities of the reservoir rim, independent of type or size of the dam. The selection of specific spillway type will be influenced by the magnitudes of the floods to be passed. On streams with large flood potential, the spillway takes on even greater design significance.

The preferred locations would be on the dam abutments or through the reservoir rim.

Follows images of various existing spillway types adopted in earthfill and rockfill dams of Italy.

Figure 8.5: Ungated Spillway of Bilancino Dam (Italy)

Figure 8.6: Morning Glory of Nicoletti Dam (Italy)

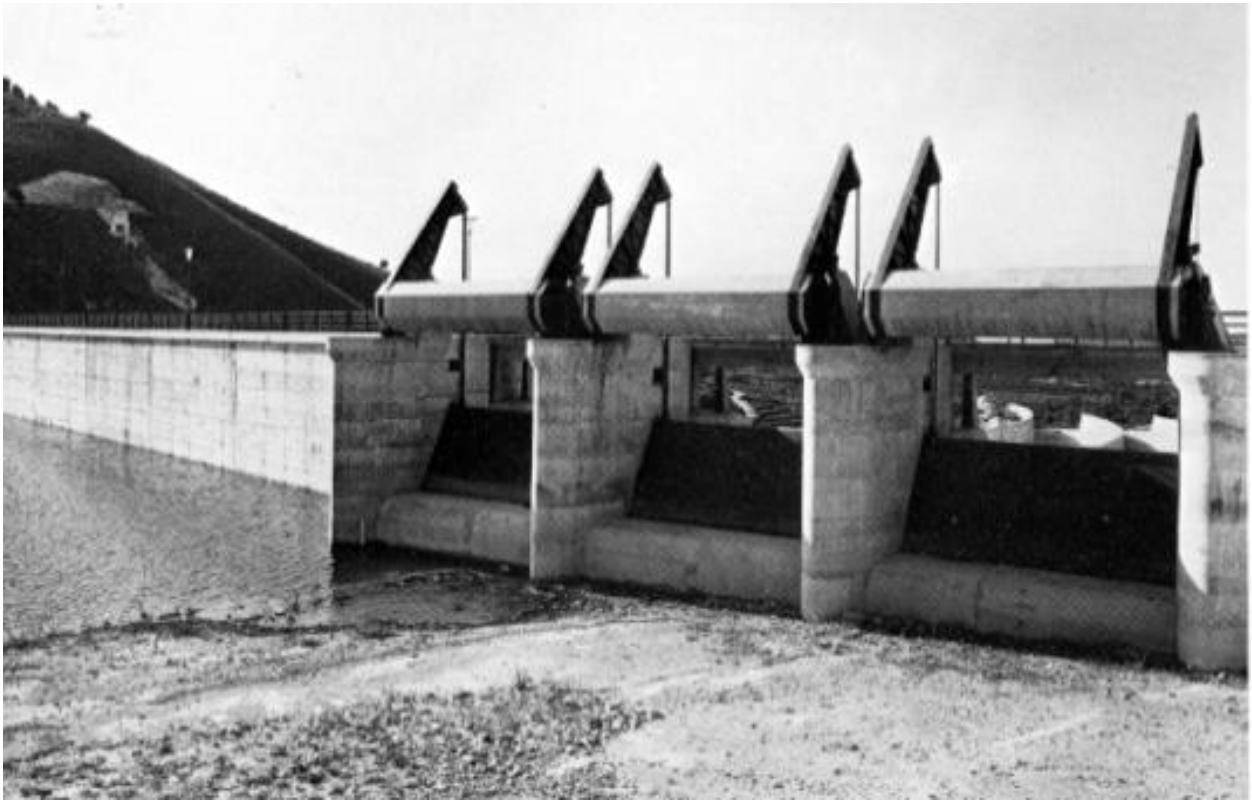


Figure 8.7: Gated Spillway – Ragoleto Dam (Italy)

8.3.2 Bottom Outlet

Outlet works regulate or release water impounded by a dam. It can release incoming flows at a retarded rate, as does a detention dam; it can divert incoming flows into canals or pipelines, as does a diversion dam; or it can release stored waters at rates dictated by downstream needs, by evacuation considerations, or by a combination of multiple-purpose requirements.

Outlet works are so named because they release out water from the reservoir. Some of these are equipped with an Intake Structure if the water is fed into a canal or a conduit for serving some specific purpose like meeting irrigation water requirement or hydropower generation, etc.

Outlet works structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation.

Outlet works may be described according to whether it consists of an open-channel or closed-conduit waterway, or whether the closed waterway is a conduit in cut-and-cover or in a tunnel. Outlet works may also be classified according to its hydraulic operation: whether it is gated or ungated or, for a closed conduit, whether it flows under pressure for part or all of its length or only as a free flow waterway.

Conduits and/or tunnels for embankment dams may have circular, rectangular, horseshoe, or oblong cross sections and their length is primarily determined by the base width of the embankment. Due to the greater length, it is usually more economical to construct a single large conduit than a number of small conduits. Conduits should be tunnelled through the abutment as far from the embankment as practicable or placed in an open cut through rock in the abutment or on the valley floor. Gates and/or valves in an intake tower in the reservoir, in a central control shaft in the abutment or embankment, or at the outlet portal are used to control the flow.

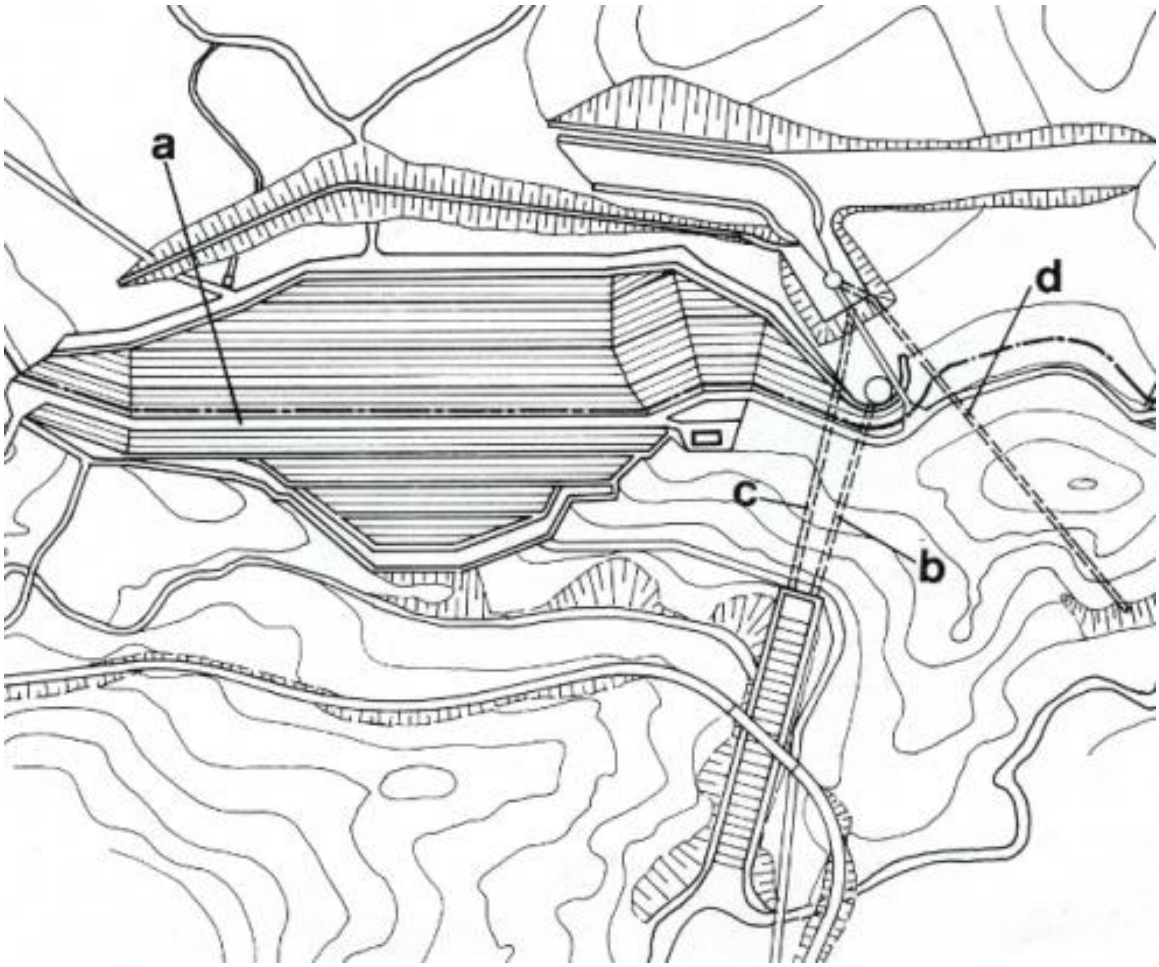


Figure 8.8: Outlet works of Scanzano Earthfill Zoned Dam (Italy) a) Main Dam; b) Spillway tunnel; c) Bottom discharge tunnel; d) diversion tunnel

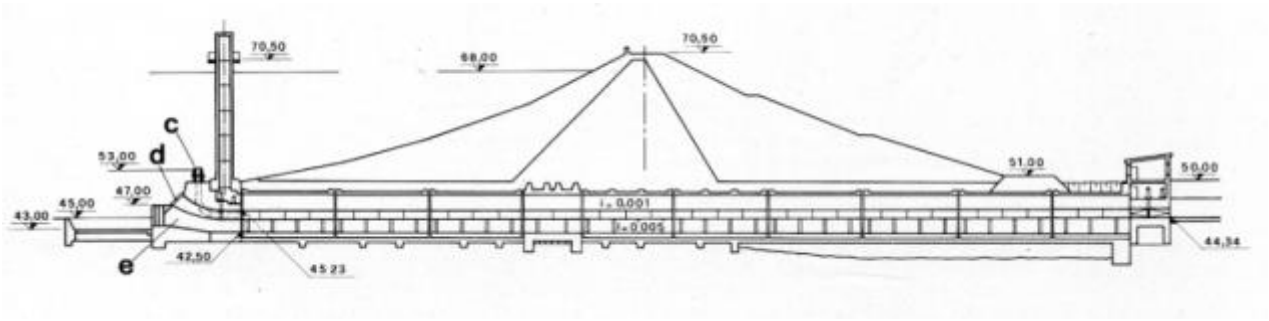


Figure 8.9: Outlet works of Trinità Earthfill Zoned Dam (Italy) c) Intake; d) Bottom discharge inlet; e) Diversion tunnel inlet

9 DRINKING WATER TREATMENT PLANT

9.1 INTRODUCTION AND WATER DEMAND

Appropriate design criteria and technical parameters have been used in the water supply aspects of the conceptual design for the Olooloitikosh-Kitengela-Kajiado Water and Sewerage Project. The Consultant has reviewed several documents and has also used their wide experience in the Water Supply and Sanitation Sector to ensure integration of local and international conditions, economics and engineering practices.

In order to come up with appropriate design criteria and technical parameters the following documents, among others, have been a reviewed:

- WHO Report No. 4 – Design and Selection Criteria for Communal Water Supply (1972)
- MWI Design Manual for Water Supply in Kenya (2005)

The estimated total water demand for the project is estimated to be 23,234 m^3/d, 30,588 m^3/d, 60,504 m^3/d, 101,117 m^3/d in the Present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Consumer	Component	Consumer Unit	Rate	Water Demand (m^3/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
Domestic (Urban)	High Class Housing	l/h/d	250	1.589	2.149	4.508	8.046
	Medium Class Housing	l/h/d	150	9.537	12.896	27.046	48.277
	Low Class Housing (IC)	l/h/d	75	429	580	1.217	2.172
	Low Class Housing (NC)	l/h/d	20	1.030	1.393	2.921	5.214
Domestic (Rural)	Medium Potential (IC)	l/h/d	50	42	54	205	654
	Medium Potential (NC)	l/h/d	15	113	147	246	294
Schools	Boarding Schools	l/h/d	50	1.195	1.628	3.229	5.881
	Day Schools with WC	l/h/d	25	658	889	1.727	3.082

Consumer	Component	Consumer Unit	Rate	Water Demand (m^3/day)			
				Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
	Day Schools without WC	l/h/d	5	15	20	38	68
Health Facility		l/bed/d	200	92	112	195	319
Hotels	High Class	l/bed/d	600	63	85	179	319
	Medium Class	l/bed/d	300	32	43	90	160
	Low Class	l/bed/d	50	4	5	11	18
	Unclassed	l/bed/d	50	112	152	318	568
Bars and Restaurants		l/d	500	500	76	102	214
Shops	Trading Centres	l/d	500	5	7	15	26
	Retail Traders	l/d	100	87	117	246	439
	Wholesale Traders	l/d	500	85	112	218	361
	Megastores	l/day	10000	25	34	70	123
Administrative Offices		l/h/d	25	25	60	81	169
Internal Demand in the Water Works (Backwashing of rapid sand filters)				823	1,112	2,320	4,131
SUB-TOTAL for drinking water treatment plant				**16.072**	**21.718**	**45.182**	**80.835**
Industries	Manufacturing	l/d	90,000	509	686	1,432	2,544
	Bakeries	l/d	40,000	8	11	24	42
	Jua Kali Artisans	l/d	100	4	5	10	19
Livestock Unit		l/h/d	50	691	934	2,073	3,313
Crop Irrigation				5,951	7,234	11,783	14,364
TOTAL				**23,234**	**30,588**	**60,504**	**101,117**

For the design it was decided to take into account the future water needs to realize a plant that is as modular as possible and able to manage through two autonomous process lines the current water requirement.

The maximum water demand considered for the sizing of the water treatment plant is (as shown in the previous table) 80.835 m^3/day which corresponds to 3.368 m^3/h and 0.94 m^3/s. From the data reported in the previous paragraph, this flow rate can only be reached downstream Kitengela

dam site, the design of the treatment plant described hereafter only take into account the foreseen maximum capacity.

However the consultant in order to have an idea of the different construction costs on the 5 different dam sites regarding the drinking water treatment plant developed separate analysis for each individual site. The feasibility study of the plants allows the consultant to identify the best design solution that permits, for future potential needs, the maximum treatment capacity. Each plant was designed with a double treatment line thus with the aim to have greater flexibility in use in relation to water demand and a reliable system to count on. The results of the preliminary analysis are summarized in the following table:

Treatment plant flow rate			
DAM SITE	**process lines**	**flow rate (mc/s) single line**	**Total flow rate (mc/s)**
Ereteti	2	0,28	0,57
Airfeld	2	0,11	0,21
Kitengela	2	0,47	0,94
Oloyankilani	2	0,02	0,04
NooPopong	2	0,06	0,12

9.2 WATER QUALITY

The treatment plant described will be fed with water coming from the artificial reservoir of Kitengela. The reservoir is the reference water resource for feeding the plant designed and therefore, for an in-depth study of the characteristics of the raw water to be carried out at the treatment plant, analysis campaigns carried out in the aforementioned territory have been taken into consideration.

9.2.1 Water Quality Standards and Regulatory Setting

In accordance with the Practice Manual for Water Supply Services in Kenya, the basic requirements for drinking water are:

- free from pathogenic (disease causing) organisms,

- containing no compounds that have an adverse acute or long-term effect on human health,
- fairly clear (i.e. low turbidity),
- not saline,
- containing no compounds that cause an offensive taste or smell,
- unable to cause corrosion or encrustation of the water supply system and not staining clothes washed it.

The specific standards for drinking water can be found in annex.

9.2.2 Existing Water Quality Data and Monitoring

Water samples were taken at the following locations within the project area:

- Main borehole tank site for Kajiado Township (1No.)
- Tank site for water coming from Nol Turesh (1No.)
- Kitengela (2 No.)

9.2.3 Water Quality Assessment

The samples were taken to The University of Nairobi for testing and the results of the analysis can be found in annex.

A summary of the results is listed below

- Both the results from samples taken from the tank sites within Kajiado Township were found to meet the standards for drinking water.
- Both results taken from Kitengela River were found to be acceptable for development as a domestic water source.

Of concern are the following parameters:

- True Colour
- Turbidity
- Iron
- Total Coliforms
- Total Faecal Coliforms
- Suspended Solids

- Hardness
- Fluoride levels
- Heavy metals presence

9.2.4 Anticipated Changes to the Physio-Chemical Character on Water Quality

The expected changes to the water quality results are as follows:

Parameter	'Flowing' Test Result	'Still' Test Result	Standard	Change Required - Flowing	Change Required - Still
True Colour (TCU)	70	80	15	55	65
Turbidity (NTU)	360	370	5	355	365
Iron (mg/l)	0.8	1.2	0.3	0.5	0.9
Total Coliforms/100ml	260	460	0	260	460
Total Feacal Coliforms/100ml	5	10	0	5	10
Suspended Solids (mg/l)	200	300	0	200	300

9.3 TREATMENT CYCLE

The interventions needed to build up the new water treatment plant is described in this report with regards to the aspects most closely related to the Water treatment process, from the Raw Water adduction Channel to the last stage of the final treatment. The plant designed consists of two independent modules to cope with current and future needs.

The treatment cycle consists of the following phases:

9.3.1 Water line

i. Duct and fine Grilling
ii. Equalization Chamber and Primary roughing
iii. Clariflocculation with single basin system "Accelerator"
iv. Quick filtration on sand beds
v. Final disinfection by sodium hypochlorite
vi. Treated water storage
vii. Backwashing of filters
viii. Accumulation of backwash water

9.3.2 Sludge Line

i. Thickening Sludge Treatment
ii. Chemical conditioning
iii. Mechanized dehydration
iv. Drying beds

9.4 WATER LINE

9.4.1 Duct and fine Grilling

The outlet will consist of an artificial dam and therefore, it requires a special downstream grate that prevents the entry of coarse material in plant, with a redundant double adduction channel.

Surface water contains fishes and debris which may clog or damage the pumps, occlude the piping and cause problems in the treatment plant. They can also contain high concentrations of suspended sediments. Grilling is necessary to avoid solid bodies of different dimensions, both suspended and floating from entering the clarification system.

The grid is normally a metal bar with free light from 15 to 65 mm.

The objective of reducing manual cleaning interventions increasingly requires the automation of grilling. This is inevitable when there is a risk of the sudden arrival of plant material that tends to clump on the grate and quickly cause complete occlusion of the water intake. To remove the material that is too small to be retained by the bar racks, the use of the fine grids is required.

The grilling system under consideration foresees the use of a vertical hydraulic grid with the aim of intercepting the solid debris even in the presence of deep adduction units. Its debris removal capacity is extremely high and makes it suitable for heavy duty.

The equipment consists of a structure of guide and containment with bunks, a fixed bar grille, an oscillating wiper comb, a comb cleaning system, a grating mouth, a pair of comb transport chains, moto-electric gearbox, reverse motion switch, local Control Panel for manual and automatic operation. According to the Standard, the grid is made of carbon steel, for which a surface protective treatment of hot galvanizing is foreseen.

9.4.1.1 Adduction Channel Sizing

For the sizing of the adduction channel we consider a speed water flow in arrival of V = 1 m/s. Assuming the input value Q max, equal to 0.94 m^3/s, you will find the useful section A for the channel:

$$A = Q / V = 0.94 \text{ m2}$$

Once the section area of the channel is defined, height and width values can be assigned

: H = 1,2 m L = 1,2 m

Channel Slope: J = i = Q / (Kst * A * (R^ 2/3))^2

where:

Kst = Coefficient by roughness by Strickler Equal 70

A = Duct section

R= Q / A

It is obtained: : i = 0,000880

9.4.1.2 Sizing Fine Grilling

The following dimensional parameters are considered for sizing:

5) Spacing between bars S = 0.020 m
6) Thickness B ' = 0.8 cm

The number of bars is thus identified:

$$(n + 1) * s = b \quad => \quad n = (B - s) / s = = 59 \text{ bars}$$

from which the useful width of the channel where the grilling system is placed can be obtained:

$$B' = (n + 1) * S + n * b' = 1,67 \text{ m.}$$

To ensure an efficient and redundant guarantee of functionality, No. 2 channels of addition and grilling are envisaged in parallel.

9.4.1.3 Flow measurement

The dosage of chemical products in the treatment process is proportional to the flow of the incoming raw water. It is therefore envisaged to use a flow meter for pressure piping with flow measurement for differential pressure of the Venturi type. From the selected flow meter the signal is transmitted to the control center of the treatment plant.

9.4.1.4 Equalizing chamber and Roughing Primary

Downstream of mechanical pre-treatment an equalization tank for the correction of the pH is expected together with the dosage of reagents needed for the process and roughing of the sedimentable material.

The correct pH values can obtain the maximum elimination of the turbidity of water from the Flocculants through destabilization, complexation and absorption on hydroxides metal. In the subsequent destabilization unit the pH of the raw water is modified because of the addition of inorganic flocculants.

The optimum pH control for destabilization through the addition of acid or base will be done automatically, if enabled by the operator by means of a pH probe, placed immediately downstream of that unit. The pH of the raw water arriving at the plant should be between 7.5 and 10.0 units. The optimization of the pH and the maintenance of the ideal value will be done with the addition of soda in the water before the destabilization.

In the whole treatment of drinking water, particular importance takes on the processes of destabilization and aggregation of colloidal particles.

In fact, the task of removing suspended solids is assigned to these processes, solids that are the main responsible for turbidity (particles of diameter 0.1- 0 micron) and color (particle diameter 50 – 10,000 Angstrom).

The optimization of this removal leads to a contemporary appreciable reduction in some pollutants, taste, unpleasant odors, bacterial load and microalga.. Destabilization and aggregation are two processes that occur necessarily in the following times and in different conditions.

The destabilization, in fact, enhanced through a very thorough thrust agitation, causes an intimate contact between the colloidal particles present in the water and the reagent and the consequent neutralization of the superficial electrostatic charges of the colloidal particles. The annulment of these repulsion forces makes possible their subsequent agglomeration In the Flocculation and separation phase.

The aggregation, favoured by a slow agitation, works the rapprochement of the particles destabilized to allow them to:

- To reunite in larger aggregates,
- To be adsorbed on flocs consisting of hydroxides or polymers of electrolytes added in the water.

Adsorption of the insoluble hydroxide clots on the same particles, and consequently the subsequent separation from the water.

As flocculant agent, the polychloride of aluminum (pac) will be used. The optimum pH range for the use of PAC is between 6 and 9.

In any case, it should be kept in mind that the same equipment used for aluminum polychloride, are suitable for the possible use of other liquid flocculants.

At the head of the mixing tank, the portion of the water from the backwash of the filters can be recirculated.

The reagent mixing phase is placed in reinforced concrete tanks, equipped with mechanical agitators and cranes for the elimination of sediments.

9.4.1.5 Tank Sizing

It is envisaged the creation of a double line of tanks for which it is estimated a time of detention (TD) within each single tank of 50 min. This will involve:

$$V=Q*td= 2.806\ m^3$$

To determine the height of the tanks it is expected, as a first approximation, the use of only one bathtub. It is then calculated the surface, imposing a fixed value of the surface hydraulic load (ratio between the flow rate and the surface of the tank) :

$$S = Q / Ci \qquad \text{where } Ci = 3\ m^3/m^2{*}h.$$

Located (h) as the ratio between the previously calculated volume and the ideal surface S:

$$S = 1.122\ m^2 \qquad \text{Height } h = 2.5\ m$$

Knowing H the size of the tanks can be derived taking into account that the tanks will equal and that the L/B ratio must be between 3 and 4.

No. 2 tanks of the following dimensions have been adopted:

- L = 45 m
- B = 13 m
- h = 2.5 m
- (L / B = 3,46)

Check of holding times and hydraulic loads:

- V = B * L * h * 2 = 2.295 m3
- td = V / Q = 52.1 min
- Ci = Q / (2 * B * L) = 2,90 $m^3/m^2{*}h$

9.4.2 Clariflocculation

For this phase of treatment, a clariflocculator with single basin tanks "Accelator" type will be adopted. In this system the suspension is recirculated in a primary reaction zone, where it mixes with the raw water to be treated and the reagents. The suspension then passes into a secondary reaction zone and finally into the concentration zone, and from here still to the primary.

The purge of the mud takes place by closing the preplaced interception organs, which prevent the return of the mud from the concentration zone to the one of reaction. The mud instead of recirculating, it is deposited at the bottom of the hopper from where, opening of the drain valves comes out by gravity towards a well of collection. Clarified waters are collected superiorly with radial ducts.

Features:

1) Single tank coupling of three functions (coagulation, flocculation, sedimentation)

2) Formation of an intermediate layer of mud ("bed of sludge"), consisting of SS + coagulants, which exerts a filter function for the smaller particles
3) Hydraulic Load: Ci = 2.5 – 3 m/hr
4) Hydraulic detention Time T = 1.00-1.5 hours
5) Mud recirculation

Advantages:

- Flocculation + better sedimentation, due to the presence of large flakes
- Better adaptability to change in flow rate
- Better use of reagents
- Smaller size
- More global clarification yields

9.4.2.1 Tanks Sizing

The number of tubs n = 2, have been fixed after checking the operating conditions.

The sizing has been carried out on the basis of the following parameters:

- Hydraulic detention Time TD = 1.2 h
- Sedimentation volume (Ws)/Total Volume (Wt) = 2/3

A hydraulic load of 5 m3/m2 * h is considered and, moreover, that the tank is cylindrical. The total volume of the tank is calculated:

- Q' = Q / n = 1.684 m^3/h
- Wt = Q * td = 2.021 m^3

The volume of the sedimentation compartment is equal to 2/3 of the total amount and then:

$$Ws = 2/3 * Wt = 1.347\ m^3$$

The volume destined for the mixing and flocculation phases is obtained by the difference:

$$Wt - Ws = 674\ m^3$$

To calculate the surface of the sedimentation compartment sI divide the flow rate by the velocity of the stream:

$$Ss = Q / Ci = 337\ m^2$$

S(i) determines the height h as the ratio between the volume of the sedimentation compartment Ws and the respective base surface Ss:

$$h = Ws / Ss = 4\ m > 3.5\ m$$

This height allows to respect an appropriate freeboard between the level of the mud deposited at the bottom of the tank (approximately ½ of the total height) and the bottom of the channel which collects the clarified water. The recommended minimum value of that freeboard (1.5 m) is abundantly exceeded.

The characteristics of the designed tanks are:

- Larger diameter: D = 30 m
- Smaller diameter: D = 21 M
- Height: h = 4 m
- Total sediment volume: 1,375 M3
- Total Surface: 706.5 Sqm
- Total Sediment Surface: 471 Sqm
- Total Volume. : 2,062 M3
- Two tanks will be used

Checks: Q = 3,368 m3/h

- n = 2 tanks
- St = 1,413 m2
- Ss = 942 m2
- Wt = 4,125 m3
- Ws = 2,750 m3
- td = 1.2 h = 1h 12 min

9.4.3 Rapid gravity filtration

Settled supernatant water from the settling tanks will be conveyed through a pipeline to the rapid gravity sand beds. Filters will be cleaned by backwashing with water only.

The Consultant will propose the use of sand filter media with an effective size and uniformity coefficient that are readily available locally.

Wash-water will be drawn from elevated water tanks. A butterfly valve along the wash-water pipeline will be used to control the backwash water flow and dissipate the excess head. A flow meter with flow-rate indicators located strategically inside the filter gallery shall be used to ensure that the required backwash rate is maintained.

All filter controls are housed in the filter gallery adjoining the filters. The pipe-work layout in the gallery has been arranged to facilitate ease of operations.

A sight glass will be fixed on the wash-water outlet pipe at each filter to enable the operator to see when the filter is clean and thus terminate backwashing operations.

The filtered water outlet will be fixed at a higher level than the top of the filter media in order to maintain a minimum water depth on the sand, to avoid air binding caused by complete draining of the sand.

Wash Water Tank

Elevated steel panel tanks will provide enough storage for the washing of the filters. The tanks shall be filled up with wash water pumps located in the pump house next to the clear water reservoir.

The wash water pipeline will be sized with flow capacity to backwash one filter bed at a time.

9.4.3.1 Sizing

The rapid filtration phase is dimensioned according to the flow rate: Q = 3.368 m^3/h.

The quick filters have been proportioned considering a filtration speed v = 8,50 m/h obtaining a surface equal to:

$$S = Q / v = 396,25 \ m^2$$

Are made 6 Filters from 10* 10 sqm and H = height 3.6 m.

The actual filtration speed will be:

$$v = Q / (6 * 10 * 10) = 5.6 \text{ m/h}$$

Check the operation during the washing phase during which they are in use 4 Filters on 6.

$$v = Q / (4 * 10 * 10) = 8.4 \text{ m/h}$$

9.4.3.2 Washing fase

The washing of the filters is carried out with the European system with air and water with one washing per filter every 12 h. Consequently there is a total number of daily washes equal to:

Num. Washes per day = 2 * 6 = 12

With intervals between washing and the other:

$$Dt = 24 \text{ h} / 12 \text{ washing cycle} = 2 \text{ h}$$

The wash is carried out for a time of about 8 ', of which about 5 ' for the water and 3 ' for the air. The speed of the washing water is 25 m /h

- Washing capacity of a filter Ql = v * Sf = 2.500 m3/h = 0,69 m3 / s
- Washing water volume of a filter Wl = Ql * tlav = 208 m3

Insert In a tank a volume equal to twice the volume of washing:

$$Wa = 2 * Wl = 416 \text{ m3}$$

There will be a wash every hour, therefore, in this time interval, the filter will have to be filled with a volume of washing water (a volume is held in the tank for a possible emergency).

$$Q \text{ fill.} = 104 \text{ m3} / h = 0.028935 \text{ m3} / s.$$

So we use a power pump:

$$P = 9.81 * Q * H / \text{Performance rate} = 5,3 \text{ kW}$$

A similar reserve pump is foreseen

The air demand for each filter is:

- A = 60 m3 / m2 * h
- tl Air = 3 ' = 180 s
- Ql Air = A * Sf = 6.000 m3 / h = 1,67 m3 / s
- W Air = Ql Air * tl Air = 300 m3.

9.4.4 Disinfection

The final disinfection is intended to guarantee a minimum coverage of disinfectant agent that prevent unwanted bacterial growth in the network of adduction and distribution of treated water. The expected dosage is 1 ppm. The disinfectant agent will be dosed directly to the entrance into the tub.

The assay is automatically adjusted on the basis of the measured input flow to the system and Integrated on the basis of the residual chlorine measurement performed on the outlet manifold from the tanks.

9.4.4.1 Sizing

It is done by chlorination in the contact tank:

$$Cl_2 + H_2O = HClO + HCl$$

$$HClO = HCl + O$$

The bactericidal action comes from the oxidizing action, from the toxic to the bacterial cell, From the inhibitor of enzymatic processes. The reactions are influenced by the PH, the temperature, the contact time, the presence of organic substances. The addition of disinfectant occurs upstream of the proportioned contact tank for a holding time of TD = 30 min. This gives rise to a slow mixing of the slurry by virtue of its typical form with serpentine.

The total volume will be

- W = Q * td = 1,684 m3

It is assumed a residual chlorine concentration of D = 5 mg/L. Disinfectant used NaCl to 12% Cl.

The Clorine required is:

- Clreq. = D * Q = 16.841 g/h

Amount of NaCl at 12% to be Insert in an hour:

- NaClO12% = (Kg/h)Cl / 0.12 = 208 l/h = 0.14 m3/h

Heigth Channel of 3.5 m ----- Stot = V / h 241 m2

Considering that the length is recommended to be four times the width it results :

- Width = 8 m
- Length.= 4 * 8 = 32 m

The volume of the storage tank is now defined:

Vtot. = [NaClO]12% * 24 * 7 = 23.6 m3

9.5 SLUDGE LINE

For the treatment of the sludge, a line of process a technical stabilization has been adopted, the treatment to which the sludge is subjected makes themselves suitable for their subsequent mechanical dehydration, without this entailing, in normal working times, further degradation processes and the consequent occurrence of undesirable phenomena.

Consider a sludge production equal to 30 grSS/m3.

Qs =80.835 m3 / d ==> 80.835 * 0.03 = 2.425 kgSS/g (Q solid)

Assuming a humidity value of 99%, the volumetric flow rate will be obtained by dividing the solid flow rate by 0.01 and multiplying it by the density (assumed equal to the water one).

Volumetric flow = Qs / 0.01 = 243 m3 / g

9.5.1 Thickening

The sludge in excess, before starting the dehydration, is subjected to thickening, in order to reduce its volume and humidity.

For this phase, a double thickener is foreseen by mean of two circular baths with comb and

scraper bridge. The waters in excess are returned to the head of the plant.

With this system is obtained not only the safety to induce to dehydration a mud with characteristics of humidity almost constant over time, but also the possibility to make independent the methods of entry to the treatment.

9.5.1.1 Sizing

Assuming a load Cs = 2.5 KgSS/M2 * H = 60 KgSS/sqm * g it can be got a useful surface:

Stot = (KgSS/g) / Cs = 40 m2

$S_{1/2}$ vasca = (KgSS/g) / Cs = 20 m2

Fixed height h = 3 m The volume for a single tank will be:

$V_{1/2}$ tank = 60 m3

A double tank is expected, to be used.

- Diameter d = 4.5 m ,
- total area S = 15.9 m2,
- Height H = 3 m and
- volume V = 71.5 M3.

Checks: Cs = 76,3 kgSS/m2 * g td = 12 h '

It is expected a performance of thickeners such as to bring the sludge out from a humidity of 99% to 98%, consequently the volume of sludge output per unit of time is obtained from the report:

Qu / Qi = (100 - 99) / (100 - 98) = ·½

The quantity of sludge to be disposed of in subsequent treatments is therefore equal to half of the sludge produced:

Qu = 121 m3 / g

Therefore, a water flow of mud equal to half the flow of mud produced (in turn equal to Qu) is sent to the plant head.

Chemical conditioning

The chemical conditioning of the mud is achieved by the addition of Lime Ca (OH), the consumption of the expected reagent is equal to 160 Kg TSS for a total of 388 Kg/d

Tank Conditioning

Foreseeing that the sludge dehydration plant works for only 8 hours a day, the flow rate to be disposed off is:

$$Qu / 8 = 15,2 \text{ m3 / h}$$

Assuming to have a mixing time of 15 ', it will results:

$$V = 3.8 \text{ m3}$$

9.5.2 Dehydration

Dehydration is entrusted to a system of mechanical type, the press filters. The press filter is an industrial machine used to dehydrate sludge through liquid solid separation processes, It is a system of volumetric reduction of liquid substances that present inside suspended solids.

9.5.2.1 Sizing

The daily incoming volume is equal to:

$$V = Qu * 1 = 121 \text{ m3}$$

Assuming that the filters only work five days a week, and by providing 2 daily operating cycles the actual volume is equal to:

$$Veff = (7 * V) / (5 * 2) = 85 \text{ m3}$$

The parameter by which the sizing is used is the capacity of the single room that is posed equal to 3.75 m3:

$$n = Veff / 3,75 = 23$$

We consider therefore a number of rooms equal to 11 .

Assuming a moisture content at the end of the treatment equal to 65%, the daily sludge flow is given by the report:

$$Qf / Qe = (100 - 98) / (100 - 65) = 0.057$$

Where Qf is the output flow at the end of dehydration, and Qe is the daily input volume in the press filters, equal to 2 * vrms (since the daily cycles are 2). At the head of the system is sent a flow of mud water Qr = Qe-Qf. The following values are obtained:

Qf = 19 m3 / g Qr = 320 m3/g

9.5.3 Drying beds

Drying beds are the most common (but also less effective) method for dehydration of sludge and represent the final treatment of the sludge line. They come to this place to cope with eventual out of service of the mechanized system (filter).

The digested sludge-from the bottom of the stabilization tanks are rich in water: to make cheaper and easier their disposal it is necessary to reduce the volume by cutting down the level of Humidity.

To dry the sludge, these are conveyed to the Two Concrete tanks of rectangular shape with very low walls, of the Total area of 364 sqm. The bottom of each element is fitted with sloping and drained.

On the bottom is foreseen a drainage layer made up of layers of coarse gravel or rubble, on which a second layer of material is extended and finally a layer of sand of about 5 cm. In Total the thickness of the 3 layers will be about 25 cm. The height of the walls of the tubs is about 30 cm above the sand.

The digested sludge is placed on the beds, for a uniform height of about 20 – 30 cm, where it is allowed to dry naturally for drainage and natural evaporation.

A drawback of the drying beds is the fact that the rain returns to the mud a good part of the lost water during the drying; it is possible to remedy by covering-like a greenhouse-the beds with transparent covers, which let pass the solar heat, thus it is possible to increase by 50% the yield of the beds. In this case it is necessary to cure the ventilation.

The removal of dried sludge (with concentration of solids between 50-70%) is carried out manually or by small mechanical means.

9.5.4 Administration Building

The administration building is designed to house an office, laboratory, store, workshop and a washroom. This is a type drawing.

9.5.5 Workshop and Filter Sand Storage

A workshop and filter sand storage facility has been provided consisting of the following provisions:-

1) Filter sand washing area
2) Sand drying bay
3) Clean sand storage bay
4) Workshop area
5) Lockable room to store spare parts and tools

9.5.6 Staff Houses

Double standard Ministry of Works and Housing Grade 9 type houses will be provided for the staff operating the works at the treatment works site. The houses are supplied with domestic water, power and foul drainage system.

9.5.7 Domestic Water Supply

Domestic water is supplied through domestic water pumps located at the pump house next to the clear water tank. Water is pumped to the elevated wash water tanks.

9.5.8 Site Drainage

Washouts and overflows from the treated water reservoir will be collected into the site drainage system discharging back into the river.

Foul water from the administration building and staff houses will be channeled into a separate system discharging into a septic tank and soak away pit.

All site storm water will be collected through lined open drains and discharged into the site drainage system for onward transmission to the natural drainage channels.

Sludge from the flocculation chambers and sedimentation tanks, and filter backwash water will be collected and handle by the water services provider separately.

9.5.9 Power Supply

Electrical power already exists in the proximity of the treatment work sites. However it will be needed to regulate it with transformers for use at the sites such that all power requirements will be tapped from the existing supply lines. Solar energy option will be also explored.

9.5.10 Site Security

Security at both the intake and treatment work will be addressed by perimeter fence and lockable gates. The fence will be made of pre-cast fencing posts, 2m high with barbed wire. A gate house will be located at the entrance of the sites. Street lighting and flood lighting will be provided on site to assist during night operations, and also to enhance night security.

10 WATER SUPPLY NETWORK

The hydraulic calculations have been elaborated considering the maximum flow rates related to the foreseen scenario at the year 2042, corresponding to the maximum design flow of m3/d 101,117. Operation conditions at the present conditions were also verified.

In both scenarios, the calculations confirmed that the pressure does not exceed the threshold limit value of 30 ATM, equal to the nominal pressure guaranteed by the chosen piping material. The flow speed, in any operational condition does not exceed the limit value.

The following elevations have been considered for the main towns to be included as scope of this project:

- Kitengela Town elevation 1,570.00 m.a.s.l.
- Isinya Town elevation 1,650.00 m.a.s.l.
- Kajado Town elevation 1,730.00 m.a.s.l.

The following elevations have been considered for each dam site:

- Ereteti Dam, elevation 1,682.00 m a.s.l.
- Ololotokish Airfield Dam, elevation 1,696.00 m a.s.l.
- Kitengela Dam, elevation 1,630.00 m a.s.l.
- NOOPOPONG River, elevation 1,570.00 m a.s.l.
- Oloyonkilani Dam, elevation 1,800.00 m a.s.l.

10.1 PUMPING STATION

The hydraulic calculation was carried out to size the total pumping energy needed for each configuration and the volume of the storage tank of the lifting system, as well as the diameter of the relative water to the final delivery (Kitengela Town-Kajiado Town).

The sizing of the above-mentioned works was made, considering the maximum water flow (under operational conditions) coming from the dam, downstream the treatment plant:

Maximum Flow	m3/d 101,117	m3/s 1.17	l/sec 1,170

The design material for the hydraulic network is HDPE with variable diameter, identified by the formula of Bresse D = 1.50 $\sqrt{q}$ and then verified with the hydraulic calculation, although it will be

possible in the next phase to choose a different one according to different information that could come from the investigation and the deepening of the hydraulic calculation.

For the determination of the Head Loss, Darcy – Weisbach formula has been applied. This formula is generally adopted for ductile iron piping. The formula is shown below:

$$\mathbf{J} = \frac{\lambda}{\mathbf{D}} \mathbf{x} \frac{\mathbf{V}^2}{2g}$$

Where:

- λ = Friction factor
- D = Diameter in m
- v = Average flow speed in M/sec
- g = Coefficient of gravity acceleration.

The Friction factor was calculated with the Colebrook – White formula:

$$\frac{1}{\sqrt{\lambda}} = -2\lg\left(\frac{2.51}{Re\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right)$$

of which an approximate version was used:

$$\lambda = \frac{1}{4}\left(\lg 3.71\frac{D}{\varepsilon}\right)^{-2}\left(1 + \frac{8}{Re\,\varepsilon/D}\right)$$

Where:

- Re = Reynolds number;
- ε = roughness equivalent;
- *D*= Inner diameter of the pipe
- ε/*D* = Relative roughness

For the coefficient of roughness, a value of 0.02, corresponding to slightly used piping, has been prudently inserted in the calculation,

The conditions of the flow type in conduct according to the maximum speed in an operational condition were studied. The speed value is derived from the relationship

$$Q = AxV \rightarrow V = \frac{Q}{A}$$

Where

- Q *(max Flow)* = m3/sec 1.17
- A (Piping area)

The speed is equal to 1.00 m/sec. This value is acceptable because the water flow under these conditions, cannot cause abrasion phenomena on the wall of the pipes. However, lesser values of velocity, corresponding to lower flow rates, are neglected, because no deposits can occur in the absence of suspended materials.

For each location, considering the different altitude, we proceed to calculate the electric power necessary to lift the flow.

The calculations carried out are hereafter developed:

$$P = 9,81 \times \frac{\gamma}{1000} \times \frac{Q \times P}{1000 \times R} = \frac{Q \times P \times 9,81}{1000 \times R} = [Kw]$$

It has been verified of the performance curves of the existing pumps in the market, to select a set of pumps in rotation, each pump with the same designed nominal electrical power.

The operation of the electric pumps will be regulated by using inverters-controlled motors, managed by a central PLC and this will allow a progressive start of the electric pumps, according to the requirements of the water supply, thus obtaining significant energy savings.

10.1.1 Results

DAM	Kitengela	Ereteti	Airfield	NOOPOPONG	Oloyonkilani
First trunk from drinking water treatment plant to disconnection tank design on the Kitengela-Kajado trunk	9,10	18,423	18,423	9,85	15,359

Kitengela-Kajado trunk (Km) Diam =700	39,44	39,42	39,42	39,44	39,44
Total head (geodetic + continuous head loss)	320,45	247,20	233,20	352,30	-84,18
Total power needed (Kw)	8,376	4,815	4,542	6,862	-----

10.2 DISTRIBUTION PIPELINES

These will be designed as follows:

- Pipes will be designed on a peak hourly factor of 1
- The pipe materials will be as follows:
  - GI or Steel for pipes 250 mm diameters or greater
  - HDPE for pipes 250mm or smaller
- Friction coefficient to be used in the Hazen-Williams equation will be as follows:
  - GI or steel lined pipes = 120
  - HDPE = 150
- Maximum pressure in the system = 60 m
- Minimum pressure in the system will be 5 m
- Velocity in the pipes will be limited as follows:
  - 50 –100 mm dia. = 0.6-1.0 m/s
  - 150 - 250 mm dia. = 1.0-1.5 m/s
  - 300 – 600 mm dia = 1.2 –2.0 m/s
- Minimum cover to the crown of pipes = 0.6 m

11 PROPOSED SEWERAGE PROJECT

11.1 DESIGN REFERENCES

Appropriate design references and technical parameters have been used in this sewerage project feasibility study. The Consultant has reviewed several documents and has also used his wide experience in the sector to ensure integration of local and international conditions and engineering practices.

The Feasibility Study Design of the sewerage system has been carried out on the basis of the following design Codes and Standards:

- Ministry of Water and Irrigation – Practice Manual for Water Supply services in Kenya, 2005.
- BS EN 1916 Concrete Pipes and Fittings, Unreinforced, Steel Fibre and Reinforced, BS EN 1917 Concrete Manholes and Inspection Chambers, Unreinforced Steel Fibre and Reinforced. However, the above Eurocodes are still used in parallel with BS 5911: Part 100:1988 – Specification for Unreinforced and Reinforced Pipes and Fittings with Flexible Joints, as some parts of BS 5911 have not been covered by an equivalent BS EN.
- BS 8005: Part 1:1987 Guide to New Sewer Construction.
- KS 06-217 – Specification for uPVC Pipes for Buried Drains and Sewer Pipes.
- WHO Report No. 9 – Selection and Design Criteria for Sewerage Projects.
- Nairobi City Council Adoptive Standards for Infrastructure Works.
- MOPW&H Manual for Civil Works Details – 1983
- Sewage Treatment in Hot Climates, D. Mara, 1975
- Waste Stabilization Ponds – A Design Manual for Eastern Africa, D. Mara et al, 1992
- Environmental Management and Co-ordination Act of 1999 – National Environment Management Authority (NEMA)

11.2 DESIGN CRITERIA

11.2.1 Type of Sewerage System

A separate sewerage system has been designed that is not expected to take any direct storm water.

11.2.2 Wastewater Components

The sewerage collection system has been designed to carry wastewater mainly composed of:

- Sewage from domestic (residential) establishments,
- Sewage from commercial establishments,
- Sewage from institutional establishments,
- Industrial wastewater flows,
- Infiltration Inflow (I/I) to sewer lines and manholes.

11.2.3 Sewerage Network

11.2.3.1 Sewer Pipes Materials

The sewer pipe materials have been selected taking into consideration availability, durability and cost.

Circular pipes have been adopted in this project.

uPVC pipes have been recommended for sewers with bore less or equal to 300mm. Un-plasticised PVC pipes are manufactured in Kenya in metric sizes up to 450mm diameter. A number of firms are involved in production, including Polypipes Ltd and Eslon.

GRP pipes have been recommended for sewers with bore between 375 to 450mm. Glass fibre reinforced polyester pipes are imported and sizes range between nominal diameters 250mm to 4,000mm. They are lighter, corrosion resistant and are much easier to install when compared to Steel and Concrete pipes. They however have considerably higher costs associated with importation.

Precast Concrete pipes have been adopted for sewers greater than 450mm lines. Spun Concrete Pipes to relevant Kenyan and British Standards are manufactured locally by a number of Companies. They form the most common choice for Sewer Pipes. Spigot and socket jointed pipes are manufactured in sizes ranging from 100mm to 915mm diameter, and ogee jointed pipes are available in sizes from 100mm to 1525 mm diameter. Rubber ring, flexibly jointed pipes are manufactured in sizes ranging from 150mm to 975mm diameter and strength classes M and H, or reinforced. Pipes are vertically cast in vibrated moulds and supplied and supplied moulds and supplied with rubber rings. Rigid jointed pipes require jointing with tarred hessian and cement mortar.

Steel pipes have been recommended for all river aerial crossings, all tarmacked road crossings, rail crossing and very deep sewers. Steel pipes are manufactured locally. They are generally used

in locations where sewers are exposed or in situations such as river crossings. Protection against corrosion is required internally and externally. Bitumen sheathing is normally used, the external sheathing generally being reinforced with glass fibre windings.

Different pipe bedding types have been used depending on the nature of the ground. These have been classified as bedding type A, B, C, and D.

11.2.3.2 Hydraulic Design of Sewers

Hydraulic design of the sewers for the Project has been based on the Manning Equation with n value of 0.013. The following expressions have been used;

$$Q = \frac{A}{n}\left(\frac{A}{P}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where;

- Q = The wastewater flow rate (m3/s)
- A = The cross-sectional area of flow, m2
- n = Manning's coefficient
- P = Wetted perimeter (m)
- S = Sewer slope (mm/m)

In order to minimize the deposition of solid matter in sewers and the corrosion of sewers and manholes resulting from the evolution of hydrogen sulphide, a minimum velocity of flow of 0.75m/s have been provided in the sewers to be achieved.

To avoid excessive abrasion from grit and sediments, peak velocities have been limited to a maximum of 3.0m/s at peak flow.

11.2.3.3 Peak Factors (FR) and Peak Flows (QR)

The following Peak Factors as proposed in the Nairobi Masterplan for sewers, sanitation and drainage (1997) report have been adopted.

Table 11-1: Peak Factors

Average Flows	Peak Factor

Less than 500m^3/day	6.9
1000m^3/day	6.2
5000m^3/day	5.1
10,000m^3/day	4.6
50,000m^3/day	3.4
More than 100,000m^3/day	2.7

The peak flow rate (QR) has been calculated using the following equation:

$$QR = FR\ (DWF - INF) + INF$$

Where;

- FR is the peak flow factor
- DWF is the Dry Weather Flow (l/s)
- INF is the Total Infiltration Rate (l/s)

11.2.3.4 Infiltration Inflow (I/I)

This study has adopted a 15% allowance for unauthorized but unavoidable entry.

11.2.3.5 Minimum Size of Sewers

In order to reduce the risk of blockages and to simplify maintenance, the minimum size of all sewers has been set to be 300mm diameter.

11.2.3.6 Depth of Sewers

The depths of sewers have been provided to be sufficient to take the gravity flows of sewage from premises. Maximum economic sewer depths have been used depending on the ground conditions, depth of the water table, local topography, size of the sewer and its proximity to buildings.

The normal minimum depth to the crown of sewers has been maintained at 1m. Where it‘s less than 1m, then haunching of pipes using concrete surround has been used.

11.2.3.7 Pipe Gradients

The following sewer pipe gradients have been used in this study;

- Minimum slope of sewer = 0.2% (1:500)
- Absolute minimum slope of sewer = 0.1% (1:1000)
- Minimum slope of lateral connection = 3% (1:33)

11.2.3.8 Manhole Spacing and Manhole Diameter

Manhole sizes and spacing are generally in accordance with NCC standards. Manholes are provided on sewers at all changes of direction, sewer level, at every junction and throughout the systems at intervals sufficiently close to simplify sewer cleaning.

The maximum distances at which manholes should be spaced and the minimum sizes of manholes are given in Table 11-2 and Table 11-3 respectively.

Table 11-2: Maximum Spacing of Manholes

Sewer Pipe Diameter (mm)	Maximum Spacing (m)
450 or less	60
525 – 825	90
900 and above	120

Table 11-3: Minimum Sizes of Manholes

Diameter of Largest Sewer (mm)	Diameter of Manhole Rings (mm)
Up to 300	1050
375 to 600	1200
675 to 900	1500

11.2.4 Waste Water Treatment Plants

11.2.4.1 General

The following sewage treatment methods have been found appropriate for use in Juja and Thika South Project area;

- Waste Stabilisation Ponds
- Aerated Lagoons
- Oxidation Ditches
- Percolating/Trickling Filters
- Activated Sludge Process
- Upflow Anaerobic Sludge Blanket Reactors (UASBs)

The design criteria for the various treatment units is discussed in the following sub-sections.

11.2.4.2 Influent BoD_5 Loading

As there is no sewerage system within the project area, the influent BOD_5, Li, in mg/l has been calculated using the following expression;

$$Li = \frac{1000B}{q}$$

Where; B is the per capita BOD_5 contribution, g/capita/d and q is the per capita wastewater contribution.

11.2.4.3 Anaerobic Ponds

Anaerobic Ponds have been designed based on volumetric BOD loading. The equation for calculating the volume of the pond, VA is as follows:-

$$V_A = \frac{Li\,Q}{\lambda v}$$

Where:

- Li is Influent BOD,
- mg/l (=g/m3),
- Q is Influent flow rate, m3/day and
- λv is Volumetric loading (g/m3/day).

The permissible volumetric BOD loading in anaerobic ponds, λV, in Kg/Ha/day is given by;

λV= 2T -100 where T is the design Temperature

11.2.4.4 Facultative Ponds

Facultative Ponds have been designed on the surface loading basis. The following formula has been adopted for sizing the mid-depth area.

$$A = \frac{10Q\,L_i}{\lambda s}$$

Where;

- ƛs is the design BOD5 loading,
- kg/ha/day,
- Q is the Average Flow in the pond, m3/day and
- Li is the influent BOD5 loading.

The permissible surface loading rate value increases with temperature, ToC. The global permissible value of the surface-loading rate to be used in design will be estimated from the following equation:

$$ƛ_s = 350(1.107-0.002T)^{(T-20)}$$

11.2.4.5 Maturation Ponds

Maturation ponds have been designed on basis of faecal coliform removal by use of first order kinetics as recommended by D. Mara (1986) as follows;

$$N_e = \frac{N_i}{(1 + k_T \emptyset a)(1 + k_T \emptyset f)(1 + k_T \emptyset m)^n}$$

Where

- Ni and N_e are number of coliform per 100 ml,
- K_t is the first order rate constant for FC removal,
- d^{-1} and θ is the retention, time, d.

11.2.4.6 Oxidation Ditches

The volume of the oxidation ditch has been calculated using the following expression;

$$V = \frac{Li\ Q}{S}$$

Where Li is the influent BOD, mg/l and S is the ditch liquor suspended solids concentration, mg/l

The mean hydraulic retention time, t, is given by;

$$t = \frac{V}{Q}$$

The energy requirement, Ro, is given by;

$$Ro = 2\ x\ BOD5\ Load$$

11.2.4.7 Percolating Filters

The percolating filters have been designed on the basis of Surface BOD_5 loading rate, λ_s, applied to the filters. This is given by the following expression;

$$\lambda s = Li\ x\ Q$$

The effective surface area of the filters, A_S, is given by

$$As = \frac{\lambda s}{OLR\ X\ D}$$

Where OLR is the design Organic BOD_5 loading rate, and D is the depth of the filter bed

The hydraulic loading rate, HLR, is calculated by the following expression;

$$HLR = \frac{Q + Qr}{As}$$

Where Qr is the recirculated liquor

11.2.4.8 Aerated Lagoons

The volume of the aerated lagoons, V_l, has been calculated using the following expression;

$$Vl = Qd$$

Where, Q is the wastewater flow rate per day d is the design retention time

The first order rate constant at a certain temperature is given as follows;

$$K_T = 5(1.035)^{T-20}$$

The soluble BOD_5 in the effluent is given by;

$$F_e = \frac{Li}{1+KT\ d}$$

The effluent BOD_5, Le is given by;

$$Le = Fe + 0.95X$$

Where X is the the cell concentration in the lagoon given by;

$X = \frac{Y(Li-Fe)}{1+bd}$ with b typically being $0.07d^{-1}$ and Y being 0.6

The total amount of oxygen required for bio-oxidation has calculated using the following expression;

$$R_{o2} = 1.5(Li - Le)Q$$

11.2.4.9 Settling Tanks

The surface area required for the Settling tanks has been calculated by;

$$As = \frac{Q}{SOR}$$

The weir loading rate, WLR, for the settling tanks has been calculated using;

$$WLR = \frac{Q}{\text{Weir Length}}$$

The design parameters used for sizing of the treatment works is as shown below;

11.2.4.10 Design Parameters

The various design parameters adopted in this study is as shown in the table below;

Table 11-4: Various design criterion for sewage treatment works

PARAMETER	UNIT	CRITERION
1. Per Capita BOD_5 loading	**g/capita/d**	**50**
2. Raw Sewage Faecal Coliform Count	**FC/100ml**	**25,000,000**
3. Design Temperature (Mean of coolest month)	**^0C**	**18**
4. Net evaporation rate	**mm/day**	**3.98**
5. Peak Flow Factor for Sewage Treatment Works	**-**	**2.45**
6. Freeboard		
All ponds, ditches and lagoons	m	1
Settling Tanks, UASBs and aeration Tanks	m	0.3
7. Length-Breadth ratio of ponds	**-**	**1:3**
8. Side slope ratio of ponds, lagoons and ditches	**-**	**1:3**
9. Design Depth		
Anaerobic Ponds	m	4.0
Facultative Ponds	m	1.75
Maturation Ponds	m	1.5
Settling Tanks	m	2.8
Percolating/Trickling Filters	m	2.0
10. Aerated Lagoons		
Retention Time	days	4
Aeration method	-	Surface aerators
Oxygen Transfer Rate in aerated Lagoons	Kg O_2/kWH	1.2
11. Primary Settling Tanks		
Surface overflow rate SOR	$m^2/m^3/d$	22
Weir Loading Rate WLR	$m^3/m/d$	<250
Minimum Retention Time	hrs	2
12. Secondary and Final Settling Tanks		
Surface overflow rate SOR	$m^2/m^3/d$	44
Weir Loading Rate WLR	$m^3/m/d$	<300
13. Oxidation Ditches		
Ditch Volatile Suspended Solids, S	mg/l	3000
Aeration Method	Surface brush aerator	

PARAMETER	UNIT	CRITERION
Return Sludge capacity	-	1.5 X ADWF
Capacity of mammoth rotors	Kg O_2/m/hr	7
Length – Breadth ratio	-	1:5
14. High Rate Percolating/Trickling Filters		
Organic BOD_5 loading rate, OLR	g/m^3/d	1,700
Hydraulic Loading Rate, HLR	m^3/m^2/d	<40
Recirculation ratio	-	1.5
15. Low Rate Percolating/Trickling Filters		
Organic BOD_5 loading rate, OLR	g/m^3/d	350
Hydraulic Loading Rate, HLR	m^3/m^2/d	<100
Recirculation ratio	-	1.5
16. Sludge Drying Beds		
Amount of Solids in domestic sewage	g/Capita/d	55
Water Content in sludge	-	90%
Density of Sludge	Kg/m^3	1,050
Sludge Drying Beds loading rate as dry solids	Kg/m^2/yr	175

11.3 KITENGELA SEWERAGE PROJECT

11.3.1 Natural Drainage Pattern and Sewerage Zones

The drainage pattern of the Kitengela Project Area is such that it naturally drains into four (4) different natural drainage patterns. These natural drainage patterns are as follows;

i) Kitengela River Drainage Basin

This basin drains the North to West of the Project Area and forms the north-western boundary of the project area. The main drainage river in this basin, the Kitengela River drains in to the Athi River some 2.5km downstream. The following sub-basins have been identified in this basin;

- Township Sub-drain: This serves the entire area enclosed by the Nairobi-Namanga Road, the Machakos-Kajiado Counties boundary and the area around the GK Prisons. This area is densely populated with multi storey residential and commercial developments.

- Noonkopir sub-drain: This serves the area behind the Main GK Prisons and the Western part of the Project Area. This area is medium populated with mostly residential single dwelling units.

ii) EPZA-Mavoko Stream Drainage Basin

The stream drains into Athi River about a kilometre after the confluence of the Kitengela River and Athi River in Athi River Town. Within the project area, the basin covers the North of Kitengela. The estates include those enclosed by the EPZ-Athi River boundary fence and the Nairobi-Namanga Road. This area is highly populated.

iii) Stony Athi River Drainage Basin:

This basin drains the Southern and Eastern parts of the Kitengela Project Area. The main river in this basin is the Stony Athi River which eventually empties into the Athi River 7km North of the Kitengela Area in Mavoko. The entire Stony Athi River is outside the Project Area but the following sub-basins within the project area directly drain in to it;

- Imambariswani Stream Sub-Basin: This serves the southern and Eastern parts of the developed area. The areas covered by this system include the Milimani, fountain view, Kitengela Girls school and Kitengela International School residential areas.
- Olooloitikoshi River Sub-Basin: This basin is further south of the core Kitengela project area and is characterised by open grasslands and a few industries along the Namanga-Kajiado Road.

11.3.2 Sewerage Systems Alternatives

The natural drainage system has been used as a basis of design of the sewerage system. The trunk sewers have been located at the lower drainage boundaries as much as permissible so as to maximize sewage collection from feeder sewer lines.

Locations of the waste water treatment plants dictate the trunk sewers routes, lengths and cost.

The following alternatives have been considered;

OPTION I: Centralized Sewerage System

In this option, one (1) waste water treatment works is proposed to be built in the Kinanie Area of Mavoko Constituency in Machakos County. This is a site located next to the existing EPZ-Athi

River Sewage Treatment Works located on coordinates $1^0$22'4"S $37^0$2'57"E along the Athi River,11.7km North-East of the Nairobi-Mombasa Road and 16km from Kitengela Township.

In this option, the following trunk sewer systems will be constructed;

v. 19,250m long Athi River Trunk Sewer – This will start at the proposed Wastewater Treatment Works inlet works to the Confluence of the Kitengela River to Athi River in Athi River Town.
vi. 7,350m Kitengela River Trunk Sewer – This will start at the Confluence of the Kitengela River with the Athi River to the the edge of the urban area of Western side of Kitengela Town.
vii. 5,800m long Noonkopir Trunk Sewer.
viii. 2,900m long Kitengela Prisons Trunk Sewer.
ix. 580m EPZA Trunk Sewer Extension to the existing EPZA WWTW.
x. 8,000m Stoney Athi River Trunk Sewer.
xi. 9,800m Imambariswani River Trunk Sewer.

The size of the waste water treatment works under this option will include sewage flows from all the area that can be drained by the above trunk sewers.

The advantages of this sewerage system option are;

- The proposed WWTP site neighbours an existing sewage treatment plant.
- Cost of acquisition of land for the WWTP is lower as the site is 13km from a major road.
- The land proposed for construction of WWTP is currently unutilised hence no compensation for loss of livelihood.
- The entire Kitengela Project Area will be served by one treatment plant.

The Main Disadvantages of this option are;

- A very long trunk sewer will be required along the Stoney Athi River and Athi River.
- Cost of compensation along the long trunk sewer for disturbance of economic activities will have to be included.
- The land is owned by a private individual and will require acquisition.
- Sizing of the Sewage Treatment Plant will incorporate sewage originating from areas outside of the project area.

- Management of the sewage treatment plant and some trunks sewers will be outside Kajiado County boundaries.

OPTION II: De-Centralized Sewerage System

In this option, Two (2) waste water treatment works are proposed to be built in the following locations;

a) Area at the Confluence of Imambaraswani River and Stoney Athi River located on coordinates 1028'55"S 376059'42"E. This site is referred to as stoney Athi in this report.
b) Area along the Kitengela River behind the East African Portland Cement Factory located on coordinates 1027'13"S 36057'17"E. This site is referred to as Noonkopir WWTW in this Report.

In this option, the following trunk sewer systems will be constructed;

i. For the Imambaraswani WWTW

- 8,000m Stoney Athi River Trunk Sewer
- 9,800m Imambariswani River Trunk Sewer

ii. For the Noonkopir WWTW

- 5,850m Kitengela River Trunk Sewer
- 5,800m long Noonkopir Trunk Sewer
- 2,900m long Kitengela Prisons Trunk Sewer

iii. 580m EPZA Trunk Sewer Extension to the existing EPZA WWTW

The advantages for this option are;

- The trunk sewers along the Stoney Athi River and Athi River will not be required.
- Cost of compensation along the trunk sewers significantly reduced.

The Main Disadvantages of this option are;

- Cost of compensation along the long trunk sewer for disturbance of economic activities will have to be included.
- The land is owned by a number of private individuals and will require acquisition.

- The sites are very close to the already built up area and has been be overtaken by urban settlement.
- The Kitengela River site is very rocky with a large position of it at an elevation that will require levelling of the site to enable gravity flow.
- The Kitengela River site is located immediately upstream of the Mavoko Water Intake Weir.

11.3.3 Reticulation Sewers

Provision of a sewerage system will require the construction of a reticulation system to serve areas that are either already developed or are in the process of development.

The secondary sewers have been proposed as collectors for the laterals and are planned such that they discharge the flows at appropriate points to the proposed trunk sewers.

Tertiary sewers have been located on the residential/estates road network so as to facilitate ease of property connection.

At total of 21km of the reticulation sewers have been identified to be implemented.

11.3.4 Waste Water Treatment Technologies Alternatives

So as to establish a method of treatment that can achieve the required effluent standards in the most economical and technically appropriate way for this project area, the following table summarises the operating characteristics, advantages and disadvantages of common alternative sewage treatment process.

Description and Comparison of Alternative Waste Water Treatment Processes

Treatment process	Brief Description	Advantages	Disadvantages
Waste stabilization ponds	▪ The most suitable method of wastewater treatment in hot climates ▪ Unit processes are Anaerobic ponds, Facultative ponds and Maturation ponds.	▪ No power requirements ▪ Very low O&M requirements ▪ No mechanical installations ▪ Very simple method of treatment	▪ High land requirement ▪ Odour nuisance, particularly from the anaerobic ponds ▪ Sensitive to temperature variations

Treatment process	Brief Description	Advantages	Disadvantages
Aerated lagoons	▪ A more intensive system of treatment resulting in greater removal of organics per unit volume of treated wastewater than in WSPs. ▪ Oxygen is supplied to wastewater by aerators floating or fixed in the lagoons.	▪ Detention time less than in stabilization ponds ▪ Less land requirement as compared to WSP	▪ High O&M requirements
Oxidation ditches	▪ An extended aeration plant best suited to small communities. ▪ The aeration tank is in the form of a long continuous channel, oval in plan and with rotor suspended over the channel for aeration, mixed liquor propulsion and prevention of suspended particles from settling out.	▪ Relatively low land requirement and suitable where land is at a premium price.	▪ High O&M requirements ▪ Not effective in removing Faecal Coli form.
Percolating/ Trickling filters	▪ Sewage is evenly distributed on the surface and effluent collected through under drains that also help in aeration. ▪ The units are preceded by settling tanks to collect the settleable organic solids delivered from the filters.	▪ Durability in all weather conditions and in the presence of corrosive effluents ▪ Does not upset for variation of hydraulic or organic loading	▪ Limitation in volumetric loading ▪ Significant loss of head ▪ Nuisance due to flies ▪ Medium O&M cost
Activated sludge process	▪ Involves aeration of settled sewage mixed with return sludge within an aeration tank during which time the micro-organisms in the sewage multiply through assimilation of the organics in the influent wastewater.	▪ Any degree of treatment is possible ▪ Final effluent is clear and odourless ▪ Low land requirement ▪ Freedom from odour nuisance ▪ Freedom from nuisance due to flies	▪ Sensitive to organic loading particularly for industrial wastes which result in bulking of sludge ▪ High O&M requirements ▪ High skilled management ▪ Sensitive to shock loading

Treatment process	Brief Description	Advantages	Disadvantages
	▪ Biomass generated is normally collected in settlement tanks.	▪ Recommended for treatment of sewage from large communities (not small works)	▪ Produces large quantity of sludge whose disposal is a problem

For comparison purposes, preliminary sizing of treatment units has been carried out for each of the alternative treatment methods including a hybrid system combining various treatment processes. The salient features and land requirements of each of the treatment method are presented in the following table;

Table 11-5: Land Requirements for different methods of Waste Water Treatment

Method of Treatment	Process Units	Land Needed (Ha)
Waste Stabilization Ponds	Anaerobic Ponds Facultative Ponds Maturation Ponds	61.63
Aerated Lagoons	Aeration Tanks Clarifiers Maturation Ponds	48.45
Oxidation Ditches	Oxidation Ditches Clarifiers Maturation Ponds	43.91
Percolating Filters	1^0 settling tanks Low rate filters 2^0 clarifiers High rate filters Maturation Ponds	44.77
Activated Sludge Process	1^0 Settling tanks Aeration tanks 2^0 settling tanks Maturation Ponds	43.44
Hybrid Sytem	Anaerobic Ponds Low Rate Filters	41.11

	Settling Tanks Maturation Ponds	

11.3.5 The Recommended Sewerage Project

11.3.5.1 Method of Waste Water Treatment

A waste stabilization ponds (WSP) system is recommended for the waste water treatment due to its low operation and maintenance, and the proven effectiveness in the local climate. The capital cost of buying land required for WSPs by far outweighs the operational and maintenance cost of the other alternatives.

Together with the recommended WSP system, a comprehensive inlet works is recommended. The inlet works will comprise coarse and fine screens, grit removal mechanism and flow measuring mechanism as well as an emergency by-pass.

A schematic profile of a typical waste stabilization ponds system is shown in drawings submitted.

11.3.5.2 Sewerage System and Location of Waste Water Treatment Plant

The Consultant recommends a centralized sewerage system as the most cost effective to construct, maintain and operate, as well the most technically viable sewerage system for the project area for immediate implementation (Year 2022) and also future extensions at the year 2032 or there about.

Staging of the sewerage project is proposed in to two (2) phases as follows;

i. Phase I – This will entail the following;

- Acquisition of all the 75 Hectares required for the present and future construction of the waste water treatment plants.
- Construction of an 18,250m3/d waste stabilization ponds system.
- Construction of Trunk Sewers up to the currently settled areas.
- Construction of Reticulation Sewers in the already densely settled areas of the project area. These areas include the main Kitengela Business Center, Milimani area, Noonkopir, residential area bordering the EPZA and the Yukos area.

ii. Phase II – This will entail construction of the following;

- An additional 18,250m3/d waste stabilization ponds system.
- Extension of the Imambaraswani River Trunk Sewer and Kitengela River Trunk Sewer to cover the additional areas currently not settled but predicted to be settled in the future.
- Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions. These areas include the area south west of the project area along the Imambaraswani river and the area south (upstream) of Kitengela River.

11.4 KAJIADO TOWN SEWERAGE PROJECT

11.4.1 Natural Drainage Pattern and Sewerage Zones

i. All natural drainage systems within Kajiado Town eventually drain in to the Olkejuado River. The sub-basins within the area are as follows;
ii. The Airstrip sub-basin
iii. The Town Center Sub-Basin
iv. The Lower Town Center Sub-Basin
v. The Administration area sub-basin
vi. The District Hospital Area Sub-Basin
vii. The Maasai Technical Institute Area Sub-Basin
viii. The Ministries Areas Sub-Basin
ix. The Mabatini Area Sub-Basin

11.4.2 Sewerage Systems Alternatives

With the location of the waste water treatment plants at different locations, the following alternative sewerage systems have been identified;

OPTION I: <u>Centralized Sewerage System with the WWTP at Iltareto</u>

In this option, one (1) WWTP is proposed to be constructed at Iltareto Area. This site is 4.5km South-east of the Athi River-Kajiado-Namanga Road along the Olkejuado River. The site is located on coordinates 1^0 54’ 17” S, 36^0 47’ 46” E.

With this site, the following trunk sewers are proposed.

i. 11,900m long Olkejuado River Trunk Sewer
ii. 3,900m long Kajiado Town Northern Sub-Trunk Sewer

iii. 2,900m long Kajiado Town Southern Sub-Trunk Sewer
iv. 3,000m long Administration zone Sub-Trunk Sewer
v. 4,400m long Maasai Technical College Sub-Trunk Sewer
vi. 7,350m long Mabatini Sub-Trunk Sewer

The main advantages of this option are;

- Cost of acquisition of land for the WWTP is lower as the site is in the rural area.
- The land proposed for construction of WWTP is currently not developed with permanent structures.
- The entire Kajiado Township Project Area will be served by single treatment plant by gravity system which is easy and cheaper to manage.

The disadvantages of this option are;

- A longer trunk sewer will be required along the Ol kejuado River.
- Additional Cost of crossing the A1 Road (Kajiado – Namanga).
- The land is owned by private individuals and will require acquisition.
- The Olkejuado River is a seasonal river; hence, the output of the treatment process will have to be of superior quality to qualify for disposal in dry river beds.

OPTION II: De-Centralized Sewerage System with two (2) WWTP at Kajiado Town and Mabatini

In this option, two (2) separate waste water treatment plants are proposed at Kajiado Town and Mabatini areas. These sites are described as follows;

c) The Kajiado Town Site: This site is proposed to be located on coordinates 1^0 52' 19" S, 36^0 46' 27" E located next bridge on Olkejuado River for the Kajiado-Namanga Road and 2km behind the Kajiado District Hospital.
d) The Mabatini Site: This site is proposed to be located along the seasonal river that drains from the Mabatini area to the Olkejuado River. It is proposed to be located on coordinates 1^0 52' 24" S, 36^0 48' 30" E.

In this option, the following trunk sewer systems will be constructed;

i. For the Kajiado Town WWTW

- 7,640m Olkejuado River Trunk Sewer

- 3,900m long Kajiado Town Northern Sub-Trunk Sewer
- 2,900m long Kajiado Town Southern Sub-Trunk Sewer
- 3,000m long Administration zone Sub-Trunk Sewer

ii. For the Mabatini WWTW

- 5,000m long Mabatini Sub-Trunk Sewer

The advantages for this option are;

- A shorter trunk sewer along the Olkejuado River that does not require a major road crossing.
- Easily accessible site as it is located next to a major road.

The Main Disadvantages of this option are;

- The land is owned by private individuals and will require acquisition.
- The sites are very close to the already built up area and will be overtaken by urban settlement within the near future.
- The Kajiado Town site is close to a major County hospital.
- Some areas of the project area, the Ministry of Public Works and Maasai Technical Institute, will not be served by a gravity system.

11.4.3 Reticulation Sewers

A total of 15km of secondary and tertiary sewers are proposed to be implemented in the currently settled areas with the major concentration been in the Main Township area, Majengo, the Kajiado Hospital area, Governor's office area and Law Courts.

11.4.4 The Recommended Sewerage Project

11.4.4.1 Method of Waste Water Treatment

A waste stabilization ponds (WSP) system is recommended for this area as the cost of land is relatively low and the energy cost is very high.

Together with the recommended WSP system, a comprehensive inlet works is recommended. The inlet works will comprise coarse and fine screens, grit removal mechanism and flow measuring mechanism as well as an emergency by-pass.

A schematic profile of a typical waste stabilization ponds system is shown in the drawings submitted.

11.4.4.2 Sewerage System and Location of Waste Water Treatment Plant

The Consultant recommends a centralized sewerage system with the WWTP at the Iltareto Area. This option is a most effective and efficient way of handling the waste water originating from the project area in the long run.

11.4.4.3 Staging of the Project

It is proposed that the implementation of this recommendation be staged and implemented in to two (2) phases as follows;

i. Phase I – This will entail the following;

- Acquisition of all the Thirty (30) Hectares required for the present and future construction of the waste water treatment plants.
- Construction of a 7,350m3/d waste stabilization ponds system.
- Construction of the 11.9km Olkejuado Trunk Sewers.
- Construction of the sub-trunk sewers and Reticulation Sewers in the already densely settled areas of the project area.

ii. Phase II – This will entail construction of the following;

- An additional 7,350m3/d waste stabilization ponds system.
- Extension of the Olkejuado River Trunk Sewer upstream to cover the additional areas next to the airstrip that is currently not settled but predicted to be settled in the future.
- Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions.

11.5 SANITATION IN AREAS OUTSIDE THE CORE URBAN CENTERS

It is proposed to use on-site sanitation facilities in the peri-urban and rural areas within the project. These facilities include;

i. Pit Latrines in Rural Areas:

This consists of a slab over a pit which may be 2 m or more in depth The slab should be firmly supported on all sides and raised above the surrounding ground so that surface water cannot enter the pit. If the sides of the pit are liable to collapse they should be lined. A squat hole in the slab or a seat is provided so that the excrete fall directly into the pit.

ii. Ventilated Pit Latrine in Rural Areas with higher Population:

A ventilated improved pit (VIP) latrine is an improvement of the simple latrine by providing a ventilated pipe for reducing the fly and odour nuisance. The pipe should extend above the latrine roof and have a fly-proof netting at the top. The superstructure should be kept dark causing the insects entering the pit to be attracted towards the light at the top of the vent pipe where they are trapped by the fly screen. A modification of the VIP latrine is the ventilated improved double pit (VIDP) latrine. This includes a single superstructure over two pits. One pit is used until it is filled to within about a metre of the top. The vent pipe and defecation holes are then completely sealed and the other pit used. The contents of the first pit are dug out after a period of at least 2 years once the contents have becomes less harmful.

iii. Pour Flush Latrines in Institutions in Rural Areas:

The latrine is fitted with a trap providing a water seal, which is cleared of faeces by pouring in of water to wash the solids into the pit and replenish the water seal. The water seal prevents flies, mosquitos and odours reaching the latrine from the pit. The pit may be offset from the latrine by providing a short length of pipe or covered channel from the pan to the pit. The pan of an offset pour flush latrine is supported by the ground and the latrine may be within or attached to a house.

iv. Septic Tanks and Package Waste Water Treatment Plants in Peri-Urban Areas:

A septic tank is an underground watertight settling chamber into which raw sewage is delivered through a pipe from plumbing fixtures inside a house or other building. The sewage is partially treated in the tank by separation of solids to form sludge and scum. Effluent from the tank infiltrates into the ground through drains or a soakpit. The system works well where the soil is permeable and not liable to flooding or waterlogging, provided the sludge is removed at appropriate intervals to ensure that it does not occupy too great a proportion of the tank capacity. Sizing of the septic tank depends on the number of users and frequency of emptying.

Where soil drainage capability is poor, use of advanced package plants is recommended.

11.6 USE OF WASTEWATER FOR IRRIGATION

In order to supplement the available water for irrigation from the proposed dam, the Consultant recommends the use of treated wastewater for unrestricted irrigation. Mara (2003) points out that as treated wastewater contains scarce water and valuable plant nutrients, the crop yields are higher when crops are irrigated with wastewater than with freshwater as indicated in Figure below.

Irrigation water	*Wheat*	*Moong beans*	*Rice*	*Potato*	*Cotton*
Raw wastewater	3.34	0.90	2.97	23.11	2.56
Settled wastewater	3.45	0.87	2.94	20.78	2.30
Waste stabilization pond effluent	3.45	0.78	2.98	22.31	2.41
Fresh water + NPK	2.70	0.72	2.03	17.16	1.70

Figure 11.1: High of crop yeld based on type of irrigation water.

NPK refers to Nitrogen, Phosphorous and Potassium additives such as would be found in fertilizer.

The losses from evaporation, seepage/saturation into the pond walls and floors and gains from rainfall are taken to be negligible. The expected effluent quantities from the Kitengela and Kajiado wastewater treatment plants are therefore taken to be equivalent to the influent, that is, 36,423 m^3/day and 14,742 m^3/day, respectively, for the year 2042. This translates to a total amount of irrigation water of 51,165 m^3/day or about 18,675,225 m^3/year.

12 ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT FOR THE PROPOSED DAM, WATER TREATMENT AND WASTE WATER TREATMENT PLANTS

12.1 INTRODUCTION

Like any other development, the proposed Dam, Waste Water and Water Treatment plant projects will bring in their awaken certain impacts. These projects are listed in the Second Schedule of Environment Management and Coordination Act (EMCA, Revised 2015) among projects that must undergo Environmental Impact Assessment (EIA) before they can commence. Impacts identification is followed by impact prediction in which case the significance of the potential impacts is determined. Predictions are normally based on explicit assumptions about environmental processes, professional judgment and different value judgments expressed by various stakeholders during consultations.

Environmental impacts manifest at all stages of a project. That is Preconstruction, Construction, Operation and Decommissioning Phases. This is because of the phase specific project activities that inform particular actions which in turn act on environmental factors in different ways generating the impacts. While some impacts are phase specific, others persist throughout the entire lifespan of the project. Conversely, some impacts are project specific mainly due to inputs and outputs. The significance of these impacts is also varied. The impacts vary in severity, duration, spatial extent, and reversibility among others. Potential impacts of the proposed projects are as follows.

12.2 POTENTIAL ENVIRONMENTAL IMPACTS OF THE PROPOSED DAM SITES

12.2.1 Kitengela River Dam Site

a) Construction Phase Impacts

- Dust from excavation and transportation of construction materials.
- Air pollution from poorly serviced construction machines and equipment.
- Excessive noise and vibration from construction machinery.
- Change in vegetation composition may affect fauna species the area.
- Loss of vegetation due to dam site clearance.
- Uncovered borrow bits pose health and safety risks to humans, livestock and game animals.
- Displacement (Physical and economic) of population to create room for the reservoir.
- Accidents during construction leading to injury, loss of lives or damage to property.

- Spread of HIV and AIDS due to presence of large numbers of migrant workers.
- Solid wastes such as empty cement bags degrade environmental aesthetics and can harm animals if eaten.
- Disruption of social bonds and support networks from displacement and subsequent resettlement.
- Sanitation problems in the contractor's camp.

b) Operation phase impacts

- Human wildlife conflicts which could result from: blockage of dispersal areas, lost and reduced grazing range and blockage of migration routes as a result of the reservoir.
- Sedimentation in the dam will increase the erosive capacity of the stream downstream.
- Fragmentation of the riverine ecosystem with effects on biodiversity.
- Likelihood of seismic activity being triggered by the weight of the reservoir in the area.
- Risk of drowning in the reservoir.
- Emergence of invasive species.
- Risk of dam failure with attendant injuries, loss of life and property destruction.
- Impacts on human health from upsurge water related diseases such as malaria cases and helminthic infections.
- Modification of weather due to the presence of a large water body may induce local climatic variations.
- Impacts on the ground water recharge thus affecting downstream abstraction from boreholes.
- Loss of livestock grazing land.

12.2.2 Noonpopong River Dam Site

a) Construction Phase Impacts

- Dust from excavation and transportation of construction materials.
- Air pollution from poorly serviced construction machines and equipment.
- Excessive noise and vibration from construction machinery.
- Human wildlife conflicts which could result from: Poaching for game meat, blockage of dispersal areas lost and reduced grazing range and blockage of migration routes.
- Change in vegetation composition may affect fauna species the area.
- Loss of vegetation due to dam site clearance.

- Uncovered borrow bits pose health and safety risks to humans, livestock and game animals.
- Cultural conflicts due to presence of migrant workers from different backgrounds.
- Accidents during construction leading to injury, loss of lives or damage to property.
- Spread of HIV and AIDS due to presence of large numbers of migrant workers.
- Solid wastes such as empty cement bags degrade environmental aesthetics and can harm animals if eaten.
- Sanitation problems in the contractor's camp.

b) Operation phase impacts

- Human wildlife conflicts which could result from: blockage of dispersal areas, lost and reduced grazing range and blockage of migration routes as a result of the reservoir.
- Sedimentation in the dam will increase the erosive capacity of the stream downstream.
- Fragmentation of the riverine ecosystem with effects on biodiversity.
- Likelihood of seismic activity being triggered by the weight of the reservoir in the area.
- Risk of drowning in the reservoir.
- Emergence of invasive species.
- Risk of dam failure with attendant injuries, loss of life and property destruction.
- Impacts on human health from upsurge water related diseases such as malaria cases and helminthic infections.
- Modification of weather due to the presence of a large water body may induce local climatic variations.
- Impacts on the ground water recharge thus affecting downstream abstraction from boreholes.
- Loss of livestock grazing land.

12.2.3 Ereteti River Dam Site

a) Construction Phase Impacts

- Dust from excavation and transportation of construction materials.
- Air pollution from poorly serviced construction machines and equipment.
- Excessive noise and vibration from construction machinery.
- Change in vegetation composition may affect fauna species the area.
- Loss of vegetation due to dam site clearance.

- Uncovered borrow bits pose health and safety risks to humans, livestock and game animals.
- Cultural conflicts due to presence of migrant workers from different backgrounds.
- Displacement (Physical and economic) of population to create room for the reservoir.
- Accidents during construction leading to injury, loss of lives or damage to property.
- Spread of HIV and AIDS due to presence of large numbers of migrant workers.
- Solid wastes such as empty cement bags degrade environmental aesthetics and can harm animals if eaten.
- Loss of grounds which have cultural significance.
- Disruption of social bonds and support networks from displacement and subsequent resettlement.
- Sanitation problems in the contractor's camp.

b) Operation phase impacts

- Human wildlife conflicts which could result from: blockage of dispersal areas, lost and reduced grazing range and blockage of migration routes as a result of the reservoir.
- Sedimentation in the dam will increase the erosive capacity of the stream downstream.
- Fragmentation of the riverine ecosystem with effects on biodiversity.
- Likelihood of seismic activity being triggered by the weight of the reservoir in the area.
- Risk of drowning in the reservoir.
- Emergence of invasive species.
- Risk of dam failure with attendant injuries, loss of life and property destruction.
- Impacts on human health from upsurge water related diseases such as malaria cases and helminthic infections.
- Modification of weather due to the presence of a large water body may induce local climatic variations.
- Impacts on the ground water recharge thus affecting downstream abstraction from boreholes.
- Loss of livestock grazing land.

12.2.4 Airfeld at Olooloitikosh River Dam Site

a) Construction Phase Impacts

- Dust from excavation and transportation of construction materials.

- Air pollution from poorly serviced construction machines and equipment.
- Excessive noise and vibration from construction machinery.
- Human wildlife conflicts which could result from: Poaching for game meat, blockage of dispersal areas lost and reduced grazing range and blockage of migration routes.
- Change in vegetation composition may affect fauna species the area.
- Loss of vegetation due to dam site clearance.
- Uncovered borrow bits pose health and safety risks to humans, livestock and game animals.
- Cultural conflicts due to presence of migrant workers from different backgrounds.
- Displacement (Physical and economic) of population to create room for the reservoir.
- Accidents during construction leading to injury, loss of lives or damage to property.
- Spread of HIV and AIDS due to presence of large numbers of migrant workers.
- Solid wastes such as empty cement bags degrade environmental aesthetics and can harm animals if eaten.
- Loss of grounds which have cultural significance.
- Disruption of social bonds and support networks from displacement and subsequent resettlement.
- Sanitation problems in the contractor's camp.

b) Operation phase impacts

- Human wildlife conflicts which could result from: blockage of dispersal areas, lost and reduced grazing range and blockage of migration routes as a result of the reservoir.
- Sedimentation in the dam will increase the erosive capacity of the stream downstream.
- Fragmentation of the riverine ecosystem with effects on biodiversity.
- Likelihood of seismic activity being triggered by the weight of the reservoir in the area.
- Risk of drowning in the reservoir.
- Emergence of invasive species.
- Risk of dam failure with attendant injuries, loss of life and property destruction.
- Impacts on human health from upsurge water related diseases such as malaria cases and helminthic infections.
- Modification of weather due to the presence of a large water body may induce local climatic variations.
- Impacts on the ground water recharge thus affecting downstream abstraction from boreholes.

12.2.5 Olooloyankilani River Dam Site

a) Construction Phase Impacts

- Dust from excavation and transportation of construction materials.
- Air pollution from poorly serviced construction machines and equipment.
- Excessive noise and vibration from construction machinery.
- Change in vegetation composition may affect fauna species the area.
- Loss of vegetation due to dam site clearance.
- Uncovered borrow bits pose health and safety risks to humans, livestock and game animals.
- Cultural conflicts due to presence of migrant workers from different backgrounds.
- Accidents during construction leading to injury, loss of lives or damage to property.
- Spread of HIV and AIDS due to presence of large numbers of migrant workers.
- Solid wastes such as empty cement bags degrade environmental aesthetics and can harm animals if eaten.
- Disruption of social bonds and support networks from displacement and subsequent resettlement.
- Sanitation problems in the contractor's camp.

b) Operation phase impacts

- Human wildlife conflicts which could result from: blockage of dispersal areas, lost and reduced grazing range and blockage of migration routes as a result of the reservoir.
- Sedimentation in the dam will increase the erosive capacity of the stream downstream.
- Fragmentation of the riverine ecosystem with effects on biodiversity.
- Likelihood of seismic activity being triggered by the weight of the reservoir in the area.
- Risk of drowning in the reservoir.
- Emergence of invasive species.
- Risk of dam failure with attendant injuries, loss of life and property destruction.
- Impacts on human health from upsurge water related diseases such as malaria cases and helminthic infections.
- Modification of weather due to the presence of a large water body may induce local climatic variations.
- Impacts on the ground water recharge thus affecting downstream abstraction from boreholes.

- Loss of livestock grazing land.

12.3 POTENTIAL ENVIRONMENTAL IMPACTS OF THE PROPOSED SEWERAGE AND WASTE WATER TREATMENT PLANT PROJECTS

a) Construction Phase Impacts

- Air pollution from emissions generated by construction machines and heavy vehicles.
- Water pollution from grease and oils.
- Dust from the site excavations and transportation of construction materials such as sand.
- Solid wastes such as used cement bags and pieces of metal
- Excessive noise and vibration from construction equipment
- Loss of flora and fauna species as a result of vegetation clearance on the proposed site.
- Disturbance of vehicular and foot by trenches on road crossings.
- Difficulty of access due to open trenches in residential and commercial areas.
- Damage of underground infrastructure such as water pipes, power, telephone and fiber optic cables.
- Effect on the stability due to excavation near houses along the sewer lines.
- Effect on stability and integrity of houses, roads and power lines due to rock blasting in the STW site.
- Influx of migrant workers leading to social and cultural conflicts.
- Increase in HIV and AIDS cases due to the presence of migrant workers.
- Occupational accidents leading to injuries and death of workers or passersby.
- Population displacement.

b) Operation phase impacts

- Contamination of drinking water from sewage leaks into burst water pipes.
- Water pollution from accidental or deliberate release of untreated effluents into the receiving rivers.
- Solid waste such as sludge.
- Offensive odours due to malfunction or poor maintenance of the STW.
- Public health issues related to irrigation with untreated sewage.
- Water and soil pollution from leaks and sewage overflow. Leaks may result from failure of sewer lines while overflow may be due to surges caused by overloading of the ponds.
- Exposure of workers to hazardous chemicals.

- Risk of wildlife such as hippos and crocodiles invading the site posing danger to workers and members of the public.
- Risk of drowning during maintenance.
- Colonization of the sites by scavenger birds which pose aviation hazards.
- Growth of invasive water weeds in the waster stabilization ponds.
- Blockage of sewer lines leading to bursts in residential areas with severe health consequences.
- Breeding of malaria transmitting mosquitoes in the ponds.
- Public health issues downstream due to consumption of sewage contaminated **water.**

12.4 POTENTIAL ENVIRONMENTAL IMPACTS OF THE PROPOSED WATER TREATMENT PLANT AND WATER SUPPLY PROJECTS

Many impacts in this project especially the construction phase compare favourably with impacts on the STW's construction impacts.

a) Construction Phase Impacts

- Human wildlife conflicts due to blockage of dispersal corridor, migration and grazing ranges.
- Air pollution from emissions generated by construction machines and heavy vehicles.
- Water pollution from grease and oils.
- Dust from the site excavations and transportation of construction materials such as sand.
- Solid wastes such as used cement bags and pieces of metal
- Excessive noise and vibration from construction equipment
- Loss of flora and fauna species as a result of vegetation clearance on the proposed site.
- Disturbance of vehicular and foot by trenches on road crossings.
- Difficulty of access due to open trenches in residential and commercial areas.
- Damage of underground infrastructure such as water pipes, power, telephone and fiber optic cables during excavation of trenches for water pipelines.
- Effect on the stability due to excavation near houses along the water supply pipelines.
- Effect on stability and integrity of houses, roads and power lines due to rock blasting in the water treatment site.
- Influx of migrant workers leading to social and cultural conflicts.
- Increase in HIV and AIDS cases due to the presence of migrant workers.

- Occupational accidents leading to injuries and death of workers or passers-by.
- Population displacement from the site of the proposed water treatment plant.
- Fire outbreak in contractor's camp.

b) Operation phase impacts

- Sludge.
- Risk of fire outbreak in the stores and offices of WTP.
- Spread of water borne diseases due to supply of poorly treated water.

12.5 ENVIRONMENTAL AND SOCIAL MANAGEMENT PLAN

The purpose Environmental and Social Management Plan (ESMP) is to provide mitigation measures for the significant potential negative environmental impacts. The objectives of the ESMP are:

- To provide evidence of practical and achievable plans for the management of the proposed project.
- To provide the Proponent and the relevant Lead Agencies with a framework to confirm compliance with relevant laws and regulations.
- To provide community with evidence of the management of the projects in an environmentally acceptable manner.

Conversely, Environmental monitoring provides feedback about the actual environmental impacts of a project. Monitoring results help judge the success of mitigation measures in protecting the environment. They are also used to ensure compliance with environmental standards, and to facilitate any needed project design or operational changes. A monitoring program, backed up by powers to ensure corrective action when the monitoring results show it necessary, is a proven way to ensure effective implementation of mitigation measures. By tracking a project's actual impacts, monitoring reduces the environmental risks associated with that project, and allows for project modifications to be made where required.

An ESMP will be prepared when the actual EIA has been carried out after exhaustive field work, documents review and adequate public consultations.

13 RESSETLEMENT ACTION PLAN

A Draft RAP report will be submitted alongside the Preliminary Design Report and a final RAP report submitted at the end of the assignment along the final design documents.

The RAP report will be prepared in line with the Kenyan guidelines and international best practices including the AfDB's safeguards that have been designed to mitigate against impoverishment risks associated with Involuntary Resettlement and the restoration or improvement of income-earning capacity of the Project Affected Persons (PAP). The policy requires full public participation in resettlement planning and implementation and describes the conditions that borrowers are obliged to meet in operations involving involuntary resettlement.

In preparing the RAPs, the Consultant will:

- Review relevant Kenyan law and procedures regarding displacement, land acquisition, and compensation;
- Review RAPs prepared for other recent similar projects in Kenya;
- Undertake a reconnaissance field survey of the proposed civil works to assist in defining the content of the RAP;
- Consult with the appropriate AWSB /TAWSB staff, and their consultants; and
- Consult with persons potentially affected by civil works.

The RAP Report will include the following:

1) Description of project area and area of influence: The Consultant will provide a brief description of the project but will provide more details for components and activities for which land acquisition and impact on livelihood are required.
2) Principles and Objectives Governing Resettlement Preparation and Implementation: The Consultant will describe the basic principles and vision of the resettlement program and demonstrate that the commitment is to ensure that affected people are meaningfully consulted, compensated fully and fairly for their losses before works in these areas can start, and assisted in their efforts to improve their livelihoods and standards of living or at least to restore them. The Consultant will describe any particular conditions in the Project that may present special problems or opportunities, and show how the resettlement will be done, in principle, so as to overcome risks or take advantage of such opportunities.

3) Legal Frameworks for Resettlement: The Consultant will review the Kenyan laws governing the taking of land or other assets, set out any requirements for resettlement that applies to the types of cases that will arise in the project and analyze all gaps and recommend how such gaps may be bridged.
4) Eligibility Criteria for Various Categories of Affected People: The Consultant will determine the method for setting a cut-off date for eligibility for compensation. Different categories of people affected by the project will be set out, and the types of losses such people may suffer highlighted. Criteria that are to be used to identify the eligibility for compensatory measures for each category of affected people, whether losses are partial or total, whether people have their own land or also rent land will be defined
5) Impacts, Land Acquisition, Resettlement and Mitigation: Land acquisition or restrictions of access that may be required will be discussed. The study will discuss extensively on the impacts of the project to people livelihoods including assets and economic activities including what assets and economic activities are likely to be affected. The RAP will further propose appropriate compensation measures to be adopted.
6) Mapping: The proposed project area will be mapped in detail. The Consultant will prepare a detailed large-scale map on which individual affected properties are demarcated. The Consultant will undertake land surveys and map different types of land according to the different land uses.
7) Methods of Valuing Affected Assets : The Consultant will describe the GoK's methods of valuing those assets that deemed eligible for compensation, and those that must be compensated under the policy principles that meet the international best practices. The methods for inventorying assets, assigning values to each type of asset, and coming to agreement with each affected person or group on the total profile of losses and compensation will be explained. An "entitlement matrix" which shows the types of affected people, the types of losses, and the forms and amounts of compensatory actions that will be taken for each type will be prepared.
8) Methods for Consultation with and Participation of Affected People: The Consultant here will show that meaningful consultation was carried out with a broad array of stakeholders including both client officials at every level and the types of people who may be the beneficiaries of, and affected by, typical civil works. A record of all such consultations will be included.
9) Implementation Schedule: The Consultant will in the RAP to set out implementation schedule for the resettlement. The RAP shall in addition to the implementation schedule

identify potential risks that could militate against the smooth implementation of the resettlement actions and suggest plausible mitigation measures to serve as a guide to the client and the team who will be working on the implementation.

10) Grievance Redress Mechanisms: The Consultant will propose and describe the mechanisms available to the affected people to report grievances and show how the mechanism will be accessible (in terms of language, distance, and cost) to affected people, and what recourse/appeal from the local grievance mechanism may be available.
11) Budget and Funding Arrangements: The Consultant will provide an estimate the overall costs of resettlement.
12) Monitoring and Evaluation: The Consultant will propose an appropriate mechanism for monitoring the effective implementation of resettlement practices.

No	Proposed Project Component	Resettlement Issues	Recommendation/ Comments
1.	**Kitengela Centralized Sewerage System** In this option, one (1) waste water treatment works is proposed to be built in the Kinanie Area of Mavoko Constituency in Machakos County. This is a site located next to the existing EPZ-Athi River Sewage Treatment Works located on coordinates 1022'4"S 3702'57"E along the Athi River,11.7km North-East of the Nairobi-Mombasa Road and 16km from Kitengela Township.	- Land to be acquired for the STWs is far from the residential areas and urban centers - Land is privately owned - Cost of compensation along the long trunk sewer for disturbance of economic activities will have to be included that is for crops and trees -Management of the sewage treatment plant and some trunks sewers will be outside Kajiado County boundaries.	- Compensation will be at full replacement cost, PAPs will be allowed to harvest and use the raw materials -Land Acquisition (Compulsory) is envisaged for the Waste Water Treatment Plant which is privately owned. -The owners of the structures will be given notice three months prior to demolish their structures during construction and compensation at full replacement and PAPs will be allowed to salvage. -Need for both county government of Kajiado and Machakos to have a discussion on the issue of management of the System
	Decentralized Sewerage System (Kitengela) In this option, Two (2) waste water treatment works are proposed to be built in the following locations; - Area at the Confluence of Imambaraswani River and Stoney Athi River located on coordinates 1028'55"S 376059'42"E. This site is referred to as stoney Athi in this report. -Area along the Kitengela River behind the East African Portland Cement Factory located on	-The trunk sewers along the Stoney Athi River and Athi River will not be required. - Cost of compensation along the trunk sewers significantly reduced. -Cost of Land Acquisition will be high compared to the above	-Land Acquisition (Compulsory) is envisaged for the Waste Water Treatment Plant which is privately owned and already subvided -Compensation at Full Replacement for crops, trees and structures that will be affected during construction

	coordinates 1027'13"S 36057'17"E. This site is referred to as Noonkopir WWTW	system its near an urban area, land has already been subdivided; it's also close to the industrial and commercial centers.	
2.	**Kajiado Decentralized Sewerage System** One (1) WWTP is proposed to be constructed at Iltareto Area located 4.5km south-east of the Namanga Road along the OlKejuado River.	- Cost of Land acquisition will be lower compared to the other options as it's in rural area. - Minimal relocation of PAPs - No presence of structures within the site - Presence of a Primary school and human settlement 200m from the proposed site	- Compensation for land is envisaged approximately 17Ha - Minimal relocation of PAPs
	Centralized System : Kajiado Two (2) separate WWTP are proposed to be constructed at Kajiado Town and at the Mabatini areas.	-Both Lands are private and necessitate acquisition - Proximity to town makes the site not feasible due to pollution and increase in land prices	-Compensation at Full Replacement for crops, trees and structures that will be affected during construction -
3.	**DAMS and WATER SUPPLY** i. Ereteti Dam site 4,800,000.00 m^3 and DWTP	-The proposed land is near Olooloitikosh Town, main Isinya – Kiserian Road hence its predominately rural – urban -There exists some settlement neighboring the dam site who will be affected by the effects of construction, hence permanent relocation would be warranted -Land is privately owned approximately 70 Ha will be needed for the project	- Land has to be acquired for the Water reservoir and Drinking Water Treatment Plant. - Permanent relocation of approximately 20 HH

		-Cost of Land is estimated to be 3.4Billion as its proximity to the main road	
	ii. Airfield Dam 7,300,000.00 m^3 and DWTP	- Land is near a town, airstrip ,main Isinya-Kiserian Road hence the cost can go up considering the area is growing -Land is privately owned in the area approximately 52Ha will be needed for the project	- Permanent Relocation of several households is envisaged - Presence of an Active airstrip which makes the site not feasible for construction -Cost of land acquisition will high as most land is subdivided
	iii. Kitengela 34,200,000.00 m^3 and DWTP	-Land is Predominately Rural and Agricultural No settlement within the site -There were some 5 households observed at the buffer zone who might necessitate relocation - Approximately 420Ha of land will be acquired	- There need to acquire land - Neglible relocation of people
	iv. Noopopong 1,800,000.00 m^3 and DWTP	-Land is Rural -No settlement within the site - 40Ha of land to be acquired	-Need for land acquisition
	v. Oloyankilani 4,400,000.00 m^3 and DWTP	-The area is predominately rural and considered Agricultural -Presence of a primary school within the reservoir area which will need to be relocated	-Need to relocate the Primary school - Land acquisition is necessary

		-Secondary school who might suffer from the effects of construction -Approximately 41ha of land will be needed	

14 SOCIO-ECONOMIC STUDY

14.1 BACKGROUND

The Water Act 2016 which has been updated from Water Act 2002 makes provision for the provision of clean and safe water in adequate quantities and to reasonable standards of sanitation for all citizens. The Act gives power to Water Works Development Agencies such as AWSB and TAWSB in accordance to Article 152 of the Act, under whose jurisdiction the proposed project falls and one of its service providers Olekejuado Water and Sewerage Company (OWSCO).

OWSCO is mandated to provide water and sewerage services to consumers spread over Kajiado County. Approximately, a total of 20,000 households in Kajiado town and its environs currently benefit from the company's services. OWSCO core business is the provision of water and sewerage services. For the company to sustain itself, it has to ensure that the Kajiado Town and its adjacent areas are well served with clean water and sanitation services.

The survey found that currently the capacity of the Company to meet the current demand is curtailed by the current storage capacity that is far much below the current consumer needs. It is for this reason that AWSB /TAWSB intent to develop a Dam, Water Reservoir, Sewerage System and Waster Water Ponds in order to extent and expand its supply coverage to the areas underserved as well as unserved. This proposed project is intended to augment the Company's current water and sanitation services to be able to serve to more consumers and especially the poor and vulnerable communities within the Kajiado, Kitengela and its environs.

This report provides an outline of the socio-economic issues within the service provider area of jurisdiction as well as other areas of intended supply. Sample of Filled data collection questionnaires are included in **Appendix.**

14.2 OBJECTIVES OF THE SOCIO – ECONOMIC SURVEY

Objectives of the socio-economic assessment through household surveys was to collect information on

- Family size, age and education
- Income and poverty levels
- Existing levels of water and sanitation services and the requirement and demand for the service

- Knowledge attitude and practices on water, sanitation and hygiene
- Gender issues (access and control of wealth and health)
- Willingness to pay cash/labour for investment
- Desire to participate in ownership and management service

14.3 METHODOLOGY AND APPROACH USED IN THE STUDY

The following chapter contains the demographic and social information which has been used to establish current and preferred patterns of usage of Water and Sanitation facilities in Kajiado, Kitengela and Olooloitikosh areas, the biggest problems that people currently face and to determine peoples' preferences for new types of services.

An empirical socio-economic survey was carried out within the said areas to ascertain information relating to the current water and sanitation situation in the town, including issues such as demands for water and sanitation, gender issues, preferences for type of water delivery, ability and willingness to pay, current problems faced and other social economic information.

Sample size for household surveys was calculated using standard statistical formula to achieve a confidence level of 90% and an error margin of 10%. A total of 100 households were surveyed in the 3 areas covering all the towns. A systematic random sampling was chosen for its ability to produce maximally heterogeneous results.

Data was collected using other complimentary methods such as a household survey, key informant interviews, observations on the ground and study of previous literature views on the topic of water and sanitation issues in the study area. Results were coded and entered in Statistical Package for Social Sciences (SPSS) tool whereby the data was analyzed by running frequencies and cross tabulations.

The study was carried out between the month of April 2018 to mid May 2018 where by; 3 enumerators were selected based on their previous experience and a half day training program was given to them on the survey objective and methodology. Community leaders and other key stakeholders were consulted before starting the surveys.

14.4 SURVEY RESULTS

14.4.1 Demography and Education

The 2009 Kenya Population and Housing Census placed Kajiado County at 687,312 people with the male and female populations standing at 345,146 and 342,166 respectively. The County Government of Kajiado gives the annual population growth rate for the county as 5.5 percent with the population in 2017 projected to have been 999,819 people of which the male and female populations are 502,077 and 497,742 respectively. The total population estimates to be considered for this project are approximately 164,404 persons.

3% of the surveyed households belong to joint and extended families (where parents, children and other relatives live together) while 97% households are nuclear families. It was also evident that there are more male headed households (68%) than female headed at 32% respectively.

14.4.1.1 Literacy Status of the household heads

One of the outcomes of basic education is literacy that is the ability to read and write meaningfully in any language and understand. In the survey, respondents were asked about their highest level of education. Those household heads that had completed primary school and above were assumed to be literate.

Education levels of male headed households is higher than that of their counterparts hence most of the households heads whether male or female headed are able to read and write which translates to 20 % have attended primary school, 48% have gone through secondary school education and 48% have been to college or university as shown in the following table. The average size of each family is 4.

Table 14-1: Annual demand forecasted Education Details of the Surveyed Households

Level of Education		% of Highest level Education Attained
	Primary	20%
	Secondary	48%
	College/University	32%

	Total	100.0

14.4.2 Economic Status

The survey results indicate that there is a strong correlation between household income and some household characteristics such as gender, sector of employment, type of business and education attainment of the household head among others.

Business and trade are the main source of income for 66% of the households followed by employment at 27% and farming at 4%. Some of business activities engaged by the respondents are small scale trading in household goods, hardware, mpesa shops, saloons, water vending, and hotels among others. Most of the respondents did not have a secondary source of income but the few who had incorporated business and farming at 4% respectively. Primary sources of income are shown in the following**Errore. L'origine riferimento non è stata trovata.** table.

Table 14-2: Annual demand forecasted Primary Sources of Income

Main Primary Sources of Income		**% of household as a Main Source of Income**
	Business	66%
	Employed	27%
	Farming	4%
	Unemployed	3%

Asset ownership has a strong association with poverty levels. 80 % of households live in rented houses while 20% live in their own self built houses. 85% of the surveyed households were of permanent nature either built with stones or iron sheets and 13% reported to have temporary structures.

Average family income of the surveyed households is give in the Table 14-3**,** 47% of the surveyed households have an income of less than Ksh 20,000.The various sources of income are sourced from business activities, salaries , farming and small scale trading in the area.

Table 14-3: Annual demand forecasted Ranges of Households Income

Monthly Income (Ksh)		% of Monthly Income
	Less than 20,000	47%
	20,001 - 30,000	33%
	30,000 - 40,000	6%
	40,001 - 50,000	6%
	Above 50,000	2%

14.4.3 Agriculture and Food Security

The UN shows that agriculture accounts for 70% of world water use, with large quantities used for irrigation and production of both food and non-food crops. Water is key to food security as such crops and livestock need it to grow. The World Food Summit of 1996 defined food security as existing when all people , at all times, have physical, social and economic access to sufficient, safe and nutritious food to meet dietary needs for a productive and healthy life.

20% of the surveyed households reported to own land within the area and minimum acreage being reported to be 1 acre. Most of the households reported to practicing livestock keeping, poultry keeping, growing vegetable and other crops for own consumption and sale within the area.

Purchase from the market is the main source of food for about 84% of the households. Combination of market and sourcing food within the farms is the second most source of food (8%). About 2% of the households rely solely on their farm production. Various food sources shown in figure below.

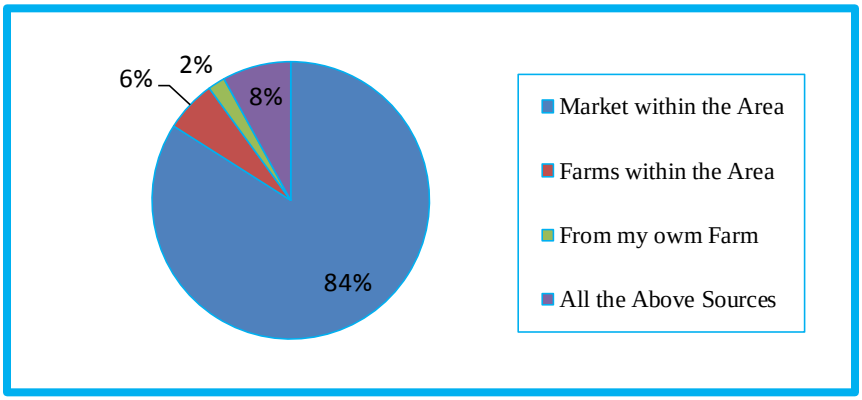


Figure 14.1: Sources of Food

Only 2% of the households reported to have 1 meal a day. 32 % households reported to have 2 meals per day and remaining 66% have three meals in a day. The responses to how often food shortage is experienced, 65% reported to have experienced food shortage sometimes while, 32% have never had cases of food shortage.

Reasons for food shortage are reported as harsh climatic conditions (16%), high food prices which entails high cost of living at 54% and 30% of households attributed the causes of food shortage as both high climatic conditions like drought and high cost of living. Water and livelihoods analysis therefore indicates that improved access to water at household level is not only a health issue but is a key determinant of food and livelihood security.

14.4.4 Status of Women

Despite their considerable contributions to both family income and rural economy, women continue to be faced with inhibitive cultural traditions relating to divisions of labor, lack of access to land and property, exclusion of women in decision making and restrictions on family inheritance. The result is that rather than being able to concentrate on activities that earn income, many women must spend the majority of their time undertaking domestic activities.

In the study are it was observed that women engaged in business and trade, agricultural labor especially in the many flower farms in the area, house work and other minor income generating activities.

Business and trade is the main economic activity (68%), followed by farming (6%), household activities at 5% and combination of farming and trade at 5% respectively. Ownership of property by women was reported to be 29% while women involvement in decision making matters in the household was more of taking care of the children (raising the kids), education and day to day running of the homestead was reported to be at 48% respectively.

14.4.5 Water Supply Service Levels

The Sustainable Development Goal 6 goes beyond drinking water, sanitation and hygiene among others at halving the proportion of the population without sustainable access to safe drinking water. The survey collected information on the main sources of water that households use

Various water sources of the surveyed households are listed below, 62% of the household reported to using borehole water, 18% sourced their water from public kiosks/ tap water, 8%

reported to have tap water inside their households while the rest sourced from other traditional sources such as rainwater (1%), canal, river and pond (3%), rainwater and canal at 3%.

Table 14-4: Annual demand forecasted Sources of Water

Main Sources of Water	% of water sources used
Rain Water	1%
Tap Water Inside House	8%
Public Tap Water (Water Kiosk)	18%
Canal,River,Pond	3%
Borehole	62%
Others	2%
Tap Water Inside House and Borehole	5%
Rainwater and Canal, River, Pond	1%

40% of the households have access to water less than 500m from their houses. Distance to water sources from the households is shown on Figure 14.2**.**

14.4.5.1 Distance to Source of Water

Bringing water closer to the households to reduce on the time taken to walk to safe water points is the ideal situation for the key players and stakeholders in the water sector. Long distances to water sources and long queues at even nearby water points mean that a lot of valuable time that would be spent on other activities is wasted in collection of water. The average distance people travel in search of water is approximately 10km from the homesteads. Water access in urban centres is better than in rural areas because of high water connectivity by the service providers (Kajiado CIDP 2013).

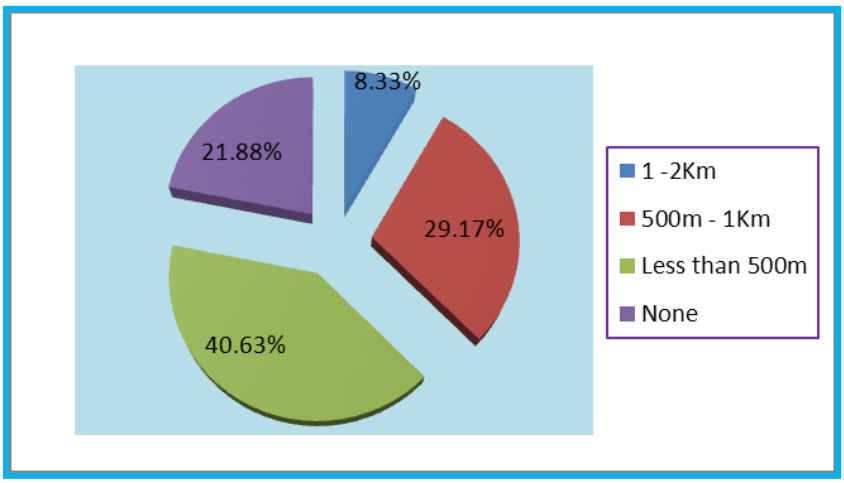


Figure 14.2: Distance covered to water source

Average time spent by the household to collect or fetch water is 40% for between 10 to 30 minutes, while 12% of the households reported to using an hour and above (more for the traditional sources)

The burden of water collection falls mostly on women. Adult females are mostly responsible for collection of water in 42% of households followed by water vendors (34% of households). Only 8% of adult males 14% schools going children are reported to be responsible collecting water.

14.4.6 Water Treatment

60% of the surveyed households think the drinking water is not safe and 20% think it is safe, while 20% are not sure if the water is safe or not for drinking. About 56% of the households further reported to be treating their drinking water. The various methods being used by these households to treat water are boiling at 41% for most households, addition of water guard at the households (22%), chlorine (12% whereby the water is treated by the service provider) and others reported to be buying treated water or purified water from the shops at 13%. Figure below shows representation of the various methods used for water treatment.

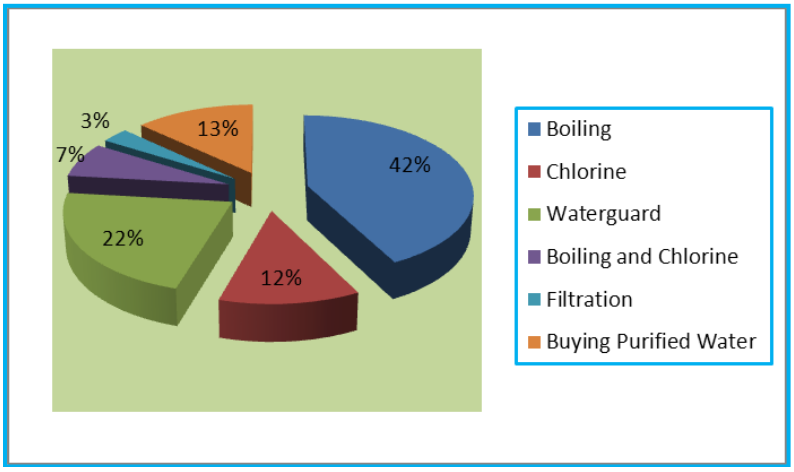


Figure 14.3: Water treatment methods

The average daily water consumption is shown in table below. The average daily water consumed is between 0 and 100 litres in 55% of the households, between 100 and 200 litres in 36% of the households, and between 200 to 500litres in 6% households.

Table 14-5: Annual demand forecasted Average Water Consumption per Household per Day

Water Quantity		% of households
	0 - 100litres	55%
	100 - 200litres	36.4%
	200 - 500litres	6.1%
	More than 500litres	3.0%
	Total	100

Water needs for various domestic uses were drinking and cooking, personal hygiene, washing clothes, utensils, floor, cattle and others such as gardening and building.

14.4.7 Preferences for Water Sources and Distance

95% of households want improvement in existing water supply and almost all of them would like to water connected in their households or within the compound. Regarding preferred water source, 97% preferred tap water connected to or within their households, 2% preferred more dams to be built while the rest (1%) preferred other sources.

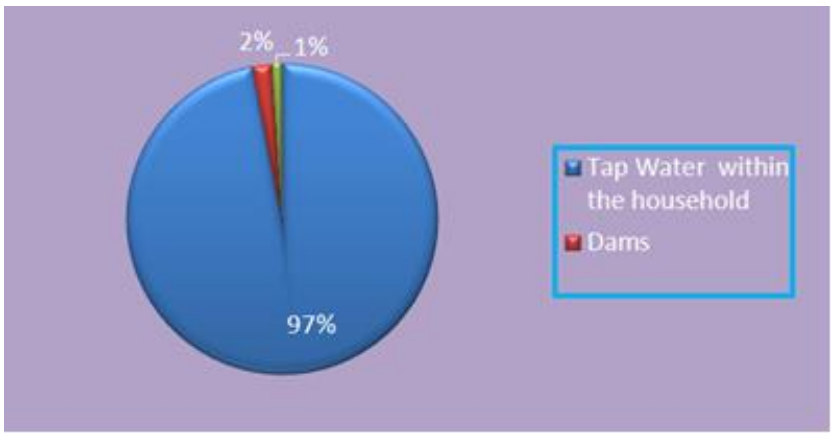


Figure 14.4: Preferred Water Source of Water Supply

14.4.8 Willingness to Pay for Water Supply

91% of the surveyed households are willing to pay for current water supply while 9% are not willing. In terms of better water supply 78% of respondents are willing to pay for better water services and are willing to contribute in labour for the project, and cash contribution.

Willingness to pay monthly charges towards improved water services are shown in the following table. 47% are willing to pay Ksh 100 to 500 per month for improved water supply and 23% are willing to pay less than Ksh 100 per month.

Table 14-6: Annual demand forecasted Willingness to Pay For Monthly User Charges For Improved Water Supply

Monthly Charges(Ksh)		% of household willing to pay for better water supply
	No Answer	20.4%
	Less than 100	22.6%
	100 - 500	47.3%
	500 - 1000	5.4%
	More than 1000	4.3%

	Total	**100**

Generally, all the households interviewed demonstrated a willingness and ability to pay for an improved service as long as there were good management practices in place. Moreover willingness to pay was more tied to reliability and continuous supply throughout the year.

14.4.9 Sanitation

Use of appropriate sanitation facilities is important in hygiene related illnesses like diarrhea, intestinal infections and cholera among others. According to the Kajiado County CIDP 2013 Report it describes sanitation issues in the county to be inadequate; Only 2,407 out of 87,120 urban households is connected to the main sewer while 17,157 use septic tanks and cesspools. In the rural areas, 44, 203 out of 86,344 households practice open defecation representing 50 percent of the total households in the county.

It is with this regard that the survey collected information about the use of toilet facilities and the actual type that the household used. Almost all the surveyed households confirmed to have access to a sanitation facility and sharing a toilet with other households was mentioned often. 60% of surveyed households use simple latrines, 29% use flush toilets and 10% of the households surveyed use a combination of simple latrines and flush toilets. Figure below shows they type of sanitation facilities used within the households.

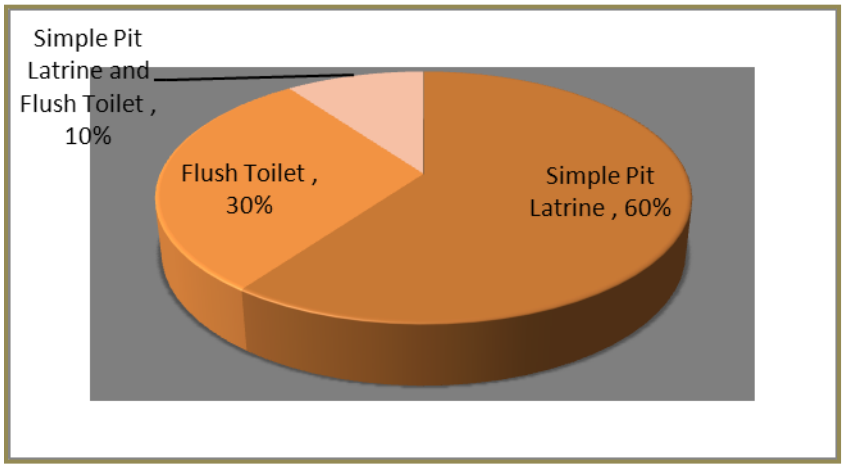


Figure 14.5: Type of existing sanitation facilities

Wastewater from the households is usually disposed in the latrines / toilet for most households at 22%, 19% have reported to using soak pits and 19% pour on the ground within the compound, while 17% use septic tanks respectively.

Collection of solid waste to dump at the dumping site and burning are the most common methods of solid waste disposal in 62% and 28% of the households respectively while the others reported use of the pits and roadside.

About 95% of the households are willing to pay for improved sanitation services. Some of the areas that they need improvement on are a better sewerage system (32%), improved drainage and designated dumping sites (49%).

14.4.10 Health

Poor hygiene and sanitation contribute to more than 80% of communicable diseases in many parts of Kenya (MOPHS 2016). The survey sought out to know the frequency of occurrence of water and sanitation related disease particularly, whether a household member fell sick in the last month (30 days) preceding the date of the survey. Diarrhea is said to be the most prevalent disease contracted by the households at 29%, followed by Typhoid 26%, Cholera 18% and finally Amoeba and Typhoid at 9% respectively. Table below shows a representation of the common water and sanitation related diseases among the households.

Table 14-7: Annual demand forecasted Common Water and Sanitation Diseases Reported by Households

Common Diseases	% of households
Diarrhea	29%
Typhoid	27%
Cholera	18%
Upper Respiratory Tract Infection	2%
Typhoid and Amoeba	9%
Others	13%
Total	**100**

At least 30% of the households contracted the diseases in the last six months. It was reported that children were the most vulnerable at 56%, followed by all other members of the household (38%).

14.4.11 Sources of information on Water, Sanitation and Hygiene

Common sources of information on water, sanitation and hygiene to the surveyed households are shown in figure below. 27% of households received information from Internet and another 27% confirmed to use general knowledge on issues of hygiene, while (14%) Television, (8%) health center, (4.1%) Radio while the rest are a combination of two or more of the said sources.

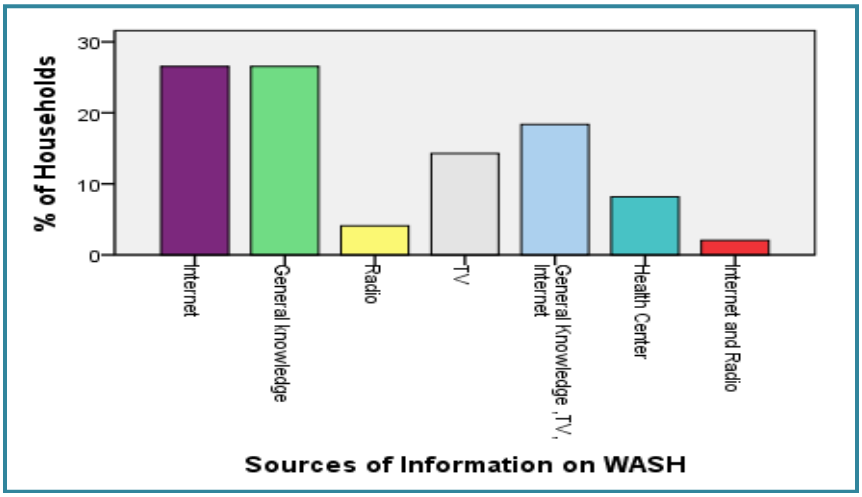


Figure 14.6: Common Sources of Information on Water, Sanitation and Hygiene

15 FINANCIAL AND ECONOMIC ANALYSIS

15.1 INTRODUCTION TO FINANCIAL ANALYSIS

The financial and economic analysis contained in this feasibility study follows the incremental method. The project is analysed based on data locally collected and on the difference of the costs and revenues between the baseline situation (without the project) and the investment situation (with the project). Therefore, all incremental costs and benefits following implementation of the project will be quantified.

With regards to the methodology, the analysis focuses on two main project components and their design options.

1) Dams;
2) Water supply network;

As total, it evaluated 5 alternatives of Dams, 1 alternative of Water supply network.

Therefore, the analysis gives evidence on a system composed by Dam, water supply network.

In order to select the most appropriate system, it recommends using both financial criteria and socioeconomic indicators.

In this sense, a table (multi-criteria analysis) placed at end of the paragraph may drive the decision-making process underling all main project costs and benefits from different perspectives.

15.2 FINANCIAL COSTS

Summary of capital costs and operation costs for project components is detailed into the paragraph.

A) Construction Costs

A.1 Dams

According to the project design and its evaluation, we have two options for each dam site proposed. The figure summarises the capital costs for the dams and its option.

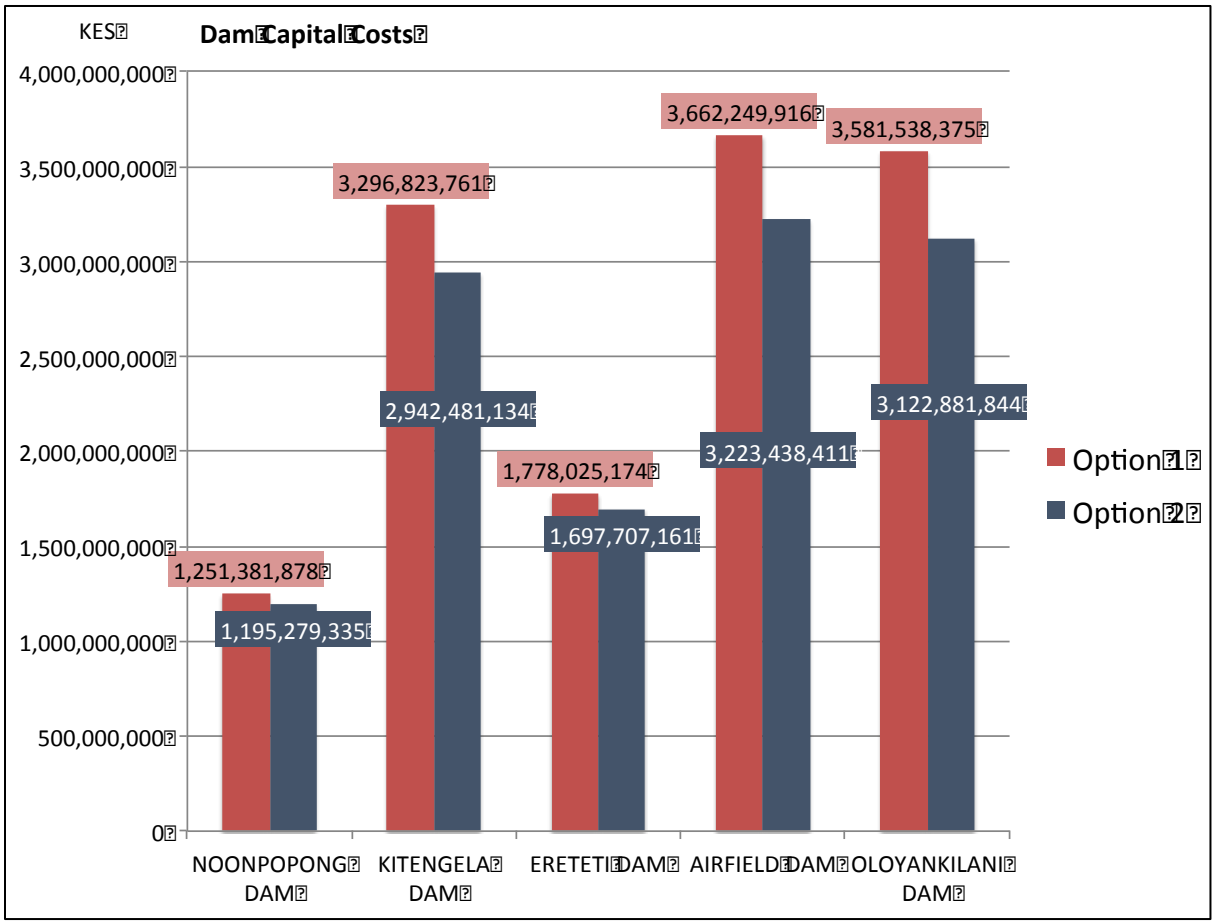


Figure 15.1: Dam Capital Costs – Consultant evaluation.

B.1 Water Supply network

According to the project design features, the water supply network capital costs calculation takes into consideration all main components: water supply network - main distribution, pumping station, and drinking water treatment plant.

The figure below summarises the total capital costs including all above components.

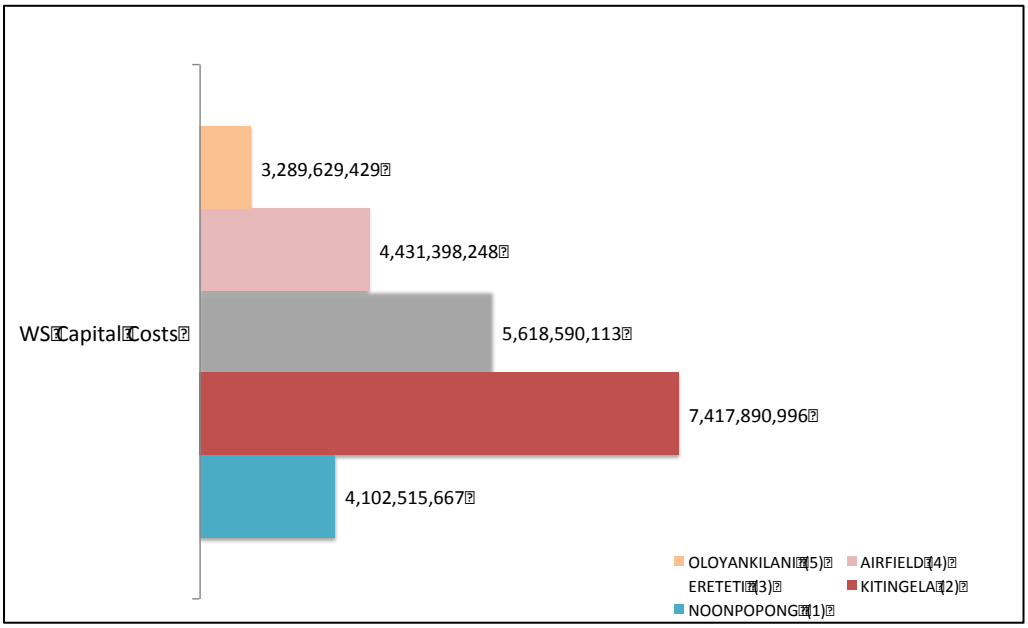


Figure 15.2: Water supply Capital Costs – Consultant evaluation.

B) Operation and Maintenance

The Operation and Maintenance costs are evaluated based on data locally collected and parametric estimation.

On following the tables underlines the average annual O&M cost for each dam option, including the water supply system.

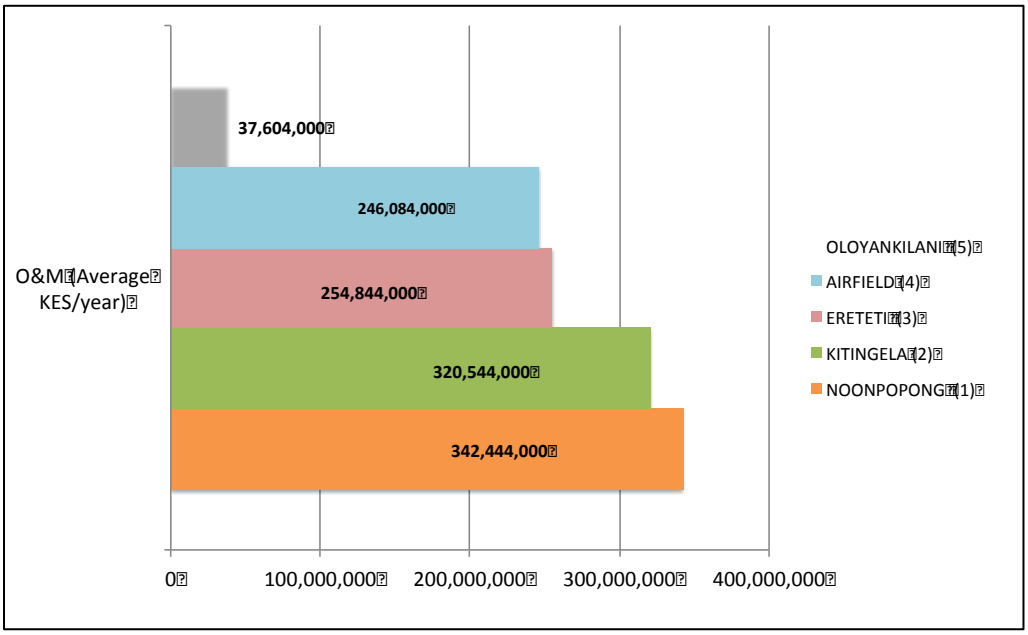


Figure 15.3: Operation and Maintenance costs Dams and Water Supply system – Consultant evaluation.

15.3 FINANCIAL REVENUES

Financial revenues are estimated based on per capita consumption forecasted in in the present (2018), Initial (2022), Future (2032) and Ultimate (2042) design years respectively.

Therefore, it focussed on the water demand future growth in order to estimate the capacity of each project option to raise revenue and cover the operating and maintaining costs.

To validate the analysis of revenue, the data surveyed gives evidence on people willingness to pay more for a better water service in town.

Following the assumptions of hedonic price[2], it may argue that people living in targeted project areas expressed the willingness to charge more.

Table 15-1: Annual demand forecasted

Consumer	**Component**	**Tariff (m3)**	**Annual Water Demand (m3/year)**			
			Present Year (2018)	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
Domestic (Urban)	High Class Housing	55	579,985	784,385	1,645,420	2,936,790
	Medium Class Housing		3,481,005	4,707,040	9,871,790	17,621,105
	Low Class Housing (IC)		156,585	211,700	444,205	792,780
	Low Class Housing (NC)		375,950	508,445	1,066,165	1,903,110
Domestic (Rural)	Medium Potential (IC)	45	15,330	19,710	74,825	238,710
	Medium Potential (NC)		41,245	53,655	89,790	107,310
Schools	Boarding Schools	50	436,175	594,220	1,178,585	2,146,565

2 In regards to the theoretical foundations, the hedonic model is based on Lancaster's (1966) theory of consumer's demand. He recognized composite goods whose units are homogeneous, such that the utilities are not based on the goods themselves but instead the individual "characteristics" of goods—its composite attribute. Thus, the consumers make their purchasing decision based on the number of good characteristics as well as per unit cost of each characteristic. Recently, this postulations are extend to public goods.

	Day Schools with WC		240,170	324,485	630,355	1,124,930
	Day Schools without WC		5,475	7,300	13,870	24,820
Health Facility		55	33,580	40,880	71,175	116,435
Hotels	High Class	65	22,995	31,025	65,335	116,435
	Medium Class		11,680	15,695	32,850	58,400
	Low Class		1,460	1,825	4,015	6,570
	Unclassed		40,880	55,480	116,070	207,320
Bars and Restaurants		65	27,740	37,230	78,110	139,065
Shops	Trading Centres	65	1,825	2,555	5,475	9,490
	Retail Traders		31,755	42,705	89,790	160,235
	Wholesale T raders		31,025	40,880	79,570	131,765
	Megastores		9,125	12,410	25,550	44,895
Administrative Offices		65	21,900	29,565	61,685	110,230
Industries	Manufacturing	65	185,785	250,390	522,680	928,560
	Bakeries		2,920	4,015	8,760	15,330
	Jua Kali Artisans		1,460	1,825	3,650	6,935
Livestock Unit		50	252,215	340,910	756,645	1,209,245
Crop Irrigation		45	2,172,115	2,640,410	4,300,795	5,242,860
TOTAL			8,180,380	10,758,740	21,237,160	35,399,890

This result gives recommendations on tariff policy and thus willingness of people to pay for a public good. On the other side, the financial analysis is focussed on local market water supply tariff structure.[3] The table above summarised the forecasted water demand for each project milestone. This table remarks the maximum revenues can be reached according to tariffs and data consumptions assumptions.

With this regards, our analysis uses these numbers, as benchmarks to estimate project option would be more profitable and sustainable.

In accordance with estimation, the table below give evidence on the average water volume released for each option and its percentage compared to the water demand forecasted.

[3] Available from Athi water service board.

Table 15-2: Water provided vs water forecasted.

Water provided vs Water forecasted					
DAM	Average Annual Water (m3)	Present Year (2018)	Initial Year (2022)	Future Year (2032)	Ultimate Year (2042)
NOO POPONG DAM	1,800,000	22%	17%	8%	5%
KITENGELA DAM	16,000,000	196%	149%	75%	45%
ERETETI DAM	4,800,000	59%	45%	23%	14%
AIRFIELD DAM	3,700,000	45%	34%	17%	10%
OLOYANKILANI DAM	700,000	9%	7%	3%	2%

As shown, Kitengela dam is the only infrastructure abled to reach valuable percentage of water forecasted demand satisfied, reaching more than 100% of forecasted water demand. This percentage might be higher if it considers only three main demand segments: domestic, health and school as shown in table.

Table 15-3: water provided vs water forecasted for primary uses

Water provide vs Water forecasted for priorital uses					
DAM	**Average Annual Water (m3)**	**Present Year (2018)**	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
NOOPOPONG DAM	1,800,000	34%	25%	12%	7%
KITENGELA DAM	16,000,000	298%	221%	106%	59%
ERETETI DAM	4,800,000	89%	66%	32%	18%
AIRFIELD DAM	3,700,000	69%	51%	25%	14%
OLOYANKILANI DAM	700,000	13%	10%	5%	3%

The table below is related to the potential revenue as benchmark in case of all water demand will be satisfied.

Table 15-4: Potential Revenue

Potential annual Revenue (KES/year)					
Consumer	**Component**	**Present Year (2018)**	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
Domestic (Urban)	**High Class Housing**	31,899,175	43,141,175	90,498,100	161,523,450
	Medium Class Housing	191,455,275	258,887,200	542,948,450	969,160,775
	Low Class Housing (IC)	8,612,175	11,643,500	24,431,275	43,602,900
	Low Class Housing (NC)	20,677,250	27,964,475	58,639,075	104,671,050
Domestic (Rural)	Medium Potential (IC)	689,850	886,950	3,367,125	10,741,950
	Medium Potential (NC)	1,856,025	2,414,475	4,040,550	4,828,950
Schools	Boarding Schools	21,808,750	29,711,000	58,929,250	107,328,250
	Day Schools with WC	12,008,500	16,224,250	31,517,750	56,246,500
	Day Schools without WC	273,750	365,000	693,500	1,241,000
Health Facility		1,846,900	2,248,400	3,914,625	6,403,925
Hotels	High Class	1,494,675	2,016,625	4,246,775	7,568,275
	Medium Class	759,200	1,020,175	2,135,250	3,796,000
	Low Class	94,900	118,625	260,975	427,050
	Unclassed	2,657,200	3,606,200	7,544,550	13,475,800
Bars and Restaurants		1,803,100	2,419,950	5,077,150	9,039,225

Shops	Trading Centres	118,625	166,075	355,875	616,850
	Retail Traders	2,064,075	2,775,825	5,836,350	10,415,275
	Wholesale T raders	2,016,625	2,657,200	5,172,050	8,564,725
	Megastores	593,125	806,650	1,660,750	2,918,175
Administrative Offices		1,423,500	1,921,725	4,009,525	7,164,950
Industries	Manufacturing	12,076,025	16,275,350	33,974,200	60,356,400
	Bakeries	189,800	260,975	569,400	996,450
	Jua Kali Artisans	94,900	118,625	237,250	450,775
Livestock Unit		12,610,750	17,045,500	37,832,250	60,462,250
Crop Irrigation		97,745,175	118,818,450	193,535,775	235,928,700
	TOTAL REVENUE	**426,869,325**	**563,514,375**	**1,121,427,825**	**1,887,929,650**

The evidence below compares the total revenue estimated in case all water demand will be satisfied with the expected revenues generated from the water will be provided by each dam option.

Table 15-5: Revenue Comparison among the project dam options

	Present Year (2018)	**Initial Year (2022)**	**Future Year (2032)**	**Ultimate Year (2042)**
DAM	(KES)			
TOTAL REVENUE (All water demand satisfied)	**426,869,325**	**563,514,375**	**1,121,427,825**	**1,887,929,650**
NOOPOPONG DAM	93,927,762	94,279,244	95,048,965	95,996,721
KITENGELA DAM	426,869,325	563,514,375	844,879,692	853,304,188
ERETETI DAM	250,474,032	251,411,318	253,463,908	255,991,256
AIRFIELD DAM	193,073,733	193,796,224	195,378,429	197,326,594
OLOYANKILANI DAM	36,527,463	36,664,150	36,963,487	37,332,058

15.4 RESULTS OF FINANCIAL ANALYSIS

For each of projects options identified in the project area we will calculate the project's financial profitability based on the investment costs, the operating and maintenance costs and the revenue over the project horizon, i.e. the project's cash flow. The financial profitability is best assessed in terms of the project's net present value (NPV). The NPV is the difference between the present value of cash inflows and the present value of cash outflows and is used in capital budgeting to analyse the profitability of a projected investment or project. When the cash flow is discounted (with the weighted average cost of capital) back to a net present value, we obtain the project's NPV[4].

[4] The discounted rate calculated for our analysis is based on the interest rate used for twenty five year fixed coupon treasury bond issue from Central Bank of Kenya.

The main results of the financial analysis (see annexures) can be summarised as it follows:

- Kitingela dam is the only option abled to cover the higher percentage of water demand expected by the forecast. In these terms, it will be cover 100% (2018); 100%(2022); 75% (2032); 45% (2042).
- If we give the priority to satisfy the demand for domestic, school and health as project priority Kintegala dam will be abled to cover 100% (2018); 100%(2022); 100% (2032); 59% (2042).
- In terms of operating and maintenance costs, the most suitable options are Kitingela and Ereteti. For all these options, the net cash flow is positive according to the demand and tariff considered as assumptions.
- The Net Present Value for Kitingela and Ereteti net cahs flow is positive considering two-discounted rate as parameters (8%; 13%).

15.5 INTRODUCTION TO ECONOMIC COSTS

15.5.1 Economic Costs

The economic benefits of the project investment will accrue to the population living in the service area. They will benefit from the water supply and sewerage capacity and service and an improved sanitation situation.

A number of direct and indirect economic benefits to the current and future society and population will follow. In both townships and its satellite towns, these are considered to be as follows:

- Economic growth in the service area
- Employment opportunities generated (directly and indirectly)
- Development of small local enterprises
- Improvement of health conditions and reduced water borne diseases
- Avoided health care costs
- Improvement of living conditions for the population increases in value of land

The economic costs following the implementation of an investment project can be due e.g. to: resettlement costs if people have to be moved to new locations, inconveniences to households

as well as commercial and industrial entities in connection with rehabilitation/construction activities. These are quantifies and monetised based on data locally collected.

15.5.2 Relocation and resettlement costs

World Bank suggests that:

"Involuntary resettlement and land acquisition will be avoided where feasible, or minimized, by exploring all viable alternatives. (ii) Where involuntary resettlement and land acquisition is unavoidable, resettlement and compensation activities will be conceived and executed as sustainable development programs, providing sufficient investment resources to give the persons displaced by the project the opportunity to share project benefits. Displaced and compensated persons will be meaningfully consulted and will have opportunities to participate in planning and implementing resettlement and compensation programs. (iii) Displaced and compensated persons will be assisted in their efforts to improve their livelihoods and standards of living or at least to restore them, in real terms, to pre- displacement levels or levels prevailing prior to the beginning of the project implementation, whichever is higher"[5].

This is the basic policy principle of a relocation and resettlement program. It is strongly recommended that both Kenya laws and the infrastructure project should be adhered to the statement mentioned.

In these terms, the project will follow this guideline in order to *give the persons impacted by the project the opportunity to share project benefits in the long run.*

The tables summarises the social costs related to expected resettlement impacts for each of dam site option.

[5] Source: Resettlement policy framework: Regional communication Infrastructure program (RCIP) phase 1 of 5th Feb 2007.

Table 15-6: Summary Relocation and resettlement costs- Kitengela - Kajiado

OPTION	**Dam Sites and Drinking Water Treatment Plants**	**Estimated Land Size in Ha.**	**Estimated Land Acquisition Costs (Ksh)**
1	Kitengela River Site and DWTP	103	Ksh.2Billion
2	Noo Popong River Site and DWTP	98	Ksh.200M
3	Ereteti River Site and DWTP	172	Ksh .3.4Billion
4	Olooloitikosh River /Airfield Site and DWTP	128	Ksh.2.5 Billion
5	Ololyankilani Site and DWTP	41	Ksh. 200M
OPTION	**Waste Water Treatment Plant**	**Estimated Land Size in Ha.**	**Estimated Land Acquisition Costs (Ksh)**
	KITINGELA		
1	Kinanie WWTW Site	37.14	Ksh.900M
2	NoonKopir WWTW Site	17.94	Ksh.600M
3	Stoney Athi WWTW Site	21.65	Ksh.900M
OPTION	**Waste Water Treatment Plant**	**Estimated Land Size in Ha.**	**Estimated Land Acquisition Costs (Ksh)**
	KAJADO		
	Iltareto WWTW Site - Option 1	16.25	Ksh.300M

	Kajiado Town WWTW Site – Option 2	9.38	Ksh.275M
	Mabatini WWTW Site – Option 3	7.4	Ksh.165M

Source: local market data[6]

15.6 ECONOMIC BENEFITS

As part of implementing this project in Kitingela and Kajado, this chapter presents economic benefits that accrue to persons within and around these project areas.

To estimate the benefits generated by the project a literature review from official sources supported the analysis, which is integrated by the results of the survey.

In this framework the main issues investigated are related to (1) health sector; (2) patient benefits, and (3) other benefits.

The following benchmarks table is extracted by official reports and survey conducted.

[6] All the costs above are as per the current socio – economic conditions hence they may vary in the future.

Table 15-7: Project Benefits

Benefit by sector	**Variable**	**Data source**	**Data Value (+ range)**
1.Health sector			
Direct Expenditures avoided, due to less illness from diarrhoeal disease	Unit cost per treatment	World Health Organisation regional unit cost data	US$ 4.3-9.7 (Cost/ visit) US416.1-39. (cost/day) Varying by WHO region
	Hospitalisation	World Health Organisation data	91.8% of cases ambulatory 8.2% of cases hospitalized
2. Patients			
Benefit by sector	**Variable**	**Data source**	**Data Value (+ range)**
Direct expenditures avoided, due to less illness from diarrhoeal disease	Transport cost per visit	Survey	US$ 0.70 per visit
Income gained, due to days from work avoided	Days off work/ episode	Expert opinion	2 days (1-4)
Days of school absenteeism avoided	Absent days/episode	Expert opinion	3 (1-5)
Productive parent days lost avoided, due to less child illness	Days sick	Expert opinion	5 (3-7)

	Opportunity cost of time	World Bank data	50% minimum wage rate
3. Other benefits			
	Variable	**Data source**	**Data Value (+ range)**
Time spent to collect water	Hours	Survey	30 minutes a day

15.7 ECONOMIC COST-BENEFIT ANALYSIS

At the global level, most water and sewerage interventions have yielded higher benefits compared to associated Economic costs. In these studies, the costs of the interventions included the full investment and annual running costs. The benefits of the interventions included time savings associated with better access to water and sanitation facilities, the gain in productive time due to less time spent ill, health sector and patients' costs saved due to less treatment of diarrhoeal diseases. The results show that all water and sanitation improvements were found to be cost-beneficial, and this applied to all world regions.

In developing regions, the return on a US$1 investment was in the range US$5 to US$28[7]. The main contributor to benefits was the saving of time associated with better access to water supply and sanitation services. When different cost and benefit assumptions were used, the cost-benefit ratios changed considerably, but even under pessimistic scenarios the potential economic benefits generally outweighed the costs.

[7] Source: World Bank

The cost-benefit ratio of water and sanitation interventions is high when all benefits are included, standing at between US$5 and US$11 economic benefit per US$1 invested for most developing world sub-regions. In brief, the calculation of the total societal economic benefit is the sum of:

- Health sector benefit due to avoided illness
- Patient expenses avoided due to avoided illness
- Value of time savings due to access to water and sanitation
- Value of productive days gained of those with avoided illness
- Value of days of school attendance gained of those with avoided illness
- Value of child days gained of those with avoided illness

15.8 ANALYSIS CONCLUSIONS

To evaluate the available options, best international standards recommend the use of both financial and socioeconomic parameters. The table below summarises all main driver abled to lead the selection of the best option, considering the future population demand and the relevant effectiveness, sustainable and equity criteria.

In this sense, the main items included in the table are:

- Construction Costs;
- Average Operation and Maintenance costs;
- Net Present Value;
- Environmental and Resettlements costs;
- Percentage of water demand satisfied;
- Revenue Generated.

It is worth to point out that the project in subject and the water provided by the foreseen hydraulic infrastructure will generate positive externalities in terms of health and social wellness for the people living in the project areas.

According to international benchmarks and recommendations from World Health Organisation, it is possible to resume the main project externalities as follows:

- Direct Expenditures avoided, due to illness diarrhoeal disease reduction: US$ 4.3-9.7 as Unit cost per treatment.
- Income gained, due to the reduction of day-off due to disease: 2 days (1-4) as days off work/ episode.
- Days of school absenteeism avoided: 3 (1-5) as absent days/episode.
- Productive parent days lost avoided, due to less child illness: 50% minimum wage rate8.
- Time spent every day to collect water: about 30 minutes/day9.

In developing regions, the return on a US$1 investment was in the range US$5 to US$28 (Source: World Bank). This return is related to (1) health sector benefits + (2) Patient expenses avoided + (3) Value of time saving due to access to water and sanitation + (4) Value of productive days gained.

The table below give evidence on the community benefits monetised in accordance with World Bank data and estimated annual based for each of project option.

Table 15-8: Project Benefits – Monetised

		Economic Costs KES (KES/Year)	Benefits (KES/year)	Benefits/Population (Annual, KES)	Benefits/Population (2042, KES)
1	NooPopong	6,666,667	3,360,000,000	1,904	4,291
2	Kitengela	66,666,667	33,600,000,000	19,039	42,907
3	Ereteti	113,333,333	57,120,000,000	32,366	72,942
4	Olooloitikosh	83,333,333	42,000,000,000	23,799	53,634
5	Ololyankilani	6,666,667	3,360,000,000	1,904	4,291

[8] World Bank benchmark.
[9] Own Survey.

15.9 MULTI-CRITERIA ANALYSIS AND OUTCOMES

Following the Terms of References, the Consultant has adopted a Multi-criteria analysis with the aim to support the decision-making process of the Client in prioritizing project options according to engineering, financial, socio-economic and environmental benchmarks.

In doing so, World Bank criteria was used.[10]

As project screening and ranking procedure, multi-criteria analysis takes into account eight main indicators and gives a final score for each of the evaluated options.

The indicators are:

1. **Capital costs:** all construction costs for all project components (dam and water supply system, including pumping stations and drinking water treatment plant).
2. **Operation & Maintenance costs:** opex costs for all project components (O&M).
3. **Resettlement cost:** It regards the costs due to the impact to the population affected by the project, to be resettled for the infrastructure construction.
4. **Community Benefits:** As mentioned into the relevant section, each water infrastructure will benefit the population in terms of (1) health sector benefits + (2) illness expenses avoided + (Value of time saving due to access to water and sanitation + (4) Value of productive days gained. All these items are evaluated by using World Bank benchmarks.
5. **Financial Sustainability:** It considers the capacity of the infrastructure to generate financial revenue abled to cover annual operation and maintenance costs.
6. **Water Demand satisfied:** It considers the percentage of water demand covered by each option.
7. **Site Suitability for Dam Construction:** It considers the availability of construction materials nearby the dam site, site accessibility and site surroundings.

10 http://documents.worldbank.org/curated/en/540451468154490419/Where-to-use-cost-effectiveness-techniques-rather-than-cost-benefit-analysis

8. **Environmental Factors:** It takes into account the proposed project impacts on endemic plants and animals, effects on wildlife dispersal corridors and dispersal areas, as well as their effects on conservation status of the land they are proposed to be built.

As recommended by the World Bank, all the selected indicators shall be weighted in terms of percentage. A project team composed by dam engineers, hydraulic engineers, socioeconomic, environmentalist, natural resource manager and financial experts evaluated each option referring to each indicator. The said experts gave a "score" to each indicator selecting among three possible options (0-low; 0.5 Medium; 1 High).

The outcomes of such evaluation and analysis remark that Kintegela and Ereteti site options are the most suitable and present the major benefits for the involved population. They present the best indicators from social, engineering and financial perspectives.

In these terms, the ranking is (from the higher score to lower):

v. **Kitengela (70%);**
vi. Ereteti (60%);
vii. Noopopong (35%);
viii. Oloyankilani (30%);
ix. Airfield (20%).

Considering the foreseen water demand and the reservoir capacity of the high ranked sites (Kitengela and Ereteti), only Kintegela site option should be considered as the most recommended.

In the table below are reported the due details regarding the **multi-criteria analysis** carried out by the project team.

Table 15-9: Multi-criteria Analysis

			Score (0-low; 0.5 Medium; 1 High)					Ranking				
	CRITERIA	**Weighting**	**NOOPOPONG**	**KITENGELA**	**ERETETI**	**AIRFIELD**	**OLOYANKILANI**	**NOOPOPONG**	**KITENGELA**	**ERETETI**	**AIRFIELD**	**OLOYANKILANI**
1	Capital Cost	**15%**	1	0.5	1	0	0	15.0%	7.5%	15.0%	0.0%	0.0%
2	O&M	**10%**	0	0	0.5	0.5	1	0.0%	0.0%	5.0%	5.0%	10.0%
3	Resettlement Costs	**10%**	1	0.5	0	0.5	1	10.0%	5.0%	0.0%	5.0%	10.0%
4	Community Benefits	**10%**	0	1	1	0.5	0	0.0%	10.0%	10.0%	5.0%	0.0%
5	Financial Sustainability	**15%**	0	1	0.5	0	0	0.0%	15.0%	7.5%	0.0%	0.0%
6	Water demand satisfied	**25%**	0	1	0.5	0	0	0.0%	25.0%	12.5%	0.0%	0.0%
7	Site Suitability for Dam Construction	**5%**	1	0.5	1	0	0	5.0%	2.5%	5.0%	0.0%	0.0%
8	Environmental effect	**10%**	0.5	0.5	0.5	0.5	1	5.0%	5.0%	5.0%	5.0%	10.0%
	TOTAL	**100%**						**35.0%**	**70.0%**	**60.0%**	**20.0%**	**30.0%**

15.10 FEASIBILITY STUDY - CONCLUSIONS AND RECOMMENDATIONS

Considering all the above considerations, analysis, proposals and studies carried out, in this paragraph Consultant briefly resume conclusions and recommendations of the present Feasibility Study Report, and more in detail:

- ✓ **Legal, Social and Environmental feasibility:**

All the five dam sites are viable from legal, social and environmental point of view. The same can be declared for the treatments ponds proposed.

- ✓ **Technical Feasibility:**

All the five dam sites present an acceptable technical complexity, and they shall be considered technically suitable. Both the sewerage and potable water networks are feasible and they do not present great complexity. Special attention shall be dedicated to the energy consumption reduction strategy in order to reduce the operation and maintenance costs of the whole infrastructure.

- ✓ **Financial and Economic Feasibility:**

The five dam sites assessed present different financial and economic indicators, and more in detail:

- Kitengela dam site presents the best indicators, considering a construction cost around 96.0 KSH per m3 of reservoir water capacity, and considering the great quantity of water that could be harvested.

- Oloyonkilani dam site presents the worst indicators, considering a construction cost around 800.0 KSH per m3 of reservoir water capacity and considering the extremely reduced reservoir capacity.

- ✓ **Suggested Dam Site**

Considering the main goals of the present project, and considering all the all the above mentioned reasons, **Consultant recommend to proceed with Kitengela Dam site (34.00 Mm^3 reservoir capacity) detailed design and construction.**

Considering that construction time-frame is estimated in about 7 (seven) years, Consultant suggest to proceed with a new assessment of the «real water needs» of the project area during the year 2025. If population grow-rate will be confirmed, Consultant suggest to proceed with the design & construction of one additional dam, Erreteti Dam (5.00 Mm^3), in order to better cover the population water needs.

It is worth to point out that the proposed "new" Erreteti dam will be hydraulically connected to the Drinking Water Treatment plant already provided for Kitengela dam, no additional cost will be necessary for other infrastructures.

- ✓ **Wastewater Treatment**

With regards to the method of wastewater treatment, **a waste stabilization ponds (WSP) system is recommended for both towns due to its low operation and maintenance cost, and the proven effectiveness in the local climate.** The capital cost of buying land required for WSPs by far outweighs the operational and maintenance cost of the other alternatives considered. Together with the recommended WSP system, comprehensive inlet works are recommended for both townships. The inlet works will comprise coarse and fine screens, grit removal mechanisms and flow measuring mechanisms as well as emergency by-passes. Additionally, in order to supplement the available water for irrigation from the proposed dam, the Consultant recommends the use of treated wastewater for unrestricted irrigation.

- ✓ **Kitengela Town Sewerage Project**

The Consultant proposes a centralized sewerage system as opposed to a decentralized one as discussed within this feasibility report. The centralized system will have its waste water treatment plant at the Kinanie Area of Machakos County, immediately next to the existing Athi River EPZA Sewage Treatment Plant. The Consultants further proposes phasing of the sewerage project as follows;

Phase I – This will entail the following:

Acquisition of all 75 Hectares required for the present and future construction of the waste water treatment plants.

Construction of an 18,250m3/d waste stabilization ponds system.

Construction of Trunk Sewers up to the currently settled areas.

Construction of Reticulation Sewers in the already densely settled areas of the project area. These areas include the main Kitengela Business Center, Milimani area, Noonkopir, residential area bordering the EPZA and the Yukos area.

Phase II – This will entail construction of the following;

An additional 18,250 m3/d waste stabilization ponds system.

Extension of the Imambaraswani River Trunk Sewer and Kitengela River Trunk Sewer to cover the additional areas currently not settled but predicted to be settled in the future.

Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions. These areas include the area south west of the project area along the Imambaraswani river and the area south (upstream) of Kitengela River.

- ✓ **Kajiado Town Sewerage Project**

A centralized sewerage system is also proposed for Kajiado Town, with a waste stabilization ponds (WSP) system being recommended as the cost of land is relatively low and the energy cost is very high required for the other technologies.

The Consultant proposes that the sewerage project in Kajiado be implemented into two (2) phases as follows;

Phase I – This will entail the following;

Acquisition of all Thirty (30) Hectares required for the present and future construction of the waste water treatment plants.

Construction of a 7,350 m3/d waste stabilization ponds system.

Construction of the 11.9km Olkejuado River Trunk Sewers.

Construction of the sub-trunk sewers and Reticulation Sewers in the already densely settled areas of the project area.

Phase II – This will entail construction of the following;

An additional 7,350m3/d waste stabilization ponds system.

Extension of the Olkejuado River Trunk Sewer upstream to cover the additional areas next to the airstrip that is currently not settled but predicted to be settled in the future.

Extension of Reticulation Sewers in the other areas not covered by Phase I including newly settled areas that can be served by the added trunk sewers extensions.

- ✓ **Sanitation in peri-urban and rural Areas within the Project Areas**

It is proposed to use on-site sanitation facilities in the peri-urban and rural areas within the project that are not covered by the sewerage networks as designed. These facilities include:

i. Pit Latrines in Rural Areas,

ii. Ventilated Pit Latrine in Rural Areas with higher Population,

iii. Pour Flush Latrines in Institutions in Rural Areas,

iv. Septic Tanks and Package Waste Water Treatment Plants in Peri-Urban Areas.

Where soil drainage capability is poor, use of advanced package plants is recommended.

- ✓ **Recommended "minimum tariff" for potable Water supply.**

As requested by the Client, this paragraph will try to assess what is the minimum tariff for every cubic meter of water that the above recommended hydraulic system will provide to the customer within the project area.

Considering that:

- ✓ The **overall construction costs** of the recommended option can be presently estimated **between 18,982,125,344.00 KES and 16,454,512,082.00 with an average construction cost around 17.75 Billion of KES;**
- ✓ The **operation and maintenance costs** have been estimate equal to about **320,500,000.00 KES per year;**
- ✓ The **average volume of water** that can be distributed and sell to customers is estimated around **16 Mm3 per year,** to be reduced of about the 20% in order to consider the volumes of water for crops irrigation and water for livestock;
- ✓ The **life cycle (full life)** of the overall infrastructure can be evaluated in about **30 years**.

For all the above mentioned reasons ,the minimum tariff that shall be considered in order to fully recovery in 30 years the over all hydraulic infrastructure capital and operational expenditures, it has been evaluated in about **75.00 KES per each m^3 of water** to be sell to the customers.

15.11 ANNEXURE

Appendix (A) STUDY Summary

Option1									
SUMMARY	FINANCIAL INDICATORS		ECONOMIC COSTS		ECONOMIC BENEFITS				
INDICATORS	CAPITAL COSTS	AVERAGE MAINTENANCE COSTS PER YEAR	Environmental and social mitigation measure	RESETTLEMENT COSTS	Forecasted water demand m3/year (2042)	% water demand satisfied (2042)	% water demand satisfied for domestic, school and health scope (2042)	Revenue forecasted (all forecasted demand satisfied)	Revenue generated (2042)
Project components									
NOOPOPONG DAM (1)	1,251,381,878	342,444,000	52,300,000	2,000,000,000	35,399,890	5%	7%	2,347,889,875	95,996,721
Water supply network	4,102,515,667								
KITENGELA DAM (2)	3,296,823,761	320,544,000.00	52,300,000	200,000,000	35,399,890	45%	59%	2,347,889,875	853,304,188
Water supply network	7,417,890,996								
ERETETI DAM (3)	1,778,025,174	254,844,000.00	52,300,000	3,400,000,000	35,399,890	14%	18%	2,347,889,875	255,991,256
Water supply network	5,618,590,113								
AIRFIELD DAM (4)	3,662,249,916	246,084,000.00	52,300,000	2,500,000,000	35,399,890	10%	14%	2,347,889,875	197,326,594
Water supply network	4,431,398,248								
OLOYANKILANI DAM (5)	3,581,538,375	37,604,000	52,300,000	200,000,000	35,399,890	2%	3%	2,347,889,875	37,332,058
Water supply network	3,289,629,429								

Option1									
SUMMARY	FINANCIAL INDICATORS		ECONOMIC COSTS		ECONOMIC BENEFITS				
INDICATORS	CAPITAL COSTS	AVERAGE MAINTENANCE COSTS PER YEAR	Environmental and social mitigation measure	RESETTLEMENT COSTS	Forecasted water demand m3/year (2042)	% water demand satisfied (2042)	% water demand satisfied for domestic, school and health scope (2042)	Revenue forecasted (all forecasted demand satisfied)	Revenue generated (2042)
Project components									
NOOPOPONG DAM (1)	1,251,381,878	342,444,000	52,300,000	2,000,000,000	35,399,890	5%	7%	2,347,889,875	95,996,721
Water supply network	4,102,515,667								
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Water supply network	7,417,890,996								
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Water supply network	5,618,590,113								
AIRFIELD DAM (4)	3,662,249,916	246,084,000.00	52,300,000	2,500,000,000	35,399,890	10%	14%	2,347,889,875	197,326,594
Water supply network	4,431,398,248								
OLOYANKILANI DAM (5)	3,581,538,375	37,604,000	52,300,000	200,000,000	35,399,890	2%	3%	2,347,889,875	37,332,058
Water supply network	3,289,629,429								

Annexure (B)

Financial Analysis: NOOPOPONG DAM (option 1 -2)

Financial Analysis								
	NOOPOPONG DAM (1)		Water supply network	O&M	Total Revenue	Net Cash Flow for Equity	NPV 8%	NPV 13%
		Capital costs	Capital costs					
		Option 1						
1	2018	375,414,563	1,230,754,700					
2	2019	375,414,563	1,230,754,700					
3	2020	500,552,751	1,641,006,267	136,977,600	93,927,762	-136,977,600.00	-108,737,235.18	-94,932,347.91
4	2021			342,444,000	93,927,762	-342,444,000.00	-251,706,562.91	-210,027,318.38
5	2021			342,444,000	93,927,762	-248,516,238.09	-169,135,975.83	-134,884,657.48
6	2023			342,444,000	94,279,244	-248,516,238.09	-156,607,385.03	-119,366,953.52
7	2024			342,444,000	94,279,244	-248,516,238.09	-145,006,837.99	-105,634,472.14
8	2025			342,444,000	94,279,244	-248,164,755.87	-134,075,695.83	-93,349,620.25
9	2026			342,444,000	94,279,244	-248,164,755.87	-124,144,162.80	-82,610,283.41
10	2027			342,444,000	94,279,244	-248,164,755.87	-114,948,298.89	-73,106,445.49
11	2028			342,444,000	94,279,244	-248,164,755.87	-106,433,610.08	-64,695,969.46
12	2029			342,444,000	94,279,244	-248,164,755.87	-98,549,638.97	-57,253,070.32
13	2030			342,444,000	94,279,244	-248,164,755.87	-91,249,665.71	-50,666,433.91
14	2031			342,444,000	94,279,244	-248,164,755.87	-84,490,431.21	-44,837,552.14
15	2032			342,444,000	94,279,244	-248,164,755.87	-78,231,880.75	-39,679,249.68
16	2033			342,444,000	95,048,965	-248,164,755.87	-72,436,926.62	-35,114,380.25
17	2034			342,444,000	95,048,965	-248,164,755.87	-67,071,228.35	-31,074,672.78
18	2035			342,444,000	95,048,965	-247,395,034.65	-61,910,367.23	-27,414,415.84

19	2036			342,444,000	95,048,965	-247,395,034.65	-57,324,414.10	-24,260,544.99
20	2037			342,444,000	95,048,965	-247,395,034.65	-53,078,161.20	-21,469,508.84
21	2038			342,444,000	95,048,965	-247,395,034.65	-49,146,445.56	-18,999,565.35
22	2039			342,444,000	95,048,965	-247,395,034.65	-45,505,968.11	-16,813,774.64
23	2040			342,444,000	95,048,965	-247,395,034.65	-42,135,155.66	-14,879,446.59
24	2041			342,444,000	95,048,965	-247,395,034.65	-39,014,033.02	-13,167,651.85
25	2042			342,444,000	95,048,965	-247,395,034.65	-36,124,104.65	-11,652,789.24
26	2043			342,444,000	95,996,721	-247,395,034.65	-33,448,245.04	-10,312,202.87
27	2044			342,444,000	95,996,721	-247,395,034.65	-30,970,597.26	-9,125,843.25
28	2045			342,444,000	95,996,721	-246,447,278.82	-28,566,621.04	-8,045,028.92
29	2046			342,444,000	95,996,721	-246,447,278.82	-26,450,575.04	-7,119,494.62
30	2047			342,444,000	95,996,721	-246,447,278.82	-24,491,273.18	-6,300,437.72

Financial Analysis								
	NOOPOPONG DAM (1)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 1						
1	2018	375,414,563	1,230,754,700					
2	2019	375,414,563	1,230,754,700					
3	2020	500,552,751	1,641,006,267	136,977,600	93,927,762	-136,977,600.00	-108,737,235.18	-94,932,347.91
4	2021			342,444,000	93,927,762	-342,444,000.00	-251,706,562.91	-210,027,318.38
5	2021			342,444,000	93,927,762	-248,516,238.09	-169,135,975.83	-134,884,657.48
6	2023			342,444,000	94,279,244	-248,516,238.09	-156,607,385.03	-119,366,953.52
7	2024			342,444,000	94,279,244	-248,516,238.09	-145,006,837.99	-105,634,472.14
8	2025			342,444,000	94,279,244	-248,164,755.87	-134,075,695.83	-93,349,620.25
9	2026			342,444,000	94,279,244	-248,164,755.87	-124,144,162.80	-82,610,283.41

10	2027			342,444,000	94,279,244	-248,164,755.87	-114,948,298.89	-73,106,445.49
11	2028			342,444,000	94,279,244	-248,164,755.87	-106,433,610.08	-64,695,969.46
12	2029			342,444,000	94,279,244	-248,164,755.87	-98,549,638.97	-57,253,070.32
13	2030			342,444,000	94,279,244	-248,164,755.87	-91,249,665.71	-50,666,433.91
14	2031			342,444,000	94,279,244	-248,164,755.87	-84,490,431.21	-44,837,552.14
15	2032			342,444,000	94,279,244	-248,164,755.87	-78,231,880.75	-39,679,249.68
16	2033			342,444,000	95,048,965	-248,164,755.87	-72,436,926.62	-35,114,380.25
17	2034			342,444,000	95,048,965	-248,164,755.87	-67,071,228.35	-31,074,672.78
18	2035			342,444,000	95,048,965	-247,395,034.65	-61,910,367.23	-27,414,415.84
19	2036			342,444,000	95,048,965	-247,395,034.65	-57,324,414.10	-24,260,544.99
20	2037			342,444,000	95,048,965	-247,395,034.65	-53,078,161.20	-21,469,508.84
21	2038			342,444,000	95,048,965	-247,395,034.65	-49,146,445.56	-18,999,565.35
22	2039			342,444,000	95,048,965	-247,395,034.65	-45,505,968.11	-16,813,774.64
23	2040			342,444,000	95,048,965	-247,395,034.65	-42,135,155.66	-14,879,446.59
24	2041			342,444,000	95,048,965	-247,395,034.65	-39,014,033.02	-13,167,651.85
25	2042			342,444,000	95,048,965	-247,395,034.65	-36,124,104.65	-11,652,789.24
26	2043			342,444,000	95,996,721	-247,395,034.65	-33,448,245.04	-10,312,202.87
27	2044			342,444,000	95,996,721	-247,395,034.65	-30,970,597.26	-9,125,843.25
28	2045			342,444,000	95,996,721	-246,447,278.82	-28,566,621.04	-8,045,028.92
29	2046			342,444,000	95,996,721	-246,447,278.82	-26,450,575.04	-7,119,494.62
30	2047			342,444,000	95,996,721	-246,447,278.82	-24,491,273.18	-6,300,437.72

Financial Analysis: KITENGELA DAM (option 1 -2)

Financial Analysis								
	KITENGELA DAM (2)		Water supply network	O&M	Total Revenue	Net Cash Flow for Equity	NPV 8%	NPV 13%
		Capital costs	Capital costs					
		Option 1						
1	2018	989,047,128	2,225,367,299					
2	2019	989,047,128	2,225,367,299					
3	2020	1,318,729,504	2,967,156,398	128,217,600	426,869,325	298,651,725.00	237,079,368.14	206,980,626.48
4	2021			320,544,000	426,869,325	106,325,325.00	78,152,287.98	65,211,313.05
5	2022			320,544,000	426,869,325	106,325,325.00	72,363,229.61	57,709,126.59
6	2023			320,544,000	563,514,375	242,970,375.00	153,112,550.56	116,703,172.73
7	2024			320,544,000	563,514,375	242,970,375.00	141,770,880.15	103,277,144.01
8	2025			320,544,000	563,514,375	242,970,375.00	131,269,333.47	91,395,702.66
9	2026			320,544,000	563,514,375	242,970,375.00	121,545,679.14	80,881,152.80
10	2027			320,544,000	563,514,375	242,970,375.00	112,542,295.50	71,576,241.41
11	2028			320,544,000	563,514,375	242,970,375.00	104,205,829.16	63,341,806.56
12	2029			320,544,000	563,514,375	242,970,375.00	96,486,878.86	56,054,696.07
13	2030			320,544,000	563,514,375	242,970,375.00	89,339,702.64	49,605,925.73
14	2031			320,544,000	563,514,375	242,970,375.00	82,721,946.89	43,899,049.32
15	2032			320,544,000	563,514,375	242,970,375.00	76,594,395.27	38,848,716.21
16	2033			320,544,000	844,879,692	524,335,692.01	153,048,590.30	74,191,529.74
17	2034			320,544,000	844,879,692	524,335,692.01	141,711,657.69	65,656,221.01
18	2035			320,544,000	844,879,692	524,335,692.01	131,214,497.86	58,102,850.45
19	2036			320,544,000	844,879,692	524,335,692.01	121,494,905.42	51,418,451.73

20	2037			320,544,000	844,879,692	524,335,692.01	112,495,282.80	45,503,054.63
21	2038			320,544,000	844,879,692	524,335,692.01	104,162,298.89	40,268,189.93
22	2039			320,544,000	844,879,692	524,335,692.01	96,446,573.04	35,635,566.31
23	2040			320,544,000	844,879,692	524,335,692.01	89,302,382.45	31,535,899.39
24	2041			320,544,000	844,879,692	524,335,692.01	82,687,391.16	27,907,875.57
25	2042			320,544,000	844,879,692	524,335,692.01	76,562,399.22	24,697,235.02
26	2043			320,544,000	853,304,188	532,760,188.23	72,030,117.14	22,207,119.68
27	2044			320,544,000	853,304,188	532,760,188.23	66,694,552.90	19,652,318.30
28	2045			320,544,000	853,304,188	532,760,188.23	61,754,215.65	17,391,432.13
29	2046			320,544,000	853,304,188	532,760,188.23	57,179,829.31	15,390,647.90
30	2047			320,544,000	853,304,188	532,760,188.23	52,944,286.40	13,620,042.39

Financial Analysis								
	KITENGELA DAM (2)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 2						
1	2018	882,744,340	2,225,367,299					
2	2019	882,744,340	2,225,367,299					
3	2020	1,176,992,454	2,967,156,398	128,217,600	426,869,325	298,651,725.00	237079368.1	65211313.05
4	2021			320,544,000	426,869,325	106,325,325.00	78152287.98	57709126.59
5	2022			320,544,000	563,514,375	242,970,375.00	72363229.61	116703172.7
6	2023			320,544,000	563,514,375	242,970,375.00	153112550.6	103277144
7	2024			320,544,000	563,514,375	242,970,375.00	141770880.1	91395702.66

8	2025			320,544,000	563,514,375	242,970,375.00	131269333.5	80881152.8
9	2026			320,544,000	563,514,375	242,970,375.00	121545679.1	71576241.41
10	2027			320,544,000	563,514,375	242,970,375.00	112542295.5	63341806.56
11	2028			320,544,000	563,514,375	242,970,375.00	104205829.2	56054696.07
12	2029			320,544,000	563,514,375	242,970,375.00	96486878.86	49605925.73
13	2030			320,544,000	563,514,375	242,970,375.00	89339702.64	43899049.32
14	2031			320,544,000	563,514,375	242,970,375.00	82721946.89	38848716.21
15	2032			320,544,000	844,879,692	524,335,692.01	76594395.27	74191529.74
16	2033			320,544,000	844,879,692	524,335,692.01	153048590.3	65656221.01
17	2034			320,544,000	844,879,692	524,335,692.01	141711657.7	58102850.45
18	2035			320,544,000	844,879,692	524,335,692.01	131214497.9	51418451.73
19	2036			320,544,000	844,879,692	524,335,692.01	121494905.4	45503054.63
20	2037			320,544,000	844,879,692	524,335,692.01	112495282.8	40268189.93
21	2038			320,544,000	844,879,692	524,335,692.01	104162298.9	35635566.31
22	2039			320,544,000	844,879,692	524,335,692.01	96446573.04	31535899.39
23	2040			320,544,000	844,879,692	524,335,692.01	89302382.45	27907875.57
24	2041			320,544,000	844,879,692	524,335,692.01	82687391.16	24697235.02
25	2042			320,544,000	853,304,188	532,760,188.23	76562399.22	22207119.68
26	2043			320,544,000	853,304,188	532,760,188.23	72030117.14	19652318.3
27	2044			320,544,000	853,304,188	532,760,188.23	66694552.9	17391432.13
28	2045			320,544,000	853,304,188	532,760,188.23	61754215.65	15390647.9
29	2046			320,544,000	853,304,188	532,760,188.23	57179829.31	13620042.39
30	2047			320,544,000	853,304,188	532,760,188.23	57,179,829	13,620,042

Financial Analysis: ERETETI DAM (option 1 -2)

Financial Analysis								
	ERETETI DAM (3)		Water supply network	O&M	Total Revenue	Net Cash Flow for Equity	NPV 8%	NPV 13%
		Capital costs	Capital costs					
		Option 1						
1	2018	1,685,577,034	2,142,860,139					
2	2019	1,685,577,034	2,142,860,139					
3	2020	2,247,436,045	2,857,146,852	101,937,600	250,474,032	148,536,432	117,913,008.50	102,943,198.14
4	2021			254,844,000	250,474,032	-4,369,968	-3,212,056.94	-2,680,183.21
5	2022			254,844,000	250,474,032	-4,369,968	-2,974,126.79	-2,371,843.55
6	2023			254,844,000	251,411,318	-3,432,682	-2,163,171.94	-1,648,780.76
7	2024			254,844,000	251,411,318	-3,432,682	-2,002,936.98	-1,459,098.02
8	2025			254,844,000	251,411,318	-3,432,682	-1,854,571.27	-1,291,237.19
9	2026			254,844,000	251,411,318	-3,432,682	-1,717,195.62	-1,142,687.78
10	2027			254,844,000	251,411,318	-3,432,682	-1,589,995.95	-1,011,228.12
11	2028			254,844,000	251,411,318	-3,432,682	-1,472,218.47	-894,892.14
12	2029			254,844,000	251,411,318	-3,432,682	-1,363,165.25	-791,939.95
13	2030			254,844,000	251,411,318	-3,432,682	-1,262,190.05	-700,831.81
14	2031			254,844,000	251,411,318	-3,432,682	-1,168,694.49	-620,205.14
15	2032			254,844,000	251,411,318	-3,432,682	-1,082,124.53	-548,854.11
16	2033			254,844,000	253,463,908	-1,380,092	-402,835.81	-195,277.89
17	2034			254,844,000	253,463,908	-1,380,092	-372,996.02	-172,812.24
18	2035			254,844,000	253,463,908	-1,380,092	-345,366.68	-152,931.19
19	2036			254,844,000	253,463,908	-1,380,092	-319,783.97	-135,337.33
20	2037			254,844,000	253,463,908	-1,380,092	-296,096.26	-119,767.55
21	2038			254,844,000	253,463,908	-1,380,092	-274,163.21	-105,988.98

22	2039			254,844,000	253,463,908	-1,380,092	-253,854.82	-93,795.56
23	2040			254,844,000	253,463,908	-1,380,092	-235,050.76	-83,004.92
24	2041			254,844,000	253,463,908	-1,380,092	-217,639.59	-73,455.68
25	2042			254,844,000	253,463,908	-1,380,092	-201,518.14	-65,005.03
26	2043			254,844,000	255,991,256	1,147,256	155,111.03	47,821.24
27	2044			254,844,000	255,991,256	1,147,256	143,621.33	42,319.68
28	2045			254,844,000	255,991,256	1,147,256	132,982.71	37,451.04
29	2046			254,844,000	255,991,256	1,147,256	123,132.14	33,142.52
30	2047			254,844,000	255,991,256	1,147,256	114,011.24	29,329.66

Financial Analysis								
	ERETETI DAM (3)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 2						
1	2018	509,312,148	2,142,860,139					
2	2019	509,312,148	2,142,860,139					
3	2020	679,082,864	2,857,146,852	101,937,600	148,536,432	148,536,432	117,913,009	102,943,198
4	2021			254,844,000	-4,369,968	-4,369,968	-3,212,057	-2,680,183
5	2022			254,844,000	-4,369,968	-4,369,968	-2,974,127	-2,371,844
6	2023			254,844,000	-3,432,682	-3,432,682	-2,163,172	-1,648,781
7	2024			254,844,000	-3,432,682	-3,432,682	-2,002,937	-1,459,098
8	2025			254,844,000	-3,432,682	-3,432,682	-1,854,571	-1,291,237
9	2026			254,844,000	-3,432,682	-3,432,682	-1,717,196	-1,142,688
10	2027			254,844,000	-3,432,682	-3,432,682	-1,589,996	-1,011,228
11	2028			254,844,000	-3,432,682	-3,432,682	-1,472,218	-894,892

12	2029			254,844,000	-3,432,682	-3,432,682	-1,363,165	-791,940
13	2030			254,844,000	-3,432,682	-3,432,682	-1,262,190	-700,832
14	2031			254,844,000	-3,432,682	-3,432,682	-1,168,694	-620,205
15	2032			254,844,000	-3,432,682	-3,432,682	-1,082,125	-548,854
16	2033			254,844,000	-1,380,092	-1,380,092	-402,836	-195,278
17	2034			254,844,000	-1,380,092	-1,380,092	-372,996	-172,812
18	2035			254,844,000	-1,380,092	-1,380,092	-345,367	-152,931
19	2036			254,844,000	-1,380,092	-1,380,092	-319,784	-135,337
20	2037			254,844,000	-1,380,092	-1,380,092	-296,096	-119,768
21	2038			254,844,000	-1,380,092	-1,380,092	-274,163	-105,989
22	2039			254,844,000	-1,380,092	-1,380,092	-253,855	-93,796
23	2040			254,844,000	-1,380,092	-1,380,092	-235,051	-83,005
24	2041			254,844,000	-1,380,092	-1,380,092	-217,640	-73,456
25	2042			254,844,000	-1,380,092	-1,380,092	-201,518	-65,005
26	2043			254,844,000	1,147,256	1,147,256	155,111	47,821
27	2044			254,844,000	1,147,256	1,147,256	143,621	42,320
28	2045			254,844,000	1,147,256	1,147,256	132,983	37,451
29	2046			254,844,000	1,147,256	1,147,256	123,132	33,143
30	2047			254,844,000	1,147,256	1,147,256	114,011	29,330

Financial Analysis: AIRFIELD DAM (option 1 -2)

Financial Analysis								
	AIRFIELD DAM (4)		Water supply network	O&M	Total Revenue	Net Cash Flow for Equity	NPV 8%	NPV 13%
		Capital costs	Capital costs					
		Option 1						
1	2018	1,098,674,975	1,329,419,474					
2	2019	1,098,674,975	1,329,419,474					
3	2020	1,464,899,966	1,772,559,299	98,433,600	193,073,733	94,640,133	75,128,388.87	65,590,359.53
4	2021			246,084,000	193,073,733	-53,010,267	-38,964,128.75	-32,512,189.51
5	2022			246,084,000	193,073,733	-53,010,267	-36,077,896.99	-28,771,849.12
6	2023			246,084,000	193,796,224	-52,287,776	-32,950,168.26	-25,114,787.55
7	2024			246,084,000	193,796,224	-52,287,776	-30,509,415.09	-22,225,475.73
8	2025			246,084,000	193,796,224	-52,287,776	-28,249,458.41	-19,668,562.59
9	2026			246,084,000	193,796,224	-52,287,776	-26,156,905.94	-17,405,807.60
10	2027			246,084,000	193,796,224	-52,287,776	-24,219,357.35	-15,403,369.56
11	2028			246,084,000	193,796,224	-52,287,776	-22,425,330.88	-13,631,300.49
12	2029			246,084,000	193,796,224	-52,287,776	-20,764,195.26	-12,063,097.78
13	2030			246,084,000	193,796,224	-52,287,776	-19,226,106.72	-10,675,307.77
14	2031			246,084,000	193,796,224	-52,287,776	-17,801,950.67	-9,447,175.02
15	2032			246,084,000	193,796,224	-52,287,776	-16,483,287.66	-8,360,331.88
16	2033			246,084,000	195,378,429	-50,705,571	-14,800,472.83	-7,174,647.72
17	2034			246,084,000	195,378,429	-50,705,571	-13,704,141.51	-6,349,245.77
18	2035			246,084,000	195,378,429	-50,705,571	-12,689,019.91	-5,618,801.57
19	2036			246,084,000	195,378,429	-50,705,571	-11,749,092.51	-4,972,390.77
20	2037			246,084,000	195,378,429	-50,705,571	-10,878,789.36	-4,400,345.81

21	2038			246,084,000	195,378,429	-50,705,571	-10,072,953.11	-3,894,111.34
22	2039			246,084,000	195,378,429	-50,705,571	-9,326,808.44	-3,446,116.23
23	2040			246,084,000	195,378,429	-50,705,571	-8,635,933.74	-3,049,660.38
24	2041			246,084,000	195,378,429	-50,705,571	-7,996,234.94	-2,698,814.50
25	2042			246,084,000	195,378,429	-50,705,571	-7,403,921.24	-2,388,331.41
26	2043			246,084,000	197,326,594	-48,757,406	-6,592,087.29	-2,032,361.98
27	2044			246,084,000	197,326,594	-48,757,406	-6,103,784.53	-1,798,550.42
28	2045			246,084,000	197,326,594	-48,757,406	-5,651,652.34	-1,591,637.54
29	2046			246,084,000	197,326,594	-48,757,406	-5,233,011.43	-1,408,528.80
30	2047			246,084,000	197,326,594	-48,757,406	-4,845,380.95	-1,246,485.66

Financial Analysis								
	AIRFIELD DAM (4)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 2						
1	2018	967,031,523	1,329,419,474					
2	2019	967,031,523	1,329,419,474					
3	2020	1,289,375,364	1,772,559,299	98,433,600	193,073,733	94,640,133	75,128,389	65,590,360
4	2021			246,084,000	193,073,733	-53,010,267	-38,964,129	-32,512,190
5	2022			246,084,000	193,073,733	-53,010,267	-36,077,897	-28,771,849
6	2023			246,084,000	193,796,224	-52,287,776	-32,950,168	-25,114,788
7	2024			246,084,000	193,796,224	-52,287,776	-30,509,415	-22,225,476
8	2025			246,084,000	193,796,224	-52,287,776	-28,249,458	-19,668,563
9	2026			246,084,000	193,796,224	-52,287,776	-26,156,906	-17,405,808
10	2027			246,084,000	193,796,224	-52,287,776	-24,219,357	-15,403,370

11	2028			246,084,000	193,796,224	-52,287,776	-22,425,331	-13,631,300
12	2029			246,084,000	193,796,224	-52,287,776	-20,764,195	-12,063,098
13	2030			246,084,000	193,796,224	-52,287,776	-19,226,107	-10,675,308
14	2031			246,084,000	193,796,224	-52,287,776	-17,801,951	-9,447,175
15	2032			246,084,000	193,796,224	-52,287,776	-16,483,288	-8,360,332
16	2033			246,084,000	195,378,429	-50,705,571	-14,800,473	-7,174,648
17	2034			246,084,000	195,378,429	-50,705,571	-13,704,142	-6,349,246
18	2035			246,084,000	195,378,429	-50,705,571	-12,689,020	-5,618,802
19	2036			246,084,000	195,378,429	-50,705,571	-11,749,093	-4,972,391
20	2037			246,084,000	195,378,429	-50,705,571	-10,878,789	-4,400,346
21	2038			246,084,000	195,378,429	-50,705,571	-10,072,953	-3,894,111
22	2039			246,084,000	195,378,429	-50,705,571	-9,326,808	-3,446,116
23	2040			246,084,000	195,378,429	-50,705,571	-8,635,934	-3,049,660
24	2041			246,084,000	195,378,429	-50,705,571	-7,996,235	-2,698,814
25	2042			246,084,000	195,378,429	-50,705,571	-7,403,921	-2,388,331
26	2043			246,084,000	197,326,594	-48,757,406	-6,592,087	-2,032,362
27	2044			246,084,000	197,326,594	-48,757,406	-6,103,785	-1,798,550
28	2045			246,084,000	197,326,594	-48,757,406	-5,651,652	-1,591,638
29	2046			246,084,000	197,326,594	-48,757,406	-5,233,011	-1,408,529
30	2047			246,084,000	197,326,594	-48,757,406	-4,845,381	-1,246,486

Financial Analysis: AIRFIELD DAM (option 1 -2)

Financial Analysis								
	OLOYANKILANI DAM (5)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 1						
1	2018	1,074,461,513	986,888,829					
2	2019	1,074,461,513	986,888,829					
3	2020	1,432,615,350	1,315,851,772	15,041,600	36,527,463	21,485,863	17,056,170.78	14,890,780.84
4	2021			37,604,000	36,527,463	-1,076,537	-791,286.83	-660,260.30
5	2022			37,604,000	36,527,463	-1,076,537	-732,672.99	-584,301.15
6	2023			37,604,000	36,664,150	-939,850	-592,264.92	-451,427.37
7	2024			37,604,000	36,664,150	-939,850	-548,393.45	-399,493.25
8	2025			37,604,000	36,664,150	-939,850	-507,771.71	-353,533.85
9	2026			37,604,000	36,664,150	-939,850	-470,158.99	-312,861.81
10	2027			37,604,000	36,664,150	-939,850	-435,332.40	-276,868.86
11	2028			37,604,000	36,664,150	-939,850	-403,085.56	-245,016.69
12	2029			37,604,000	36,664,150	-939,850	-373,227.37	-216,828.93
13	2030			37,604,000	36,664,150	-939,850	-345,580.89	-191,884.01
14	2031			37,604,000	36,664,150	-939,850	-319,982.31	-169,808.86
15	2032			37,604,000	36,664,150	-939,850	-296,279.92	-150,273.32
16	2033			37,604,000	36,963,487	-640,513	-186,959.64	-90,630.18
17	2034			37,604,000	36,963,487	-640,513	-173,110.78	-80,203.70
18	2035			37,604,000	36,963,487	-640,513	-160,287.76	-70,976.73
19	2036			37,604,000	36,963,487	-640,513	-148,414.59	-62,811.26
20	2037			37,604,000	36,963,487	-640,513	-137,420.92	-55,585.19

21	2038			37,604,000	36,963,487	-640,513	-127,241.59	-49,190.43
22	2039			37,604,000	36,963,487	-640,513	-117,816.29	-43,531.36
23	2040			37,604,000	36,963,487	-640,513	-109,089.15	-38,523.32
24	2041			37,604,000	36,963,487	-640,513	-101,008.48	-34,091.44
25	2042			37,604,000	36,963,487	-640,513	-93,526.37	-30,169.41
26	2043			37,604,000	37,332,058	-271,942	-36,767.04	-11,335.40
27	2044			37,604,000	37,332,058	-271,942	-34,043.55	-10,031.33
28	2045			37,604,000	37,332,058	-271,942	-31,521.81	-8,877.28
29	2046			37,604,000	37,332,058	-271,942	-29,186.86	-7,856.00
30	2047			37,604,000	37,332,058	-271,942	-27,024.87	-6,952.21

Financial Analysis								
	OLOYANKILANI DAM (5)		**Water supply network**	**O&M**	**Total Revenue**	**Net Cash Flow for Equity**	**NPV 8%**	**NPV 13%**
		Capital costs	Capital costs					
		Option 1						
1	2018	1,074,461,513	986,888,829					
2	2019	1,074,461,513	986,888,829					
3	2020	1,432,615,350	1,315,851,772	15,041,600	36,527,463	21,485,863	17,056,170.78	14,890,780.84
4	2021			37,604,000	36,527,463	-1,076,537	-791,286.83	-660,260.30
5	2022			37,604,000	36,527,463	-1,076,537	-732,672.99	-584,301.15
6	2023			37,604,000	36,664,150	-939,850	-592,264.92	-451,427.37
7	2024			37,604,000	36,664,150	-939,850	-548,393.45	-399,493.25
8	2025			37,604,000	36,664,150	-939,850	-507,771.71	-353,533.85
9	2026			37,604,000	36,664,150	-939,850	-470,158.99	-312,861.81
10	2027			37,604,000	36,664,150	-939,850	-435,332.40	-276,868.86

11	2028			37,604,000	36,664,150	-939,850	-403,085.56	-245,016.69
12	2029			37,604,000	36,664,150	-939,850	-373,227.37	-216,828.93
13	2030			37,604,000	36,664,150	-939,850	-345,580.89	-191,884.01
14	2031			37,604,000	36,664,150	-939,850	-319,982.31	-169,808.86
15	2032			37,604,000	36,664,150	-939,850	-296,279.92	-150,273.32
16	2033			37,604,000	36,963,487	-640,513	-186,959.64	-90,630.18
17	2034			37,604,000	36,963,487	-640,513	-173,110.78	-80,203.70
18	2035			37,604,000	36,963,487	-640,513	-160,287.76	-70,976.73
19	2036			37,604,000	36,963,487	-640,513	-148,414.59	-62,811.26
20	2037			37,604,000	36,963,487	-640,513	-137,420.92	-55,585.19
21	2038			37,604,000	36,963,487	-640,513	-127,241.59	-49,190.43
22	2039			37,604,000	36,963,487	-640,513	-117,816.29	-43,531.36
23	2040			37,604,000	36,963,487	-640,513	-109,089.15	-38,523.32
24	2041			37,604,000	36,963,487	-640,513	-101,008.48	-34,091.44
25	2042			37,604,000	36,963,487	-640,513	-93,526.37	-30,169.41
26	2043			37,604,000	37,332,058	-271,942	-36,767.04	-11,335.40
27	2044			37,604,000	37,332,058	-271,942	-34,043.55	-10,031.33
28	2045			37,604,000	37,332,058	-271,942	-31,521.81	-8,877.28
29	2046			37,604,000	37,332,058	-271,942	-29,186.86	-7,856.00
30	2047			37,604,000	37,332,058	-271,942	-27,024.87	-6,952.21

16 PROJECT COST ESTIMATES

MAXIMUM & MINIMUM PROJECT COSTS			
Construction costs			
DAM SITE		**MAXIMUM**	**MINIMUM**
ID	**NOO POPONG DAM**	Amount KES	Amount KES
1	Dam costruction costs	KES 290,252,354.44	KES 234,149,811.31
2	Outlet works	KES 961,129,523.64	KES 961,129,523.64
3	Pumping station - conctruction costs	KES 718,585,000.00	KES 718,585,000.00
4	Water supply network - connection trunks	KES 323,807,360.77	KES 323,807,360.77
5	Land acquisition costs	KES 200,000,000.00	KES 200,000,000.00
	SUB-TOTAL 1	**KES 2,493,774,238.85**	**KES 2,437,671,695.71**
6	Drinking water treatment plant	KES 476,463,590.89	KES 476,463,590.89
7	Water supply network - Main distribution from Kajiado to Kitengela	KES 1,898,018,698.40	KES 1,898,018,698.40
8	Water supply network - Kitengela Town	KES 450,065,700.00	KES 450,065,700.00
9	Water supply network - Kajiado Town	KES 235,702,341.20	KES 235,702,341.20
	SUB-TOTAL 2	**KES 3,060,250,330.50**	**KES 3,060,250,330.50**
10	Sewerage network - Trunk sewers :		
	Kitengela sewerage network	KES 1,584,858,421.25	KES 815,069,291.25
	Kajiado sewerage network	KES 718,482,549.13	KES 550,420,392.13
11	Waste Stabilization Ponds :		
	Kitengela treatment pond	KES 1,122,586,424.00	KES 1,703,023,656.00
	Kajiado treatment pond	KES 1,061,045,961.00	KES 841,063,845.00
12	Land acquisition costs :		
	Kitengela	KES 900,000,000.00	KES 600,000,000.00
	Kajiado	KES 300,000,000.00	KES 165,000,000.00
	SUB-TOTAL 3	**KES 5,686,973,355.38**	**KES 4,674,577,184.38**
TOTAL AMOUNT Construction		**KES 11,240,997,924.72**	**KES 10,172,499,210.58**
Operational and Maintainance costs			
13	Dam	KES 5,526,400.00	KES 5,526,400.00
14	Drinking water treatment plant	KES 7,286,400.00	KES 7,286,400.00
15	Sewerage network:		
	Kitengela	KES 12,346,600.00	KES 9,506,600.00
	Kajiado	KES 10,684,600.00	KES 7,474,600.00
16	Pumping station	KES 306,600,000.00	KES 306,600,000.00
TOTAL AMOUNT O&M		**KES 342,444,000.00**	**KES 336,394,000.00**

MAXIMUM & MINIMUM PROJECT COSTS			
Construction costs			
DAM SITE		MAXIMUM	MINIMUM
ID	KITENGELA DAM	Amount KES	Amount KES
1	Dam costruction costs	KES 1,833,228,924.76	KES 1,478,886,297.74
2	Outlet works	KES 1,463,594,836.56	KES 1,463,594,836.56
3	Pumping station - conctruction costs	KES 667,257,500.00	KES 667,257,500.00
4	Water supply network - connection trunks	KES 324,398,442.56	KES 324,398,442.56
5	Land acquisition costs	KES 2,000,000,000.00	KES 2,000,000,000.00
	SUB-TOTAL 1	**KES 6,288,479,703.88**	**KES 5,934,137,076.86**
6	Drinking water treatment plant	KES 3,842,448,313.64	KES 3,842,448,313.64
7	Water supply network - Main distribution from Kajiado to Kitengela	KES 1,898,018,698.40	KES 1,898,018,698.40
8	Water supply network - Kitengela Town	KES 450,065,700.00	KES 450,065,700.00
9	Water supply network - Kajiado Town	KES 235,702,341.20	KES 235,702,341.20
	SUB-TOTAL 2	**KES 6,426,235,053.24**	**KES 6,426,235,053.24**
10	Sewerage network - Trunk sewers :		
	Kitengela sewerage network	KES 1,584,858,421.25	KES 815,069,291.25
	Kajiado sewerage network	KES 718,482,549.13	KES 550,420,392.13
11	Waste Stabilization Ponds :		
	Kitengela treatment pond	KES 1,703,023,656.00	KES 1,122,586,424.00
	Kajiado treatment pond	KES 1,061,045,961.00	KES 841,063,845.00
12	Land acquisition costs :		
	Kitengela	KES 900,000,000.00	KES 600,000,000.00
	Kajiado	KES 300,000,000.00	KES 165,000,000.00
	SUB-TOTAL 3	**KES 6,267,410,587.38**	**KES 4,094,139,952.38**
TOTAL AMOUNT Construction		**KES 18,982,125,344.50**	**KES 16,454,512,082.48**
Operational and Maintainance costs			
13	Dam	KES 5,526,400.00	KES 5,526,400.00
14	Drinking water treatment plant	KES 7,286,400.00	KES 7,286,400.00
15	Sewerage network:		
	Kitengela	KES 12,346,600.00	KES 9,506,600.00
	Kajiado	KES 10,684,600.00	KES 7,474,600.00
16	Pumping station	KES 284,700,000.00	KES 284,700,000.00
TOTAL AMOUNT O&M		**KES 320,544,000.00**	**KES 314,494,000.00**

MAXIMUM & MINIMUM PROJECT COSTS			
Construction costs			
DAM SITE		MAXIMUM	MINIMUM
ID	ERETETI DAM	Amount KES	Amount KES
1	Dam costruction costs	KES 415,533,707.14	KES 335,215,693.70
2	Outlet works	KES 1,362,491,466.90	KES 1,362,491,466.90
3	Pumping station - conctruction costs	KES 513,275,000.00	KES 513,275,000.00
4	Water supply network - connection trunks	KES 203,357,076.61	KES 203,357,076.61
5	Land acquisition costs	KES 3,400,000,000.00	KES 3,400,000,000.00
	SUB-TOTAL 1	**KES 5,894,657,250.64**	**KES 5,814,339,237.21**
6	Drinking water treatment plant	KES 2,318,149,067.62	KES 2,318,149,067.62
7	Water supply network - Main distribution from Kajiado to Kitengela	KES 1,898,018,698.40	KES 1,898,018,698.40
8	Water supply network - Kitengela Town	KES 450,065,700.00	KES 450,065,700.00
9	Water supply network - Kajiado Town	KES 235,702,341.20	KES 235,702,341.20
	SUB-TOTAL 2	**KES 4,901,935,807.22**	**KES 4,901,935,807.22**
10	Sewerage network - Trunk sewers :		
	Kitengela sewerage network	KES 1,584,858,421.25	KES 815,069,291.25
	Kajiado sewerage network	KES 718,482,549.13	KES 550,420,392.13
11	Waste Stabilization Ponds :		
	Kitengela treatment pond	KES 1,703,023,656.00	KES 1,122,586,424.00
	Kajiado treatment pond	KES 1,061,045,961.00	KES 841,063,845.00
12	Land acquisition costs :		
	Kitengela	KES 900,000,000.00	KES 600,000,000.00
	Kajiado	KES 300,000,000.00	KES 165,000,000.00
	SUB-TOTAL 3	**KES 6,267,410,587.38**	**KES 4,094,139,952.38**
TOTAL AMOUNT Construction		**KES 17,064,003,645.24**	**KES 14,810,414,996.81**
Operational and Maintainance costs			
13	Dam	KES 5,526,400.00	KES 5,526,400.00
14	Drinking water treatment plant	KES 7,286,400.00	KES 7,286,400.00
15	Sewerage network:		
	Kitengela	KES 12,346,600.00	KES 9,506,600.00
	Kajiado	KES 10,684,600.00	KES 7,474,600.00
16	Pumping station	KES 219,000,000.00	KES 219,000,000.00
TOTAL AMOUNT O&M		**KES 254,844,000.00**	**KES 248,794,000.00**

MAXIMUM & MINIMUM PROJECT COSTS			
Construction costs			
DAM SITE		MAXIMUM	MINIMUM
ID	AIRFIELD DAM	Amount KES	Amount KES
1	Dam costruction costs	KES 2,270,237,567.49	KES 1,831,426,062.42
2	Outlet works	KES 1,392,012,348.48	KES 1,392,012,348.48
3	Pumping station - conctruction costs	KES 492,744,000.00	KES 492,744,000.00
4	Water supply network - connection trunks	KES 481,583,800.96	KES 481,583,800.96
5	Land acquisition costs	KES 2,500,000,000.00	KES 2,500,000,000.00
	SUB-TOTAL 1	**KES 7,136,577,716.93**	**KES 6,697,766,211.86**
6	Drinking water treatment plant	KES 873,388,501.69	KES 873,388,501.69
7	Water supply network - Main distribution from Kajiado to Kitengela	KES 1,898,018,698.40	KES 1,898,018,698.40
8	Water supply network - Kitengela Town	KES 450,065,700.00	KES 450,065,700.00
9	Water supply network - Kajiado Town	KES 235,702,341.20	KES 235,702,341.20
	SUB-TOTAL 2	**KES 3,457,175,241.29**	**KES 3,457,175,241.29**
10	Sewerage network - Trunk sewers :		
	Kitengela sewerage network	KES 1,584,858,421.25	KES 815,069,291.25
	Kajiado sewerage network	KES 718,482,549.13	KES 550,420,392.13
11	Waste Stabilization Ponds :		
	Kitengela treatment pond	KES 1,703,023,656.00	KES 1,122,586,424.00
	Kajiado treatment pond	KES 1,061,045,961.00	KES 841,063,845.00
12	Land acquisition costs :		
	Kitengela	KES 900,000,000.00	KES 600,000,000.00
	Kajiado	KES 300,000,000.00	KES 165,000,000.00
	SUB-TOTAL 3	**KES 6,267,410,587.38**	**KES 4,094,139,952.38**
TOTAL AMOUNT Construction		**KES 16,861,163,545.60**	**KES 14,249,081,405.52**
Operational and Maintainance costs			
13	Dam	KES 5,526,400.00	KES 5,526,400.00
14	Drinking water treatment plant	KES 7,286,400.00	KES 7,286,400.00
15	Sewerage network:		
	Kitengela	KES 12,346,600.00	KES 9,506,600.00
	Kajiado	KES 10,684,600.00	KES 7,474,600.00
16	Pumping station	KES 210,240,000.00	KES 210,240,000.00
TOTAL AMOUNT O&M		**KES 246,084,000.00**	**KES 240,034,000.00**

MAXIMUM & MINIMUM PROJECT COSTS			
Construction costs			
DAM SITE		MAXIMUM	MINIMUM
ID	OLOLYANKILANI DAM	Amount KES	Amount KES
1	Dam costruction costs	KES 2,372,907,901.28	KES 1,914,251,370.14
2	Outlet works	KES 1,208,630,473.49	KES 1,208,630,473.49
3	Pumping station - conctruction costs	KES -	KES -
4	Water supply network - connection trunks	KES 547,063,834.75	KES 547,063,834.75
5	Land acquisition costs	KES 200,000,000.00	KES 200,000,000.00
	SUB-TOTAL 1	**KES 4,328,602,209.52**	**KES 3,869,945,678.38**
6	Drinking water treatment plant	KES 158,693,115.35	KES 158,693,115.35
7	Water supply network - Main distribution from Kajiado to Kitengela	KES 1,898,018,698.40	KES 1,898,018,698.40
8	Water supply network - Kitengela Town	KES 450,065,700.00	KES 450,065,700.00
9	Water supply network - Kajiado Town	KES 235,702,341.20	KES 235,702,341.20
	SUB-TOTAL 2	**KES 2,742,479,854.96**	**KES 2,742,479,854.96**
10	Sewerage network - Trunk sewers :		
	Kitengela sewerage network	KES 1,584,858,421.25	KES 815,069,291.25
	Kajiado sewerage network	KES 718,482,549.13	KES 550,420,392.13
11	Waste Stabilization Ponds :		
	Kitengela treatment pond	KES 1,703,023,656.00	KES 1,122,586,424.00
	Kajiado treatment pond	KES 1,061,045,961.00	KES 841,063,845.00
12	Land acquisition costs :		
	Kitengela	KES 900,000,000.00	KES 600,000,000.00
	Kajiado	KES 300,000,000.00	KES 165,000,000.00
	SUB-TOTAL 3	**KES 6,267,410,587.38**	**KES 4,094,139,952.38**
TOTAL AMOUNT Construction		**KES 13,338,492,651.85**	**KES 10,706,565,485.71**
Operational and Maintainance costs			
13	Dam	KES 7,286,400.00	KES 7,286,400.00
14	Drinking water treatment plant	KES 7,286,400.00	KES 7,286,400.00
15	Sewerage network:		
	Kitengela	KES 12,346,600.00	KES 9,506,600.00
	Kajiado	KES 10,684,600.00	KES 7,474,600.00
16	Pumping station	KES -	KES -
TOTAL AMOUNT O&M		**KES 37,604,000.00**	**KES 31,554,000.00**