UNIVERSITY OF NAIROBI

THE FEASIBILITY OF COMBINING THE KISERIAN AND ONGATA RONGAI TOWNSHIPS WASTEWATER TREATMENT PLANTS

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Abstract

In towns that lack proper planning, there lacks reserved land for the construction of wastewater treatment works. In most cases, the sites that are identified as technically suitable for such works are located on land already owned and registered to individuals. This makes such projects expensive due to the cost incurred in land acquisition. It also attracts resistance from residents living within and/or in close vicinity to the proposed sites.

The purpose of this study was to determine the technical and economic feasibility of combining the wastewater treatment systems for Ongata Rongai and Kiserian townships. The report concentrates on comparing the per capita cost incurred in acquiring land in both Kiserian and Ongata Rongai townships for the construction of two WWTPs to serve the townships separately to that of combining the wastewater sewerage system so that only one WWTP is constructed in Ongata Rongai to serve both townships.

To achieve this objective, various reports were reviewed so as to determine the appropriate projected population and water demand of both townships to the ultimate design year, 2036. A factor of 0.8 was applied to the water demand giving wastewater flows of 7,975m^3/day and 22,800m^3/day from Kiserian and Ongata Rongai townships respectively. The Google Earth Pro Software was used to view suitable trunk sewer routes so as to establish the suitability of the project area's terrain to the adoption of a gravity flow system. The area required for the series of waste stabilization ponds, staff houses and administration blocks was then determined for both cases.

The analysis showed that a total area of 51 acres (20.6 ha) would be required to serve Kiserian Township at a cost of 510 million while 120 acres (52.6 ha) are required for the Ongata Rongai WWTP at a cost of 2.63 billion if the plants were to be constructed separately. For the construction of the combined WWTP, 169 acres (68.4 ha) would be required at a cost of 3.14 billion. The conclusion made from data analysis is that it would be approximately 8% cheaper to construct separate WWTPs than to construct one WWTP to serve both townships.

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List of Abbreviations

BOD – Biochemical Oxygen Demand

FC – Faecal Coliform

MoWI – Ministry of Water and Irrigation

NST-WSDP - Nairobi Satellite Towns Water and Sanitation Development Programme

OWSC - Oloolaiser Water and Sewerage Company Limited

SS – Suspended Solids

WSP – Waste Stabilisation Pond

WWTP – Wastewater Treatment Plant

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CHAPTER ONE

1.0 Introduction

1.1 General

Urbanisation is a global trend which, when properly managed, can no doubt enhance economic development opportunities, particularly in developing countries. However, poorly managed urbanisation can cause extreme pressures on natural resources. The most common environmental problem in many urban and peri-urban areas without a sewerage system is water pollution caused by direct disposal of untreated wastewater from households, institutions and industries into watercourses and into the ground. This causes severe deterioration in the quality of water in water bodies and groundwater supplies. It is therefore necessary that these areas are served with adequate systems for the collection and treatment of wastewater.

The provision of a proper wastewater management infrastructure is usually very costly. A feasibility study has to be carried out to determine the factors that will make such a project a success. This includes economic (cost of land acquisition, construction, operation and maintenance) and technical feasibility. This study seeks to look into the feasibility of combining the wastewater treatment plants for Kiserian and Ongata Rongai townships as opposed to constructing separate treatment plants in both townships. The study focuses on the technical and economic feasibility of both cases. A comparison of the per capita investment expected to be made by the government in either case is used to determine the cheaper option between the two alternatives.

1.2 Background

One of the goals of the current development blueprint for this country, the Kenya Vision 2030, is to make Kenya a nation that has a clean, secure and sustainable environment by 2030. The 2030 goal for urban areas is to achieve "a well-housed population living in an environmentally secure urban environment." This means that the government must make it its objective to ensure basic sanitation facilities are provided for urban centres and satellite towns and this has to be done in the most feasible manner for the efficient accomplishment of this goal.

Athi Water Services Board, the board that is given the mandate to ensure sustainable provision of water and sewerage services in Nairobi and its environs has considered constructing a wastewater treatment plant in Rimpa area for Kiserian Town. However, the proposal was met with a lot of resistance from the residents and property owners of the area. The property owners claimed that locating the plant in the area would devalue adjacent land and deter growth. The neighbouring town, Ongata Rongai, does not have a central wastewater collection and treatment system either but plans are underway for one. If a convenient site for the location of the treatment plant for Ongata Rongai is identified, it may present an opportunity to combine the two wastewater treatment systems.

There is a possibility that this decision may lead to a more economical project as Ongata Rongai is more densely populated than Kiserian Town. A study carried out by Singhirunnusorn and Stenstrom (2010) in Thailand showed that for WSPs, the total costs of treating wastewater per m^3/day decrease with increasing plant size due to an economy of scale. In principle, the greater the density, the better and more efficient will be the utilisation of any urban infrastructure. Accordingly, lower densities may imply longer networks for fewer consumers and higher per capita investment. Conversely, the project may either be technically impossible to actualize due to the terrain or be more expensive than having separate WWTPs for each town. This is due to the fact that there may not be land available for such a project in the highly developed Ongata Rongai Township and buildings may have to be demolished and owners compensated to acquire land thus escalating cost.

1.3 Problem Statement

Kiserian Town is one of the fastest growing satellite towns in Nairobi's suburbs. It was originally occupied by the Maasai but has witnessed marked growth over the years. This is because of its proximity to Nairobi making it home to residential neighbourhoods that host people who work in the capital city. This immense growth in population has led to an increase in wastewater generation and thus calls for investment in the infrastructure needed for better wastewater management which is best achieved by having a central wastewater treatment facility. In Kenya, the most feasible method of wastewater treatment is the utilization of waste stabilization ponds whose construction requires large tracts of land.

Unfortunately, this rapid growth has attracted real estate but little environmental control. Sewage from the large population is mostly managed using pit latrines, open drains and soak pits leading to ground and surface water pollution. Property prices in the satellite town is mostly driven by demand from investment groups and land buying companies as the demand for better housing by the middle class increases. A recent report by Hass Property Index showed that the value of land in Kiserian Town appreciated by 22.8 percent between September 2014 and September 2015, having an average asking price of 5.8 million per acre. There is a general increase in value of land in the town and lesser people are willing to let go of their land for the purposes of the construction of such facilities as a wastewater treatment plant.

The decision by Athi Water Services Board to construct a wastewater treatment plant for Kiserian Town at Rimpa area was met with a lot of resistance from the property owners of the area. Such resistance usually leads to long delays in the actualization of construction projects. With plans underway to construct a wastewater treatment system for the neighbouring Ongata Rongai Town, an opportunity to combine the Kiserian Town treatment system with that of Ongata Rongai Town is presented. It is necessary to carry out a feasibility study that compares the viability of having separate sewerage systems for the two townships against that of having a combined wastewater treatment system serving both townships so as to come up with the best solution.

1.4 General Objectives

- To determine the technical feasibility of combining the wastewater collection system and treatment plant for Ongata Rongai and Kiserian townships
- To determine if it would be more economical to construct one wastewater treatment plant in Ongata Rongai serving both Kiserian and Ongata Rongai townships as opposed to constructing separate treatment plants in the respective townships

Specific Objectives

- To estimate the wastewater generation from the projected population that will be served by the proposed sewerage system in Kiserian and Ongata Rongai townships
- To establish a technically suitable site for the location of a wastewater treatment plant that will conveniently serve both Kiserian and Ongata Rongai townships
- To establish how much land would be required for the construction of the treatment plants
- To determine an estimate cost of acquiring the land required for the construction of the treatment plants
- To determine the per capita investment made in the provision of the treatment plants

1.5 Scope and Limitations of the Study

The area considered under this study is located within the service area of the Oloolaiser Water and Sewerage Company Limited (OWSC). It spreads from Ongata Rongai, along Magadi Road up to Kiserian located southwest of Nairobi. The areas to be served by the sewer reticulation are the built-up areas within the townships that can be connected to a gravity based sewerage system. This covers areas proposed for water supply and sanitation system in the Nairobi Satellite Towns Water and Sanitation Development Programme Feasibility Report. The study covers technical and economic feasibility and the estimation of the sewage generation is based on projected land use and water demand to the year 2036.

This study does not cover the design of the sewerage reticulation system, the geotechnical studies and environmental impacts of locating the treatment plant in either location. The cost comparison therefore does not factor in the variation that may be brought about by the difference in geotechnical conditions of the proposed sites or costs incurred due to environmental management.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Introduction

The feasibility of locating a wastewater treatment plant rests on the integration of many considerations. These considerations include cost, the kind, amount and characteristics of the wastewater, applicable laws and regulations, economic and socio-political factors, the characteristics of the land area itself, climate, land use of potential sites and surrounding areas, topography and drainage characteristics, soil properties and geology among other factors.

2.2 History of Wastewater Management

The first human communities were scattered over wide areas and waste produced was returned to land and decomposed using natural cycles. Until the birth of the first advanced civilization, the disposal of human excreta was managed through holes in the ground which were covered after use as explained by the Mosaic Law of Sanitation. During the agrarian revolution, mankind established permanent settlements and with human settlement came the environmental impacts. For centuries, the management of wastewater from the increased population was not given much consideration, if any. In most cultures, wastewater was disposed of in the streets near population centres. This created serious health impacts on the population and had a negative impact on the environment. This is evidenced by the numerous outbreaks which occurred throughout Europe until the nineteenth century (Lofrano and Brown, 2010).

In countries like India, waste products including human excreta used to be collected, carried and disposed of manually to a safe point of disposal by sweepers. Thanks to civilization, this archaic method of collecting and disposing of the society’s wastes has been replaced by a modernised system in which these wastes are mixed with an adequate amount of water and carried through closed conduits under conditions of gravity flow. This mixture of water and waste products, commonly referred to as sewage, automatically flows up to a place where it is treated to reduce the constituents that are considered harmful to public health and the environment in general. The common practice is to either dispose of the resulting treated effluent into in a running body of water, such as a stream, or to use it for irrigation purposes (Garg, 1979).

This modern water-carried sewerage system has completely replaced the old conservancy system of sanitation in the developed countries like the USA. However, most developing countries still use the old conservancy system at various places, particularly in the villages and smaller towns. The metropolitan cities and a few bigger towns have generally been equipped with facilities of this modern water carriage sewerage system; and attempts are usually being made to equip the remaining cities and towns with this system, as soon as funds become available.

The modern water-carried sewerage system is preferred to the old conservancy system because of the following advantages;

i. The water carriage closed conduit system is more hygienic. The old conservancy systems posed health hazards to cleaners and other residents because of the possibilities of flies and other insects transmitting disease germs from the open conduits to foods and eatables. In modern sewerage system no such danger exists because the polluted sewage is carried away in closed conduits, as soon as it is produced.
ii. In the conservancy system, the waste products are generally buried underground, which may sometimes pollute the city's water supplies, if the water supply pipes happen to pass through such areas or the wells happen to draw water through such areas.
iii. In the water carriage sewerage system, the sewage is carried through underground pipes which owing to their being underground, do not occupy floor area on roadsides or impair the beauty of the surroundings. In most conservancy systems, it is common to have open drains designed to carry less foul sewage from bathroom and sinks which will no doubt interfere with the aesthetics of the area.

Despite the numerous benefits associated with the modern water-carried system, it has not been possible to completely replace the old conservancy system. This is mainly because huge capital funds are required for the construction of such a system. Moreover, for the proper functioning of a sewerage system, sufficient amount of water must be made available to the population. For this reason, a reliable and assured water supply must first be installed before installing the sewerage system (Garg, 1979).

2.3 The Importance of having a Central Wastewater Treatment Plant

Wastewater from households, industries and combined sewers is collected and transported to the treatment plant by a sewer system. It is then treated in the plant and the common practice is to dispose of the resulting effluent into rivers, lakes or estuaries for dilution. This method of disposal is generally the only feasible method for several communities and it ensures adequate water resources for downstream users during droughts. The main purpose of wastewater treatment is therefore to prevent pollution of the receiving watercourse. Other methods of disposal include irrigation, infiltration, evaporation from lagoons, and submarine outfalls extending into the ocean.

The characteristics of a wastewater depend extensively on the type of sewer collection system and the industrial wastes entering the sewers. The degree of treatment required is determined by the beneficial uses of the receiving stream or lake (Hammer and Viessman, 1985). When untreated wastewater accumulates and is allowed to go to septic, the decomposition of the organic matter it contains will lead to nuisance conditions including the production of malodorous gases. In addition, untreated wastewater contains numerous pathogenic micro-organisms that dwell in the human intestinal tract. Wastewater also contains nutrients, which can stimulate the growth of aquatic plants, and may contain toxic compounds that may be mutagenic or carcinogenic. For these reasons, the immediate nuisance-free removal of wastewater from its sources of generation, followed by treatment, reuse, or dispersal into the environment is necessary to protect public health and the environment.

2.4 Health, Social and Environmental Impact

Rapid urbanization has immense effects on the infrastructure of supply, disposal and treatment of water, wastewater and solid waste. The environment is put under serious strain by deficient or missing wastewater and waste treatment plants. This situation not only causes worldwide environmental damage but also causes public health problems. Due to the close link that exists between water supply and sanitation, human health and development, it is important to find efficient and cost-effective ways to manage water supply and wastewater treatment. The force behind the construction of the sewerage in Kiserian and Ongata Rongai Towns is to deal with the growing population and the need to prevent ground and surface water pollution due to the reliance on pit latrines and soak pits.

In many parts of the world, environmental and health problems have often been caused by discharging inadequately treated wastewater into watercourses. Water quality issues arise when these waters are discharged to water bodies that are used as water supplies. Case in point is the wastewater treatment plant in Dandora area in Nairobi where the effluent from stabilization ponds flows into the Nairobi River through an open channel. The effluent from the stabilization ponds is discharged without undergoing disinfection and livestock drink from the open channel. This is an operation and maintenance problem that might cause harm to the livestock and human health in the long run (Wang, H. et al, 2014).

The discharge of inadequately treated effluent into water bodies has also caused the deterioration of water quality in Lake Victoria, Kisumu, Kenya. A report done by Aquaclean Services Limited (2015) pointed out that the lake is being contaminated by effluent from sewer connections that are emptied untreated into the lake due to dilapidated and faulty sewage treatment facilities and from undertreated wastewater from manufacturing industries. Such occurrences have caused an increase in waterborne diseases such as cholera and stomach cramps. The undertreated wastewater from industries also has a high nutrient level which has been identified as one of the causes for the flourishing of water hyacinth in the lake resulting in the reduction of fish and physical interference with access to water supply, commercial transportation services and provision of a habitat for the vector mosquito that causes transmission of malaria.

Wastewaters may contain either odorous compounds or compounds that produce odours during the process of treatment. Odours have been rated as the foremost concern of the public relative to the implementation of wastewater treatment facilities. The control of odours has in the past few years become a major consideration in the design and operation of wastewater collection, treatment and disposal facilities. In many areas, projects have been rejected by the public because of the concern over the potential for odours. In Kajiado County for instance, the property owners in Rimpa Estate opposed a proposed project to construct a wastewater treatment plant in the area claiming that over 1,000 residents will be affected by the stench. The residents were also concerned that the plant may result in the contamination of the river in the area which is used for irrigation downstream (Marindany, 2015). At low concentrations, odours may not cause bodily harm but the psychological stress it causes is of concern. Offensive odours can cause poor appetite, impaired respiration, nausea, vomiting and mental perturbation. In extreme situations, it may lead to the deterioration of personal and community pride, interfere with human relations, discourage capital investment, lower socioeconomic status, and deter growth (Metcalf & Eddy, 2003).

From the cases pointed out in the above paragraphs in this subsection, it is clear that the provision of water supply, sanitation and wastewater services generates substantial benefits for public health, the economy and the environment. Wastewater treatment interventions can generate significant benefits that may be difficult to assess in monetary terms except in obvious cases such as for certain economic sectors like fisheries, tourism and property markets. Protecting water resources by ensuring the effluent discharged is treated to the correct standards ensures water supply in a sustainable manner. This can deliver clear and sizeable benefits for both investors in these services and to the water users. Non-economic benefits associated with the provision of a central wastewater treatment facility include the raising of social status and improving cleanliness and overall well-being of the residents. Furthermore, if there are reliable wastewater treatment facilities in place, the discharged water can always be stored and reused in irrigation, groundwater recharge, and non-potable urban uses such as fire protection, air conditioning and toilet flushing.

2.5 Wastewater Management in Towns without a Sewerage System

In modern days, towns that do not have a sewerage system mostly utilize on-site treatment and/or disposal methods. The common methods are the use of septic tanks, cesspools and pit latrines.

Septic Tanks – this is a small-scale sewerage treatment system. It usually consists of a tank connected to an inlet pipe at one end and a septic drain field at the other end. This tank separates the wastewater into three layers. The bottom layer is comprised of large solids, better known as sludge; the middle layer is relatively clear water; and the top layer comprises floating solids.

Baffling action within the tank allows the relatively clear water in the middle layer to move into the lateral field while keeping the components of the other two layers in the tank for further bacterial breakdown.

Soak pits – a soak pit is a covered, porous-walled chamber that is filled with coarse rocks and gravel. Its purpose is to aid in the disposal of pre-settled effluent from a collection and storage/treatment technology, such as a septic tank. It allows the effluent to slowly infiltrate into the surrounding soil. A layer of sand is spread across the bottom to help disperse the flow.

Cesspools - this is a conservancy tank that is constructed underground for the reception and storage of sewage from households. The contents are transported periodically by an operator to a central site or to the nearest manhole for disposal. It is only used in cases where soils are non-porous rendering the use of septic tanks and any other forms of disposal impractical.

Pit Latrines – a pit dug in the ground beneath a toilet to collect and store human excreta until it is full after which it is emptied. There will be seepage of water into the surrounding soil through the sides and bottom of the pit. During storage and seepage, the collected waste undergoes decomposition of organic substances under anaerobic conditions, bacterial and viral die-off. The decomposition of organic matter results in the reduction of the volume of sludge accompanied by the release of malodorous gases. Special types of pit latrines known as ventilated pit latrines (VIP) are widely adopted to control odour and insects where the vent acts to draw odour and insects up and out of the latrine.

These methods of onsite treatment and disposal of wastewater are however unsuitable for towns due to one or more of the following reasons;

- space limitations
- groundwater pollution
- fly, mosquito and rodent breeding (if not carefully designed and operated)
- occasional foul odours in areas of residence
- need for regular emptying
- topography, soil type and other geologic limitations

The force behind the construction of a sewerage system for Kiserian Town is to deal with the growing population and need to prevent ground and surface water pollution due to the reliance on pit latrines and soak pits.

2.6 Analysis and Selection of Wastewater Treatment Process

The choice of the wastewater treatment process to be adopted for a particular locality should be derived from a balance between technical and economical criteria, taking into account quantitative and qualitative aspects of each alternative. Criteria for making the best choice can be attributed to the various aspects connected essentially with the situation at hand therefore common sense and experience when attributing the relative importance of each technical aspect are essential. While the economic side is fundamental, it is important to remember that the best alternative is not always the one that simply presents the lowest cost in economic-financial studies. The following aspects are considered important in the selection of wastewater treatment systems. In general, the items are organized in order of decreasing importance for developing countries like Kenya. Each of these factors must be evaluated in terms of the local conditions and the technology employed;

i. Constructions costs
ii. Sustainability
iii. Operational costs
iv. Simplicity
v. Efficiency
vi. Reliability
vii. Sludge disposal
viii. Land requirements
ix. Environmental impacts

2.7 Waste Stabilization Ponds

Waste stabilization ponds (WSP) are large shallow basins that are enclosed by earth embankments. They are the simplest method of wastewater treatment relying entirely on natural processes involving both algae and bacteria. The treatment processes are unaided by man and the work of the wastewater treatment engineers is reduced to merely allocating a properly dimensioned place for the treatment process. Since the process is unaided by man, the rate of oxidation is slow and as a result hydraulic retention times are longer than in conventional wastewater treatment (e.g. activated sludge and oxidation ditches). The long hydraulic retention times and depth limitations require that waste stabilization ponds occupy very large tracts of land.

Waste stabilization ponds are commonly employed in developing countries due to their simplicity, low-cost, high efficiency and robustness. They are without doubt preferred in developing countries because as much as they require much more land than other treatment processes, acquiring large tracts of land in these countries is generally easier and more cost effective than spending large amounts of money on energy as is necessitated by most of the conventional wastewater treatment methods.

WSP are used in most parts of the world. In New Zealand, WSP are the most common form of wastewater treatment, with 100 of the 160 plants serving populations less than 1000 being WSP (Archer and Mara, 2003). There are many WSP systems in Australia, including those at the Western Treatment Plant in Melbourne. They are common in all parts of the developing world, where they can serve large populations, for example, the Dandora WSP in Nairobi, Kenya serve a sewered population of approximately 1 million, and the Al Samra WSP near Amman, Jordan serve a population of approximately 2.6 million (Mara, 2003).

There are three types of WSPs: anaerobic, facultative and maturation ponds designed for different purposes as discussed in the following sections. These ponds are usually arranged in series starting with the anaerobic ponds, then the facultative ponds and finally the maturation ponds. Maturation ponds are only necessary if a series of the other two types of ponds do not achieve the required effluent standards. It is commonly observed that the effluent from a series of ponds is of better quality than that from a single pond of the same size. Marais (1974) proved that maximum efficiency in a series of ponds is achieved when the retention time in each pond is the same. In practice, it might not be possible to have all ponds in a series having the same retention time due to the use of differing basis for the design of anaerobic and facultative ponds (Mara, 2003).

2.7.1 Anaerobic Ponds

Anaerobic ponds are designed primarily for BOD removal and are typically the first in a series of ponds. They are constructed to a depth of 2 to 5 metres and contain neither dissolved oxygen nor algae. A depth of 3 m is commonly assumed for preliminary design. Industrial wastewaters characteristically contain compounds such as heavy metals and organic matter that are toxic to algae. This fact necessitates the treatment of wastewater in anaerobic ponds prior to treatment in facultative and maturation ponds so that these toxic substances are degraded. Floating materials, collectively known as scum, are retained in anaerobic ponds. The scum prevents penetration of light into the pond inhibiting both algal growth and photosynthesis. This ensures the ponds remain in anaerobic conditions.

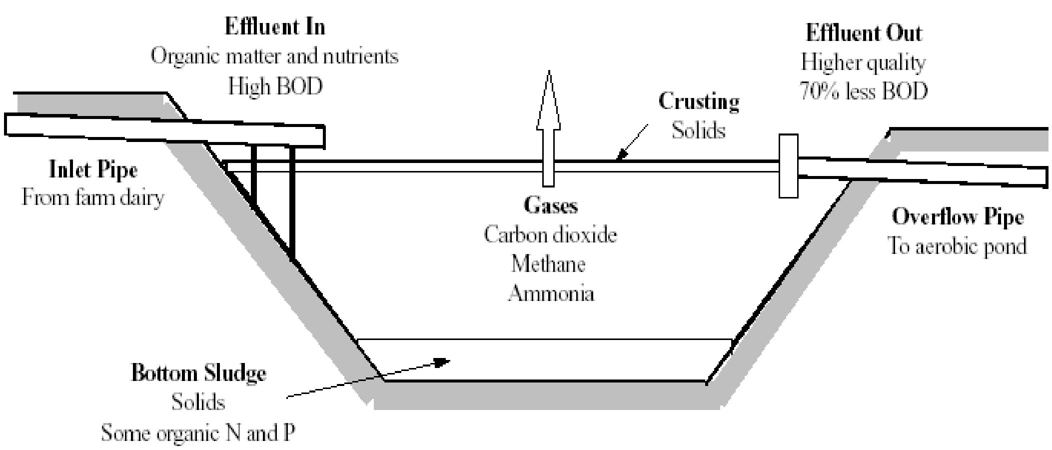

Figure 1: Operation of an anaerobic pond *(Source: Ramadan and Ponce, 2016)*

Anaerobic ponds are sized on the basis of volumetric BOD loading and receive a very high organic loading. Removal of BOD is by sedimentation and subsequent anaerobic digestion of settleable solids. A properly designed anaerobic pond that is not significantly under-loaded is expected to achieve greater than 60% BOD removal at 20^0C (Mara, 2003). They are therefore considered efficient and have a short retention time. At a temperature of 20^0C, a retention time of one day is usually sufficient. Mara and Pearson (1998) recommend the design values of λ_v given in the table below that ensure odour problems are avoided. The design temperature is taken as the mean air temperature of the coldest month.

Table 1: Recommended Design Loadings for Anaerobic Ponds

Temperature (^{0}C)	Volumetric Loading (g/m^{3}day)	BOD Removal (%)
<10	100	40
10-20	20T-100	2T+20
20-25	10T+100	2T+20
>25	350	70

Source: Mara, 2003

Anaerobic ponds are designed on the basis of volumetric BOD loading (λ_v g/m^{3}d) which is given by:

$$\lambda_v = {}^{L_i Q}/_{V_a}$$

Where L_i is the influent BOD in g/m^3; Q is the flow in m^3/d; and V_a is the anaerobic pond volume in m^3

Once the appropriate value of λ_v is selected, the pond volume is calculated from the equation:

$$t_a = {}^{\mathrm{Va}}/_{Q}$$

A value of retention time less than one day is not recommended and if the equation above gives a value less than one, a value of one day should be used and the corresponding V_a recalculated. The anaerobic pond area is then given by the equation:

$$A_a = {}^{Qt_a}/_{D_a} = {}^{L_i Q}/_{\lambda_v D_a}$$

Where D_a is the liquid depth in metres

2.7.2 Facultative Ponds

There are two types of facultative ponds; primary and secondary facultative ponds. Primary facultative ponds receive raw wastewater while secondary facultative ponds receive effluent from anaerobic ponds. The principal function of facultative ponds is BOD removal. The surface loading in facultative ponds should be relatively low to allow the growth of a healthy algal population. This is because facultative ponds are photosynthetic ponds, i.e. the oxygen needed by the bacteria in the ponds to oxidise the wastewater BOD is generated mainly by algae that grow naturally in these ponds.

Facultative ponds are best designed to a depth range between 1m and 1.8 m with the value of 1.5 m being commonly used for preliminary design. Depths below 1 m encourage the emergence of vegetation making the pond an ideal breeding ground for mosquitoes. On the other hand, depths greater than 1.8 m result in low light penetration causing a predominance of oxygen consumption over its (oxygen) production. This is undesirable as it makes the pond predominantly anaerobic rather than aerobic such that it has a low safety factor in normal operation and it would thus be less capable of handling pollution shockwaves.

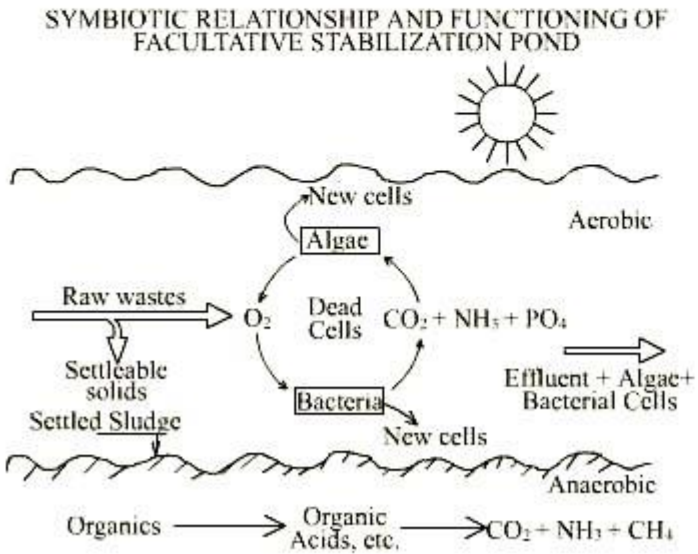

Figure 2: Functioning of Facultative Ponds (Source: http://nptel.ac.in/)

It is best to design facultative ponds on the basis of surface BOD loading because light needed for algal photosynthesis arrives from the sun at the pond's surface. Algal oxygen production is therefore a function of area so the BOD loading must also be a function of area (Mara, 2003). The surface loading (λ_s kg/ha/day) is given by:

$$\lambda s = \frac{10 * \mathrm{L_i Q}}{\mathrm{A_f}}$$

Where A_f = facultative pond area, m^2; and LiQ is the mass of BOD entering the pond, g/day

The permissible design value of the surface loading increases with temperature and the following global design equation was developed by Mara (2003) as a guide for loading facultative ponds:

$$\lambda s = 350(1.107 - 0.002\mathrm{T})^{\mathrm{T}-25}$$

The value of surface loading determined from the above equation is then used to obtain a suitable pond area. The facultative pond retention time is the calculated using the equation below;

$$t_f = {}^{\mathrm{A_f} D_f}\!/_{Q}$$

Where D_f is the pond depth, m; Q is the flow, m^3/day.

A minimum value of 5 days should be adopted for temperatures less than 20^0C and 4 days for temperatures greater or equal to 20^0C. This is to minimize hydraulic short-circuiting and to give the algae sufficient time to multiply (Mara, 2003). The BOD removal of facultative ponds can be estimated from the equation below:

$$L_e = \frac{L_i}{1 + k_1 t_f}$$

Where L_i is the BOD of either the raw wastewater in the case of primary facultative ponds or the anaerobic pond effluent in the case of secondary facultative ponds in mg/l; k_1 is the first-order rate constant for BOD removal, day^{-1} given by the equation;

$$k_{1(T)} = k_{1(20)}(1.05)^{T-20}$$

The design values $k_{1(20)}$ are 0.3day^{-1} for primary facultative ponds and 0.1 day^{-1} for secondary facultative ponds and; L_e is the unfiltered BOD which includes the BOD of the algae present in the facultative pond effluent. The value Le is the unfiltered BOD. It includes the BOD of the algae present in the facultative pond effluent. The non-algal fraction of the BOD is usually taken as 0.3 of unfiltered BOD for design purposes.

Therefore Le (filtered) = [0.3(Le) unfiltered]

2.7.3 Maturation Ponds

The primarily purpose of maturation ponds is to reduce the number of pathogens in the effluent from facultative ponds. The pathogens, predominantly faecal bacteria and viruses, should be reduced to a level suitable for aquacultural re-use in cases where the effluent is discharged in inland watercourses. BOD, suspended solids and nutrients such as nitrogen and phosphorous are also removed in maturation ponds but only at a very slow rate. Maturation ponds typically have depths of 1m as they are characteristically aerobic and shallow depths allow for great penetration of light. If lined, shallower depths than 1 m may be used as this would help achieve higher pathogen removal. However, for unlined ponds, shallower depths are likely to result in emergent macrophytes that provide a suitable shaded habitat for mosquito breeding.

The design of maturation ponds relies on the equations developed by Marais (1974) modelled on first-order kinetics in a completely mixed reactor. The equation for a single pond is given by:

$$N_e = \frac{N_i}{1 + k_{B(T)}t}$$

Where $k_{B(T)} = 2.6(1.19)^{T-20}$; N_e and N_i are the numbers of E coli per 100 ml of pond effluent and influent respectively; $k_{B(T)}$ is the first-order rate constant for E coli removal at T^0C in a completely mixed reactor, day^{-1}; t is the mean hydraulic retention time in the pond in days and; T is the design temperature, ^0C. The design temperature used is the mean temperature of the coolest month of the year.

For a series of WSP comprising an anaerobic pond, a secondary facultative pond and n equally sized maturation ponds, the equation for the design of maturation ponds can be rewritten as:

$$N_e = \frac{N_i}{\left(1 + k_{B(T)}t_a\right)\left(1 + k_{B(T)}t_f\right)(1 + k_{B(T)}t_m)^n}$$

Where N_e and N_i are now the E coli numbers per 100 ml of the final effluent and the raw wastewater, respectively; the subscripts a, f and m refer to anaerobic, facultative and maturation ponds; and n is the number of equally sized maturation ponds.

Several constraints have been suggested by Marais (1974) for the design of maturation ponds. The first one is the minimum value for the retention time in maturation ponds which is set at a value of 3 days to allow for algal reproduction and to minimize hydraulic short-circuiting. The second constraint has no theoretical justification but is based on engineering judgement. It states that the retention time in the maturation pond should not be greater than that in the facultative pond. The third constraint sets a limit on the BOD surface loading on the first maturation pond and it is recommended that the value be 75 per cent of the BOD surface loading on the facultative ponds.

The surface loading (λ_s kg/ha/day) for maturation ponds is given by

$$\lambda s = \frac{10 * L_i D_m}{t_m}$$

2.8 Evaluation Criteria of the Land Area

2.8.1 Sewage Treatment Works Location

A site comparison study is usually carried out after alternatives have been screened and rough sizing of the processes is complete. There are many parameters that need to be taken into account before deciding the most feasible situation. For instance, in highly urbanized areas, the availability of land may preclude all but one site. In a case where more than one site is available a number of major issues may need to be put into consideration.

2.8.2 Land Use

Land use is a good indicator of the available sites in terms of how much land is potentially suitable and available for waste application. For example, agricultural land can often serve or be adapted for use in waste treatment. A brief review of land use maps can avoid consideration of areas with urban or industrial development, historical value, or unique environmental features. Occasionally, the possible site will be limited to land already owned by a municipality or industrial concern. In such cases, the use of the land surrounding the site should be evaluated to determine if these uses are compatible with a waste application system. Projected land use plans, where they exist, may also eliminate certain areas from consideration. (Loehr et al, 1979)

Cost of land acquisition and the availability of land for expansion purposes are major elements in the plant location process. When some people dwelling on the proposed site are affected and have to be moved and resettled elsewhere, a serious problem often arises. Rehabilitation can be a messy affair besides the long delays which can occur if not satisfactorily resolved. The physical characteristics of the site alternatives that must be evaluated include the potential for flooding, foundation stability, groundwater intrusion, and the difficulty in preparing the site. For example, the need for blasting of rock may make the cost prohibitive for an otherwise ideal site. Other issues to be considered include wetland infringement, the availability of alternate, independent sources of power, waste disposal options, public acceptance, and security (Davis, 2010).

Waste Stabilization Ponds should be located at least 200 m (preferably 500 m) downwind from the community they serve and away from any likely area of future expansion. Odour release, even from anaerobic ponds, is most unlikely to be a problem in a well-designed and properly maintained system, but the public may need assurance about this at the planning stage, and a minimum distance of 200–500 m normally allays any fears. It is important to ensure that there is vehicular access to and around the ponds so the site should be flat or gently sloping, in order to minimize earthworks. WSP should not be located within 2 km of airports, as birds attracted to the ponds constitute a risk to air navigation (Mara, 2003).

The presence of wastewater treatment plants usually has negative effects on adjacent land use and consequently on land values. The mere knowledge that a wastewater treatment plant is located nearby is generally enough to reduce the land values in the vicinity owing to actual or imagined nuisances from odours, flies and mosquitoes (Arceivala, 2007). These factors have to be considered in selecting sites for location of sewage treatment plants. In Kajiado County for instance, the property owners in Rimpa Estate opposed a proposed project to construct a wastewater treatment plant in the area alleging that they will be affected by the unpleasant smell (Marindany, 2015). Offensive odours can cause poor appetite, impaired respiration, nausea, vomiting and mental perturbation. In extreme situations, it may lead to the deterioration of personal and community pride, interfere with human relations, discourage capital investment, lower socio-economic status, and deter growth (Metcalf & Eddy, 2003).

2.8.3 Topography

Topography of the general area to be served, that of tentative sites suitable for treatment plant location and the area for disposal works have to be determined. Contour plans of the tentative sites are necessary for making layouts. Topographic information is important in locating immediate and final pumping stations. It is desirable that the treatment plant be located in an area that allows feeding into the treatment plant through gravity to minimize pumping which is expensive at operational stages. The overall configuration of the area's landscape determines the pathway and the rate of surface water movement and often the subsurface flow and this information is therefore important in establishing how well the site will handle drainage. For instance, if a site is nearly level and the adjacent terrain is higher and sloping, the most probable case is that it will receive excess water via surface runoff from the higher ground. Such conditions might preclude additional inputs of wastewater unless good drainage is provided.

An example of problems brought about by topography is pointed out in a report done by Frame Consultants Ltd in the documentation for the Mwingi Town (in Kitui County, Kenya) water supply and sanitation project-Phase II. Frame Consultants established that the topography of Mwingi Town would not enable the usage of a single site for the treatment works without pumping or tunnelling. This is because the western side of the town discharges to a completely different basin from the eastern side and these catchments cannot meet on ground level unless deep under surface excavation works are undertaken. Consequently, the consultants recommended that 2 No. treatment works be provided at different locations; Site A, serving Zone A on the western side of the town is located near Tyaa river and serves a catchment area of 4.3km^2 while Zone B, on the eastern side of the town serves a catchment area of 2.8km^2 and is located near river Kivou and drains into it (Frame Consultants Ltd, 2013).

2.8.4 Geotechnical Considerations

Geotechnical aspects of WSP design are very important. In France, for example, a third of the WSP systems that malfunction do so because of geotechnical problems which could have been avoided at the design stage (Bernhard and Kirchgessner, 1987). Poor geotechnical design is also common in Mexico (Mantilla et al, 2002).

The main aim of carrying out geotechnical investigations is to ensure correct embankment design and to establish whether the soil is insufficiently impermeable to require lining of the pond. The maximum height of the groundwater table should also be determined during this stage, and the following properties of the soil at the proposed pond location must be determined:

i. particle size distribution;

ii. maximum dry density and optimum moisture content;
iii. Coefficient of permeability;
iv. Atterberg limits; and
v. organic content

Organic and plastic soils and medium-to-coarse sands are considered unsuitable for embankment construction. If there is no suitable local soil with which at least a stable and impermeable embankment core can be formed, it must be brought to the site at extra cost and the local soil, if suitable, used for the embankment slopes. Embankments should ideally be constructed from the soil excavated from the site. During construction, it is important to strive to achieve a balance between cut and fill, although it is worth noting that ponds constructed completely in cut may be a cheaper alternative, especially if embankment construction costs are high (Mara, 2003).

2.8.5 Climatic Feasibility

Localized areas provide little to no choice with respect to climatic conditions. Nevertheless, the climate of a particular area strongly affects the overall feasibility as well as the ultimate design of treatment systems. Suitable temperature, direct sunlight and suitable moisture conditions are necessary for organic waste decompositions and for the growth and development of vegetative cover. Temperature is a key factor in the design of most treatment processes, especially the natural-based non-mechanised ones. Warm temperatures decrease land requirements, enhance conversion processes, increase removal efficiencies and make the utilisation of some treatment processes more feasible. The fact that most warm climate regions are situated in developing countries has made it easier for these countries to favour waste stabilisation ponds as a method of wastewater treatment. When applied in lower temperature regions, stabilization ponds occupy much larger areas and are subjected to a decrease in performance during winter (Von Sperling, 2007).

2.9 Compensation for Land Acquired for Public Benefit

In Kenya, and especially in cities and satellite towns, land ownership is a great investment with hefty returns. The galloping land prices in satellite towns such as Ongata Rongai and Kiserian is being driven by rising demand as the population that works within the Nairobi Central Business Ditrict increases. Due to poor or lack of town planning, there lacks adequate wayleaves for the construction of large infrastructure such as wastewater treatment works. If such infrastructure has to be put in place in a town, the government through various parastatals has to acquire privately owned land. This leads to high land acquisition costs especially in areas where massive development has already taken place. Huge compensation costs are competing with other development needs making it difficult to provide the necessary infrastructure for public benefit.

The rights of land owners in Kenya are protected by the law and no private land can be acquired by the government compulsorily except in accordance with the law. The circumstances under which private land may be acquired for the benefit of the public and the conditions to be observed are stipulated in the constitution and the Land Acquisition Act. The Constitution expressly states that no private property shall be compulsorily acquired by the government unless, among other conditions, provision is made by a law applicable to that taking of possession or acquisition for the prompt payment of full compensation to all persons interested in the land (Sifuna, 2003). The formula for determining the amount of compensation is stipulated in the Land Acquisition Act. To arrive at the appropriate amount of compensation, the principles set out in the schedule to the Act are applied. The matters to be considered in compensation as stipulated by the Act include:

a) The market value of the land
b) Damage sustained or likely to be sustained by persons interested at the time of the commissioner taking possession of the land by reason of severing the land from his other land
c) Damage sustained or likely to be sustained by persons interested at the time of the commissioner taking possession of the land by reason of the acquisition injuriously affecting his other property, whether movable or immovable or in any other manner or his actual earnings

d) If in consequence of the acquisition, any of the persons interested is or will be compelled to change his residence or place of business, reasonable expenses incidental to the change

e) Damage genuinely resulting from diminution of the profit of the land between the date of publication in the Gazette of the notice of intention to acquire the land and the date the commissioner takes possession of the land.

Under the Land Acquisition Act, compensation can be either in kind in the form of land or money.

2.10 Design Considerations

2.10.1 Population Projections

Before designing a wastewater treatment plant, it is essential to project the future population of the communities to be served by the plant. The plant should be able to serve the community satisfactorily generally for 20 to 30 years. It is however tricky to forecast the population growth due to the economic and social factors involved. Various mathematical extrapolation methods are used to obtain future population. One of the most commonly used is the geometric progression method. In this method, population growth is estimated as a function of the existing population at every instant using the equation:

$$\mathrm{P_n} = \mathrm{P_0}(1 + \mathrm{r})^{\mathrm{n}}$$

$Where\ P_n = sewered\ population\ in\ n\ years\ time$

$P_0 = present\ sewered\ population$

$r = the\ annual\ population\ growth\ rate\ expressed\ as\ a\ decimal\ fraction$

$n = number\ of\ years$

The value of the present population that is to be sewered and the anticipated value of r should be obtained from the local municipal planning department. These values should not only be consistent with values from the recent past, but they also need to take into account any major developments expected to occur in the planning period of n years. The value of n is usually taken as 20 years, but it is better to determine Pn in steps of 5 years (i.e for n= 5, 10, 15 and 20 years) in order to decide the way to phase the development of the proposed wastewater treatment facilities. Phasing is important since, rather than building a complex waste stabilization pond to serve the population expected in 20 years time, it is financially more sensible to build it for the population expected in, say, 5 years time and then expand it in 4 years time to serve the population anticipated in 10 years time, and so on (Mara, 2003). It is however important to note that all the land required to adequately serve the population expected in 20 years must be bought at the beginning of the project.

2.10.2 Average Sewage Flow

Generally, the amount of sewage corresponds to the water consumption of the community. The fraction of the supplied water that enters the sewerage system varies due to the fact that part of the water consumed may be incorporated into storm water or infiltrate due to activities such as watering of gardens and various cleaning activities. The fraction of the supplied water that enters the sewerage in the form of sewage is called Return Co-efficient, denoted R. Mathematically;

$$R = \frac{\text{sewage flow}}{\text{water flow}}$$

Typical values of R vary between 60% and 100%. A value of 80% is usually adopted for most African countries. The values of water consumption per capita vary from locality to locality due to factors such as water availability, climate, level of industrialization, water cost, economic level of the community et cetera.

2.10.3 Design Flow Rates

The average daily flow, expressed in volume per unit time, maximum daily flow, peak hourly flow, minimum hourly and daily flows, and design peak flow are generally used as the basis of design for sewers, lift stations, wastewater treatment plants, treatment units, and other wastewater handling facilities. The design average flow is the average of the daily volumes to be received for a continuous 12-month period of the design year. The average flow may be used to estimate pumping and chemical costs, sludge generation, and organic-loading rates. The maximum daily flow is the largest volume of flow to be received during a continuous 24-hour period. It is employed in the calculation of retention time for equalization basin and chlorine contact time. The peak hourly flow is the largest volume received during a 1hour period, based on annual data. It is used for the design of collection and interceptor sewers, wet wells, wastewater pumping stations, wastewater flow measurements, grit chambers, settling basins, chlorine contact tanks, and pipings.

The design peak flow is the instantaneous maximum flow rate to be received. The peak hourly flow is commonly assumed as three times the average daily flow. The minimum daily flow is the smallest volume of flow received during a 24-h period. The minimum daily flow is important in the sizing of conduits where solids might be deposited at low flow rates. The minimum hourly flow is the smallest hourly flow rate occurring over a 24-h period, based on annual data. It is important to the sizing of wastewater flow meters, chemical-feed systems, and pumping systems (Dar Lin, 2007).

2.10.4 Design Period

The design period of a facility is the estimated length of time it is expected to meet the demand, that is, the design capacity. This should not be confused with the life expectancy which is determined by wear and tear. The design period selected is dependent on the following:

i. Environmental and Regulatory constraints
ii. Wastewater Characteristics
iii. The estimation of population, commercial and industrial growth
iv. Facility limits
v. The useful life of the structures and equipment
vi. The ease or difficulty of expansion
vii. Performance in early years of life under minimum hydraulic load.

More often than not, the financing of water and wastewater works is done by the state. This fact also influences the selection of the design period, often resulting in the selection of a design period that is substantially less than the useful life of the plant. Population data and forecast estimates are also important for numerous policy decisions. Historic records do provide a basis for developing trendlines and making forecasts of future population growth. For short-range forecasts of 10 to 15 years, data extrapolation is of sufficient accuracy for planning purposes. For long-range forecasts of 15to 50 years, more sophisticated techniques are employed. Consideration of the flow rates during the early years of operation is often overlooked, and over sizing of equipment and inefficient operations can result (Metcalf & Eddy, Inc., 2003).

2.11 Description of Project Area

2.11.1 Project Area

The project area is located within the service area of the Oloolaiser Water and Sewerage Company Limited (OWSC). It spreads from Ongata Rongai, along Magadi Road up to Kiserian located southwest of Nairobi. Kiserian and Ongata Rongai are situated in Kajiado County, in the former Rift Valley Province, Kenya. The two towns are approximately 7km apart. The population of both towns has been constantly increasing mainly due to their proximity to the Nairobi Central Business District, the availability of affordable housing as well as access to transport and electricity.

The project area covers an area of approximately 29.3km^2 and spreads over thirteen sub-locations. The towns' administrative boundaries have changed over the years as they cover a much larger area with their ever increasing population. Population densities vary significantly within the project area and can be divided up in urban and peri-urban areas with population densities ranging from 170 people to nearly 25,000 people per km^2. It is estimated that approximately 60% of the population in the project area resides in medium and high-cost houses while about 40% of the population lives in low-cost areas. Over the last five years, the population in the project area grew by 5.6% annually which is significantly higher than the national average of 3% in urban areas. It is expected that the annual growth rate in these areas will remain high at approximately 4% for the next 30 years and that this growth will remain higher in Ongata Rongai than in Kiserian due to its proximity to Nairobi.

2.11.2 Ongata Rongai

Ongata Rongai is a fast-growing, mainly residential area at the outskirts of Nairobi close to The Nairobi National Park. It is a multi-class area which is dominated by the middle-class living in middle-class housing situated mostly around the urban centre. The town is located between the Kaputei plains and the western slopes of Ngong hills approximately 17km from Nairobi. According to Majidata, there are three low-income areas in Ongata Rongai, namely; Mosoi Range (Kisumu Ndogo), Kware A and Kware B. Ongata Rongai urban centre comprises four sub-locations. Ongata Rongai sub-location is located south of Magadi Road and comprises the main commercial area surrounded by low-cost apartment houses as well as high and medium-cost housing areas located between Magadi Road and Kandisi River.

On the other side of Magadi Road are Kware and Mosoi Range sub-locations which are densely populated areas with mainly low-cost housing. Medium-cost housing estates have developed mainly in Kandisi sub-location. The project targets both the urban and peri-urban population that is rapidly developing along Magadi Road. Ongata Rongai is densely populated in its eastern side where there are multi-storey apartment buildings while the western side is settled with individual medium to high-cost houses. The population on the eastern side as well as that that of Kware and Mosoi Range is not expected to grow at the same rate it has been over the coming years as the areas are already densely packed.

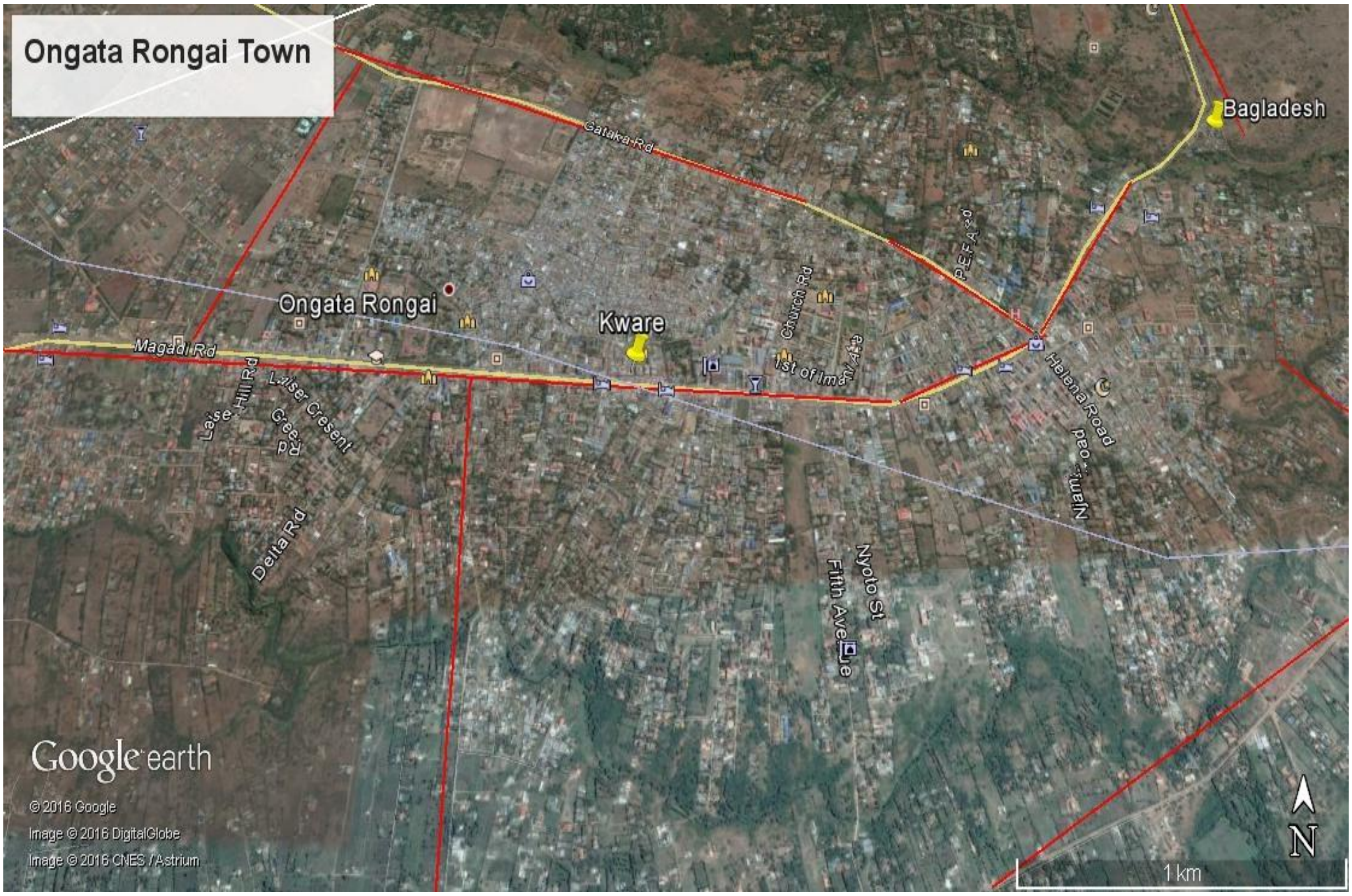

Figure 3: An Aerial View of Ongata Rongai Township *(Source: Google Earth)*

2.11.3 Kiserian

Kiserian Town borders Ongata Rongai and stretches in the direction of Ngong town. It lies at the foot of Ngong Hills and is situated north of Kiserian Dam which is the main raw water source sustaining the water supply in the project area. There are four sub-locations in this township namely; Naserian, Olchorro-Onyore, Olekasasi and Oloosurutia. The only low-income area recorded by Majidata in this town is the Oloosurutia (Gichagi) area but informal areas with low-income characteristics have developed on both sides of Kiserian Dam. The density in Olchorro-Onyore on the south of Kiserian Dam is low because the area is largely rural and is used mainly for agricultural purposes. The population density in Olekasasi area is still comparatively low due to a large number of non-developed plots and empty spaces between individual medium-cost houses that are arranged in clusters. It is expected that the population density in this area will still grow over the coming years.

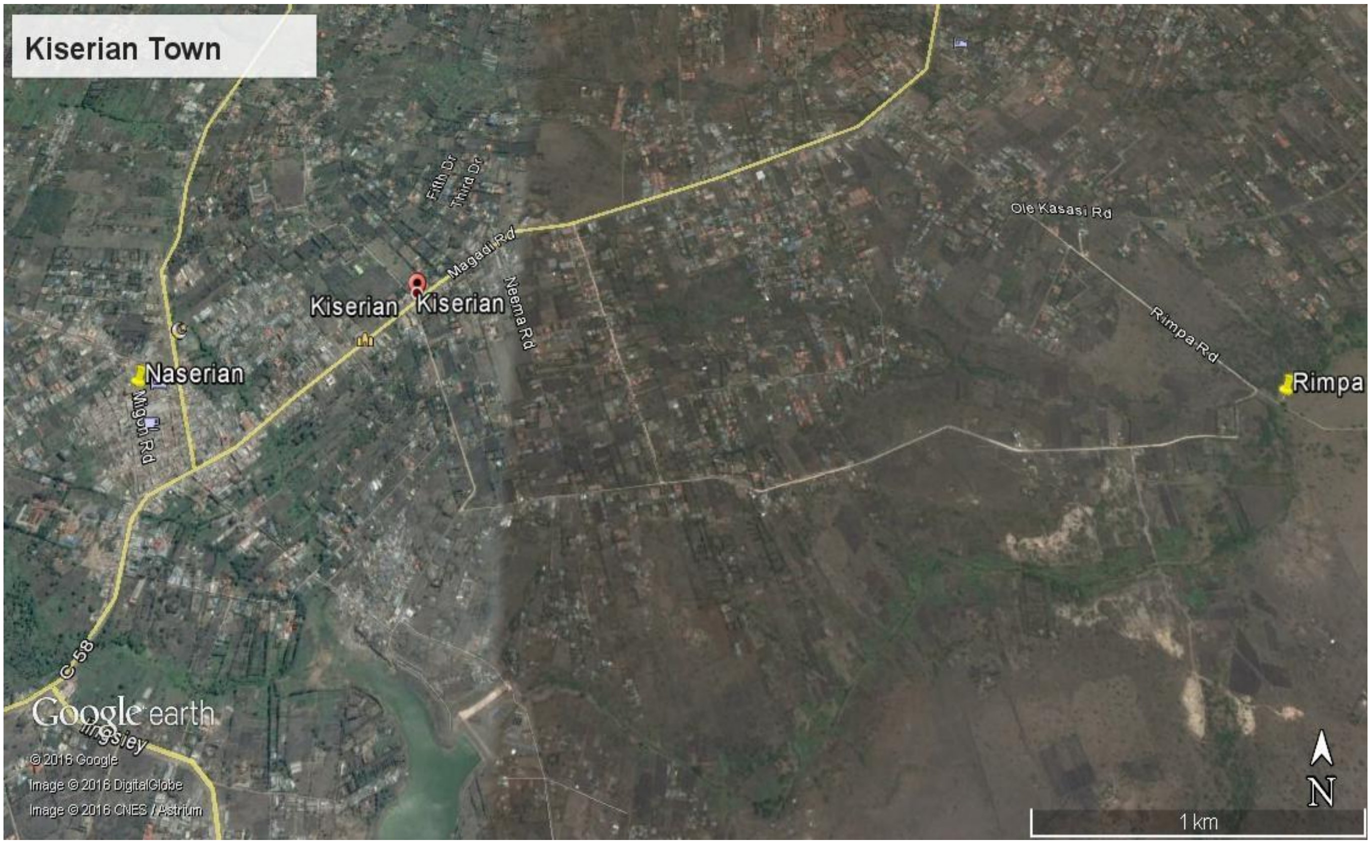

Figure 4: An Aerial View of Kiserian Township *(Source: Google Earth)*

2.11.4 Climate

Kajiado County is to a large extent semi-arid. Both Ongata Rongai and Kiserian areas lie in mainly sub-humid and semi-arid climate with small pockets of humid climate. The rainfall pattern in the project area is bimodal, with the occurrence of short rains from October to December and long rains from March to May. The rainfall in Kajiado District is greatly influenced by the altitude thus heavy rains occur around Ngong Hills, Chyulu Hills, Nguruman escarpment and the foothills of Mt. Kilimanjaro. The mean annual rainfall varies from 400mm to about 1200mm.

Temperatures in Kajiado District vary with altitude and range from a mean maximum of about 30^0C around Lake Magadi to a mean minimum of 16^0C on the slopes of Ngong Hills. Temperatures in the project area generally vary between a mean maximum of 24^0C and a mean minimum of 11^0C. The potential evaporation ranges between 500mm to 800mm per annum.

2.11.5 Topography and Geology

The project area can be described as having a non-homogenous character. The main physical features in the district are plains, with occasional volcanic hills and valleys. The land rises from 500m above sea level around L. Magadi to 2,500m above sea level in the Ngong' hills area. Ongata Rongai lies at an altitude of approximately 1,770m above sea level while Kiserian is located at an elevation of about 1,860m above sea level such that the area slopes as one moves towards Ongata Rongai Town from Kiserian Town. The terrain in Ongata Rongai area slopes from west to the southeast while that in Kiserian Town slopes from west to east. The Kiserian River is located south of Kiserian Town.

Kiserian's geology reflects its location on the slopes of Ngong Hills, and is covered to a large extent by tertiary volcanic soils. The types of rock formations found near Ngong' hills are tertiary volcanic rocks including phonolite, trachyte, basalt, agglomerates and the Kapiti phonolite formed as a result of volcanic eruptions which are associated with the formation of the Great Rift Valley. These have since been overlain by a succession of trachyte phonolitic lava and volcanic sediments. Ongata Rongai area is mainly covered by tertiary volcanic material of the Pliocene period, collectively known as the Ngong Volcanic. The tephries and banasites overlie the Kandisi phonolite. Below the phonolite are the Mbagathi phonolitic trachytes which in turn overlie the Athi tuffs and lake beds which directly overlie the basement system (Gauff Inginieure, 2014).

2.11.6 Current Sanitation Situation in the Project Area

According to the Census 2009, approximately 80% of the population in the project area use pit latrines while 20% of the households, mainly medium-to-high-cost housing, use a flush or pour flush toilet connected to either a communal or individual septic tank or cesspool. It is also reported that pit latrine emptying is not usually carried out. According to Majidata, sanitation coverage using an improved sanitation facility in the low-income-areas is fairly low at approximately 15% in Kware A and 61% in Kware B. In most cases, households share their sanitation facility with their neighbours. On average, only 3% of the households have a toilet inside their residence. Only 8% to 30% of the households use improved pit latrines while most households use unimproved latrines. Flush toilets connected to septic tanks are rare.

2.11.7 Water and Sanitation Related Health Problems Area

According to the Kenya Health Information System (2013), 7.1% of the population living in Ongata Rongai and Kiserian areas suffer from waterborne diseases including diarrhoea, dysentery, cholera, typhoid and bilharzias. Approximately 81% of the cases were diarrhoea followed by typhoid at approximately 18% marginally. Dysentery and bilharzias cases occurred marginally. According to a survey done by Gauff Inginieure (2014), it was established that diarrhoea and amoeba are common in the area and de-worming is done at primary schools. An improvement in the sanitation infrastructure is expected to result in a decrease in the incidence of waterborne diseases within the area.

2.11.8 Population and Water Demand

The per capita consumption rates in this report are done in the same manner as was done in the Feasibility Study and Master Plan for Developing New Water Sources for Nairobi and Satellite Towns. The domestic consumers in the project area are divided into the following four categories and their respective per capita consumption shown:

- High cost (or low density formal housing) – 150 litres/capita/day
- Medium cost (or medium density formal housing) – 100 litres/capita/day
- Low cost (high density formal housing) – 50 litres/capita/day
- Low-income (receiving water from kiosks/ stand posts) – 30 litres/capita/day

For institutional and commercial categories, the water demand estimation is done according to the (MoWI) Practice Manual for Water Supply (2005) and Services as follows:

- Education Sector – 50 litres/capita/day
- Health Sector – 100 litres/capita/day

- Administrative Sector – 25 litres/capita/day
- Commercial & Industrial – 20,000 litres/ha/day

The population and water demand projections are summarised in the following table borrowed from the Nairobi Satellite Towns Water and Sanitation Development Programme (NST-WSDP) – Feasibility Report – Volume 2 (2014) done by Gauff Ingenieure in association with GFA Consulting Group. The feasibility report was done for the water distribution network system in Kiserian and Ongata Rongai areas.

Table 2: Kiserian and Ongata Rongai Townships water demand to the year 2035

Design Demands		Water Demand (m^3/day)					
Description	**Factor**	**Consumption**	**2015**	**2017**	**2020**	**2030**	**2035**
Total Population			**150,700**	**168,000**	**194,300**	**282,200**	**336,100**
Domestic							
High cost area	0.0%	150 l/c/d	-	-	-	-	-
25% High/75% medium cost area	25.9%	113 l/c/d	4,389	4,893	5,659	8,219	8,656
Medium cost area	25.8%	100 l/c/d	3,881	4,327	5,004	7,268	8,656
75% medium/25% low cost area	9.7%	88 l/c/d	1,278	1,425	1,648	2,393	2,850
50% Medium/ 50% low cost area	3.8%	75 l/c/d	424	473	547	794	945
Low cost area	25.4%	50 l/c/d	1,911	2,130	2,464	3,278	4,261
Low income area	9.6%	30 l/c/d	432	482	557	809	964
Total Domestic Water Demand			**12,315**	**13,728**	**15,878**	**23,060**	**27,465**
Education Sector	19,700	50 l/c/d	985	1,098	1,270	1,845	2,197
Health Sector	3,690	100 l/bed/d	369	412	476	692	824
Administrative Sector	9,850	25 l/c/d	246	275	318	461	549
Commercial & Industrial	40	20,000 l/ha/d	739	824	953	1,384	1,648
Overall Water Demand (m^3/day)			**14,654**	**16,337**	**18,894**	**27,442**	**32,683**

Source: Gauff Ingenieure (2014)

The population data is based on the official census from the Central Bureau of Statistics for 2009 and recommended future growth rates. The domestic, institutional and commercial category demands were cumulated to obtain the overall water demand to which the distribution system losses were added to obtain the average Gross Network Water Demand. The population distribution given is per the estimation from the field assessment done by the two consulting groups.

2.11.9 Existing Water Supply in the Project Area

Existing sources of water in Ongata Rongai include the Mbagathi Water Treatment Plant which produces up to 3,000m^3/day, 160 m^3/day borehole at Ongata Rongai Health Centre, and two boreholes at Olekasasi, one producing 80m^3/day and the other 380 m^3/day totalling to a supply of 3,620m^3/day. Kiserian area is served by various boreholes. These are;

- The Kiserian office borehole producing 512 m^3/day
- A borehole of Magadi Road producing 416 m^3/day
- A 360 m^3/day borehole at Narumoru Primary School
- Silanga borehole supplying 360m^3/day
- Kahara borehole supplying 360m^3/day

The other water sources are two springs at Ngong Hills supplying 150m^3/day and 50m^3/day. This totals to a water supply of 2,208m^3/day.

The recently completed Kiserian Dam project is expected to provide 15,700m^3/day treated water to supply Kiserian, Ongata Rongai and its environs. However, only approximately 4,000m^3/day of the dam's capacity is utilized due to the inadequate distribution pipe network infrastructure. Plans are underway to construct additional infill and extension distribution pipelines and to rehabilitate problematic sections of the existing distribution system so that effective utilization of the water sources is realized. There are also a number of privately owned systems that serve the unserved and underserved areas.

CHAPTER THREE

3.0 METHODOLOGY

3.1 Introduction

The various data needed for determining the feasibility of combining the wastewater treatment system for Kiserian and Ongata Rongai townships was collected and analyzed to produce an outcome that satisfied the objective of the report. The data collected included:

i. The population to be served by the wastewater treatment plant in both towns
ii. Existing water supply
iii. Projected average per capita water consumption rates
iv. Land use
v. Topographical data

3.2 Data collection methods

The data collection methods listed below were employed to collect the various data required.

i. Interviews
ii. Literature review/Desk Research
iii. Google Earth Pro Software
iv. Direct Observation

3.2.1 Interviews

Face to face interviews were conducted to collect information on the possible sites for the location of a WSP in Ongata Rongai area. The interviews also gave an insight on the possible compensation costs that would be incurred in land acquisition. These interviews were carried out with the residents living in the proposed sites for the location of the WWTP, the staff of Athi Water Services Board and the staff of Oloolaiser Water and Sewerage Company Limited.

3.2.2 Literature Review/Desk Research

Various reports were reviewed to get information on population and its growth rate, water demand and supply in both townships and land use in the project area. The reports that were reviewed include:

i. Feasibility Study and Preliminary Design Report-Kiserian Sewerage Project-Volume 1&2 –Report (2008);
ii. Nairobi Satellite Towns Water and Sanitation Development Programme (NST-WSDP) – Feasibility Report – Volume 2 (2014)

Information on land use, population projection and water consumption rates was based mainly on reports (i) and (ii) above as they provided estimates from both the 2009 census and field visits. Data from the two reports was used to estimate an appropriate annual growth rate and to come up with sewage generation values for both towns in the ultimate design year of 2036.

3.2.3 Google Earth Pro Software

The Google Earth Pro Software was used to obtain supplementary information on land use and topography of the project area. The surface profile of the project area was viewed from the Google Earth Pro application. This enabled the researcher to determine and insert the spot heights of several points along the likely sewer line routes. The spot heights also give a general impression of the terrain of both towns. The information from Google Earth Pro is used to establish the technical feasibility of locating the wastewater treatment plant in the proposed sites.

3.2.4 Direct Observation

Direct observation of the possible sites in Ongata Rongai for the construction of the WSPs was done so as to obtain information on settlement and land use. This was necessary for the purposes of land acquisition and compensation costs estimation and for the determination of the suitability of these areas for the location of a wastewater treatment plant.

3.3 Challenges in collection of data

The challenges faced during data collection include:

i. Resistance from residents to answering interview questions
ii. Inadequate literature for review
iii. Obtaining up-to-date maps of the project area

The steps that were taken to deal with these challenges include:

i. Booking appointments with the relevant persons to be interviewed in advance
ii. Use of guidelines from Ministry of water and Irrigation for estimation purposes so as to come up with figures that are as accurate as possible
iii. Use of direct observation and the Google Earth Pro software to map out sections of the project area and to evaluate land use and topography

CHAPTER FOUR

4.0 DATA ANALYSIS AND RESULTS

4.1 Introduction

In this chapter, analysis of the data collected is presented and the results of the analysis established. The section of the analysis that deals with sizing of the WSPs is presented in the Appendices.

4.2 Development Control

All land parcels in and around the project area is privately owned and there is no development plan for either of the townships. Each landowner develops his/her land without reference to a central planning authority in respect of the type of housing that should be developed. In the feasibility report done by Norken, (2008) an assessment of the land-use was carried out for the sewage generation zones of the project area and projections made based on:

- ✓ How far the zone is from the town centre
- ✓ The accessibility of the area
- ✓ The existing infrastructure, like roads, water, power lines etc
- ✓ Topography
- ✓ Soils' suitability for building foundations
- ✓ Observed subdivisions as shown by the presence of fenced plots
- ✓ The type of housing already constructed, etc

The land-user categories and the respective population densities adopted in the Feasibility Study and Preliminary Design Report; Kiserian Sewerage Project and accordingly in this report are shown in the following table:

Table 3: Land-user Categories and the Respective Population Densities

Land-User Categories	Population Density (Persons per Hectare)
Open farm land	10
High Class Medium Density (HCMD)	125
High Class Low Density (HCLD)	50
Medium Class High Density (MCHD)	250
Medium Class Low Density (MCLD)	175
Low Class High Density (LCHD)	350
Low Class Low Density (LCLD)	275
Commercial Areas	350
Institutions such as schools and churches	250

Source: Norken, 2008

4.3 Design/Sewered Population

The design population consists of the domestic, commercial and institutional population so that the treatment plant is designed to handle sewage from these sources only. Industrial wastewater, such as that from slaughterhouses in Kiserian, will be assumed to be pre-treated to satisfactory standards and discharged to the nearest streams. For Kiserian Township, the final figures found in the Feasibility Study and Preliminary Design Report-Kiserian Sewerage Project-Volume 1 – Report (2008) are adopted. These figures are shown in the table below. The figures given are projected to the years 2010, 2020 and 2030.

Table 4: Kiserian Township Sewered Population and Water Demand to the year 2030

Location Kiserian Town	**Sewered Population**				
	2010	**2020**	**2030**	**Growth rate in %**	
				2010-2020	**2020-2030**
	5,258	30,715	70,107	19.3	8.6
Average per capita Water Consumption	150	165	175		

Source: Norken (2008)

For the design of the combined wastewater treatment plant, the figures adopted are borrowed from the Nairobi Satellite Towns Water and Sanitation Development Programme Report (2014). These figures are considered appropriate as they were computed with a vision of providing a water distribution system for Kiserian and Ongata Rongai Towns. The figures are based on the official census data from the Central Bureau of Statistics for 2009 and recommended future growth rates. The population and water demand projections are summarised in the table below:

Table 5: Ongata Rongai and Kiserian Townships Sewered Population and Water Demand to the year 2035

Design Demands		Water Demand (m^3/day)					
Description	**Factor**	**Consumption**	**2015**	**2017**	**2020**	**2030**	**2035**
Total Population			**150,700**	**168,000**	**194,300**	**282,200**	**336,100**
Domestic							
High cost area	0.0%	150 l/c/d	-	-	-	-	-
25% High/75% medium cost area	25.9%	113 l/c/d	4,389	4,893	5,659	8,219	8,656
Medium cost area	25.8%	100 l/c/d	3,881	4,327	5,004	7,268	8,656
75% medium/25% low cost area	9.7%	88 l/c/d	1,278	1,425	1,648	2,393	2,850
50% Medium/ 50% low cost area	3..8%	75 l/c/d	424	473	547	794	945
Low cost area	25.4%	50 l/c/d	1,911	2,130	2,464	3,278	4,261
Low income area	9.6%	30 l/c/d	432	482	557	809	964
Total Domestic Water Demand (m^3/day)			**12,315**	**13,728**	**15,878**	**23,060**	**27,465**
Education Sector	19,700	50 l/c/d	985	1,098	1,270	1,845	2,197
Health Sector	3,690	100 l/bed/d	369	412	476	692	824
Administrative Sector	9,850	25 l/c/d	246	275	318	461	549
Commercial & Industrial	40	20,000 l/ha/d	739	824	953	1,384	1,648
Overall Water Demand (m^3/day)			**14,654**	**16,337**	**18,894**	**27,442**	**32,683**

Source: Gauff Ingenieure (2014)

The following formula is used to obtain the population growth rate to the years 2016, 2026 and the ultimate design year, 2036.

$$P = P_O(1+r)^n$$

$$r = (P / P_o)^{1/n} - 1$$

Where: *P – Projected population after n years*

P_o – Population during the reference year

r – Population growth factor

n – Projection period in years

The projected figures are presented in the tables below:

Table 6: Kiserian Town Projected Sewered Population and Water Demand for the years 2016, 2026 and 2036

Location	**Growth Rate**		**Growth Rate**		**Growth Rate**	
Kiserian Town	**19.3%**		**8.6%**		**3.6%**	
Year	**2010**	**2016**	**2020**	**2026**	**2030**	**2036**
Sewered Population	5,258	15,161	30,715	50,396	70,107	86,680
Average per capita Water Consumption (l/p/d)	100	100	100	100	100	100
Overall Water Demand (m^3/d)	526	1,516	3,072	5,040	7,011	8,668

Table 7: Population and Water Demand Projections for both Kiserian and Ongata Rongai Towns for the years 2016, 2026 and 2036

Location			**Growth Rate**		**Growth Rate**		**Growth Rate**	
Kiserian and Ongata Rongai Town			**5.6%**		**8.6%**		**3.6%**	
Year	**Factor**	**Consumption**	**2015**	**2016**	**2020**	**2026**	**2030**	**2036**
Sewered Population			**150,700**	**159,140**	**194,300**	**243,028**	**282,200**	**348,912**
Total Domestic Water Demand (m^3/day)			12,315	13,005	15,878	19,860	23,060	28,510
Education Sector	19,700	50 l/c/d	985	1040	1,270	1590	1845	2280
Health Sector	3,690	100 l/bed/d	369	390	476	595	1692	2090
Administrative Sector	9,850	25 l/c/d	246	260	318	400	461	570

Equivalent Population								45,358
Total Equivalent Sewered Population								394,270
Overall Water Demand (m^3/day)			13,915	14,695	17,942	22,445	27,058	**33,450**

4.4 Sewage Treatment Works Design

The waste stabilization ponds have been designed to meet the 20/30 Royal Commission standard for BOD_5, suspended solids and Coliform count, with the permissible limits being:-

- BOD5 at 20^0C not to exceed 20 mg/l
- Suspended solids not to exceed 30 mg/l
- Coliform count not to exceed 1,000 per 100 ml

The waste stabilization ponds sizing and configuration is carried out with guidance from Mara (2003). The influent will be pre-treated at the inlet works which will comprise of screening bars and a rapid settling chamber. The series of ponds will comprise anaerobic, facultative and maturation ponds.

4.5 Population Projections

It is assumed that the land for the construction of the treatment plant will be acquired in 2016 and the design period of the plant is twenty years. The design population is taken as the projected population for the year 2036 and the WSPs are designed to treat the wastewater flow in the same year. The population projection was based on the initial, future and ultimate periods as follows;

- Initial Design Year – 2016
- Future Design Year – 2026
- Ultimate Design Year – 2036

4.6 Wastewater Treatment Works Design Data

4.6.1 Components of the Works

The land requirement is determined from the sewage treatment works which will comprise of the following components:

i. Preliminary treatment comprising of coarse and fine screens
ii. Anaerobic, facultative and maturation ponds whose number and size is determined from the calculations in the Appendices section;
iii. An Administration Building that includes laboratory facilities;
iv. 3 No. Staff houses (Grade 9)

4.6.2 Sewage Characteristics

The 5-day 20^0C biochemical oxygen demand (BOD5) of the sewage from residences is usually taken to be 55g per capita per day with an average of 505mg/l. The per capita contribution of total suspended solids (SS) is assumed to be 80mg/cap/d with an average of 730 mg/l. Sewage from institutions, such a schools and hospitals, and commercial sewage shall be assumed to have similar characteristics to sewage from residences.

4.6.3 Influent Parameters

The design influent sewage parameters adopted are as follows:

- Per Capita BOD5 = 55g/per person per day, taken to be approximately 505mg/l;
- Faecal Coliform – a suitable design value of $4*10^7$ per 100ml is adopted

4.6.4 Temperature

The design temperature adopted is based on the pond temperature during the coldest period of the year. The applicable value is 16^0C based on data from Dagoretti Weather Station (Norken, 2008). The design temperature is taken as 3^0C above the temperature of the coldest month, giving a value of 19^0C.

4.7 Sewage Generation Analysis and Ponds Design

The design flow is estimated by applying a sewage factor of 0.80 to the water consumption rates. Infiltration into the sewer shall be assumed to be 15% of sewage generation. The values are given in the tables below and the influent BOD loading from the projected population calculated.

4.7.1 Design of Kiserian Township Waste Stabilisation Ponds

Table 8: Sewage Generation for Kiserian Township

Location **Kiserian Township**	**Initial Year** **(2016)**	**Future Year** **(2026)**	**Ultimate Year** **(2036)**
Sewered Population	15,161	50,396	86,680
Average per capita Water Consumption (l/p/d)	100	100	100
Overall Water Demand (m^3/d)	1,516	5,040	8,668
Sewage Generation Factor	0.8	0.8	0.8
Sub-total Design Flow	1,213	4,032	6,934
15% for Infiltration	181.932	605	1,040
Total Sewage Flow (Q m^3/d)	**1,395**	**4,636**	**7,975**

BOD Loading

$$L_i = {1000 * B}/{Q}$$

Where: Li = wastewater BOD, mg/l

B = Per capita BOD Load, g/p/d (55g/p/day)

Q = Wastewater flow, (l/p/day)

$$L_i = \frac{(55 * 1000 * 86680)}{(7{,}975 * 1000)} = 600\ mg/l$$

Details of the design of the anaerobic, facultative and maturation waste stabilisation ponds required to treat the wastewater flow from Kiserian Township to the recommended effluent standards are included in Appendix-1. The table below shows the pond sizes required, the retention time in each pond and the effluent parameters from the ponds that will treat wastewater from Kiserian Township.

Table 9: Removal of BOD and Faecal Coliform from Kiserian Township WSPs

Pond	Pond Area (m^2)	Retention Time (days)	Residual BOD (mg/l)	Residual FC/100 ml
Raw Wastewater			600	$4*10^7$
Anaerobic	3,418	2.2	252	
Facultative	85,690	18.8	39.5	
First Maturation	31,038	5.8	25	
Second Maturation	31,038	5.8	15.8	865

4.7.2 Design of Ongata Rongai Township Waste Stabilisation Ponds

Table 10: Sewage Generation for Ongata Rongai Township

Location **Kiserian and Ongata Rongai Townships**	**Initial Year** **(2016)**	**Future Year** **(2026)**	**Ultimate Year** **(2036)**
Sewered Population	143,979	192,632	307,590
Overall Water Demand (m^3/d)	13,179	17,405	24,782
Sewage Generation Factor	0.8	0.8	0.8
Sub-total Design Flow	10,543	13,924	19,826
15% for Infiltration	1,581	2,089	2,974
Total Sewage Flow (Q m^3/d)	**12,125**	**16,013**	**22,800**

BOD Loading

$$L_i = 1000 * B/_{Q}$$

Where: Li = wastewater BOD, mg/l

B = Per capita BOD Load (55g/p/day)

Q = Wastewater flow (l/p/day)

$$L_i = \frac{(55 * 1000 * 307{,}590)}{(22{,}800 * 1000)} = 742\ mg/l$$

Details of the design of the anaerobic, facultative and maturation waste stabilisation ponds required to treat the wastewater flow from Ongata Rongai Township to the recommended effluent standards are shown in Appendix-2. The table below shows the pond sizes required, the retention time in each pond and the effluent parameters from the ponds that will treat wastewater from the township.

Table 11: BOD and Faecal Coliform Removal from Ongata Rongai Township WSPs

Pond	Pond Area (m^2)	Retention Time (days)	Residual BOD (mg/l)	Residual FC/100 ml
Raw Wastewater			742	$4*10^7$
Anaerobic	12,084	2.7	311.6	
Facultative	302,961	23.3	40.8	
First Maturation	73,267	4.8	27.5	
Second Maturation	73,267	4.8	18.5	841

4.7.3 Design of the Combined Waste Stabilisation Ponds

Table 12: Sewage Generation for Combined Kiserian and Ongata Rongai Townships

Location Kiserian and Ongata Rongai Townships	Initial Year (2016)	Future Year (2026)	Ultimate Year (2036)
Sewered Population	**159,140**	**243,028**	**394,270**
Overall Water Demand (m^3/d)	14,695	22,445	33,450
Sewage Generation Factor	0.8	0.8	0.8
Sub-total Design Flow	11,756	17,956	26,760
15% for Infiltration	1,763	2,693	4,014
Total Sewage Flow (Q m^3/d)	**13,519**	**20,649**	**30,774**

BOD Loading

$$L_i = {}^{1000 * B}/_{Q}$$

Where: Li = wastewater BOD, mg/l

B = Per capita BOD Load (55g/p/day)

Q = Wastewater flow (l/p/day)

$$L_i = \frac{(55 * 1000 * 394{,}270)}{(30{,}774 * 1000)} = 705\ mg/l$$

The design of the waste stabilisation ponds required to treat the wastewater flow from the combined sewerage system to the recommended effluent standards is done in Appendix-3. The table below shows the pond sizes required, the retention time in each pond and the effluent parameters from the ponds that will treat wastewater from both townships.

Table 13: BOD and Faecal Coliform Removal from Combined WSPs

Pond	Pond Area (m^2)	Retention Time (days)	Residual BOD (mg/l)	Residual FC/100 ml
Raw Wastewater			705	$4*10^7$
Anaerobic	15,497	2.5	296.1	
Facultative	388,148	22.1	40.5	
First Maturation	104,724	5.1	26.8	
Second Maturation	104,724	5.1	17.7	849

4.8 Land Requirement Analysis

According to Mywage.org/Kenya Website (2012), a grade nine officer of the Local Authorities and the Councils' Water Companies who are appointed to pensionable posts within a permanent establishment is entitled to a house allowance that includes the cost of land not exceeding 0.4 hectares (one acre). This figure is adopted in the estimation of land required for the three staff houses. The laboratory and administration block will occupy an extra acre. This gives a total land area requirement of 4 acres. If an additional 6 acres is acquired to cater for inlet works, access routes and areas between the ponds and the fence, a total of 10 extra acres will be acquired in addition to the area to be occupied by the WSPs.

4.8.1 Pond Geometry

The following pond geometry shall be adopted for all the ponds.

Where:

- L = pond length at top water level, TWL, m
- B = pond breadth at top water level, TWL, m
- n = horizontal slope factor (i.e. a slope of 1 in s) taken as 2 for all ponds
- D = pond liquid length, m
- F = freeboard

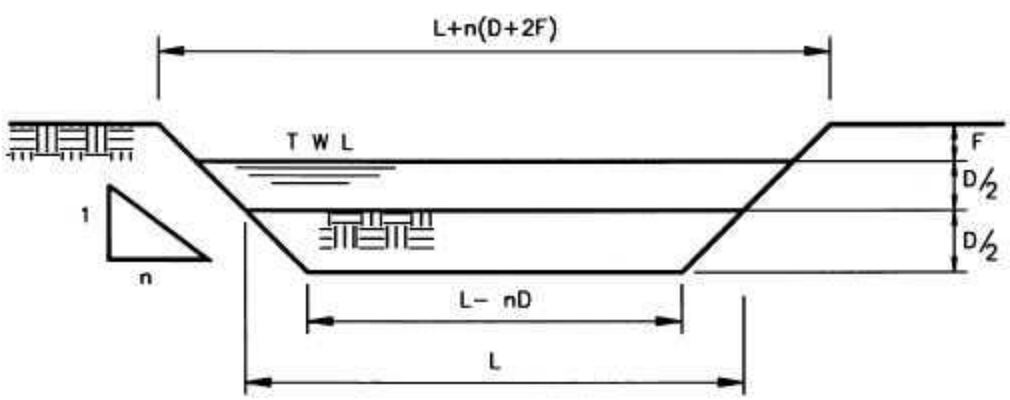

Figure 5: Geometry of Pond *(Mara and Pearson, 1998)*

Table 14: Effective Pond Areas for Kiserian Township WWTP

	Anaerobic	**Facultative**	**Maturation**
Total number	1	2	2
n	2	2	2
Freeboard, F, m	1	1	1
Depth, m	5	1.75	1.5
n(D+2F)	14	7.5	7
L_{TWL}, m	82	294	250
W_{TWL}, m	42	146	125
L_{TOP}, m	96	301.5	257
W_{TOP}, m	56	153.5	132
Area Required	1*96*56 **= 5,376 m^2**	1*301.5*153.5 **= 92,560.5 m^2**	2*257*132 **= 67,848 m^2**
Total Area Required	**165,784.5 m^2**		

4046.86 m^2 = 1 acre

165,784.5 m^2 = 41 acres

Total acreage required = 41 + 10 = 51 acres (20.6 hectares)

Table 15: Effective Pond Areas for Ongata Rongai Township WWTP

	Anaerobic	**Facultative**	**Maturation**
Total number	1	2	2
n	2	2	2
Freeboard, F, m	1	1	1
Depth, m	5	1.75	1.5
n(D+2F)	14	7.5	7
L_{TWL}, m	155	550	382
W_{TWL}, m	78	276	192
L_{TOP}, m	169	557.5	389
W_{TOP}, m	92	283.5	199
Area Required	1*169*92 **= 15,548 m^2**	2*557.5*283.5 **= 316,102.5 m^2**	2*389*199 **= 154,822 m^2**
Total Area Required	**486,472.5 m^2**		

4046.86 m^2 = 1 acre

486,472.5 m^2 = 120 acres

Total acreage required = 120 + 10 = 130 acres (52.6 hectares)

Table 16: Effective Pond Areas for Combined Kiserian and Ongata Rongai Townships WWTP

	Anaerobic	**Facultative**	**Maturation**
Total number	1	2	2
n	2	2	2
Freeboard, F, m	1	1	1
Depth, m	5	1.75	1.5
n(D+2F)	14	7.5	7
L_{TWL}, m	176	623	458
W_{TWL}, m	89	312	229
L_{TOP}, m	190	630.5	465
W_{TOP}, m	103	319.5	236
Area Required	1*190*103 **= 19,570 m^2**	2*630.5*319.5 **= 402,889.5 m^2**	2*465*236 **= 219,480 m^2**
Total Area Required	**641,939.5 m^2**		

4046.86 m2 = 1 acre

641,939.5 m^2 = 159 acres

Total acreage required = 159 + 10 = 169 acres (68.4 hectares)

4.9 Site for Location of Kiserian Township Wastewater Treatment Plant

Two technically feasible sites have been identified in the Feasibility Study and Preliminary Design Report for Kiserian Sewerage Project. One site is about 1 km downstream of the Kiserian Dam Site while the other is approximately 2 km downstream of the dam. The Rimpa Estate site is the one that is located approximately 2 km downstream of Kiserian Dam. Because of the terrain of the project area, only wastewater from Kiserian Township can flow to this site by gravity. The map below shows possible streets that the main sewers can follow to the Rimpa Estate site. The surface profile of the routes was viewed on Google Earth Pro Software and the spot heights above sea level of points along the routes shown on the map to give a general impression of how the ground slopes.

Figure 6: Map showing possible flow routes to proposed site at Rimpa Estate *(Source: Google Earth)*

From the calculations done in the preceding section, it was established that the required acreage for the construction of the Kiserian Township wastewater treatment plant is approximately 51 acres. If a 455*455 m area is purchased, it will be sufficient for the project. A closer look at Rimpa Estate shows that the land is mostly bare with a few scattered settlements in the proximity of the proposed site. The land is however privately owned and the client will need to purchase it at the current land rates. According to a report by Marindany, (2015), the average price of land in Rimpa Estate is KShs. 10 million per acre. This figure shall be used for the estimation of cost incurred in land acquisition.

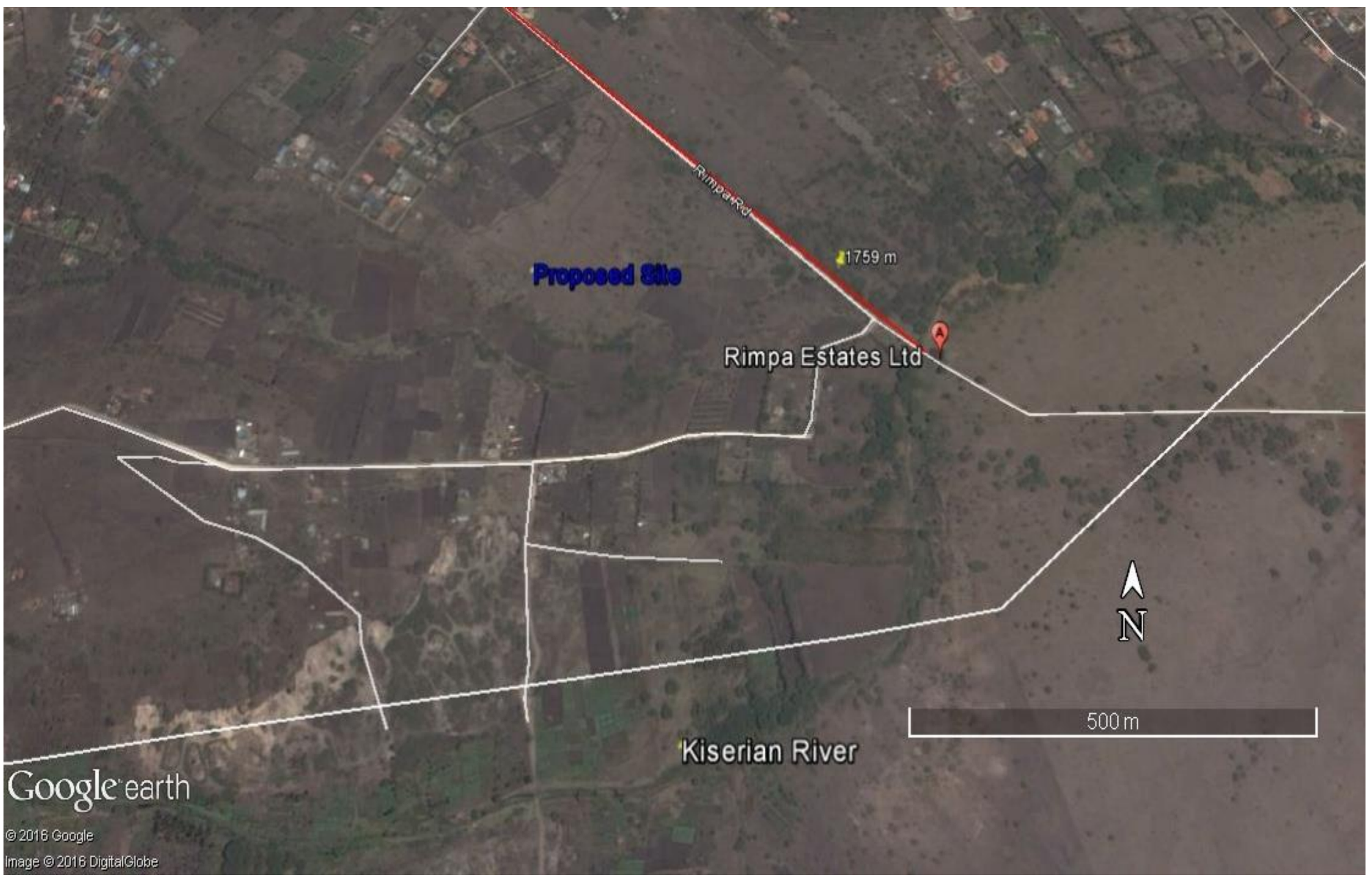

Figure 7: Map showing the proposed site at Rimpa area *(Source: Google Earth)*

4.10 Site for Location of Ongata Rongai Wastewater Treatment Plant

The interview with the OWSC led to the identification of a suitable site for the location of either the Ongata Rongai Township WWTP or the combined treatment plant. The proposed site is to the south of the Nairobi National Park, just past Leleshwa Getaway where the Kiserian River and Kandisi River meet and flow into the Mbagathi River. The site is technically feasible as the wastewater from both Kiserian and Ongata Rongai townships can flow by gravity to the identified site. The price of land within the identified site has an average asking price of KShs. 20 million per acre. There are a few scattered settlements within the identified site and the client will need to compensate the owners of the establishments before construction begins.

From the calculations done in the preceding section, it was established that the required acreage for the construction of the treatment plant to treat wastewater from Ongata Rongai Township is 130 acres. If a 725*725 m area is purchased, it will be sufficient for the project. For the treatment of wastewater from a combined sewerage system serving both Kiserian and Ongata Rongai Townships, 169 acres of land are required. Land measuring 827*827 will be sufficient for the project.

The map below shows possible channels that the main sewer lines can follow to the proposed site. The surface profile of the routes was viewed on Google Earth Pro and the spot heights above sea level of points along the routes shown on the map to give a general impression of how the ground slopes. The trunk sewer from Ongata Rongai Township will flow along Gataka Road and later adjacent to Kandisi River while the trunk sewer from Kiserian Township will flow adjacent to Kiserian River.

According to the guideline developed by the Institute of Quantity surveyors of Kenya as presented in the Architecture Kenya Media Limited Website (2015), the cost of constructing a square metre of a single unit residential building is approximately Kshs. 32,000 in Nairobi and its environs. The site identified for the combined treatment plant has ten houses with a combined area of 1,021 m^2. A fairly accurate estimate of the compensation costs is determined in the following section by multiplying the rate of construction per square metre by the total area of buildings in the site.

Figure 8: Map showing possible flow routes to proposed site south of the Nairobi National Park in Ongata Rongai Township *(Source: Google Earth)*

Figure 9: Map showing proposed site in Ongata Rongai Township *(Source: Google Earth)*

4.11 Estimation of per Capita Cost Incurred in Land Acquisition

For the Plant to Serve Kiserian Township: The site identified is bare land and the cost incurred in acquisition does not include any compensation costs for buildings to be demolished. The total cost is given by:

Cost incurred in acquiring land = 51 acres * 10,000,000 = KShs. 510,000,000

For the Plant to Serve Ongata Rongai Township: The total compensation cost for the proposed site to be occupied by the treatment plant is given by the cost incurred in acquisition of land plus the cost incurred in compensation for the houses that are to be demolished. This is given by:

Cost incurred in acquiring bare land = 130 acres * 20,000,000 = KShs. 2,600,000,000

Plus cost incurred in compensation of demolished houses = 1,021 m^2 * 32,000 = 32,672,000

Giving a total of KShs. 2,632,672,000

The estimated number of people served by the project in the ultimate year of 2036 = 394,270 people; therefore the per capita investment in acquisition of land is given by:

$$KShs.\,(2{,}632{,}672{,}000 + 510{,}000{,}000)/_{394{,}270} = KShs.\,7{,}971\ per\ person$$

For the Combined Plant to Serve Both Townships: The total compensation cost for the proposed site to be occupied by the treatment plant is given by the cost incurred in acquisition of land plus the cost incurred in compensation for the buildings that are to be demolished. This is given by:

Cost incurred in acquiring bare land = 169 acres * 20,000,000 = KShs. 3,380,000,000

Plus cost incurred in compensation of demolished buildings = 1,021 m^2 * 32,000 = 32,672,000

Giving a total of KShs. 3,412,672,000

The estimated number of people served by the project in the ultimate year of 2036 = 394,270 people; therefore the per capita investment in acquisition of land is given by:

$$KShs.\,3{,}412{,}672{,}000/_{394{,}270} = KShs.\,8{,}656\ per\ person$$

CHAPTER FIVE

5.0 DISCUSSION OF RESULTS, RECOMMENDATIONS AND CONCLUSIONS

5.1 Discussion of Results

The first objective of this study was to find an estimate of the projected population that will be served by the proposed sewerage system in Kiserian and Ongata Rongai townships in the ultimate year of 2036. This was achieved by adopting the population growth rates presented in various reports with either the intention of provision of sewerage systems or water distribution in the two townships. The projected population in the ultimate year is 86,680 people and 307,590 people for Kiserian and Ongata Rongai townships respectively, giving a total population of 394,270 people for both townships. The water demand in these areas was obtained using guidance from the MoWI Practice Manual for Water Supply (2005) depending on population densities and expected water consumption rates. A factor of 0.8 was applied to the water demand to obtain the sewage generation from both townships. A factor of 0.15 was then applied to the obtained values to cater for infiltration into the sewerage system. This gave wastewater generation values of 7,975m^3/day and 22,800m^3/day from Kiserian and Ongata Rongai townships with a combined wastewater flow of 30,774m^3/day in the ultimate year.

The sewage generation was used to size the anaerobic, facultative and maturation waste stabilization ponds. The pond areas combined with the areas needed for inlet works, administration blocks, the laboratories, staff housing, access routes and bare land between the ponds and the fences gave the total required acreage for the construction of the wastewater treatment plants. For the Kiserian Township treatment plant, a total of 51 acres (20.6 ha) was deemed sufficient while the treatment plant to serve Ongata Rongai Township requires a total acreage of 120 acres (52.6 ha). The combined wastewater treatment plant serving both townships requires a total area of 169 acres (68.4 ha).

Two technically feasible sites that would make it possible to adopt gravity based sewer systems were identified. The site for the location of the Kiserian Township wastewater treatment plant is situated in Rimpa Estate, 2 km downstream of the Kiserian Dam. The site for the location of the treatment plant to serve either Ongata Rongai Township or the combined wastewater treatment plant is situated to the south of the Nairobi National Park immediately after Kandisi and Kiserian Rivers meet and flow into Mbagathi River. The technical feasibility of both sites was determined by drawing lines on Google Earth Pro software along the streets where the main sewer lines would possibly follow and viewing the surface profiles of these lines to confirm that the paths slope in such a manner as to ensure gravity flow. Point elevations showing the heights above sea level are marked on the proposed streets to give a general impression of how the trunk sewers would flow to the proposed sites.

After the sites were identified, the suitable cost of land acquisition was estimated. A cost of KShs.10 million per acre was used for the Rimpa Estate site while a value of KShs. 20 million per acre was adopted for the Ongata Rongai site. Compensation costs for the buildings that occupy the areas in the proposed site were then factored in giving a total value of KShs. 510 million and 2.63 billion for the Rimpa Estate and Ongata Rongai sites respectively for the separate wastewater treatment plants. For the combined treatment plant, the total cost of land acquisition was found to be 3.41 billion. These values were then divided by the population to be served by the treatment plants in the ultimate year to give an estimation of the per capita investment and to allow for a comparison of the cost incurred in land acquisition. The per capita cost for constructing separate plants in Rimpa Estate and Mbagathi River site was found to be KShs. 7,971 per person while that for constructing a combined treatment plant was determined as KShs. 8,656 per person. This makes constructing separate treatment plants to serve either township considerably cheaper than constructing one treatment plant for both townships.

Construction and maintenance costs are not factored in this assessment as these costs are not expected to vary widely with different site locations. Other factors that may lead to varying cost but are not featured in the cost comparison include costs caused by varying geotechnical conditions that determine the workability of the sites, environmental costs and professional consultation fees. The cost of conveying the wastewater from the source to the location of the treatment plant should also be factored in if more precise values are desired.

5.2 Conclusion

This study was conducted to establish the economic and technical feasibility of combining the Kiserian and Ongata Rongai townships wastewater treatment plants. The analysis has led to the conclusion that it would be technically feasible to combine the wastewater treatment systems for the two townships. This is evidenced by the sloping terrain of the project area to the site identified to the south of the Nairobi National Park in Ongata Rongai.

The per capita cost of constructing separate wastewater treatment plants for Kiserian and Ongata Rongai townships is 8% cheaper than that of constructing a combined wastewater treatment plant in Ongata Rongai to serve both townships. Further economic assessment can be done to establish the suitability of having a combined wastewater treatment plant as the economy of scale may not only be achieved in land acquisition but also in other project costs that may be considered.

5.3 Recommendations

From the calculations, it is evident that within the limits of estimation errors, it is much cheaper to have separate treatment plants serving the two towns than to have one combined treatment plant serving both townships. It would therefore be more prudent to construct separate wastewater treatment plants in the respective townships in the recommended sites.

The trunk sewers from Ongata Rongai Township will flow close to Gataka Road and later adjacent to Kandisi River while the trunk sewer from Kiserian Township will flow adjacent to Kiserian River to the proposed sites. This will allow gravity flow eliminating the need for pumps along the sewer lines.

5.3.1 Areas for Further Study

The study has concentrated only on the technical and economic feasibility aspects of locating the treatment plants in either of the locations identified. Further research should be done on the social and environmental implications of the same. The government would also find it easier to handle one group of opposing residents than handling two groups. This may lead to very long delays in the commencement of the project and result in wastage of resources and escalated costs. A comparison of the social and environmental costs the society may have to pay will lead to a better judgement of the more feasible option between the two alternatives presented from a more specific viewpoint.

There are two sites recommended in Feasibility Study and Preliminary Design Report; Kiserian Sewerage Project (2008) as suitable sites for the location of the Kiserian wastewater treatment plant. This study concentrates on the Rimpa Estate site which is located 2 km downstream of the Kiserian Dam. Further studies can be carried out on the economic suitability of the site 1 km downstream of the dam.

Further studies can also be carried out to determine the technical and economic feasibility of using a trunk sewer to carry the wastewater from Kiserian and Ongata Rongai townships to the Ruai Treatment Plant to take advantage of the unused capacity of the plant. It is also likely that the proposal to construct a WWTP that close to the Nairobi National Park will be faced with a lot of resistance from the Kenya Wildlife Service. As it has been established that it would be cheaper to have separate WWTPs, further feasibility studies can be carried out to determine the suitability of constructing the WWTP for Kiserian Township alone and using a trunk sewer to carry the wastewater from Ongata Rongai Township to the Ruai Treatment Plant.

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APPENDICES

6.0 Appendix 1: Design of Kiserian Township Waste Stabilisation Ponds

Table 17: Kiserian Town Anaerobic Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Average Flow (Q) m^3/day	7,975 m^3/day
Depth (D_a) m	5 m
Temperature ^{0}C	19^{0}C
Wastewater BOD (mg/l)	600 mg/l
Organic Breakdown Rate, Kt	0. 3*1.05^(19-20) = 0.29
Permissible volumetric BOD loading, λ_v	20T-100 = ((20*19)-100)) = 280 g/m^3/day

Anaerobic ponds are designed on the basis of volumetric BOD loading (λ_v, g/m^3d) which is given by:

$$\lambda_v = {}^{L_i Q}/_{V_a}$$

Where L_i is the influent BOD in g/m^3; Q is the flow in m^3/d; and V_a is the anaerobic pond volume in m^3

Therefore

$$V_a = {}^{L_i Q}/_{\lambda_v} = \frac{600 * 7,975}{280} = 17,089\, m^3$$

and

$$t_a = {}^{\mathrm{Va}}/_{Q} = {}^{17,089}/_{7,975} = 2.2 > 1 day$$

Assuming a pond depth of 5m the plan area of pond = $^{17,089}/_{5} = 3{,}418\ m^2$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 82 m

Pond Width = 42 m

Allow a freeboard of 1m

Anaerobic Pond Area = 82*42 = 3,444 m^2

BOD Removal: For a temperature of 19^0C, %BOD removal = 2T+20 = 2*19+20=58%

Effluent BOD from the anaerobic pond = 42%*600=252 mg/l

6.1 Facultative Ponds Design

Table 18: Kiserian Town Facultative Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Depth (D_f)	1.75m
Organic breakdown rate, Kt = $k_{1(20)}(1.05)^{T-20}$	0. 3*1.05^(19-20) = 0.29
Li (mg/l)	252 mg/l

The permissible design value of the surface loading increases with temperature and the following global design equation was developed by Mara (2003) as a guide for loading facultative ponds:

$$\lambda s = 350(1.107 - 0.002\mathrm{T})^{\mathrm{T}-25}$$

$\lambda s = 350(1.107 - 0.002 * 19)^{19\text{-}25}$= 234.5 Kg/ha/day

The surface loading (λ_s kg/ha/day) is given by:

$$\lambda s = \frac{10 * L_i Q}{A_f}$$

$$234.5 = \frac{10 * 252 \ * 7{,}975}{A_f}$$

$$A_f = \frac{10 * 252 * 7{,}975}{234.5} = 85{,}690 m^2$$

$$t_f = {A_f D_f}/{Q} = {85{,}690 * 1.75}/{7{,}975} = 18.8$$

$$L_e = \frac{L_i}{1 + k_1 t_f}$$

$$L_e = \frac{252}{1 + (0.29 * 18.8)} = 39.5\ mg/l$$

Adopt two facultative ponds in parallel and use a length: breadth ratio of 2:1. The two facultative ponds will have a retention time of 18.8 days each.

Area of each pond (required) $= {85{,}690}/{2} = 42{,}845\ m^2$

Pond Length = 294 m;

Pond Width = 146 m

Allow a freeboard of 1m

Facultative Pond Area (provided) = 294*146 = 42,924 m^2

6.2 Maturation Ponds Design

Table 19: Kiserian Township Maturation Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Influent Faecal Coliform	$4*10^{7}$ mg/100ml
Effluent Faecal Coliform	1,000 mg/100ml
Depth (D_m) m	1.5 m
Li = Le($_{fac}$) (mg/l)	39.5 mg/l
FC removal rate $K_B(_T)$ = 2.6(1.19)^(T-20)	=2.6(1.19)^(19-20) = 2.2
Retention time in anaerobic pond (t_a)	2.2 days
Retention time in facultative ponds (t_f)	18.8 days

For a series of WSPs comprising an anaerobic pond, a secondary facultative pond and n equally sized maturation ponds, the equation for the design of maturation ponds can be written as:

$$N_e = \frac{N_i}{(1 + k_{B(T)}t_a)(1 + k_{B(T)}t_f)(1 + k_{B(T)}t_m)^n}$$

Try two maturation ponds;

$$1{,}000 = \frac{4 * 10^7}{(1 + 2.2 * 2.2)(1 + 2.2 * 11.2)(1 + 2.2 * t_m)^2}$$

Gives t_m = 5.8 days < t_f (18.8 days)

The surface loading (λ_s kg/ha/day) for maturation pond is given by;

$$\lambda s = \frac{10 * \mathrm{L_i D_m}}{\mathrm{t_m}} = \frac{10 * \mathrm{L_i Q}}{\mathrm{A_m}}$$

$$\lambda s = \frac{10 * 39.5 * 1.5}{5.8} = 101.6$$

$$101.6 = \frac{10 * 60 * 7{,}975}{\mathrm{A_m}}$$

$$\mathrm{A_m} = 31{,}038\ \mathrm{m}^2$$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 250 m

Pond Width = 125 m

Allow a freeboard of 1m

Maturation Pond Area (provided) = 250*125 = 31,250 m^2 each

7.0 Appendix 2: Design of Ongata Rongai Township Waste Stabilisation Ponds

Table 20: Ongata Rongai Town Anaerobic Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Average Flow (Q) m^3/day	22,800 m^3/day
Depth (D_a) m	5 m
Temperature ^{0}C	19^{0}C
Wastewater BOD (mg/l)	742 mg/l
Organic Breakdown Rate, Kt	0. 3*1.05^(19-20) = 0.29
Permissible volumetric BOD loading, λ_v	20T-100 = ((20*19)-100)) = 280 g/m^3/day

Anaerobic ponds are designed on the basis of volumetric BOD loading (λ_v, g/m^3d) which is given by:

$$\lambda_v = {}^{L_i Q}/_{V_a}$$

Where L_i is the influent BOD in g/m^3; Q is the flow in m^3/d; and V_a is the anaerobic pond volume in m^3

Therefore

$$V_a = {}^{L_i Q}/_{\lambda_v} = \frac{742 * 22{,}800}{280} = 60{,}420\ m^3$$

and

$$t_a = {}^{\mathrm{Va}}/_{Q} = {}^{60{,}420}/_{22{,}800} = 2.7 > 1 day$$

Assuming a pond depth of 5m the plan area of pond = ${}^{60{,}420}/_{5} = 12{,}084\ m^2$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 155 m

Pond Width = 78 m

Allow a freeboard of 1m

Anaerobic Pond Area = 155*78 = 12,090 m^2

BOD Removal: For a temperature of 19^0C, %BOD removal = 2T+20 = 2*19+20=58%

Effluent BOD from the anaerobic pond = 42%*742 = 311.64 mg/l

7.1 Facultative Ponds Design

Table 21: Ongata Rongai Town Facultative Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Depth (D_f)	1.75m
Organic breakdown rate, Kt = $k_{1(20)}(1.05)^{T-20}$	0. 3*1.05^(19-20) = 0.29
Li (mg/l)	311.64 mg/l

The permissible design value of the surface loading increases with temperature and the following global design equation was developed by Mara (2003) as a guide for loading facultative ponds:

$$\lambda s = 350(1.107 - 0.002\mathrm{T})^{\mathrm{T}-25}$$

$\lambda s = 350(1.107 - 0.002 * 19)^{19-25} = 234.5$ Kg/ha/day

The surface loading (λ_s kg/ha/day) is given by:

$$\lambda s = \frac{10 * \mathrm{L_i Q}}{\mathrm{A_f}}$$

$$234.5 = \frac{10 * 311.64 * 22{,}800}{\mathrm{A_f}}$$

$$\mathrm{A_f} = \frac{10 * 311.64 * 22{,}800}{234.5} = 302{,}961\ \mathrm{m}^2$$

$$t_f = {}^{\mathrm{A_f} D_f}\!/_{Q} = {}^{302{,}961 * 1.75}\!/_{22{,}800} = 23.3$$

$$L_e = \frac{L_i}{1 + k_1 t_f}$$

$$L_e = \frac{311.64}{1 + (0.29 * 23.3)} = 40.8\ mg/l$$

Adopt two facultative ponds in parallel and use a length: breadth ratio of 2:1. The two facultative ponds will have a retention time of 23.3 days each.

Area of each pond (required) = $^{302,961}/_{2} = 151{,}480.5\ m^2$

Pond Length = 550 m;

Pond Width = 276 m

Allow a freeboard of 1m

Facultative Pond Area (provided) = 550*276 = 151,800 m^2

7.2 Maturation Ponds Design

Table 22: Ongata Rongai Town Maturation Pond Design Guideline

Parameters	**Ultimate Year (2036)**
Influent Faecal Coliform	$4*10^7$ mg/100ml
Effluent Faecal Coliform	1,000 mg/100ml
Depth (D_m) m	1.5 m
Li = Le($_{fac}$) (mg/l)	40.8 mg/l
FC removal rate $K_B(_T)$ = 2.6(1.19)^(T-20)	=2.6(1.19)^(19-20) = 2.2
Retention time in anaerobic pond (t_a)	2.7 days
Retention time in facultative ponds (t_f)	23.3 days

For a series of WSPs comprising an anaerobic pond, a secondary facultative pond and n equally sized maturation ponds, the equation for the design of maturation ponds can be written as:

$$N_e = \frac{N_i}{\left(1 + k_{B(T)}t_a\right)\left(1 + k_{B(T)}t_f\right)(1 + k_{B(T)}t_m)^n}$$

Try two maturation ponds;

$$1{,}000 = \frac{4 * 10^7}{(1 + 2.2 * 2.7)(1 + 2.2 * 23.3)(1 + 2.2 * t_m)^2}$$

Gives $t_m = 4.8$ days < t_f (23.3 days)

The surface loading (λ_s kg/ha/day) for maturation pond is given by;

$$\lambda s = \frac{10 * \mathrm{L_i D_m}}{\mathrm{t_m}} = \frac{10 * \mathrm{L_i Q}}{\mathrm{A_m}}$$

$$127 = \frac{10 * 40.8 * 1.5}{4.8}$$

$$127 = \frac{10 * 40.8 * 22{,}800}{\mathrm{A_m}}$$

$$\mathrm{A_m} = 73{,}267\ \mathrm{m}^2$$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 382 m

Pond Width = 192 m

Allow a freeboard of 1m

Maturation Pond Area (provided) = 382*192 = 73,344 m^2 each

8.0 Appendix-3: Design of the Combined Waste Stabilisation Ponds

8.1 Anaerobic Pond Design

Table 23: Guideline for Combined Anaerobic Pond Design

Parameters	**Ultimate Year (2036)**
Average Flow (Q) m^3/day	30,774
Depth (D_a) m	5 m
Temperature ^{0}C	19^{0}C
Wastewater BOD (mg/l)	705 mg/l
Organic Breakdown Rate, Kt	0. 3*1.05^(19-20) = 0.29
Permissible volumetric BOD loading	20T-100 = ((20*19)-100)) = 280 g/m^3/day

Anaerobic ponds are designed on the basis of volumetric BOD loading (λ_v g/m^3d) which is given by:

$$\lambda_v = {}^{L_i Q}/_{V_a}$$

Where L_i is the influent BOD in g/m^3; Q is the flow in m^3/d; and V_a is the anaerobic pond volume in m^3

Therefore

$$V_a = {}^{L_i Q}/_{\lambda_v} = \frac{705 * 30{,}774}{280} = 55{,}503.11\ m^3$$

and

$$t_a = {}^{\mathrm{Va}}/_{Q} = {}^{55{,}503.11}/_{30{,}774} = 2.5\ days > 1 day$$

Assuming a pond depth of 5m the plan area of pond = $^{77,485}/_{5} = 15,497\ m^2$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 176 m

Pond Width = 89 m

Anaerobic Pond Area (provided) = 176*89 = 15,664 m^2

Allow a freeboard of 1m

BOD Removal; For a temperature of 19^0C %BOD removal = 2T+20 = 2*19+20 = 58%

Effluent BOD from the anaerobic pond = 42%*705=296.1 mg/l

8.2 Facultative Ponds Design

Table 24: Guideline for Combined Facultative Pond Design

Parameters	**Ultimate Year (2036)**
Depth (D_f)	1.75m
Organic breakdown rate, Kt = $k_{1(20)}(1.05)^{T-20}$	0. 3*1.05^(19-20) = 0.29
Li (mg/l)	296.1 mg/l

The permissible design value of the surface loading increases with temperature and the following global design equation was developed by Mara (2003) as a guide for loading facultative ponds:

$$\lambda s = 350(1.107 - 0.002\mathrm{T})^{\mathrm{T}-25}$$

$\lambda s = 350(1.107 - 0.002 * 19)^{19-25} = 234.5$ Kg/ha/day

The surface loading (λ_s kg/ha/day) is given by:

$$\lambda s = \frac{10 * L_i Q}{A_f}$$

$$234.5 = \frac{10 * 296.1 * 30{,}744}{A_f}$$

$$A_f = \frac{10 * 296.1 * 30{,}744}{234.5} = 388{,}148\ m^2$$

$$t_f = {}^{A_f D_f}\!/_{Q} = {}^{388{,}148 * 1.75}\!/_{30{,}744} = 22.1$$

$$L_e = \frac{L_i}{1 + k_1 t_f}$$

$$L_e = \frac{296.1}{1 + (0.29 * 22.1)} = 40.5\ mg/l$$

Adopt two facultative ponds in parallel and use a length: breadth ratio of 2:1. The two facultative ponds will have a retention time of 22.1 days each.

Area of each pond (required) = ${}^{388{,}148}\!/_{2} = 194{,}074\ m^2$

Pond Length = 623 m;

Pond Width = 312 m

Allow a freeboard of 1m

Facultative Pond Area (provided) = 623*312 = 194,376 m^2

8.3 Maturation Ponds Design

Table 25: Guideline for Combined Maturation Pond Design

Parameters	**Ultimate Year (2036)**
Influent Faecal Coliform	$4*10^7$ mg/100ml
Effluent Faecal Coliform	1,000 mg/100ml
Depth (D_m) m	1.5 m
Li = Le($_{fac}$) (mg/l)	40.5 mg/l
FC removal rate $K_B(_T)$ = 2.6(1.19)^(T-20)	=2.6(1.19)^(19-20) = 2.2
Retention time in anaerobic pond (t_a)	2.5 days
Retention time in facultative ponds (t_f)	22.1 days

For a series of WSP comprising an anaerobic pon, a secondary facultative pond and n equally sized maturation ponds, the equation for the design of maturation ponds can be written as:

$$N_e = \frac{N_i}{\left(1 + k_{B(T)}t_a\right)\left(1 + k_{B(T)}t_f\right)\left(1 + k_{B(T)}t_m\right)^n}$$

Try 2 maturation ponds;

$$1{,}000 = \frac{4 * 10^7}{(1 + 2.2 * 2.5)(1 + 2.2 * 22.1)(1 + 2.2 * t_m)^2}$$

Gives t_m= 5.1 days < t_f (22.1 days)

The surface loading (λ_s kg/ha/day) for maturation pond is given by;

$$\lambda s = \frac{10 * \mathrm{L_i D_m}}{\mathrm{t_m}} = \frac{10 * \mathrm{L_i Q}}{\mathrm{A_m}}$$

$$\lambda s = \frac{10 * 40.5 * 1.5}{5.1} = \text{ 118.9 Kg/ha/day}$$

$$118.9 = \frac{10 * 40.5 * 30{,}774}{A_m}$$

$$A_m = 104{,}724\ m^2$$

Adopting a length: breadth ratio of 2:1 then:

Pond Length = 458 m

Pond Width = 229 m

Allow a freeboard of 1m

Maturation Pond Area (provided) = 458*229 = 104,882 m^2 each