Improvement of Decentralised Wastewater Treatment in Asaba, Nigeria

Master’s Thesis by

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July 2010

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Picture on front page:

River Niger - by the Niger bridge connecting Asaba and Onitsha (photo from Google map)
SUMMARY

Decentralised approach to wastewater treatment is the handling of wastewater close to the point of generation. This concept which employs onsite or clusters system is gaining more relevance in the developing world as opposed to the centralised wastewater treatment regime applied in industrialised nations. However, decentralised wastewater treatment is not without its share of challenges resulting from choice of inappropriate technology, improper siting of infrastructure, inadherence to correct design concepts and lack of proper maintenance. These bring about negative public health and environmental impacts including groundwater nitrate contamination, eutrophication of surface water bodies and contribution to global warming through the emission of greenhouse gases.

This work reviews some existing technologies applied in decentralised wastewater treatment and disposal in developing countries such as the septic tank, imhoff tank, soil infiltration device etc. Technologies that are potentially sound and beneficial for wastewater treatment are also explored. Emphasis is placed on fecal sludge management which is grossly inadequate and in some cases totally lacking in rural, peri-urban and some urban areas of developing countries.

A case study is undertaken for a portion of Ezenei Quarters, Asaba, an urban settlement in Nigeria, West Africa with a population of about 1656 persons. This study area is representative of many suburban sprawls in developing countries in terms of wastewater treatment, population size and economics. An attempt is made to select the most appropriate combination of decentralised technologies to improve on the present wastewater handling. These technologies will demonstrate the importance and feasibility of recovery and reuse of organic waste in an area with no wide spread application. The system proposed uses the combined advantage of both black and greywater for effective treatment. The combined wastewater is however treated in sedimentation tanks in series (70 m$^3$ and 60 m$^3$ respectively) for the settling and eventual discharge of wastewater sludge. The effluent from the sedimentation tanks is directed to a combined intermittent sand filter and vegetated subsurface wetland system which are both 500 m$^2$ in area. The intermittent sand filter is aerobic in function aiding nitrification while the subsurface wetland in anaerobic with enhanced denitrification. Theoretical mass balance shows discharge quality to groundwater meeting the design limits of less than 5 mg/l BOD, TSS, and N, and 1 mg/l P.

The sludge withdrawn from the sedimentation tanks is digested in a 50 m$^3$ anaerobic digester. The anaerobic digester is also fed with the organic fraction of municipal solid waste including vegetable waste from the nearby Ogbogonogo Market. There is an expected yearly methane production of about 30,000 m$^3$, 37% of which can be utilised in a generator in the production of electric power that can meet the yearly electricity consumption of about 112,000 KWh. There will be excess biogas meeting about 20% of the cooking energy requirement of the study area or more if electricity generation is cut down to provide for the area only when there is power outage from the national grid. The biosolids from the anaerobic digester is further composted in a 30 m$^3$ invessel composting unit with sawdust as bulking agent. The resulting compost is sold to farmers in nearby villages. The total treatment of wastewater incorporates also rainwater harvesting and treatment in the study area due to the inadequacy of fresh water supply from the public agency. The harvested rainwater can effectively meet 45% non potable water use such as toilet flushing, car washing, gardening in the study area.
The benefits of the proposed system include effective and improved sanitation, harnessing of energy from different biowaste to meet communal electricity and cooking needs, protecting public health, also groundwater sources and surface water bodies from contamination, the reduction of green house gases and over dependence on kerosene from fossil origin. Others include reducing the amount of solid for landfilling and providing freshwater savings.

However, for sustainability of the system, there is the necessity to promote communal involvement. It is also important to educate the community about the system, its operation and advantages, informing them in addition on the need for constant maintenance. The responsible management entity for such decentralised facilities should be the centralised government in this case; the state government which should conduct regular inspection ensuring that best management and operational practices are kept. The government can also provide subsidies for capital cost through a relevant ministerial agency. The major effect to the community in the implementation of the decentralised wastewater treatment improvement is the need to construct new pipelines for wastewater collection and the cost involvement which hopefully can be offset by the energy production and sale of compost. Further studies can however be carried out on cost implication of the proposed system and the contribution of the present septic system to groundwater contamination in the area with the view of remediation.
ACKNOWLEDGEMENTS

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I also thank my examiner Associate Professor Karin Jönsson for her support in this thesis and her role in the teaching the course `Decentralised Water and Wastewater Treatment´ which has contributed a great deal to some of the concepts applied in this thesis work.

I remember with fondness and am grateful to my children David and Bethel (the new addition) for the joy and hope their smiles and hugs have given me throughout the period of the thesis work. I thank my husband, Emmanuel for his loving inspiration, prayers and encouragement that made me strong. Thanks also for being there despite your tight schedule and enabling me complete this work. My brother, Bemigho Enonuya that supplied me with information on the study area and Toju Ejumudo, my sister that also encouraged me, thank you.

I wish to thank the professionals, who granted permission to reference their original ideas, information and diagrams in this work

Finally, I appreciate friends and family members who showed so much needed concern.

Esther Omenka
July, 2010
**NOMENCLATURES AND ACRONYMS**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>BOD</td>
<td>Biochemical oxygen demand</td>
</tr>
<tr>
<td>COD</td>
<td>Chemical oxygen demand</td>
</tr>
<tr>
<td>DO</td>
<td>Dissolved Oxygen</td>
</tr>
<tr>
<td>EAWAG</td>
<td>Swiss Federal Institute for Environmental Science and Technology</td>
</tr>
<tr>
<td>FRN</td>
<td>Federal Republic of Nigeria</td>
</tr>
<tr>
<td>HRT</td>
<td>Hydraulic retention time</td>
</tr>
<tr>
<td>NTU</td>
<td>Nephelometric turbidity unit</td>
</tr>
<tr>
<td>OWTS</td>
<td>Onsite wastewater treatment system</td>
</tr>
<tr>
<td>PVC</td>
<td>Poly vinyl chloride</td>
</tr>
<tr>
<td>RME</td>
<td>Responsible Management Entity</td>
</tr>
<tr>
<td>SANDEC</td>
<td>Department for Water and Sanitation in Developing Countries</td>
</tr>
<tr>
<td>SS</td>
<td>Suspended solids</td>
</tr>
<tr>
<td>SWIS</td>
<td>Subsurface wastewater infiltration system</td>
</tr>
<tr>
<td>TS</td>
<td>Total solids</td>
</tr>
<tr>
<td>UN</td>
<td>United Nations</td>
</tr>
<tr>
<td>USEPA</td>
<td>United State Environmental Protection Agency</td>
</tr>
<tr>
<td>VFA</td>
<td>Volatile fatty acid</td>
</tr>
<tr>
<td>VS</td>
<td>Volatile solids</td>
</tr>
<tr>
<td>WHO</td>
<td>World Health Organisation</td>
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1. INTRODUCTION

In Nigeria and many developing countries the bulk of the domestic and industrial wastewater are discharged into the receiving waters without proper treatment in both rural and urban areas. There is presently more emphasis on improving drinking water quality than on safe and sustainable wastewater treatment and re-use. However, World Health Organisation has predicted in the Guidelines for safe use of wastewater, excreta and greywater that “As freshwater becomes increasingly scarce due to population growth, urbanisation and, probably, climate change, the use of wastewater, excreta and greywater in agriculture and aquaculture will increase” (WHO, 2006). Also, the millennium development goals of the United Nations aim to improve the socio-economic conditions of the low income countries by spurring development in areas including water supply and sanitation (UN, 2000). In pursuance of these objectives, it is necessary to develop methods of domestic wastewater treatment that are especially suitable in developing countries. The emphasis should be on low-cost, low-energy, and low-maintenance, high-performance systems that contribute to environmental sustainability by producing effluents or end products that can be safely and profitably reused. Making a choice of an appropriate technology will necessitate a detailed survey of the prevailing local conditions. The technology will only be relevant and function well if they are installed in areas with appropriate soils and hydraulic capacities for soil based infrastructure. The design should aim to treat the incoming waste load to meet public health, groundwater, and surface water performance standards, and environmental protection ideal. They should be installed properly and maintained to ensure long-term performance and sustainability (USEPA, 2002). Other factors for consideration include composition of the wastewater, availability of land, availability of funds, expertise and social acceptance.

The wastewater management concept, which has been applied over several decades in the industrialised countries, has received widespread success by improving conditions of hygiene. This concept consists essentially of a pipe network for the collection of wastewater and uses water as a transport medium and subsequently leads to a central treatment plant (EAWAG, 2007). This highly engineered and centralised wastewater management system, although has resulted in the positive reforms in sanitation in industrialised nations in the 20th century but has not been nearly successful when replicated in developing nations (Rose, 1999) due to constraints including lack of technical knowhow, funds, mindset, lack of governmental involvement and commitment etc.

The trend in wastewater treatment that is presently relevant and most appropriate in most developing world context is the decentralised system. The decentralised wastewater treatment option may be low cost and employs naturally based infrastructure which uses processes such as physical, biological and solar elements in their operation. This system is currently receiving global attention (Rose, 1999). Moreover, the decentralised wastewater management concept solves sanitation issues close to the source of waste generation and it better views human and other waste matters as potential resources than the centralised system (EAWAG, 2007).
1.1 OBJECTIVE

The objective of this thesis is to improve on the decentralised domestic wastewater handling presently practised in some developing countries. The work will focus on the impacts this treatment and disposal method have on the water courses, groundwater, health and environment. A study area will be selected with an aim of choosing appropriate technology for wastewater treatment in the decentralised mode suitable for the prevalent site condition including climate, soil type and that would be cultural acceptable with consideration for the developmental conditions of the community. The combination of treatment technology will be an improvement upon the existing method and subsequently the wastewater effluent produced can be safely discharged into the environment. The reuse of wastewater as a local resource will be at the centre of the solution proffered.

1.2 METHOD

The state of wastewater treatment in the study area selected is described based on site information collected. The impacts of the present wastewater handling in the community typical of most developing nations, is studied with the aid of literature. Appropriate treatment methods are proposed for the study area based on information including average wastewater production, rainwater data, population size, local settings, area map etc. The combination of treatment selected is finally represented by a scheme while approximate computations are made to determine their sizes. Mass balance diagrams show the fate of contaminants for each treatment step. Finally the proposed system is then evaluated in the local context.
2. PRESENTATION OF LOCATION

The location for the case study for decentralised wastewater treatment and improvement is Asaba which is in Delta State of Nigeria. Nigeria is a tropical country and is located in West Africa on the gulf of Guinea, having a total area of 923,800 km$^2$ with a population of over 120 million people. It is bordered by Cameroon in the east, Benin Republic in the west and in the north by Niger, lying between 4°N and 14°N and between 3°E and 15°E. Appendix 1 shows the location of Nigeria in the world map. The mean temperature in the country is 27°C, although maximum temperature varies between 32°C in the coastal areas to 41°C in the northern parts of the country. Within the coastal area of Nigeria, the climate is very wet with annual rainfall of about 3500 mm while in the northern regions of the country annual rainfall is as low as 600 mm (FRN, 2003). There are two main seasons in Nigeria; the raining season and the dry season. The raining season in southern Nigeria ranges between 9 -12 months while in the northern parts between 3 – 4 months.

Asaba is the capital city of Delta State of Nigeria, one of the 36 states of Nigeria. The 1991 national population census places Asaba with a population of about 50,000. However, since being made the capital of the oil rich Delta State, there has been migration of people from the rural areas of Delta State and other parts of the country to Asaba. Presently the population of Asaba is estimated to be about 500000. Asaba is located at the northern part of Delta state with an area of 762 km$^2$. It lies between 6° 11’ 0” N, 6° 45’ 0” E. Asaba is located at the western edge of the River Niger which links other West African countries and flows into the Atlantic Ocean. Through the River Niger, Asaba forms a connector between the Northern, Eastern and Western Nigeria. A map of Nigeria showing the location of Asaba is seen in Figure 1 below.

Figure 1: Map of Nigeria showing Asaba (http://geography.about.com/)
2.1 PRESENT STATUS OF WATER SUPPLY AND SANITATION IN ASABA, NIGERIA

The Federal Government of Nigeria (FRN) in 2000 reported that about 30% of urban population has access to an acceptable water supply. This is mainly due to poor maintenance and unreliability of water supply. There is also lack of comprehensive sanitation strategy for the disposal of excreta; wastewater and solid waste in Nigeria in general (FRN, 2000). The sanitation delivery is therefore poor with no urban centre having sewerage system except some parts of Abuja which is the present Federal Capital Territory and Lagos which is the former Nigerian Headquarters.

Asaba is an urban centre in Nigeria and the drinking water supply is from the state owned corporation, Delta State Urban Water Board, an agency under the State Ministry of Water Resources, Asaba. The basic principle used for water supply in the state is to provide water to the towns and villages within the state through the construction of water supply schemes including such components as: groundwater source (boreholes), equipping them with submersible pumps, providing source of storage in form of overhead/ground tanks and having a ready source of power supply in the form of generators to run the schemes and a distribution network (DSUWB, 2009). Individual homes are expected to be connected to the distribution network. In rural settings standtaps are provided to bring water supply closer to the natives, some of who have to walk long distances for their water supply. Osirike (2003) studied some of these water supply schemes and found that 58% of them were functional in the rural areas. In the Asaba metropolis however, there is a main water supply scheme built in the mid 1960s with an overhead storage tank capacity of 150,000 gallons (DSUWB, 2009). This scheme functions only epileptically due to a myriad of reasons including inconsistent power supply. Although there is a standby generator, it does not function often due to lack of fuel and maintenance issues. There is also lack of skilled manpower, management, financial issues and frequent leakages from the distribution network which is worn in many places due to age and material used in construction. There are also smaller water supply schemes complimentary to this in Zappa, Ogbolie, and St Patrick’s College etc, all fraught with similar problems. Many private homes have water supply boreholes and others buy water from the private owners. These private boreholes are not state supervised for quality control.

The wastewater collection and treatment in many parts of Asaba is mainly decentralised with onsite facilities such as ventilated improved toilet (VIP), septic tanks, Imhoff tanks which are connected to soil wastewater infiltration systems which in most cases are soakaway pits. These decentralised systems are designed mainly with separate grey and black water treatment. The wastewater treatment facilities are private owned by property lot owners (landlords). These are designed and constructed together with the houses; hence specifications are approved by the lands and surveys ministry for newer buildings. The subsequent management of the onsite facilities is left in the hand of the landlords. These include dislodging of the sludge when the septic tank is full or overflowing, repair of breakages and leakage. There is no government ministry presently in charge of routine checks to ensure adequate functionality and health safety of these facilities.
2.2 THE PROPOSED SITE FOR DECENTRALISED WASTE WATER TREATMENT IMPROVEMENT IN ASABA

Asaba is divided into five main quarters or villages including Ezenei (Umuezei), Ugboriama, Agu, Ajaji and Onaie which correspond to the grandsons of one of the most famous early settlers (asaba.com). A portion of Ezenei Street within Umuezei Quarters is chosen for the hypothetical design because it is the residence of the present day traditional ruler, the Asaga of Asaba. It is also representative of the population that can be obtained in many urban settlements in Nigeria having also similar wastewater treatment. The area delineated in the map, Figure 2 below has about 92 houses with about 18 persons per house with land area of 136,877 m². The area was sectioned off as an extended family unit as most of the buildings belong to persons with family relations extending to generations. Although some buildings are owned by other settlers, it is a community where people know one another. It is also chosen for ease of design with the roads and footpaths acting as boundaries. The houses are not properly planned. The status of the people is mainly middle to low income and mostly civil servants and traders. Other activities include hairdressing salons, tailoring and restaurants, business centers.

Figure 1: Map of study area, Ezenei Asaba (Google earth: 6°11’49.86” N 6°44’04.65” E)
3. UNDERSTANDING DECENTRALISED WASTEWATER TREATMENT AND TREATMENT PARAMETERS

Wastewater is defined as effluent from domestic activities consisting of blackwater which includes excreta, urine and sludge and greywater which includes kitchen, laundry and bathroom wastewater (Hoek, 2004). Centralised wastewater includes domestic wastewater, wastewater from commercial and industrial sector, stormwater and other runoff. However, there is no clear cut definition of decentralised wastewater treatment. Otis (1999) defined decentralised wastewater treatment as a concept which involves basically management. It is the approach whereby one responsible management entity is directly responsible for a group of treatment infrastructure which are placed in such strategic locations within the operational zone of the responsible management entity, in which every resident is provided with treatment facilities that are easily affordable and very effective. He proposes a distributed treatment concept of decentralisation at the watershed level with treatment technologies ranging from a combination of clusters, onsite systems and in some cases even municipal treatment plants. Decentralised wastewater treatment in this approach can be seen as a management concept with a whole watershed in focus rather than small-scale wastewater treatment technologies. However, the wastewater treatment presently prevalent in most developing countries is the decentralised system which includes onsite and the cluster systems and these have not been fully integrated as a management principle as there is lack of inventories, accountability, proactive monitoring and co-ordination of the system.

Tchobanoglous (1995) defined the decentralised wastewater management as the collection, treatment, and disposal/reuse of wastewater from individual homes, clusters of homes, isolated communities, industries, or institutional facilities, as well as from portions of existing communities at or near the point of waste generation. In this concept, the decentralised wastewater treatment includes the onsite and the cluster systems. An onsite wastewater treatment/disposal system is one that treats wastewater from individual homes while the cluster system treats wastewater from a group of two or more homes. When considering some clusters that serves large communities, it becomes increasingly difficult to demarcate between decentralised and centralised systems. However, they can be differentiated based on volume of wastewater, sewer type and treatment type, ownership, discharge method and relative scale (Booz et al., 2004).

Considering the factors above, the decentralised wastewater treatment will handle relatively small volumes, employ small diameter pressurised pipes, on-lot settling tanks and treatment alternative such as sand filters. It is also usually owned by private entities such as individual developers, homeowners associations etc. Most onsite and cluster systems discharge the treated effluent by soil infiltration and it is intended to serve individual homes and a portion of a community.

The centralised wastewater treatment on the other hand employs processes that handle large volumes of wastewater and employs gravity sewers. They are usually public owned, idealised to serve a substantial part of a community or whole communities. The treated effluents from centralised wastewater treatment plants are often discharged to surface water bodies (Booz et al., 2004).

The foundational task of wastewater treatment is taking the wastewater as effluent from individual homes, commercial and industrial sources through different unit processes with the aim of restoring to the original or acceptable quality. Therefore the outcome of the unit
processes used to treat wastewater before discharge into receiving water are determined and controlled by the wastewater effluent quality parameters (Drinan and Nancy, 2001). These are stipulated by each country. The effluent parameters for the treatment of wastewater are such that promote safe discharge of wastewater effluent to the receiving water body in terms of quality. Wastewater parameters provide the guideline whereby the biological, chemical and physical characteristics of wastewater can be measured. Wastewater treatment parameters are dependent on the type of contaminant in the wastewater stream. The major types of contaminants can be grouped as organic or inorganic. The important organic contaminant parameters for this work include BOD (biochemical oxygen demand), COD (chemical oxygen demand) and the inorganic contaminants include P (Phosphorus), N (Nitrogen) and heavy metals. Other relevant parameters include SS (Suspended Solids) and TS (Total Solids) (Gillberg et al., 2003).

Biological Oxygen Demand (BOD) is defined as the amount of biodegradable substances present in the wastewater achieved by measuring the amount of oxygen consumed by microorganisms or bacteria in the wastewater. The micro-organisms will consume oxygen during the breakdown of biodegradable substances. The oxygen demand is measured over a period of 5 days (BOD_5) or 7 days (BOD_7) at a temperature of 20°C. BOD is measured in mg oxygen/l or g Oxygen/m^3 (Gillberg et al., 2003).

Chemical Oxygen Demand (COD) is defined as the measure of the contaminant concentration in the wastewater which can be oxidized at high temperature by oxidizing agents such as potassium dichromate. The amount of oxidizing agent used in the process for complete oxidation is the measure of the organic content of the wastewater which is then converted to equivalent oxygen concentration. The COD is measured in mg oxygen/l or g Oxygen/m^3.

Phosphorus and Nitrogen. Phosphorus occurs in wastewater as organic and inorganic form. The organic form is bonded to solids in the wastewater while the inorganic form exists in dissolved form as orthophosphates and polyphosphates which cause algae growth and eutrophication of lakes. The major contributors of phosphorus to wastewater are human waste and phosphorus-laden detergents. Nitrogen is an essential nutrient for microbes and plants and the main contributors of nitrogen to the wastewater are from human urine, faeces, atmospheric deposition and agricultural activities. The four main types of nitrogen in wastewater are organic nitrogen, ammonia nitrogen, nitrite nitrogen and nitrate nitrogen. Nitrogen can easily change forms depending on the oxidation state. Table 1 below shows the characteristic pollutants in raw residential wastewater (blackwater).

**Table 1: Characteristics of Raw Residential Blackwater (Adapted from Burubai et al., 2007)**

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Blackwater (mg/l)</th>
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<tbody>
<tr>
<td>Biochemical Oxygen demand (BOD)</td>
<td>603 – 665</td>
</tr>
<tr>
<td>Chemical Oxygen demand (COD)</td>
<td>734 – 765</td>
</tr>
<tr>
<td>Total Organic Carbon (TOC)</td>
<td>329 – 332</td>
</tr>
<tr>
<td>Suspended Solids (SS)</td>
<td>269 – 272</td>
</tr>
<tr>
<td>Fecal coliform (FC)</td>
<td>13 – 17 (10^6/100 ml)</td>
</tr>
</tbody>
</table>
4. DESCRIPTION OF EXISTING TECHNOLOGIES FOR DECENTRALISED WASTEWATER TREATMENT IN NIGERIA

There are a number of treatment modes that are presently used in Nigeria and most developing countries for decentralised wastewater treatment or handling. The technologies employed use a number of simple, cost effective, low energy and easy to maintain methods such as anaerobic processes, ponds and earth based treatment options.

4.1 ANAEROBIC DIGESTION PROCESSES

Septic tanks, Imhoff tanks, baffled septic tanks and ventilated improved toilets are some of the most commonly used treatment decentralised technology which employ the anaerobic digestion processes. Anaerobic processes take place in the absence of oxygen and can be described in four phases namely hydrolysis, acidogenesis, acetogenesis and methanogenesis (Mata-Alvarez, 2003).

In hydrolysis complex organic molecules are broken down to soluble organic monomers by bacteria, whereby, carbohydrates, fat and proteins are converted respectively to monosaccharide, fatty acids and amino acids. Substrates like wastewater sludge undergo hydrolysis. This process leads to production of organic acids which lowers the pH and reduces bacteria decomposition.

The acidogenesis phase is basically the fermentation of organic acids where the products from hydrolysis are further converted into volatile short chain acids or volatile fatty acid (VFA) such as propionic, lactic, butyric acids etc and ethanol, methanol and alcohols. This is done by acidogenic bacteria which favour the low pH from the hydrolysis phase. Ammonium is also produced by amino acids in this stage.

The third phase acetogenesis is regarded as combined with the process of acidogenesis. The organic fatty acids are degraded by acetogens which produce hydrogen and acetone. CO₂ is also a product of acetogenesis. There is also BOD and COD reduction.

The fourth phase, metanogenesis is the determinant of the anaerobic digestion process because of the slow growth rate of the matanogens, which are the bacteria which control methane fermentation. They favour a more neutral or alkaline environment and work by either reducing carbon dioxide by hydrogen or alcohol fermentation or acetate conversion forming CO₂ and CH₄.

4.2 SEPTIC TANK

The most used form of anaerobic treatment process for domestic wastewater in Nigeria is the septic tank. A septic tank is a self-contained, underground wastewater treatment system usually with two or more compartments. Septic refers to the anaerobic condition which develops in the tank. The effluent produced from a house consisting usually of blackwater (toilet waste) flows directly through the sewer pipe into the septic tank usually made of concrete or fibreglass.

The septic tank is for pre-treatment and treats the wastewater by holding it in the tank with a retention time long enough for solids and liquids to separate. Wastewater then forms three
layers inside the tank. Solids lighter than water (such as greases and oils) float to the top forming an anaerobic layer of scum. Solids heavier than water settle at the bottom of the tank forming another anaerobic layer of sludge. This leaves a middle layer of partially clarified effluent or wastewater. This layer of clarified liquid then flows from the septic tank to the drainfield or a distribution device, which helps for further purification. The semi-solid waste (fresh sludge) is digested by anaerobic organisms, which live without oxygen, in the tank. During anaerobic decomposition, there is generation of some gases such as CH$_4$ and H$_2$S which escape from the septic tanks through the vents. Emission of gases occurs sporadically and affects the resuspension of sludge and the movement of the anaerobes. In the septic tank the organic sludge and the scum are the first to be decomposed anaerobically which reduces the sludge volume. Figure 3 below shows a schematic diagram of a septic tank.

Figure 2: Schematic diagram of a septic tank (Maryland Dept of Environment, USA, 1970)

4.2.1 REMOVAL OF CONTAMINANTS IN SEPTIC TANKS

A septic tank removes many of the settleable solids, oils, greases, and floating debris in the raw wastewater. The removal efficiency of a septic tank that treats black water was studied in Bayelsa State of Nigeria and the result showed 54% BOD, 57% COD and 40% SS removal (Burubai et al, 2007). Nitrogen enters the septic tank from raw wastewater in a complex form as organic molecules which are mainly from faeces and urea in the human urine. When there is suspended solid removal through sedimentation, some particulate nitrogen can be removed. In addition, during the process of hydrolysis, particulate nitrogen is converted to soluble form which is thereafter converted to ammonium through the process of ammonification at normal pH. During a two year operation of a pilot plant at La Pine, Oregon, USA, it was observed that the septic tank effluent contained about 70% nitrogen effectively removing on the average 30% (Lombardo, 2004).

Phosphorus is a key element in the eutrophication in surface water bodies and its removal is of concern where septic tank effluent may flow directly or indirectly through subsurface flows to surface water. One of the most effective ways of reducing phosphorus from wastewater is
to use phosphate free detergents (USEPA, 2002). Onsite systems are rarely designed for phosphorus removal. Most of the processes that remove phosphorus from wastewater are additions to other pre-treatment processes. Table 2 below shows some characteristics of septic tank effluent.

**Table 2: Characteristics of domestic septic tank effluent. BOD, SS in two compartment septic tank (Seabloom et al, 2004)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>BOD (mg/l)</th>
<th>Suspended Solids (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw sewage influent</td>
<td>184</td>
<td>234</td>
</tr>
<tr>
<td>Septic tank effluent</td>
<td>99</td>
<td>123</td>
</tr>
<tr>
<td>% Removal</td>
<td>46</td>
<td>48</td>
</tr>
</tbody>
</table>

**4.2.2 DESIGN OF SEPTIC TANK**

The factors to be considered in the design of the septic tank include the tank volume, hydraulic retention time, and geometry, compartmentalisation, pumping rate and site hydrogeology.

**4.2.3 TANK VOLUME AND HYDRAULIC RETENTION TIME (HRT)**

The septic tank will be designed with the primary task of providing quiescent conditions within the tank in order to enhance sedimentation for effective removal of suspended solids, oil and grease. This is achieved by providing a suitable hydraulic retention time which is determined by the tank geometry and volume (USEPA, 2002). The volume of the septic tank is determined based on the population size and water use which will ultimately affect the peak daily flow necessary for design. USEPA (2002) and D’Amato et al. (2008) specify a design volume based on two times the design flow, which is the existing or projected daily wastewater flow or the water consumption if wastewater flow values are unavailable. The rule of thumb for the hydraulic retention time for a septic tank is at least 24 hrs after allowing for maximum sludge and scum accumulation which is taken to be about 50% of the tank size (D´Amato et al., 2008) or two-thirds the tank (USEPA, 2002). Another important factor based on flow from the population is whether or not the septic tank is combined for both grey and blackwater.

**4.2.4 TANK GEOMETRY**

It is also known that tank geometry is an important consideration that affects the residence time of solids in the tank. Septic tanks with a ratio of 3:1 length to width respectively will improve the removal of solids by reducing short-circuiting of raw wastewater and increasing the travel path of particles when they settle.

**4.2.5 COMPARTMENTALISATION**

Under real conditions, it has been observed that compartmentalisation of the septic tank enhances suspended solid and organics removal compared to single compartment tanks. Compartmentalisation causes sludge to settle out more in the first compartment hence most of the digestion of sludge takes place there. It is important that the first compartment is larger.
than the subsequent ones to dampen oscillations between compartments (D´Amato et al., 2008)

4.2.6 PUMPING RATE

The volume of the septic tank decreases as scum and sludge accumulate with the use and the effective volume for settling diminishes. The scum and sludge accumulation rate can be predicted to determine the frequency of pumping or dislodging of sludge from the septic tank. Figure 4 below shows work done by the United States Public Health Service (1955) and Bounds (1988). The graph of sludge/scum accumulation in gallons per person with years has been used to determine with a high level of confidence, the interval of pumping of sludge from the septic tank. The desired rate of pumping or dislodging of sludge from the septic tank and the estimated wastewater flow rate can thus be a guide in determining the tank dimensions. However, the interval of pumping should provide enough time for digestion of solids and be affordable. Gray (1995) determined that the mean sludge accumulation rate is 0.234 l/p.d (85.3 l/p.y) after testing the sludge accumulation in 28 septic tanks. This is irrespective of sludge age. However, the sludge age affects the accumulation as it declines with time with a significant decrease after 12 months. USEPA determines that septic tank should be pumped when the sludge and scum accumulates has exceeded 30 % of the tank volume or when it encroaches on the inlet or outlet baffles. Depending on the tank size this takes about 3 – 5 years.

Figure 4: Sludge/scum accumulation rate of septic tanks in gallons per person (Bounds, 1997)

4.3 THE IMHOFF TANK

The Imhoff tank is basically a two storey septic tank design but serves more effectively than the ordinary septic tank in separation of fresh sewage from already digested sewage. The upper chamber acts as a sedimentation tank with a bottom inclined to a slope which allows the settled sludge to flow to the bottom chamber. Here the sludge is then digested by anaerobic bacteria. The sloping funnel-like walls prevent an upflow of sludge from the lower to the upper chamber, hence preventing mixing and turbulence in the system. Digested sludge is only removed from the bottom chamber which also is separately vented. While the Imhoff tank ensures a higher quality of effluent, its drawback is that it requires more frequent emptying of digested sludge. Figure 5 below shows a schematic diagram of an Imhoff tank.
4.4 THE ANAEROBIC BAFFLED REACTOR (BAFFLED SEPTIC TANK)

The baffled septic tank, also known as the anaerobic baffled reactor (ABR), like the Imhoff tank is an upgraded form of the septic tank. It is most suited for low population areas with expectation of higher effluent quality. The ABR consists primarily of a compartment for settling just like the simple septic tank but the adjoining section has baffled reactors laid out in series. The baffle walls are configured in such a way to ensure flow of the effluent wastewater from the settling compartment in an upflow mode through a series of anaerobic sludge blanket reactors. The upflow velocity is important and its design usually results in large but shallow upflow chambers. The construction of the ABR is handled by skilled contractors and sludge production although low, removal should be done regularly. The BOD removal efficiency rate of 70-95 % is achievable with the ABR (UNESCO-IHE, 2006). Figure 6 below shows a schematic diagram of an anaerobic baffled reactor.

Figure 6: The Anaerobic baffled reactor, ABR (After Ludwig, 1998 in UNEP, 2002)
4.5 PONDS

Domestic wastewater is also treated in waste stabilisation ponds or lagoons in both developing and developed countries depending on the available space, for they usually require large areas. Waste stabilisation ponds are scattered around some major cities in Nigeria, Ile-Ife, Nsukka, Zaria etc. The ponds use natural processes with manmade basins suited for both small and large populations have low capital costs, simple in operation and maintenance and have high performance. The ponds may be anaerobic or aerobic depending on if the pond is aerated or otherwise, usually shallow. The design of ponds depends on the temperature of the area, the hydraulic retention time (HRT) and the loading rates, both organic and hydraulic. They are well suited for tropical countries but require more area in the temperate climates and have been used successfully in some temperate countries. A typical pond treatment system consists of three ponds in series, where the first pond is anaerobic, the second facultative and the third a maturation or aerobic pond. Figure 7 below shows waste stabilization pond in series. The anaerobic pond is primarily designed for removal of suspended solid and soluble organic matter or BOD, while the facultative pond works through the action of algae and heterotrophic bacteria to remove more BOD and the maturation pond finally removes pathogens and nutrients.

Figure 7: Waste stabilisation ponds in series (Van der Steen, UNESCO-IHE Institute for Water Education, 2003)

4.5.1 ANAEROBIC PONDS

In anaerobic ponds, anaerobic conditions are maintained throughout its depth. That is, there is usually no dissolved oxygen or algal growth. They have a depth of about 3 – 5 m (Münch, 2005). They can receive wastewater with organic loading of about 100 gBOD/m³/day or its equivalent. Where the wastewater is primarily treated in a septic tank before the pond, the tank can have a hydraulic retention time (HRT) of 1 day. On the contrary the anaerobic pond can act as an open septic tank. For a low loaded anaerobic pond the neutral pH maintained prevents the release of H₂S gas hence preventing odour emissions. High loaded anaerobic ponds on the other hand may have odour problems until a layer of scum is released to spread over the pond covering it. The anaerobic digestion is temperature dependent and for an
anaerobic pond which is well designed, BOD removal of 60% or more can be achieved (Kayombo et al., 1998).

4.5.2 FACULTATIVE PONDS

The facultative pond is layered and at the top layer, aerobic treatment occurs while there is anaerobic degradation in the deeper layer. There are primarily two types of facultative waste stabilisation ponds: primary and secondary. The primary facultative pond may receive raw wastewater after it has been screened and the secondary facultative pond receives pre-treated wastewater from either a septic tank or an anaerobic pond. The loading of the facultative pond is temperature dependent with a depth between 1 and 2 m. (Peña and Mara, 2004). Generally, the upper aerobic layer of the facultative pond also permits the growth of algae which gives it its predominantly dark green colour. The algal population also generates the oxygen required for BOD removal from the pond. BOD removal in this pond amounts to about 70%. The retention time for a facultative pond is between 10 and 20 days. In the bottom layer, there is a decrease of oxygen due to less penetration of light to the depth, the oxygen demand becomes more than the supply. There is also improper mix of the two layers. These factors give rise to anaerobic conditions in the bottom layer with the release of methane.

4.5.3 AEROBIC PONDS

The aerobic pond which is also known as maturation pond, usually receives effluent from the facultative pond and it maintains aerobic conditions throughout the depth of the pond. This is done either by the natural transfer of oxygen between the air water interfaces or the pond can be aerated by algae growth during photosynthesis. The aerobic pond is called a shallow pond and usually at a depth of 1 – 1.5 m. Another form of maturation pond is the aerated pond in which oxygen transfer is done by mechanical devices for aeration. This may be more expensive to operate and maintain, although it could be deeper and require less area. The size of the pond is determined ultimately by the requirement of the quality of the final effluent. The aerobic pond removes mainly virus and fecal bacteria by algae activity and photo-oxidation. Phosphorus removal is also by algae biomass uptake, precipitation and sedimentation. Nitrogen which has been converted to ammonium in the anaerobic pond is taken up by algae biomass in the maturation pond (Kayombo et al., 1998). Total nitrogen that may be removed from the waste stabilisation pond, depending on the number of ponds included in the series is often above 80%, 50% phosphorus removal and 90% ammonium removal (Peña and Mara, 2004). In the design of maturation ponds two primary considerations are time and temperature as fecal bacteria die off with increase of time and temperature. High light intensity and dissolved oxygen are also necessary for increased removal of fecal bacteria in these ponds.

4.6 SUBSURFACE WASTEWATER INfiltration SYSTEM: SOAKAWAY PITS

The subsurface wastewater infiltration system usually does not receive raw sewage but pre-treated wastewater from septic tanks, Imhoff tanks and passes it through unsaturated soil to the groundwater. As the pre-treated wastewater passes the soil strata, it is purified through physical, biological and chemical processes.

Presently in Nigeria, most septic tanks are connected to soil water infiltration system for individual buildings. The infiltration system is mainly a pit whereby the sides are used as infiltration surface. These pits are called soakaway pits and are connected directly to the
septic tanks. In Asaba, the design standards are not followed as in most Nigerian cities, whereby the quality of effluent from the septic tank which should determine the design of the soakaway device is not considered in the construction (Burubai et al., 2007). Other considerations in the design of the soakaway pit should be the depth to water table, soil type, and soil and subsoil percolation rate, volume of wastewater effluent from the septic tank. The sides of the pit are usually lined with perforated sandcrete blocks.

4.6.1 DESIGN OF RELEVANT TECHNOLOGY FOR SEPTIC TANK EFFLUENT

4.6.2 THE INTERMITTENT SAND FILTER

The intermittent sand filter is an aerobic device that can be used for the secondary treatment of septic tank effluent. It is used effectively in removal of many contaminants before discharge to the groundwater through the soil water infiltration system. The intermittent sand filter is a bed of sand (or granular material) underlain by gravel base with an impervious bottom layer lined with PVC. There is a drain underneath the washed sand gravel bed that collects treated septic tank effluent. The filtered water then flows to the soil water infiltration surface. The processes that take place in the intermittent filters include filtration or straining of suspended solids, sedimentation, biological process such as BOD removal and nitrification. Chemical adsorption of phosphorus can occur based on the media used. The design of the intermittent sand filters depends on the organic loading rate and the sand media characteristics. The sand media should be sized between 0.25 – 1 mm in diameter, the depth about 1 m or less, dosing tank volume about ½ or 1½ times the volume of septic tank effluent, dosed between 12 – 24 times daily with 76 mm (3 inch) diameter pipes. The organic loading of the intermittent sand filter should be 0.024 kg/m² per day (USEPA, 2002). The dosing system should be designed in such a way to provide maximum contact of the effluent with the media. The filter can remove up to 90% BOD and over 55% total nitrogen.

4.6.3 THE SOIL WATER INFILTRATION SURFACE: BED, TRENCHES, MOUND

The soil water infiltration system is the component by which the septic tank effluent is dispersed into the ground water. It can receive primarily treated wastewater from the septic tank or for better contaminant removal from the sand filter. There are different configurations of the system such as mounds, at grade, trenches and beds. It is usually a layer of natural soil or an imported fill which has capacity for infiltration placed over crushed rock or gravel as porous media to aid in spreading the effluent from the filtration surface. The gravel will also act as support for the dosing pipes. The infiltration surface is aerobic permitting biological degradation and nitrification. The unsaturated soil strata beneath the infiltration zone enable percolation and dispersal of the septic effluent. There is both pathogenic and phosphorus removal from this zone. USEPA (2002) specifies an organic loading of 150 lbBOD/1000ft² (0.73 kg/m²) for a sandy soil as found in Asaba. The infiltration surface should be placed at a depth less than 0.6 m below the final grade to allow for proper aeration. There can be multiple cells to permit for periods of rest, maintenance and ideal uniform spreading of effluent. The width of the infiltration bed must not be greater than between 3 m and 4.5 m while the bed length is governed by the site and dosing method for spreading of the effluent. To allow for better denitrification, there can be a mound whereby the infiltration surface is raised so as to permit the placement of a layer of about 0.3 m organic rich layer as carbon material such as saw dust (Bedessem et al., 2005). The effluent from the sedimentation tanks can be dosed either by gravity with a distribution box onto perforated pipes or there can be dosing under
pressure which is more effective. The soil wastewater infiltration system is able to remove 90 – 98% BOD, 10 – 40% Total Nitrogen, 85 – 95% phosphorus and up to 99% fecal coliform.

4.6.4 CONSTRUCTED WETLANDS

Constructed wetlands can be used for the secondary or tertiary treatment of septic tank effluent and can achieve high contaminant removal. They are either surface flow or subsurface flow wetland that use processes such as biological, chemical, physical and microbial for the treatment of wastewater. The surface flow wetlands are constructed to resemble natural wetland while the subsurface flow wetland is a bed of porous media, such as sand or gravel, usually vegetated and can either be vertical flow or horizontal flow by configuration.

The removal of contaminant in a wetland depends on the wastewater type, organic and hydraulic loading, the constructed wetland configuration or design and the climate. The plants in a wetland are important for gas transfer such as oxygen into the wastewater and methane and other gases out of the wastewater. During plant growth also, there is removal of nutrients such as nitrogen and phosphorus by the wetland plants. The influent ammonium nitrogen and organic nitrogen into the surface flow wetland can be removed by the process of nitrification as the wetland is aerobic. However it is recommended for effective nitrogen removal, the design of a surface flow wetland, that it is not fully vegetated.

The vegetated submerged wetland on the other hand, is effective for denitrification due to its anaerobic state. The rate of denitrification is however governed by the available carbon from the wastewater and decaying wetland plants. The minimum requirement for denitrification is a COD to nitrogen (nitrate) ratio of about 2.3 g (USEPA, 2000). BOD and TSS removal in a subsurface flow wetland is by flocculation, settling and filtration. Surfac ing of wastewater due to heavy rainfall can be prevented in subsurface wetland by evapotranspiration and proper drainage system design. The subsurface wetland has the advantage of no need for protection from mosquito or other vector.

The design criteria include organic or hydraulic loading and retention times for both surface and subsurface flow wetlands. The organic loading rate is given respectively as 80 kgBOD/ha.d and 75 kgBOD/ha.d by Wood (1995). A two staged subsurface flow wetland treating primary sedimentation effluent: a horizontal flow and vertical flow subsurface wetland connected in series was studied by Shrestha et al., (2001) and obtained the following contaminant removal 97% TSS, 95% NH₄-N, 97% BOD, 93% COD, 99% E coli, and 47% PO₄-P removals.
5. EFFECTS OF DECENTRALISED WASTEWATER TREATMENT ON WATER SOURCE, ENVIRONMENT AND PUBLIC HEALTH

There have been many reports of incidences of system failure and poor performances associated with the decentralised wastewater system especially the septic tanks and associated soil water infiltration systems (Carrol et al., 2006). The poor performances of these onsite or decentralised systems are related with soil and site conditions not being properly investigated before the systems are sited. This is important because soil condition is influential in determining the level of treatment, and disposal of septic tank effluent prior to its discharge to groundwater (Dawes and Goonetilleke, 2006). Also, septic tanks provide only primary treatment for wastewater and hence effluents may be discharged into groundwater without proper treatment with harmful micro-organisms including pathogens and nutrients such as nitrogen and phosphorus (USEPA, 1996)

5.1 EFFECT OF SYSTEM DENSITY ON GROUNDWATER

To effectively protect a groundwater resource from contamination from onsite wastewater treatment system, it is required that there is an adequate separation distance usually vertical between the infiltration system and the highest head or expected rise to which the groundwater may attain. In similar vein, surface water sources are also protected by applying lateral separation distance between the subsurface wastewater infiltration system and the surface water (Hantzsche and Finnemore, 1992).

Density of the septic system especially with the associated subsurface wastewater infiltration system is also a very important factor in determining the impact of decentralised onsite and cluster systems in the contamination of groundwater resource. Population density will determine ultimately, the load of effluent that is applied to or that infiltrates per unit of a land area and subsequently the amount of contaminants that may percolate and enter into the groundwater (University of Florida, 1987). It was noted also that there is higher risk of groundwater contamination with onsite wastewater treatment system densities greater than 15/km² (Geary and Whitehead, 2001). The impact of density of onsite or decentralised wastewater system is particularly relevant in the study area in Asaba, Nigeria where most of the city have onsite system density greater than 15/km². When a water budget was considered for individual private building lots it was observed that the onsite wastewater treatment and disposal systems had a potential of contributing about 60% to groundwater recharge in the area under consideration. This in effect may contribute to contamination of the aquifer if the decentralised wastewater treatment system does not remove contaminants effectively (University of Florida, 1987). System density is especially important in urban centres in developing countries where overpopulation gives rise to septic system densities that make even suitable soils incapable of assimilating the load of wastewater flow and to retain and transform the contaminants (USEPA, 2002). There are also factors such as climatic, soil and site characteristics and type of onsite systems which become more critical when there is high density of these onsite systems. The depth to groundwater is also of importance in siting of decentralised wastewater treatment systems. Carroll and Goonetilleke (2005) concluded in their study that there is significant impact on shallow groundwater with high system density when they tested the chemical and microbiological data from the aquifer under a high density of onsite wastewater treatment system in a coastal region.

Nitrate-Nitrogen build-up in groundwater resources is about the most significant effect of the long term use of onsite wastewater treatment or the decentralised system. Carroll et al. (2004)
stated that there is appreciable contamination of groundwater resource from nitrate-nitrogen due to density of onsite wastewater treatment systems. This is particularly important if the groundwater is being used as a potable water source. A large proportion of the decentralised systems are not designed to adequately treat nitrogen although they could remove some other wastewater contaminants. The infiltration device is aerobic in nature hence most of the ammonia or ammonium nitrogen and organic nitrogen from the wastewater is converted to nitrate NO$_3^-$ by nitrification in a two phase process which is shown in Equations 1 and 2 below.

**First Step:** The groups of bacteria, known as nitrite formers or ammonium oxidizers, convert ammonia under aerobic conditions to nitrates and derive energy from the oxidation:

\[
\text{Equation 1} \quad 2NH_3^+ + 3O_2 \rightarrow 2NO_2^- + 2H^+ + 2H_2O
\]

**Second Step:** Then the "nitrite nitrogen" is oxidized by a group of nitrifying bacteria, also known as the nitrate formers, into the "nitrate nitrogen" form:

\[
\text{Equation 2} \quad NO_2^- + O_2 \rightarrow 2NO_3^- + \text{(nitrite oxidizers)}
\]

The subsurface infiltration layer is usually placed at a depth where plant root cannot take up significant amount of nitrate. In addition, the microbial process that may be required to make the nitrate immobile is not met because the carbon to nitrogen ratio of the subsurface infiltration soils is low (Degen et al., 1991). Thus the nitrate can move relatively unchanged and with other effluent contents to the groundwater (Hantzsche and Finnemore, 1992). Nitrate reaching the subsurface wastewater infiltration system can only be effectively reduced by denitrification which is the biological and chemical reduction of nitrate and nitrite to nitrogen and or nitrous oxide both of which are volatile gases in the presence of high moisture condition, high soil pH and high organic carbon. However, the rate of denitrification is insignificant due to lack of anaerobic condition and organic matter that make for the presence of denitrifying bacteria especially in infiltration system with predominantly sandy characteristics (Walker et al., 1973). However, when an organic rich layer of 0.3 m, acting as carbon source was placed under the model of infiltration system or drainfield as studied by Bedessem et al. (2005), it was concluded that the rate of denitrification of the septic tank effluent increased which subsequently reduced the transport of nitrate to the groundwater. The chemical representation of the processes of denitrification is shown in equation 3 below. There is an increase in alkalinity.

\[
\text{Equation 3} \quad 2NO_3^- + H^+ + \text{(bacteria)} \rightarrow HCO_3^- + N_2 + \text{(nitrogen gas (end product))}
\]

The ability for groundwater to dilute the nitrate is limited hence, to reduce the nitrate that percolates into groundwater from septic tanks and the associated infiltration system to an
acceptable background level, a large horizontal distance of 2.9 km separation between adjacent disposal systems is required as proposed by Pang et al. (2005) in their study of effect of septic tank clusters on groundwater in a coarse gravel aquifer. Achieving this distance may not be practicable; hence there is need for adequate effluent treatment before disposal. Siting of water supply systems upstream of the wastewater treatment and disposal system may produce a viable solution. However, where there are clusters, neighboring disposal system may contaminate the groundwater source (Pang et al., 2005). A model, known as dilution model, developed by Trela and Douglas in 1978 (Equation 4) can be used to predict in acre per person, the capacity of the soil to renovate the septic tank effluent discharged without having to contribute to groundwater nitrate above recommended standards (Rogers et al., 1988).

\[
\frac{V_e C_e}{(V_i + C_i) C_q} = H
\]

where \(V_e\) = Volume of septic tank effluent entering the system  
\(C_e\) = Nitrate concentration in septic tank effluent  
\(V_i\) = Volume of infiltrating capacity  
\(C_i\) = Nitrate concentration in precipitation  
\(C_q\) = nitrate standard  
\(H\) = carrying capacity (acres/person)

### 5.2 Impact of Decentralised Wastewater Treatment Systems on Environment and Public Health

Decentralised wastewater treatment systems have been reported to result in environmental and public health impacts. These systems are a threat to surface water and the ecosystem among other sources. They function by basically providing an enabling environment for the solid contaminants to settle while digesting the organic matter but the liquid effluent may contain some pathogenic organisms such as bacteria, viruses and protozoa and nutrients which eventually are discharged into the soil (McDowell et al., 2005). In addition, the regular event of failed septic tank and outflow of raw wastewater into the stormwater drains resulting in unsanitary conditions is not unrelated to the common cases of dysentry, diarrhea, gastrointestinal illness and typhoid fever in Nigeria (FRN, 2000).

Most pathogens and some slight amount of phosphorus are removed through biological processes especially under suitable soil and groundwater conditions. Nitrates from the decentralised systems when dissolved in groundwater through the soil profile can eventually be transported to surface water. This is because of the interaction between ground and surface water. These two sources of water are interconnected as water that circulates through the hydrological circle basically gets to the groundwater and is eventually transported to surface water. The area of case study in Asaba, Nigeria is particular of interest due to its nearness to River Niger. The area slopes downwards towards the River Niger and with the density of onsite system, there can be both groundwater flows from the subsurface wastewater infiltration system and shallow subsurface water flow towards the river.

Nitrate discharged from septic systems and associated infiltration system can cause nutrient enrichment in fresh surface water. Nutrient enrichment, also known as eutrophication is the process whereby there is over fertilisation of the water body with nutrients such as nitrogen
and phosphorus. This phenomenon leads to negative ecological effects such as turbidity of surface water, increase in the production of aquatic weeds, algae blooms, and lowered level of dissolved oxygen. Nutrient enrichment by nitrogen from septic system is enhanced if there is a complementary source of phosphorus such as from storm water runoff drainages and erosion. One the other hand, there can be phosphorus outflow from septic systems and infiltration device through groundwater aquifers located close to rivers and lakes (McDowell et al., 2005).

Nitrate nitrogen in potable water supply can cause direct risk to livestock and human health. High levels of this can cause severe blood disorder known as methemoglobinemia or ‘blue baby syndrome’ which can affect people of all ages but impacts children less than six months mostly and can result in complications in pregnancy (USEPA, 2002). In many developing countries, a vast majority of the population relies on untreated groundwater as source for potable water and microbial quality may not be regulated by health, water or environmental agencies. Septic tanks can also be a source of pathogenic contamination of groundwater in these parts as well as other countries where there is widespread dependence on them.

Borchardt et al. (2003) in the study of onsite system density in relation to infectious diarrhea stated that onsite wastewater system is a store for human enteric pathogens and linked the density of onsite systems to gastrointestinal illnesses. Since the treatment of onsite wastewater depends on the soil and environmental conditions for renovation of the effluent through filtration and adsorption, the effectiveness may be limited and this will lead to disposal of bacteria and viruses into the groundwater. Carroll et al. (2005) also found fecal contamination in surface water. Although the majority of the fecal contamination in rural setting was from non human source, however, human source contamination with fecal coliform increases significantly in urban settings and this was linked with onsite wastewater treatment systems.

Sampling and analysis carried out by Karpiscak et al. (2006) showed some groundwater samples representative of individual non disinfected groundwater systems in the test area contained fecal coliform and _E. coli_. An organism, _Aeromonas hydrophila_ which is associated with human gastroenteritis was also discovered to be prevalent. This observation is relevant especially in developing countries in the tropics where the organism, _Aeromonas hydrophila_ may fester due to its preference for high temperature. Factors like high water table, improperly constructed infiltration system can permit the travel of bacteria and viruses to the groundwater where they may survive for several days (USEPA, 2002).

Onsite wastewater treatment systems and the associated subsurface wastewater infiltration systems which are not installed in suitable soils have been associated with contamination of shellfish water in coastal areas due to coliform bacteria. This trend is also related to high density of onsite wastewater treatment systems and has led to decline in economy in the area affected (Duda and Cromartie, 1982). Hagedorn (2004) also discovered when he tested non point contamination using a fluorometer, that there is harmful potential of decentralised onsite system when they leach into shellfish beds in estuarine environment. This is because pathogens from the discharge can eventually cause disease in humans when they consume contaminated shellfish.

In 2000, CBS American radio network reported the environmental challenge that Florida Keys Island, which is bordered on by the Gulf of Mexico and the Atlantic on two sides, was undergoing. There was a major threat to the marine ecosystem and the coral reefs were being ruined. The major cause was population explosion with the result that the old and in most cases faulty sewage systems including two-thirds of the onsite systems and cesspits, could no
longer threat effectively the human waste. Subsequently wastes flow to the coral reefs to the marine environment resulting in eutrophication, closure of beaches and depletion of the natural habitat for young fish and crustaceans. Shinn of the USGS (1993) also stated that in addition to natural events, human activities such as nutrient enrichment from treated sewage have led to the demise of coral reefs in this Island.

5.2.1 METHANE PRODUCTION FROM DECENTRALISED ONSITE WASTEWATER TREATMENT AND ENVIRONMENTAL IMPACT

Vastaete et al. (2009) stated that methane gas emitted to the atmosphere from septic tanks contributes an estimated 20 – 40 m³ per person per year. Under the anaerobic condition of septic tanks, wastewater is degraded producing methane and carbon dioxide. The amount of methane produced is dependent on the amount of biodegradable fraction, expressed as BOD or COD in the wastewater. When methane gas is not harnessed, it can contribute to the global warming effect and it is considered to be the most important contributor in the atmosphere to the green house gas effect through wastewater treatment (El Fadel and Massoud, 2001). Methane and other green house gases such as CO₂ and nitrous oxide (N₂O) act by trapping and retaining solar heat causing an increase in the temperature of the atmosphere. On the other hand when in the treatment of wastewater, methane gas is harnessed it can be used as an energy source.

5.3 IMPACT OF MANAGEMENT ON SYSTEM SUSTAINABILITY

The decentralised wastewater treatment is also affected by management problems especially in developing countries. Many of the systems do not employ adequate design criteria. In addition management is usually by owners of property lot or landlords who are mostly uninformed and untrained on the operation and maintenance of these systems. This management approach is based on complaints and failures (USEPA, 2002). Most of the management problems that lead to health and environmental hazards include hydraulic overloading of the system as a result of increased population, neglect to pump and dispose of sludge tanks when full. This leads to overflow of sludge to either stormwater drains and eventually to surface waters or to the infiltration device causing clogging. There should be a system of constant monitoring by appropriate health or environmental authorities and regular system upgrades.

Presently one of the issues confronting the management, operation and sustainability of these decentralised or small wastewater systems include lack of knowledge and public awareness of the value of the systems (Otis, 1999). They are usually perceived as primitive or low tech options which hardly meet effluent standards (Ujang and Hamid, 2006). It is relevant to emphasize the fact that decentralised wastewater treatment usually disperses treated effluent and is important for groundwater recharge and maintenance of baseflow in streams. It is important to also portray the fact that there is less risk associated with system failure compared to the centralized wastewater treatment option. In addition, waste water effluent and sludge should be viewed as a potential resource.

Another challenge to the decentralised wastewater system in most developing countries is the lack of accountability. In most rural and urban centers of Nigeria for example, siting criteria and system performance are not monitored. No government agency is particularly responsible for regular monitoring of the systems to evaluate them. The volumes of these systems also make them difficult to supervise. In addition, in some urban slums, there are no access roads
to these small treatment facilities to dislodge the sludge when they are full. In such cases dislodging is manual and this creates a public health nuisance. Hence the small systems are poorly operated and maintained. Also, some of these onsite-decentralised facilities in Nigeria are used for several years without upgrading nor are there any plans put in place to upgrade them.
In many world cultures wastewater had been seen as a nuisance largely because it possesses some hazards to the human and livestock hygiene, contains organic matter and nutrients that lead to eutrophication e.g. nitrogen and phosphorus, causing problems on natural water courses such as seas, lakes and rivers. However, in many regions of the world fresh water is becoming extremely scarce due to overpopulation, urbanisation, climate change and pollution. The need to conserve this resource is becoming increasingly important. In water scarce arid and semi arid regions, additional sources of water are being sought to supplement the fresh water supply. In addition, strategies are being developed to substitute less restrictive use of fresh water with lower quality water (Hespanol, 1997). The low quality water includes wastewater, drainage and brackish water. Freshwater supply, after it has been used can now be treated and subsequently reused for other purposes. The extent of treatment required for reuse is dependent on the level of pollution and the purpose to which the water will be eventually put. The World Health Organisation in 2006 predicted that the reuse of wastewater in agriculture and aquaculture will increase. There could be nutritional improvement, health and economic benefits associated with the reuse of wastewater in agriculture and aquaculture, as this contributes to significant economic activities, supports the livelihood of the irrigators and families. However good management practices should be adhered to in order to prevent public health impacts (WHO, 2006).

The main consideration of the treatment of wastewater in the conventional centralised approach to wastewater treatment is meeting the stringent requirement for final outfall to the receiving water body. The need to comply with these stringent goals and maintain a safe environment makes it absolutely necessary to apply technologies that are easily adaptable and economically viable. However in many low income countries these highly engineered technologies are not affordable. The main task of wastewater treatment is therefore shifting focus from mere disposal to reuse of wastewater streams. The decentralised wastewater treatment approach is presently adapted to the economic status of the developing countries and has the advantage of reducing the cost involved in extensive pipe network which is the main frame of centralisation of wastewater treatment. In addition, decentralised infrastructure could be properly harnessed to achieve the goal of both purifying wastewater for reuse for agriculture, aquaculture, toilet flushing etc. Also bioenergy can be generated from the organic matter in the decentralised wastewater stream and the system generally will contribute to improvement in the soil integrity, public health and sanitation as well as conserve freshwater resources for drinking water supply (Holler, 2003). Considering also the challenge of the water scare regions of the world, decentralised wastewater treatment option with efficient use and reuse of water could pose a great benefit.

The main object of decentralisation should therefore be the reuse of wastewater stream whilst the planning of wastewater treatment schemes should employ to a large extent a combination of technologies that promote reuse. This will provide a solution to the pressure mounting against water resources in many countries including Nigeria, improve food and fibre production and offer solution to the problem of environmental degradation. Contrarily, the implementation of wastewater reuse will be fraught with obstacles such as economical and social acceptance issues, technical and lack of awareness and non uniformity of standards for reuse but with constant involvement of all stakeholders in the process many of the challenges will be overcome.
In this framework, each country or region or sub communities within the country will be provided with technologies that are easily adapted to the population, geology and the social needs and located close to the community with communal participation and ownership. So inclusive in this approach is the need for long term sustainability of the decentralised wastewater treatment system, protection of both human health and the environment, minimise the waste and loss of already scare resources with the most minimal use of energy. China as an example of a developing country has been making advances in the reuse of waste both human and animal, for increased productivity in food and aquaculture (Rose, 1999).

In Nigeria therefore, there could be more public awareness on the use of wastewater as a resource for agriculture, aquaculture etc. Presently, the country relies heavily on revenue accruing from fossil fuel. In the Niger Delta region of the country; the control of crude oil windfall is a source of major intertribal conflict. In addition, there is environmental degradation associated with the activities of crude oil exploration and refining. The economy could be diversified with more focus on agriculture, aquaculture and bioenergy production. This will also improve the prospect of sustainability of the fossil fuel reserves of the country.

6.1 USE OF WASTEWATER IN AGRICULTURE AND AQUACULTURE

Agriculture is the sector that has the largest consumption of water worldwide especially in the developing world context and irrigated agriculture contributes about 40% of the food production on 17% of the world’s cultivated area (Meinzen-Dick and Appasamy, 2002). Many developing countries have practiced the use of untreated or partially treated wastewater in irrigation of food crops for centuries. However, the number of persons presently dependent on wastewater irrigation is on the increase due to rapidly increasing supply of freshwater which in turn releases larger volumes of wastewater. In addition 10% of the world population consumes crops which are a product of wastewater irrigation (Smit and Nasr, 1992). All of these are an outfall of rapid population growth and awareness on sanitary management. Wastewater use for the irrigation of crops will reduce the need for artificial fertilizers and in some cases totally eliminate it. The nutrients supplied by wastewater will also reduce harmful environmental impacts which are associated with the production of artificial fertilizers (WHO, 2006). Wastewater irrigation has been known to increase crop yield more than freshwater irrigation even when used in combination with chemical fertilizers. For some vegetables, this also reduces the time for crop production (WHO, 2006). This trend makes for preference of wastewater irrigation by farmers and can cut down the poverty level by a great margin.

The water and nutrient components of wastewater can be used beneficially for crop production, hence enhancing sustainability of the environment as the quantity of wastewater especially untreated and partially treated wastewater in developing countries that would have been discharged into the environment and open water bodies becomes reduced. This will in turn reduce disease transmission and environmental degradation. One of the United Nations millennium development goals is to halve by the year 2015, the people without access to water supply and safe and affordable sanitation (UN, 2000). In pursuance of this objective promoting the use of wastewater in agriculture and aquaculture will go a long way in the attainment of this goal. This will place better value on both the water resource and the nutrient being used for productivity and will diminish the mindset that wastewater is a nuisance while also preserving fresh water resource for drinking and domestic use.

In most developing countries, wastewater is used to irrigate agricultural land and the main objective of irrigated farming is income generation and improvement. In these countries
subsistent farmers prefer to use wastewater even if there are alternatives due to the high content of nutrient in the wastewater and for the reason that it is a less expensive option than chemical fertilizers. In low income countries this practice abounds and is unregulated, despite the health and environmental risks involved (Scott et al., 2004). In a research carried out by Cornish and Kielen (2004), it was observed that in some African countries polluted water from shallow groundwater, streams and rivers also polluted to some degree by wastewater from domestic, industrial and municipal sources, are being used in urban and peri-urban irrigation. The use of untreated dilute wastewater is quite a common practice in the sub-Sahara African region. The microbial water quality used for irrigation and the fecal coliform count were also measured and as an indication of biological health risk. This was found to exceed the WHO standards. Thus informal wastewater irrigation poses direct health risk and the wastewater that has been polluted with industrial effluent may also further pollute both soil and groundwater. However, wastewater irrigation has great positive financial capital for the irrigators in the short term but in the long term consideration, the negative impact may far outweigh the benefits of wastewater irrigation especially if left largely unregulated.

The drawbacks to health and the environment resultant of unregulated wastewater irrigation poses a challenge of the requirement of a management approach. This approach will not threaten the livelihood of those benefitting from the practice but will generate appropriate safeguards to manage the human health and environmental risks (Rijbaman and Lebel, 2002). The WHO in 2006 formulated such a guideline which objective is to ensure maximum public health benefit derived from the use of wastewater in irrigated agriculture, increase crop production while minimising negative effect to the environment and public health through less exposure to pathogens and toxic components in the wastewater. The guidelines are expected to be considered on the basis of national environment of the country or region, social, economic and cultural conditions applicable. In line with this, the guideline encourages both positive and negative incentives that will change and improve both behaviour and situations. However, Hoek (2004) on the other hand argues that many developing countries may not be able to meet the level of treatment required before wastewater is suitably applied.

In Vietnam, where freshwater is not scarce, wastewater agricultural irrigation is also widely practised with application in agriculture and aquaculture in 93% of the cities due to consumer acceptance and contribution to food security (Raschid et al., 2004). Aquaculture is the process of growing fish and other aquatic organisms for the purpose of food production. Wastewater aquaculture serves a three-fold function, for flood control, as reservoir sink for city wastewater as well as fisheries. The growth rate of some species of fish e.g. Nile Tilapia have been found to improve in growth with treated wastewater than in their natural habitat while test showed favourable with regards to public health and environmental risks (Khalil and Hussein, 2008).

6.2 MANAGEMENT OF FECAL SLUDGE

The sludge from the septic tanks is collected and disposed of untreated to disposal sites or into drainage ditches or open water bodies in developing countries, which leads to dire public health and environmental effects (EAWAG, 2000). Septic tank sludge or fecal sludge has a higher amount of pathogens than raw sewage by percentage. This is because the primary treatment of wastewater in septic tanks or most onsite decentralised systems tend to accumulate the solids together (USEPA, 2002). In the centralised wastewater treatment approach prevalent in developed countries, wastewater is collected and treated in central treatment works and sludge is treated with technologies which may not be affordable in
developing countries. Fecal sludge could be managed and used for agricultural and land purposes such as on golf courses, gardens, forests. The sludge is a rich store of nutrients like phosphorus and nitrogen which can be recycled and used for plant nutrient. In theory, the fecal sludge from 6 billion people in the world would contain 33% and 22% of the world’s mineral nitrogen and phosphorus need respectively (WHO, 2006).

Aalbers (1999), in his study on resource recovery from fecal sludge stipulates a design BOD loading range of 1.5 kg to 16 kg BOD/ha.d for fecal sludge fed fish pond. Although in Africa, the direct use of fecal sludge for fish production is not wide spread but fish are grown in lakes that have been polluted with faeces or fecal sludge (WHO, 2006). On the other hand, in Asia, human excreta or fecal sludge is used widely in aquaculture. Cities including Malaysia, Hong Kong, China, and Japan, Taiwan and India practice human excreta or fecal sludge fertilised fish ponds. Also due to its simplicity and less loss of nitrogen, it is attractive to apply fecal sludge without treatment; however for the health effects it is advisable to treat fecal sludge.

6.3 TECHNOLOGIES FOR FECAL SLUDGE MANAGEMENT

Handling of fecal sludge is complex because it cannot be discharged directly into the surface water bodies as the pathogenic content is higher than wastewater effluent and it cannot be acceptably disposed off on a landfill due to its high water content. Strauss et al. (2002) stressed the importance to utilise low cost technologies for the effective treatment and management of fecal sludge in developing countries. These technologies will involve very low or no energy cost and no chemicals employed but with high contaminant removal level, easy to operate, utilising mainly local material. The volume of fecal sludge that can be collected may be affected by the storm water infiltration, groundwater level and infiltration, soil absorption and the dislodging practice employed. The technologies suitable for handling of fecal sludge in the study area include composting, vermicomposting, drying beds and land application.

6.3.1 COMPOSTING

Composting can be described as the process whereby organic matter is aerobically decomposed in a controlled environment to produce humus (USEPA, 2003). Fecal sludge from decentralised systems can therefore be pretreated to reduce the moisture content and then composted at a temperature of 55°C in combination with bulking agents like wood chips to increase the porosity of the compost. Composting destroys pathogens from the fecal sludge and can act as a good soil conditioner. Fecal sludge can also be co-composted with domestic organic solid waste. If the solid waste is not sufficient in organic matter, it may affect the quality of the compost produced. However, for an optimal mixture of fecal sludge and solid waste a carbon – nitrogen (C: N) ratio of 30 – 35 and water content of 50 – 60% should be maintained (Strauss et al., 2002). It is important for a thermophilic condition to be maintained that the compost heap be frequently turned for aeration. The process time for composting ranges between 6 weeks to 2 months. When there is co-composting of fecal sludge and solid waste, there is ultimately a reduction of the volume of solid waste that is disposed off in landfills. There are different types of composting method that can be used including windrow, aerated static pile and invessel composting processes.
6.3.2 VERMICOMPOSTING

Vermicomposting of sludge and biosolid is achieved by the combined action of earthworms and microorganisms. There are different species of earthworms in Nigeria such as *Eudrilus eugeniae*, *Hyperodrilus africanus*, *Alma milsoni*, *Libyodrilus violaceus* etc (Dedeke et al., 2010). The species most associated with wastewater treatment are the *Eisenia fetida* and *Eudrilus eugeniae* (Bajsa et al., 2003). Worms can be used to decompose fecal sludge with a reduction in volume. In a study performed by Rodriguz-Canche et al. (2010) earthworms were used to treat fecal sludge from septic tank resulting in reduction of pathogens (fecal coliform, *Salmonella* specie and helminth ova) with production of a biosolid or compost that could be used for agricultural and soil improvement purpose. The experiment with earthworm was carried out under a temperature range of 22 – 36°C. Temperatures above 30°C were surmised to produce some biological activities in the fecal sludge that may have impacted negatively on the development of the earthworms. Hence, before this process is applied, it is necessary to check the specie of earthworm in a particular region or area, their population dynamic and the effect of temperature. Bajsa et al. (2003) associated the highest volatile solids degradation at a carbon to nitrogen content of 25.

In the process of vermicomposting the worms blend mechanically the organic material and at the same time modify the chemical, physical and biological condition. They aid in maintaining aerobic condition in the sludge, take in some of the solids to produce biomass while they give out the digested product or vermicompost, which is of a better quality than conventional compost (Suthra, 2009)

6.3.3 DRYING BEDS

Depending on the consistency of the fecal sludge and the dislodging practices, drying bed can be used for further treatment. The drying bed is basically a sand-gravel filter which operates by evaporation and percolation. A contaminant removal level of 70 - 90% COD, ≥95% SS, 100% helminth eggs and 40- 60% inorganic nitrogen can be achieved in a drying bed (Aalbers, 1999). The removal of organic nitrogen is accomplished by nitrification and ammonia removal while the removal rate of helminth eggs is largely dependent on resident time. For the design of a drying bed, USEPA (2002) recommends an anaerobic sludge loading rate of 100 to 160 kg/m².yr.

6.3.4 LAND APPLICATION

After fecal sludge has been stabilised in the biodigester, drying beds or composted, the resulting biosolid can then be used on land for soil conditioning or as a fertiliser for crop production. The physical, chemical and biological characteristics play a major role in the use of biosolids. The physical properties of the biosolid such as solid and organic matter content will affect the plants growth by limiting the amount of nutrients available to the plant. These properties affect soil structure, soil water content, infiltration and runoff. The chemical properties of the biosolids such as its pH, soluble salts, organic chemicals, trace element, plant nutrient content influence the soils chemical properties and also plant growth. The chemical properties of biosolids depend on the type and extent of treatment received such as lime stabilization, or if they have undergone primary, secondary or tertiary treatment. The addition of lime during stabilization will increase the normal pH of the biosolids from 7 or 8. The major plant nutrients in biosolids are nitrogen, phosphorus and potassium as can be seen in the Table 3 below. Nitrogen is in the organic form in the biosolids and upon soil application,
is converted to ammonium through ammonification. The ammonium nitrogen is available for plant uptake.

The biological composition of the biosolids affect the organic matter content and the amount of microbes inputted in the soil which will subsequently affect human health and environment. Microbes such as bacteria, fungi, protozoa that are present in the biosolids during decomposition contribute to enhancement of the soil biota. The organic matter content of the biosolids is important because it increases soil aggregation, exchange capacity for cation which is relevant for plant nutrient uptake, improves hydraulic conductivity of the soil and the organic carbon content of the soil. Pathogens on the other hand can survive in primarily treated wastewater but with stabilization, there is more chance of die-off. The factors that affect pathogenic survival from biosolids to soil are water content, pH, temperature and organic matter content, soil permeability and sunlight.

Low solid or liquid biosolids are better applied by injection, the dewatered or semi-solids are better placed by surface spreading or plowing while the high solids such as compost are used more like fertilizers. Anaerobically digested biosolids have been reported to improve crop production such as maize, sorghum, cotton, soybeans among others (Epstein, 2003).

**Table 3: Median concentration of several macronutrients- adapted (Epstein, 2003)**

<table>
<thead>
<tr>
<th>Nutrient</th>
<th>Type of Biosolid</th>
<th>Anaerobic</th>
<th>Aerobic</th>
</tr>
</thead>
<tbody>
<tr>
<td>%N</td>
<td></td>
<td>4.2</td>
<td>4.8</td>
</tr>
<tr>
<td>%P</td>
<td></td>
<td>3.0</td>
<td>2.7</td>
</tr>
<tr>
<td>%P</td>
<td></td>
<td>0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>%Ca</td>
<td></td>
<td>4.9</td>
<td>3.0</td>
</tr>
<tr>
<td>%Mg</td>
<td></td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>%Fe</td>
<td></td>
<td>1.2</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**6.4 USE OF URINE**

Urine can be separated at source from urine separation toilets and used as a nutrient in Nigeria. The amount of nutrients contained in excreted urine depends on the diet of the individual, water intake, level of activity and climatic condition. Urine is rich in nitrogen and when treated, it can be used effectively for plant nutrient as a fertilizer. Urine has been used successfully as fertilizer to produce crops such as leeks, barley and spinach. The amount of nitrogen produced in the urine of a person per year is between 30 – 70 kg and can be used for fertilizer on an agricultural land of about 300 – 400 m² (Jönsson et al., 2004). It can be applied either diluted or undiluted but advisably close to the soil as soon as it is produced to prevent smells, burns to the foliage and loss of ammonia. During storage of urine, there may be production of sludge due to the conversion of urea to ammonium with the accumulation of the enzyme ureas as can be seen in the equation 5 below. When ammonium is produced, then the pH of the urine during storage increases to about 9 at which the magnesium, calcium, phosphate now form precipitates. The major risk to the use of urine however is fecal contamination.

**Equation 5**

\[
\text{CO(NH}_2\text{)}_2 + 3 \text{H}_2\text{O} \rightarrow 2\text{NH}_4^+ + \text{OH}^- + \text{HCO}_3^-
\]

\text{urea} \quad \text{water} \quad \text{urease} \quad \text{ammonium} \quad \text{hydroxide} \quad \text{carbonate}
Ammonium produced can be used directly as plant nutrient. The phosphorus in urine is discharged as phosphate ions which are readily taken by plants. Urine as a fertilizer can compare favourably with urea and ammonium based chemical fertilizers. During application of urine as a fertilizer, soil incorporation rather than surface application is preferable to minimize volatization of ammonia. During storage, ventilation should not be provided to avoid smells and since urine is corrosive, it should be stored in resistant vessel, such as plastics. The major issue to the reuse of urine is the social acceptability factor especially in Northern Nigeria where the idea of handling of urine may be met with resistance based on religious grounds.

6.5 TREATMENT AND REUSE OF GREYWATER

Greywater is the wastewater that is produced from domestic activities such as laundry, sinks, hand basin, bathing and kitchen. Presently, in the study area, greywater is untreated and uncombined with blackwater. It is however, discharged in both open and closed drains which end up in the nearby Anwai Stream and River Niger or allowed to pool behind homes until it evaporates or percolates. This makes for breeding grounds for mosquitoes and other disease agents. The composition of greywater will depend on the availability of water, living standards, size of family, use of detergent and eating habits. The volume of greywater produced depends largely on the availability of water and in developing countries, it does not exceed 100 l/p.d on the maximum (WHO, 2006). Greywater, because it has been used can cause both health and environmental impacts although the concentration of contaminants especially plant nutrients and pathogen is lower than in wastewater and fecal sludge. The use of greywater comes from the need mainly to conserve freshwater reserve but in developed countries the volume of wastewater treated in the treatment plants needs to be reduced. In the reuse of greywater, its direct use without storage is recommended because storage leads to change in the quality and increases the growth of health related micro-organisms (Dixon and Fewkes, 2007). Also there is the potential for viral infection from reuse of greywater especially where there is exposure either by direct contact through bathing or indirect contact during storage or irrigation from greywater produced from infected persons. The indicator organism for pathogenic contamination is the fecal coliform (FC). Fecal contamination of greywater is associated with washing clothing items that are fecally contaminated such as for childcare, anal cleansing and showering.

The use of greywater close to the source of generation is an attractive idea. It can produce a steady source of water supply for home garden and improve food production. It can also afford year round growth of vegetables. However, the use of greywater in the developing countries is highly unregulated. Hence, to reduce soil salinization as a result of salt build up from use of greywater for irrigation, it is necessary to avoid hard soaps containing calcium but preferable is the use of soft soaps containing potassium. It is also recommended that greywater be applied subsurface to prevent contamination by direct contact. The pollutants common in greywater are COD, BOD, SS, surfactants (present in household chemicals and personal care products, phosphates from detergents, nitrogen, fecal contaminants from hand washing and diaper change (Winward et al., 2008, Widiastuti et al., 2008). The compositions of contaminants are largely dependent on different practices such as the use of phosphorus containing detergent, use of shower oils, shampoos, grease and cooking oils. Where phosphate free detergent is used the phosphorus content of greywater reduces and BOD reduction also occurs where there is less use of cooking oils.
Greywater account for about 50 - 80% of domestic wastewater, while 20 – 30% of potable water supply to household is used for toilet flushing (Widiastuti et al., 2008). If harnessed, greywater could contribute to relieving the water shortage problem the study area currently faces. If options of source for water supply are diversified, then dependency on one source; potable water, will be reduced. Studies on greywater reuse have shown freshwater savings from 30 – 40% (Ghisi and Oliveira, 2007, Mah et al., 2009). Instead of once-off use of water and then discharge into the drains, simple treatment such as gravel filters for soil infiltration, constructed wetlands or ponds may be applied. Greywater can be treated, then be combined with rainwater and recycled for non potable water use such as toilet flushing, gardening, carwash, artificial groundwater recharge etc, hence reducing the health hazards associated with greywater. To meet health based targets of 200 or 1000 FC/100 ml as stipulated by WHO (2006) less than 10 mg/l BOD and less than 2 NTU for turbidity (USEPA, 2004) for the reuse of greywater (or reclaimed water) especially for irrigation, such simple decentralised treatment techniques should be applied.

Different methods have been used in the treatment of greywater such as Ecological Sanitation “Ecosan” in Malaysia (Mah et al., 2009), combined sedimentation/sand filtration, aeration and disinfection (Godfrey et al., 2009) in India, use of natural zeolites (Widiastuti et al., 2008), disinfection (chlorine, hydrogen peroxide etc), constructed wetland (Dallas and Ho, 2005) and absorption of organics with powdered activated carbon, PAC combined with solar photocatalytic oxidation (Gulyas et al., 2009). The treatment chosen depends on the quantity, quality and the intended reuse of the greywater. The option of treatment chosen for greywater also, should be able to effectively take care of the organics and pathogenic loadings in the influent and subsequently produce effluents that can be consistently reused safely in line with the appropriate standards (Pidou et al., 2008, Winward et al., 2008).

6.5.1 CONSTRUCTED WETLAND FOR GREYWATER TREATMENT

For choice of biological treatment of greywater, the high COD/BOD ratio should be considered, especially with the low availability of nutrients which may restrict the performance. The BOD and COD of raw wastewater collected from baths, showers, and hand basins were in the range 129 – 155 mg/l and 246 – 587 mg/l respectively (Jefferson et al., 2004) while Winward et al., 2008 reported a low BOD of 20 mg/l and COD of 86 mg/l in raw greywater which correspond to high strength and low strength greywater respectively. Constructed wetland will be a suitable choice for the treatment of greywater in the study area because of ease of operation. It is important in the design of a constructed wetland to consider such factors as the level of required treatment, the objective as regards social aesthetics, land availability and the site topography (Wood, 1995). The amount of rainfall and runoff in the study area, Asaba, in the design of the wetland is relevant especially if it is a surface flow wetland as this may affect the volume of water in the storage. The wetland may be subsurface, surface or a combination of both, in order to combine both aerobic and anaerobic pollutant removal optimally. The vegetated wetland employs evapotranspiration and the plant roots support microbial growth as they provide a large surface area in addition to enhancing environmental aesthetics (Konnerup et al., 2009). Vegetated constructed wetland with crushed rock media achieves high BOD removal of less than 10 mg/l but insufficient fecal coliform contaminant removal (Dallas and Ho, 2005). Although the specification for fecal coliform and other pathogens by USEPA is non-detectable level for reuse of greywater, vegetated wetland can be used as pretreatment which can be followed by disinfection using chlorine.
Shrestha et al., 2001 and Paulo et al., 2005 on the other hand designed and studied a combined surface flow and subsurface flow wetlands for greywater treatment and obtained removal efficiencies of 95%, 35% 97%, 88%, 97%, 58%, 82% and 98% respectively for Turbidity, TS, TSS, COD, BOD, total phosphate, total nitrogen and total coliforms. Although, there was low removal of TS, however, there was practically no SS in the effluent showing a high dissolved solid fraction which may be related to the COD. For ease of treatment, the greywater from the study area can be separated in terms of kitchen and laundry wastewater (dark greywater) from greywater collected from bathrooms and other non-toilet greywater. This is because dark greywater is usually more polluted with cooking oils, food particles, and detergents. Otherwise grease traps can be constructed for the kitchen or dark greywater (Shrestha et al., 2001). Moreover, in the study area, washing machines are not being used. The effluent from the wetland can then be chlorinated and recycled to homes for non potable uses.

### 6.5.2 CHLORINATION OF GREYWATER

Winward et al. (2008) studied the effect of chlorination of greywater for reuse and observed that it is necessary to have a biological treatment or filtration stage to eliminate particle agglomeration of organics or suspended solids with fecal coliform and other micro-organisms before appropriate initial chlorine disinfection. It is important for particles in the greywater to be removed before chlorination as this prevents particles from protecting the pathogens from inactivation. The dose of chlorine will be dependent on the max particle size of the materials in the greywater, temperature, pH and contact time. 8 - 15 mg/l is considered adequate with a contact time of 30 min – 2 hours. However, a free or residual chlorine concentration of 0.5 to 1 mg/l should be maintained (Hammer and Hammer, 2004, Winward et al., 2008). The tank design that allows effective contact and dispersion is long and narrow in geometry permitting plug flow condition.

### 6.6 RAINWATER POTENTIAL IN STUDY AREA

Rainwater harvesting has great potential in the area as the annual rainfall in the study area is about 2200 mm (FRN, 2003). Presently, the rainwater collection is not harnessed on a communal level although individual collection is done as the need arises. Conscious, consistent and communal rainwater harvesting effort and treatment could enhance water sufficiency and reduce the dependence on state owned water delivery. To get the rainwater volume that can be collected or harvested from the roof in the study area, it is important to know the roof area. To calculate the potential rainwater harvest in the study area, the rational method, Equation 6 below can be used.

**Equation 6**

\[
\text{Discharge, } q = ClA
\]

*where C is the runoff coefficient, I is the rainfall intensity and A is the roof area*

Rainwater can be collected, stored above ground in individual buildings, treated minimally since rainwater is not as polluted as greywater. The minimum to mean SS, COD, BOD, and *E coli* are respectively, 6 – 32 mg/l, 11 – 14 mg/l, 6.4 – 10 mg/l and 1060/100 mL of roof and road surface harvested rain water (Nolde, 2007, Zhai Xue-dong et al., 2009). Although Mikelsen, (1993) reported that roof harvested rainwater has 5 – 50 mgSS/L with no COD
content. Simple treatment such as filtration and use of disinfection can be applied to roof harvested rain water (Nolde, 2007). However for the study area if individual water storage tanks are applied for rain collection alum which is a crystal of hydrated potassium aluminum sulphate can be used for treatment. Alum is readily available, easy to handle and acts as a coagulant that can remove suspended solids. The amount of alum that may be required for use is dependent on the volume of rainwater to be treated. The coagulant can then be removed by filtration.

6.7 BIOENERGY FROM WASTE

Bioenergy is basically energy from the sun stored in organic material including human waste streams, plant cellulose, lignin and animal fat. When biomass which is human, plants and animal wastes is degraded, there is breakdown of the sugars with the release of energy, carbon dioxide and water. These by-products when harnessed can create power or bioenergy. The outcry concerning the environmental impacts including the threat posed by global warming resultant of use of fossil fuel for energy production and the fact that this energy source is finite has led to a global fight to prevent climate change and the awareness of bioenergy as a potential source of energy for tackling this change (Akinbami et al., 2001). Biodegradation of municipal solid waste in landfills and burning of fossil fuels for the production of energy as is currently done in Nigeria and other parts of the world, release a high amount of carbon dioxide and other greenhouse gases but the use of biofuels to replace fossil fuels will contribute to the overall reduction of carbon dioxide release into the atmosphere hence helping to reduce global warming. The European Environmental Agency has reported that 19 million tons equivalent of oil (toe) in biomass will be available by 2020 for the production of bioenergy, 46% of this would be derived from biowaste (Marshall, 2007). Biogas on the other hand can be harnessed from different sources of biodegradable wastes including fecal sludge generated in decentralised sector, communal solid waste, agricultural waste or animal waste through the use of an anaerobic digester.

The production of biogas can be accomplished when the biowaste is digested anaerobically with the production of methane gas, carbon dioxide, and other minor gases. About 50 – 70% of the product is methane gas which can be harnessed to produce both electrical energy and transport fuel (Marshall, 2007). Akinbami et al. (2001) stated that 357 – 60,952 tons of CO₂ emissions will be avoided per year if biogas were to replace kerosene use in Nigeria.

6.7.1 ANAEROBIC DIGESTER AND DESIGN CONSIDERATIONS

An anaerobic digester is a tank that is sealed, possibly heated and provides an environment suitable for anaerobic bacteria which occur in nature to grow, multiply and convert organic matter to biogas while producing a low odour effluent. One major objective of anaerobic digestion is to transform the bulky organic matter in the form of wastewater, municipal solid waste etc which are usually odorous to a more inert material with the advantage of production of energy. In the design of a digester suitable for the degradation and stabilisation of domestic wastewater and municipal solid waste for the production of biogas, the type of waste, waste generation rate, local environmental condition and ambient temperature have to be considered (Igoni et al., 2008). The objectives of selecting a particular design for anaerobic digestion are to maximise the use up of the volatile solids while increasing the methane yield. It is also important to increase the rate of conversion, achieve a stable system with the most minimal energy input (Wilkie, 2005).
The biogas digester function optimally in tropical climate due to high temperatures and it will be suitable for the study area, Asaba, Nigeria. The high temperature regime will aid in the die-off of pathogens in the wastewater and increase production of biogas. Anaerobic/biogas digesters are used for production of electricity, also for cooking and lighting, to run water pump engines, agricultural machineries and can produce fertilisers in addition to sanitary enhancement. It is possible to produce biogas small to medium scale of about 100 m$^3$ or more. Although there has not been much awareness of this energy technology in Nigeria, researches are underway while countries like Tanzania, Zimbabwe, Kenya have had some practice on the technology having as many as a hundred or more 100 m$^3$ biogas digester as at 1993 (Akinbami et al., 2001).

### 6.7.2 Anaerobic Digester Types

There are different types of anaerobic digesters in use presently and can be differentiated based on operational processes, such as batch or continuous, mesophillic or thermophillic depending on the temperature, high or low solids, based on the solid content and single stage or multistage based on the operational stages or complexity.

The batch anaerobic digester is fed the substrates in batches, that is, one digestion process is completed undisturbed for a specified HRT before another batch is introduced. This process is simple, cheap but biogas production prediction will follow a normal distribution pattern over time. In the continuous anaerobic digestion processes on the other hand, the substrate is added continuously or in stages in the digester while the digestate is removed constantly with constant production of biogas e.g. the continuous stirred tank reactor (CSTR), Upflow Anaerobic Sludge Blanket (UASB). Thermophilic anaerobic digesters operate in temperature range of 44 – 57°C while mesophilic anaerobic digesters operate within a temperature range 30 – 38°C. Low solid and high solid anaerobic digesters depend on the Total Solids (TS) input. The advantage of low solids digester is the ease with which the substrate and bacteria intermix, enabling higher biogas production. In addition, there is ease of flow, hence less energy is required. On the other hand due to thicker substrate consistency of the high solid digester more energy input is required for its movement and processing. However, the low solid digesters often require more volume due to high water content.

Hammer and Hammer (2004) and Wilkie (2005) described the single stage digester or two phase digester based on their operation and solid content. In single stage, all the reactions take place in just one sealed tank. The single stage digester is of two types: the complete mix anaerobic digester or the unmixed one-stage anaerobic digester. The complete mix digester has its content stirred mostly intermittently through effluent or biogas recirculation or mechanically. It operates with a solid content of between 3 – 10%. Digesters that are high rate are usually completely mixed with no separation of components. After digestion; the content of the digester is dewatered. While the latter (unmixed) performs the process of volatile solid digestion, gravity thickening and storage. The sludge from the mix feed separates or stratifies with a scum layer on top, middle layer of supernatant and a bottom layer of sludge which is actively digested. An example of the one staged digester is the anaerobic lagoon operating with ambient temperature with a solid content of less than 2%. It is an impoundment with a cover which is gas tight and the gas is collected within the cover. Some single phased digesters are not completely mixed and in this case there is allowance for drainage of about two-thirds of the supernatant and removal of digested sludge for mechanical dewatering.
The two staged or multistage anaerobic digester uses two or more digesters in series to separate the process of biological stabilisation of the mix-feed sludge from the process of gravity thickening, digestion and storage. The first digester tank is completely mixed and heated for an optimal bacterial decomposition while the second digester tank is for gravity thickening and storage of digested sludge.

6.7.3 OPERATIONAL CONDITIONS FOR ANAEROBIC DIGESTERS: TEMPERATURE

Temperature is important in the anaerobic digestion process and affects the production of gases. The decomposition process also is temperature dependent. Both processes increase with an increase in temperature to an optimal level where further increase of temperature renders the process unstable. The optimal ranges of temperature for successful digestion are either mesophilic, that is, between 30 – 38°C or 44 – 57°C which is thermophilic (Igoni et al., 2008). Thermophilic digestion has a higher metabolic rate than the mesophilic digestion, however, there is high death rate of the thermophilic bacteria, and it is less stable and more susceptible to variation in temperature fluctuation (Wu et al., 2006). They also noted that the net energy production from thermophilic digestion was less in comparison with that of mesophilic temperature range as there is initial high energy requirement for the start up in attaining the high temperature required for the thermophilic digestion process. Chae et al. (2008) tested the ultimate methane yield of a biogas digester at various mesophilic temperatures and concluded that the 35°C mesophilic temperature gave the optimal methane production. This is particularly important for the design of biogas digester in the study area, Asaba which falls within this temperature range. This will reduce the cost of energy application for biogas production. Tchobanoglous et al. (2003) also stated that temperature approximately 20°C (psychrophilic) is not ideal for anaerobic digestion as it limits the degradation of long chain fatty acids which causes foaming in the reactor hindering the overall digestion process.

6.7.4 HYDROGEN ION CONCENTRATION (pH)

In a controlled environment like the anaerobic biogas digester, the conditions that encourage the action of both acid and methane producing bacteria include anaerobic conditions, mesophilic temperature, a pH between 6.6 to 7.6, consistent supply of organic matter and flowable liquid in the wet process with total solid content TS of about 10%. The hydrogen ion concentration is important for the process of anaerobic digestion of organic matter as high acidity leads to inhibition of the process. Igoni et al. (2008) put the desired pH between 6-8 with the optimal at 7. The first phase of anaerobic digestion resulting in the production of organic acids lowers the pH. In the final phase that leads to the production of methane the bacteria use the organic acids to form CO₂ and methane. The CO₂ is soluble in water forming HCO₃⁻ ions which buffers again the pH stabilising the process of anaerobic digestion. If there is a tilt in the digestion where the acid forming phase exceeds the methane producing phase, there is imbalance and the process is impeded. This can be corrected by the use of lime or sodium bicarbonate (Igoni et al., 2008)

6.7.5 RETENTION TIME

The amount of time the substrate spends in the digester is the hydraulic retention time (HRT). The factors that determine the choice of an appropriate retention time include the digestion phases, amount and type of substrate, temperature, the eventual use of the digestate and whether it is wet or dry process. Retention time is proportional to the size of the digester, the
longer the retention time, the bigger the digester volume needed and the less cost effective it is and vice versa. However, the longer the retention time the more complete the degradation of organic matter until an optimum is attained then a decrease in degradation of organic matter and biogas production set in. The degradation of organic matter is measured by the BOD or COD content of the digester effluent. It is possible to design a lower retention time with optimum degradation and one way to achieved this is by minimal digester mixing (Stroot et al., 2001 and Ostrem, 2004). The retention time can be represented mathematically by Equation 7.

\[ HRT = \frac{\text{volume}}{\text{daily flow}} \]

**Equation 7**

### 6.7.6 ORGANIC LOADING RATE (OLR)

The organic loading rate is the measure of the volatile solids input to the digester and it is the ratio of the waste concentration to the hydraulic retention time as shown in Equation 8.

\[ OLR = \frac{\text{Influent Concentration}}{HRT} \]

With increased input of volatile solids into the digester, there is increased bacterial demand which may not be met especially in the acidogenesis digestion phase. The outcome is production of excess organic acids which the system may not be able to handle due to less production of methanogenic organisms and acidic pH which leads to less production of methane and system inefficiency (Ostrem, 2004).

### 6.7.7 CO-DIGESTION

Co-digestion is the process of simultaneously digesting a homogenous mixture of two substrates or more in ratios (Anhuradha et al., 2007). Apart from wastewater, biogas can be produced from other feedstock including landfill waste, municipal solid waste (MSW) and other biodegradable wastes. From several studies, it has been observed that co-digestion of wastewater and other organic matter such as municipal solid waste, vegetable market waste, other agricultural plant and animal wastes have increased biogas production than just the use of wastewater (Holler, 2003, Anhuradha et al., 2007, Davidsson et al., 2007, Zupancic et al., 2008). Using different substrates improves the biogas production due to the synergistic effect of the substrates; hence the composition of the substrates in terms of its biodegradability is of importance. Another benefit associated with codigestion of different substrates is cost efficiency because one digester is used instead of a digester for each substrate. In addition there is stability of the digestate produced after the digestion.

In the study area, there is the possibility of using settled fecal sludge from sedimentation tanks, municipal solid wastes, vegetable and fruit wastes, used cooking oil, grease trap wastes from meat processing etc. Nigeria is estimated to produce 20kg of municipal solid waste per person per year (Akinbami et al., 2001). Municipal solid waste is a mixture of both organic and inorganic materials which have to be pre-treated by sorting in order to separate the organic part. The organic fraction of the municipal solid waste OFMSW is the fraction that
can be decomposed biologically which includes food, animal and garden wastes. The solid waste also has to undergo reduction in size before co-digestion. For effective pre-treatment, communal awareness can be created on the need for source separation in order to get the organic part for co-digestion. Hamzawi et al. (1998) evaluated the technical feasibility of anaerobic co-digestion using biological activity tests and identified an optimal mixture of 25% organic fraction of municipal solid waste and 75% sewage sludge for biogas production. In addition, it was discovered by laboratory testing that several pre-treatment such as chemical, thermal, biological and mechanical increased the biodegradability of the substrates.

6.7.8 PREDICTING BIOGAS GENERATION FROM AN ANAEROBIC DIGESTER.

Biogas yield was predicted from a study undertaken by Ojolo et al. (2008) on the organic fraction of municipal solid waste (OFMSW). The average biogas produced from their study was 5.15 dm$^3$/kgTS to 5.83 dm$^3$/kgTS. A regression model was developed from every substrate loading experimented upon versus the corresponding retention times and used as the predictive model. The optimal retention time was observed to be 23 days. Also, Gillberg et al. (2003) predicted a production of 0.03 m$^3$ per person per day of digester gas for a wastewater substrate. Anhuradha et al. (2007) in their kinetic study and anaerobic co-digestion of vegetable waste and sewage sludge showed that although the vegetable wastes contained more bio-degradable matter and hence higher biogas yield than the sewage sludge but co-digestion of both improved the biogas production. The specific biogas yield for co-digestion of the two substrates was 0.68 L/gVS and the HRT was 15 days. Stroot et al. (2001) also obtained specific gas yield of 0.49 lbiogas/gVSadded/day for a retention time of 17 days with high organic loading rate with minimal mixing for two substrates; wastewater sludge and organic fraction of municipal solid waste. Zupancic et al. (2008) in their full scale study of codigestion of sewage sludge and organic fraction of municipal solid waste used an organic loading rate of 1 kgm$^{-3}$d$^{-1}$of volatile suspended solid, also obtained 0.89 m$^3$ biogas/kgVSS added. Davidsson et al. (2007) predicted a methane production potential from sewage sludge of approximately 330 – 400 m$^3$/tVSadded. Finally, Singh et al. (2010) predicted the biogas production from the organic fraction of municipal solid waste to be 135 m$^3$ per tonne digested with methane fraction of 65%. The percentage of methane in the production of biogas from co-digestion of different substrates are in the range of 58 – 66% (Loustarinen et al., 2009) for sewage sludge and grease trap from meat processing, 64 – 69% (Davidsson et al., 2007) for sewage sludge and organic fraction of municipal solid waste and 65% (Anhuradha et al., 2007) for sewage sludge and mixed vegetable waste. The summary of biogas production from different substrates can be seen in Table 4 below.

### Table 4: Summary of biogas and methane potential from different substrates.

<table>
<thead>
<tr>
<th>Substrate Type</th>
<th>Biogas Potential</th>
<th>Methane %</th>
<th>Reference</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>OFMSW</td>
<td>0.15m$^3$/kgVS</td>
<td>65</td>
<td>Singh et al. (2010)</td>
<td></td>
</tr>
<tr>
<td>Wastewater sludge</td>
<td>0.03m$^3$/p.d</td>
<td>0.27 m$^3$/kgVS</td>
<td>Gillberg et al. (2003)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>65</td>
<td>Davidsson et al. (2007)</td>
<td></td>
</tr>
<tr>
<td>Wastewater sludge + Vegetable waste</td>
<td>0.68 m$^3$/kgVS</td>
<td></td>
<td>Anhuradha et al., 2007</td>
<td>15 days HRT</td>
</tr>
<tr>
<td>OFMSW + wastewater</td>
<td>0.49m$^3$/kg VS</td>
<td>0.89m$^3$/kgVS</td>
<td>Stroot et al. (2001)</td>
<td>17 days HRT</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td></td>
<td>Zupancic et al. (2008)</td>
<td></td>
</tr>
<tr>
<td>Grease trap + wastewater sludge</td>
<td></td>
<td>64 – 69</td>
<td>Davidsson et al. (2007)</td>
<td></td>
</tr>
</tbody>
</table>
However, the Ken and Hashimoto model given by the Equations 9 and 10 can be used to predict theoretically the methane yields and efficiency of the digester if all the variables are known.

**Equation 9**

\[
\frac{S}{S_0} = \frac{K}{J_f m(1+K)}
\]

**Equation 10**

\[
CH_4 = \frac{B_o S_o}{J} \left(1 - \frac{K}{J_f m(1+K)}\right)
\]

where

- \(B_o\) = ultimate methane yield in L/gVS
- \(S\) = effluent volatile solid concentration of substrate in g/l
- \(S_o\) = influent volatile solid concentration of substrate in g/l
- \(J\) = Retention time in days
- \(J_m\) = Minimum Retention time in days
- \(K\) = Kinetic Constant (-)
- \(CH_4\) = volumetric methane yield (\(r_{CH_4}\))

6.7.9 USE OF BIOGAS OR METHANE IN STUDY AREA

The most important use to which the biogas produced in the study area can be put is for the generation of electric power because of the incessant power outages from the national electric power grid distributed by the Power Holding Company of Nigeria (PHCN). Tricase and Lombardo (2009) stated that it is possible for 90% of the biogas generated to be used especially with cogeneration: 30% for heating while 60% for electricity. Two-thirds of the biogas generated nowadays in the EU is used for electricity production while one-third is used for heating. They converted by equivalence that 1 m³ of biogas will produce 3.8KWh of heat. Davidsson et al. (2007) gave the energy content of methane as 10.4KWh per cubic metre. The formulae in the Equations 11 and 12 below represent these:

**Equation 11**

\[
Electricity (KWh/yr) = \text{biogas (m}^3/\text{yr}) \times 10.4(\text{KWh/m}^3)
\]

**Equation 12**

\[
Heat \ (KWh/yr) = \text{biogas (m}^3/\text{yr}) \times 3.8(\text{KWh/m}^3)
\]

A study carried out in 2005 by the World Bank reported by the EEDRB (Energy and Environmental Data Reference Bank put per person consumption of electric power in Nigeria at 68.17 KWh. Although this amount compared to per person consumption in Sweden (14,741 KWh) is very minimal. However, in the study area, heating is of no great relevance but cooling, as it is in the tropics and temperature is over 30°C. The more important need for energy in the study area is for cooking. Biogas can therefore be purified for methane production and supplied to the community and can be used efficiently in low gas burners for cooking and other thermal needs of the household.
6.8 UPFLOW ANAEROBIC SLUDGE BLANKET (UASB) REACTOR

The UASB reactor is particularly suited to the study area due to its large population size and low land requirement. It is preferably designed for communal use basically for domestic wastewater or blackwater. The UASB will act as a biodigester septic tank, because it applies both the settling of wastewater sludge as a septic tank and it is constructed to capture the biogas that is generated from the digestion of the sludge. In the design of the UASB, the influent pipes are placed at the ground level of the reactor unit. Wastewater is then distributed in an upflow mode in the reactor. Baffles and a gas separator are placed at the sides and top of the unit respectively to separate the sludge and water phase from the biogas. At the top of the reactor is an effluent withdrawal arrangement. The main principle of the UASB is the active sludge bed or blanket at the bottom of the tank where bacteria within the sludge act on the wastewater degrading it. Then granules which are small in size form while the surface area of the granules are covered with bacteria which have aggregated. The bacteria can spontaneously aggregate together to form into dense and compact granules which have good settling properties and act as biofilms. This active granular sludge blanket can act as filters which then prevent the wastes coming in from escape but allows only the liquid portion. In the sludge blanket, anaerobic digestion occurs and biogas is produced. The biogas produced which is a mixture of methane and carbon dioxide produces a mix in the reactor content. An improvement to the UASB will be a settling compartment that is enlarged, which will increase the sludge volume in the reactor, the more the active sludge the more effective the operation of the UASB in the waste digestion and production of biogas. The Figure 8 below shows a schematic representation of the Upflow Anaerobic Sludge Blanket reactor.

Figure 8: Schematic representation of Upflow Anaerobic Sludge Blanket (UASB); Source UNEP-IHE 2004
A major advantage of the UASB reactor or septic tank is low HRT required for digestion which will save time, space and cost of building large volume or tanks for both separation and digestion. Another advantage of the UASB over the conventional septic tank is improvement in the physical removal of suspended particles and biological degradation provided by the upflow mode and the liquid, solid and gas phase separation as against the horizontal flow mode as is obtained in the septic tank, Luostarinen et al.(2007). Although some operational drawbacks in the use of the UASB have been documented by Chernicharo et al. (2009). These include blockage of inlet pipes, scum accumulation, corrosion and gas leakage. These can be taken care of by use of sieves, pressurized scum removal, use of materials in the gas, liquid, solid separator interface that help check corrosion such as fiberglass respectively.

The production rate of biogas for the UASB is 0.35 m$^3$ methane/kgCOD removed (UNEP – CEHI 2004). The COD removal in a UASB treating domestic wastewater is given as 70 – 90% (Luostarinen et al., 2007, Diamantis and Aivasidis, 2009). This depends on temperature, upflow velocity and HRT. Although not much literature is available on the UASB as compared with the anaerobic digester in treatment of domestic sewage for biogas production, there are few documented ways in which contaminant removal can be enhanced. These include co-digestion, reducing start up time by use of inoculum and coagulants e.g. water extract of Moringa oleifera seeds or incorporation of two tanks (Lin et al., 2000, Kalogo et al., 2001. Akila and Chandra, 2007, Diamantis and Aivasidis, 2009). These methods will result in both high substrate degradation and improvement of the overall system efficiency and ultimately boost biogas production.
7. CHOICE AND DESIGN OF APPROPRIATE TECHNOLOGY FOR EZENEI, ASABA

In the study area, Asaba, Nigeria, individual housing unit own a septic tank or two depending on the size of building and space available. An approach to improving decentralised wastewater treatment in this area will be the construction of a wholesome system with the production of biogas or renewable energy at the centre.

7.1 COMPONENTS FOR IMPROVED DECENTRALISED WASTEWATER TREATMENT

The treatment for domestic wastewater will combine both blackwater and greywater. The components of the improved decentralised wastewater treatment proposed will include:

- Grease trap for kitchen dark greywater
- Sedimentation units
- Intermittent sand filter
- Constructed wetland in series
- Anaerobic digester
- Biogas storage
- Gas electricity generator
- Composting unit
- Compost storage
- Rainwater treatment and storage

7.2 PROCESS DESCRIPTION

Wastewater is collected from the houses in the community, piped to the sedimentation tanks where the sludge settles. The sedimentation tanks are connected in series and the second tank is compartmentalised allowing for more effective sedimentation. This tank acts as an overflow tank and will be useful also for emergency or shock load and maintenance. The settled sludge is withdrawn from the tank bottom to the anaerobic digester which is also fed with organic fraction of municipal solid waste. The resulting biosolid is further treated in an in-vessel composting unit to produce compost that is easy to handle and useful for land application for agriculture or soil amendment. The effluent from the sedimentation tanks can be further treated in an intermittent sand filter. Vegetated subsurface constructed wetland connected in series receive the secondary effluent treating it to standard for final discharge to groundwater. Figure 9 below shows a schematic representation of the proposed treatment.
Figure 9: Schematic Representation of Ezenei Wastewater Treatment with Biogas Production

GW = Groundwater
ISF = Intermittent sand filter
HF VFS = Horizontal flow and vertical flow subsurface
CW = Constructed wetland
OFMSW = Organic Fraction of Municipal solid waste
In the production of biogas, the first step is to separate the solid fraction of the wastewater. This can be done in a sedimentation unit where all the communal wastewater is directed. There is combination of black and greywater although the dark grey wastewater from kitchen can be channeled through a grease trap (consisting of an external vented flow device and an interceptor) in order to remove fats, oils and grease wastes from cooking before entering the basin. In the design of this unit, factors including volume, geometry, soil and hydrological conditions of the site, available space, and final use of wastewater effluent are to be considered. The essence of the sedimentation unit is to enhance sludge settling and thickening in order to increase the sludge concentration but the resultant sludge should be within pumpable limit. The semi solid material from the separation process is transferred directly to an anaerobic digester to produce biogas.

The study area as has been delineated in chapter two is Ezenei Quarter, Asaba, Nigeria. The area has about 92 buildings with an average of 18 persons per building. The population of the delineated area will be about 1656 persons. For design purposes the flow should be about 200 m$^3$/d. This design flow is obtained from the daily water use per person of 120 l/p.d (Burubai et al., 2007). The required retention time and sizing of sedimentation tanks can be calculated from the Equations 13 and 14 (Hammer and Hammer, 2004).

**Equation 13**

\[ Q = V_o A = V / t \]

**Equation 14**

\[ t = \frac{24V}{Q} = \frac{24H}{V_0} \]

where \( V_o = \) surface loading or overflow rate \( (m^3/m^2.d) \)

- \( Q = \) average daily flow \( (m^3/d) \)
- \( A = \) Area \( (m^2) \)
- \( V = \) tank volume \( (m^3) \)
- \( t = \) retention time \( (hrs) \)
- 24 = number of hours per day

---

### Box 1: Communal Sedimentation Tank Size

**Sedimentation tank 1 Volume**

\( A_1 = \) Length x breadth (3:1) = 9 x 3 = 27m$^2$

\( V_1 = \) Area x height = 27 x 2.5 = 70 m$^3$

Wastewater flow generated \( (Q_1) = 1656p \times 120l/p.d = 198,720l/d \approx 200m^3/d \)

Retention time, \( t_1 = \frac{24H}{V_o} = \frac{24\text{hrs/d} \times 2.5\text{m}}{24\text{m}^3/m^2.d} \approx 3\text{hrs} \)

Sludge accumulation \( S_1 = 1656p \times 1.5 l/p.d \approx 3\text{m}^3/d \)

**Sedimentation tank 2 Volume**

\( Q_2 = Q_1 - V_1 - S_1 = 200m^3 - 70m^3 - 3m^3 = 127m^3 \)

\( V_2 = Q_2 t = 127\text{m}^3/d \times 1/2d \approx 60\text{m}^3 \)
Two sedimentation tanks can be constructed in series, and the second has an adjoining wall. The tanks will be constructed in a way that there is additional depth as the tank slopes to a hopper bottom where the sludge accumulates and is removed. The inlet to the tank will have baffles to attenuate the velocity of flow at the inlet, preventing short circuiting and hydraulic disturbance while weirs at the end will enable uniform flow. The prescribed surface loading for a primary sedimentation tank is about 24m/d. The specification for length and width for the rectangular sedimentation tank varies from the ratios 3:1 to 5:1 (Length: width), the width ranges from 3 – 6.1 m while the height is between 2 – 2.5 m. The sludge accumulation is given between 0.23 – 1.5 l/p.d. (Gray, 1995, Bounds, 1997, Gilberg et al., 2003, Hammer and Hammer, 2004).

The first sedimentation tank area and volume will be respectively 27 m² (9 m x 3 m) and 70 m³ with a height of 2.5 m. The daily sludge volume that can be collected from the sedimentation tank will be about 3 m³/d if sludge accumulation per person is taken as 1.5 l/d. The solids or sludge accumulated is then withdrawn from the hopper at the tank bottom to the digester. The overflow from the tank goes into another similar tank which acts as a compartmental sedimentation basin, having also longer retention time of ½ day for further treatment. The overflow from tank one in half a day is about 100 m³ for a design flow of 200 m³/d assuming equal hourly flow. The second tank receives about 127 m³/d of settled wastewater less sludge withdrawn daily and less tank storage. If a retention time of about half day is chosen for design then the tank volume required for the second tank will be about 60 m³.

In the selection of values for design, the organic strength of the wastewater and quantity should be determined by measurements which have been undertaken throughout a year including seasonal climatic variations, reasonable infiltration. However, in the study area, this is not available. Hammer and Hammer (2004) gave the average composition of domestic wastewater based on 450 l/p.d as in the Table 6 below.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Raw (450l/p.d)</th>
<th>After settling (450l/p.d)</th>
<th>Percentage Removal (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Solids (TS)</td>
<td>800</td>
<td>680</td>
<td>15</td>
</tr>
<tr>
<td>Total volatile solids (TVS)</td>
<td>440</td>
<td>340</td>
<td>12</td>
</tr>
<tr>
<td>Suspended solids (SS)</td>
<td>240</td>
<td>120</td>
<td>50</td>
</tr>
<tr>
<td>Volatile Suspended Solids (VSS)</td>
<td>180</td>
<td>100</td>
<td>44</td>
</tr>
<tr>
<td>Biochemical Oxygen Demand (BOD)</td>
<td>200</td>
<td>130</td>
<td>35</td>
</tr>
<tr>
<td>Total Nitrogen as N</td>
<td>35</td>
<td>30</td>
<td>15</td>
</tr>
<tr>
<td>Total Phosphorus as P</td>
<td>7</td>
<td>6</td>
<td>15</td>
</tr>
</tbody>
</table>

After primary settling in a sedimentation tank as designed above, there is some reduction in contaminant level. According to the Table 5 above, the total solid reduction is about 15%, 12% volatile solids removal, 50% SS, 15% nitrogen and 35% BOD removal. However, the second sedimentation tank is assumed to remove contaminants as a septic tank due to longer retention time. The percentage contaminant removals in the second sedimentation tank include 50% BOD removal, 70% TS, 30% Nitrogen and 40% phosphorus removal (Patterson, 1985, USEPA, 2002, Lombardo, 2004).
7.4 DESIGN OF INTERMITTENT SAND FILTER

The design of the intermittent sand filters depends on the contaminant loading rate and the sand media characteristics. The sand media is about 0.25 mm in diameter. The dosing tank volume is about ½ times the volume of septic tank effluent which is about 30 m³, dosed between 12 – 24 times daily with 76 mm (3 inch) diameter pipes. Pressured rigid pipe networks with orifices distribution method can be used. The organic and hydraulic loading of the intermittent sand filter should be 0.024 kg/m² and 0.12 m³/m² per day respectively. The required size for the intermittent filter based on the hydraulic loading with a 60 m³/d septic tank effluent flow of about 500 m² (25 m x 20 m) with a depth of less than 1 m. The dosing system should be designed in such a way to provide maximum contact of the effluent with the media. The filter can remove up to 90% BOD, over 99% fecal coliform and over 55% total nitrogen with an effluent of less than 10 mg/l TSS (USEPA, 2002). Appendix 2 shows a diagrammatic representation of the intermittent sand filter.

7.5 DESIGN OF SUBSURFACE FLOW WETLAND

The design criteria including organic, hydraulic loading, retention times in subsurface flow wetlands are summarized in the Table 6 below by Wood (1995). A two staged vegetated subsurface flow: a horizontal flow and vertical flow subsurface constructed wetland connected in series can be constructed.

Table 6: Process design criteria for constructed wetlands adapted from Wood (1995)

<table>
<thead>
<tr>
<th>Factor</th>
<th>Typical Surface flow (SF) wetland</th>
<th>Typical subsurface flow (SSF) wetland</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retention time</td>
<td>5 – 14</td>
<td>2 – 7</td>
</tr>
<tr>
<td>Max BOD Loading rate, kg/ha.d</td>
<td>80</td>
<td>75</td>
</tr>
<tr>
<td>Water/Substrate depth, m</td>
<td>0.5</td>
<td>– 1.0</td>
</tr>
<tr>
<td>Hydraulic loading rate, mm/d</td>
<td>7 – 60</td>
<td>2 – 30</td>
</tr>
<tr>
<td>Areal Requirement, ha/m³.d</td>
<td>0.002 – 0.014</td>
<td>0.01 – 0.007</td>
</tr>
<tr>
<td>Length to width ratio</td>
<td>2:1 to 10:1</td>
<td>0.25:1 to 5:1</td>
</tr>
<tr>
<td>Mosquito control</td>
<td>Required</td>
<td>Not required</td>
</tr>
</tbody>
</table>

For a loading of 4 kgBOD/d to the subsurface wetland, an area of 500 m² which can be divided into two beds of 18 m x 14 m each for horizontal flow and vertical flow wetland using length to width ratio of about 1.3:1 or Darcy’s law. The two staged wetland can obtain the following contaminant removal 97% TSS, 95% N, 97% BOD, 93% COD, 99% E coli, and 47% P removals (Shrestha et al., 2001). Appendix 3 shows a diagrammatic representation of the subsurface flow wetland.

7.6 EFFECT OF RAINFALL AND EVAPORATION IN DESIGN SYSTEM

Consideration should be given to rainfall and evaporation in the design of the sedimentation tanks or evapotranspiration in the vegetated subsurface flow wetland. The annual rainfall in the area is about 2200 m over 9 – 10 months of the year, while the evapotranspiration is 76% of rainfall (Ayoade, 1975 and Idowu, 2002). When there is substantial evaporation as a result of high temperature and wind effect over the sedimentation tank, then there will be increase in contaminant concentration. On the other hand if the storage from the rainfall excluding evaporation is high, then there might be need for increased tank volume. However, the tanks
The effect of rainfall and evapotranspiration especially in a tropical climate over the constructed wetland is important due to the effect of surfacing which is the increase in head of water over the media. Surfacing results in a lack of hydraulic capacity and it occurs when the media can no longer transport the required flow. The factors that can cause surfacing apart from the dynamics of rainfall and evaporation include clogging due to accumulation of solids, faulty design of inlet and outlet pipes, poor choice of media etc. The potential for surfacing in a subsurface wetland can be determined from Darcy’s Equation 15 and 16 below:

**Equation 15**

\[ Q = K A dh/dl \, (m^3/d) = KWD \cdot dh/dl \]

**Equation 16**

\[ dh = \frac{Q dl}{KWD} \, (m) \]

Where

- \( K \) = hydraulic conductivity of media \( (m/s) \)
- \( A \) = Area normal to wastewater flow \( (m^2) \)
- \( W \) = width of wetland \( (m) \)
- \( D \) = depth of wetland \( (m) \)
- \( dh/dl \) = hydraulic gradient

The flow into the wetland considering rainfall and evaporation, assuming no net overland runoff flow into the subsurface flow constructed wetland with and area of 500 m\(^2\) is about 1 m\(^3\)/d for the 9 – 10 months of rains. There will be no surfacing if a media depth of about 0.65 m is chosen for a 65 m\(^3\)/d flow.
7.7 MASS BALANCE OF DESIGN SYSTEM

Figure 10a below shows the mass balance of BOD for the design system. The BOD entering the system through the sedimentation tanks is about 120 kgBOD/d or 600 mg/l. After thickening of sludge in the sedimentation tank, the effluent contains 40 kgBOD. This is further treated in an intermittent sand filter and a combined horizontal flow and vertical flow subsurface wetland. The treated wastewater can be safely discharged to the groundwater having a BOD of 2 mg/l which is less than the recommended 5 mg/l for safe disposal to groundwater (USEPA, 2002). Figure 10b is the mass balance for the total suspended solids which also meets the 5 mg/l disposal limit for groundwater.

Figure 10a: Mass balance of BOD in the design system

Figure 10b: Mass balance of TSS in the design system
Figure 10c below is the mass balance of nitrogen in the design system for decentralised wastewater treatment in the study area. The nitrogen entering the system through the sedimentation tanks is about 20 kg/d or 100 mg/l. 8 kg is removed as particulate nitrogen with the suspended solids as they settle out while 12 kgN in the form of NH$_4^-$ from the septic tank effluent is first nitrified in the intermittent sand filter, leaving about 5% which is denitrified in the anaerobic vegetated subsurface wetland. The resultant nitrogen is 0.2 mg/l which is less than the 5 mg/l required for discharge to groundwater. However, 8 kgN/d goes with the sludge to the digester. The carbon to nitrogen required for anaerobic digestion is in the range of 8 – 30 (Kossmann et al., 1998, Ostrem, 2004). Hence with a 60 kgCOD in the digester, a C/N ratio of 20 is met. After digestion however the biosolid nitrogen content is about 4.2% (Epstein, 2003). For a compost C/N ratio of 30 (Nazih and Wang, 2007) to be maintained however, manure can be added with saw dust as bulking agent to enhance the nitrogen necessary for effective composting.

**Figure 10c: Mass balance of N in the design system**

7.8 DESIGN OF THE ANAEROBIC DIGESTER

From the design of the communal sedimentation unit above, there is daily removal of the thickened sludge. The sedimentation tanks are connected to the anaerobic digester which is also fed with organic fraction from municipal solid waste that is generated from the area and presently dumped behind Ezenei Street and some vegetable waste from the nearby Ogbogonogo market. The sludge from the wastewater constitutes 75% of the content of the anaerobic digester while the organic faction of municipal solid waste is 25% (Hamzawi et al., 1998). In Nigeria, a person will produce 20 kg of municipal solid waste per year (Akinbami, 2000). More than 50% of this comprise the organic fraction (Demirbas, 2006), that is about 10 kg. This implies that for the 1656 persons in the study area the organic fraction of municipal solid waste generated will be about 45 kg per day.
7.8.1 PREDICTING BIOGAS PRODUCTION

**Box 3: Predicting Biogas Production**

**Wastewater Fraction**
Incoming COD: 160kg/d (Assume COD = 2BOD)

**Organic Fraction Of Municipal Solid Waste (OFMSW)**
50% + 20kgMSW / p.yr + 1656p / 365d = 45kg OFMSW / d

**Total Organic matter**
\[\text{WW + OFMSW} = 160\text{kg/d} + 45\text{kg OFMSW / d} = 205\text{kg organic matter / d}\]

75% wastewater and 25% OFMSW optimal design

For optimal gas production a mix ratio of 75% organic matter in wastewater and 25% OFMSW is chosen for daily digester input. The total added organic matter to the digester daily therefore will be:

\[25\% \text{COD wastewater + 75}\% \text{from OFMSW} = \frac{160\text{kgCOD}}{d} + \frac{53\text{kg OFMSW}}{d} = 220\text{kg organic matter / d}\]

**Biogas Production Estimate**

If an average biogas production of 500L biogas / kg organic matter added, at a temperature of 35°C and 20days HRT, then expected gas production will be

\[220\text{kgOM / d} \times 500\text{biogas / kg} \times 110\text{m}^3\text{biogas / kg OM} \approx 110\text{m}^3\text{biogas / d}\]

The gas accumulation in 20days which is the retention time will be about 2000m³.

**Methane proportion**
The proportion of methane in the digestion of OFMSW and communal blackwater sludge could be taken as 70% which gives a theoretical methane content of approximately 80m³ per day.

In general, the biogas production for every 1 kg of organic matter input into the digester for mesophilic digestion range is between 400 to 500 L of gas (Rueil-Malmaison, 1991). More recently however, with codigestion of substrates, higher biogas production has been obtained under different organic loading rates and digestion condition as discussed in section 6.7.8. The organic matter content to the digester considering the 75% wastewater fraction and 25% OFMSW fraction will amount to about 220 kg. The additional daily 8 kg organic matter can be obtained from the vegetable wastes generated from the nearby market. If an average biogas production of 500 Lbiogas/ kg organic matter added is adopted, at a temperature of 35°C and 20 days HRT then expected gas production will be about 110 m³ with an estimate of 70% methane (80 m³/d).

7.8.2 BIODIGESTER SIZE

The volume requirement for digesting different sludge type is presented in Table 6 below. The sludge thickened in the primary communal sedimentation tank is primary sludge with a biogas digester size requirement of 30 l/p.d. In high rate mesophilic digestion with only a primary digester the organic loading rate should be between 1.3 to 2 kg/m³.d (Hammer and Hammer, 2004 and Rueil-Malmaison, 1991). The total volume required for the community digester serving 1656 persons will therefore be about 50 m³.
Box 4: Energy Generation from Biogas

**Methane generated electricity**

Yearly methane production will be $80 \text{ m}^3/\text{d} \times 365 \text{d} = 29200 \text{ m}^3$

Electricity (KWh/yr) = methane (m$^3$/yr) $\times$ 10.4 (KWh/m$^3$)

Electricity = 304000(KWh/yr)

**Communal Electricity Consumption**

$68.17 \text{ KWh/p yr} \times 1656p \approx 112000 \text{ KWh/yr}$

If the electricity consumption per person in Nigeria is 68.17 KWh then 37% of the yearly methane production can satisfy the total electricity consumption for the area (112000 KWh/yr).

**Heating/Cooking**

For cooking purpose the amount of biogas required per person per day is $0.26 \text{ m}^3$ (Adeoti et al., 2000, De Alwis, 2002) Hence for the 1656 persons in the study area, the total theoretical amount of biogas required for cooking will about be $400 \text{ m}^3$. Then about 20% of the daily kerosene use in the area can be replaced by biogas.
7.8.4 BIOGAS GENERATOR

The biogas generated is from readily available sources; wastewater, municipal solid waste and market vegetable waste. Biogas from these waste sources is reliable and sustainable and can be utilised for electrical energy generation through the use of a biogas generator whose capacity is suitable for the daily biogas production. The theoretical amount of methane that can be produced is about 80 m$^3$/d and 37% of which corresponds to about 300 KWh/d electric power generation. Different models of biogas generator for electricity production exist in the market today and some are also modified from diesel or liquefied petroleum gas generators. The generator type chosen should be easy to operate and maintain. The electricity generated can then be supplied to the community through electricity cables. On the other hand, the extra gas can be supplied to the community by the use of canisters.

7.8.5 INVESSEL COMPOSTING AND LAND APPLICATION

**Box 5: Compost Mix and unit size**

| TS to the invessel composting unit = 200 KgTS      | Assume COD = 2x BOD  |
| COD to the invessel composting unit = 60 kgCOD    | N to the invessel composting unit = 0.3 kgN |
| For a C/N ratio of 30, the additional nitrogen required = 1.7 kgN | For a sludge ratio of 40%, the required bulking agent mix/sludge ratio is about 0.8 |
| If sludge = TS = 200 kg, then total bulking agent saw dust required = 0.8 x 200 = 160 kg (160 kg - 2 kg N) | If sludge = TS = 200 kg, then total bulking agent saw dust required = 0.8 x 200 = 160 kg (160 kg - 2 kg N) |
| Water content = 60%                             | Where x is the mass of manure that will give 2 kg of N. |
| Assume total bulking agent = 160 kg             | Water content = 540 kg |
| Solids Content = 200 + 160 = 360 kg             | Density of water = 1000 kg/m$^3$, Density of saw dust = 300 kg/m$^3$ and sludge = 953 kg/m$^3$ |
| The size of invessel composting unit based on the densities and mass of water, sludge, saw dust with 21 d retention time = 30 m$^3$ | The size of invessel composting unit based on the densities and mass of water, sludge, saw dust with 21 d retention time = 30 m$^3$ |

The study area is not very close to agricultural land hence storage or further treatment may be necessary for the anaerobically stabilised sludge. In the anaerobic digestion, there is a 20% solid, 40 – 70% organic matter, 95.8% N and 97% P reductions (USEPA, 2002, Epstein, 2003, Gillberg et al., 2003, Hammer and Hammer, 2004). The organic matter that can therefore be removed from the anaerobic digester in the study area is about 50 kgBOD/d while the remaining 30 kgBOD/d can be further decomposed in an invessel composting unit converting it to a more inert substance that can be readily available to plants or for other land improvement purposes. This is necessary for the study area as there not sufficient available land space for other composting regimen such as windrows or aerated static pile. The vessel will be vertically operated in a plug flow mode with the biosolid and bulking agent applied at the top while the compost is taken from the other end. Air is also introduced from the bottom of the vessel. The carbon to nitrogen ratio should be between 25 – 35 while about 5% oxygen is required for aerobic condition with a temperature of about 55°C, optimum moisture content between 50 – 60% and 21 days retention time (Nazih and Wang, 2007). The amount of bulking agent (sawdust) needed to maintain the moisture content, porosity and structural integrity of the compost mix is a given in the Figure below 11. Figure 12 also shows the mass balance of TS in the digester and invessel composting unit.
This ratio is site specific and depends on the relative volatility and solids content of the bulking agent and sludge. Wood chips/saw dust has a density of 265 kg/m$^3$ to 333 kg/m$^3$, while digested sludge has a density of between 893 kg/m$^3$ to 1012 kg/m$^3$. For a biosolid or sludge solids content of 40%, the mixing ratio for the bulking agent is 0.8. This implies that for a TS content of 200 kg and a COD content of 60 kg, assume COD = 2 x BOD, (Gillberg et al., 2003), the bulking agent required would be about 160 kg. Since the C/N ratio required for composting is about 30, then there will be need to incorporate some manure that can give a nitrogen content of about 2 kg as a mix with the saw dust to meet up the C/N ratio of 30.

The size of the composting unit required for a retention time of 21 days and saw dust to biosolids ratio of 0.8:1 (40%TS) is about 30 m$^3$. The exhaust air from the compost is treated.
by biofilters to prevent odour emission that may result from the composting. The compost is allowed to cure and sold to farmers. The cost of the odour treatment can be recovered from the sale of the compost.

7.9 RAINWATER HARVEST

**BOX 6: Rainwater harvest**

The rainfall or stormwater collected from the roofs in the study area is calculated from the Rational method; Equation 6 below.

\[ discharge, q = CiA \]

The runoff coefficient for roof surface is in the range 0.75 – 0.95.

\[ Rain\ harvest, q = 0.85 \times 2.2m \times 14,854m^2 = 28,000m^3 or \frac{90m^3}{d} or \ 0.06 \ m^3/p.d \]

Potential rainwater harvest per person per day in the study area is 0.06 m\(^3\)/p.d assuming storage since raining season is about 9 – 10 months of the year. Individual buildings based on a population of 18 can have 2-3 day rain harvest storage of 2 – 3 m\(^3\) more or less.

**Storage Volume**

The theoretical storage volume required for each house is max

\[ \text{Storage Volume} = \frac{92 \text{ house}}{1651 \text{ persons}} \times \frac{0.05 \ m^3}{p.d} \times 2 \text{ days} = 2.0 \ m^3/\text{house. d} \]

**Toilet Flushing**

Toilet flushing requires 20% of the available freshwater supply

\[ 20% \left( \frac{120}{p.d} \right) = 0.024 \ m^3/p.d \times 1656p = 40 \ m^3/d \]

Also rainwater has great potential in the study area and can be harvested from the roofs. Rainwater harvested can be treated and used to satisfy the non potable water uses in the homes such as toilet flushing, car washing, street washing, laundry. Considering the total roof area and yearly rainfall value of about 14,800 m\(^2\) (google earth) and 2200 mm respectively the rainfall harvest can be estimated from Equation 6. Appendix 4 shows the spatial distribution of rainfall in Nigeria. The expected rainfall harvest for the area will be about 28000 m\(^3\) or 90 m\(^3\)/d. For a fresh water consumption of 200 m\(^3\) per day in the study area, the theoretical rainwater harvest can provide about 45% reduction. The required storage of harvested rainfall per building will be about 1-2 m\(^3\). This is done above ground and alum added as a coagulant.

It is necessary to note that raining season is between 9 - 10 months of the year hence, treated rainwater can be used as a supplement, contributing to water saving or sufficiency in the area. Rainwater harvesting will also help to reduce the storm water hence attenuating flooding and erosion.

7.10 APPLICABILITY OF CHosen TECHNOLOGY

There has been application of combined sedimentation with a two staged subsurface wetland in Kathmandu Valley in Nepal (Shrestha et al., 2001) The combined technology is a demonstration project for treatment of hospital wastewater and had been operated successfully for three years with about 95% major contaminant removal. There has also been transfer of this technology in other parts of Nepal. Nepal is a developing country with a summer temperature of about 28°C and a winter temperature of about 7°C. Also a regional project by joint co-operation between Israel, Palestine and Egypt was established. The purpose was to develop decentralised wastewater reuse schemes that are efficient, low cost, low impact and
replicable based on available land, climate, population and socio-economic development. The Israeli Sakhnin pilot project in the region for wastewater treatment combined such technology as UASB reactor, passive aerated vertical bed, intermittent sand filter and constructed wetland. The influent wastewater was primary effluent after sedimentation. The climate of Israel ranges from temperate to tropical. Summer temperature in Sakhnin, northern Israel is between 20°C – 27°C and the winter temperature are between 12°C – 14°C. The pilot project has been operated for 4 years.

An Asian Development Bank pilot project in East Vietnam on the development of appropriate decentralised sanitation solutions for peri-urban areas in 2009 used a combined anaerobic baffled reactor septic tank with anaerobic filter and constructed wetland. Septic tanks where used primarily for solids removal. The construction of the cluster system pilot project earmarked for 60 households started in 2008 with connection of about 80% of the households. The choice of decentralised technology required minimal operation and maintenance investments hence enabling full communal ownership with a possibility for enlargement and replication (Plaisance, 2009). Vietnam is a developing country with generally tropical climate. The temperature in southern Vietnam ranges between 22 – 35°C.

A similar Decentralised Wastewater Treatment program by EAWAG adopted from a master thesis project in 2003 for the purpose of improvement of sanitation in tropical countries was developed as a pilot project in Asian Institute of Technology, Bangkok, Thailand. The project is also aimed at determining the applicability of the decentralised technologies based on social and economic factors, their potentials and limitations. The technology combination is similar to the Vietnamese project including an anaerobic baffled reactor, sand filter and constructed wetland.

The major consideration of the applicability of the technologies chosen, especially the aquatic and ecological treatment systems, with regards to the study area is the climate. From the study of combination of technologies above, it is possible to apply these decentralised technologies in temperate and tropical climate. Anaerobic digestion on the other hand will be well suited to the study area due to high temperature. There may be no need for heating or relatively low heat application will be required for optimum mesophilic digestion. However, the biogas digester, intermittent sand filter and constructed wetland are totally novel idea that will take a while for adaptation and effective running. Since the technology combination is a relatively new concept, it is important that a pilot project be initiated in the study area to demonstrate the potential benefit of the system. With the success of the pilot plant can wastewater handling with biogas production be introduced in the Asaba metropolis in clusters.
8. DISCUSSION ON THE IMPLICATION OF THE CHOSEN SYSTEM

The health and environmental concerns associated with the present decentralised wastewater implementation make it imperative to develop new decentralised wastewater treatment concepts which will minimise environmentally harmful emissions and encourage the reuse of waste for energy production. There are different treatment types that may be potentially sound for decentralised wastewater treatment in developing countries. This study encourages the choice of treatment technology or a group of treatment options that are easy to operate, with low energy consumption, relevant to a community for the treatment of wastewater with the most minimal or no negative impact. The choice of treatment technologies should also be predicated upon the social and cultural acceptance of the community, taking into cognisance the climatic conditions.

8.1 MEETING ENERGY REQUIREMENT

Although Delta State (where the study area is located) has about 40% of total natural reserve of crude oil and gas in Nigeria, the inhabitants are very energy deficient (Obueh, 2006). Presently, the study area is mainly dependent on electric power supply from the national grid, generated mostly from hydro-power. Electricity generation is grossly inadequate and supply is epileptic leading to reliance on kerosene for both lighting and cooking and on fuel wood especially in rural settings. Projections done in this study for electricity production and cooking from the proposed biogas generation showed the possibility of electricity generation that will adequately meet the present yearly per person consumption in the study area. Although this is a far cry from per person consumption in developed countries such as Sweden, Denmark, USA, it will be a great improvement in electricity supply. On the other hand, biogas can be a supplement as a cooking fuel, meeting 20% of the cooking gas requirement in the area. Also, the energy requirement for cooking in the area can be met if the generation of electricity is done only in the event of power outage from the national grid.

The production of biogas from wastes is futuristic, wholesome and will ultimately in combination with hydro-power from Kainji dam, wind generated power and solar energy reduce fossil fuel dependence. This is important due to the need to conserve energy of fossil origin while also minimising the negative impact of gas flaring from wellheads. Gas flaring during crude oil production and refining has been associated with negative public health impacts and environmental pollution.

The biogas production however can be improved upon with a view to meeting the total energy needs for both electricity and cooking. By co-digesting wastewater with higher energy substrates such as animal by products from meat processing, grease trap waste such as fats, oils and grease, waste from slaughter houses biogas production can also be improved. Co-digestion of animal by products and wastewater sludge in ratio of 1:7 has been found to improve methane yield substantially (Luste and Luosterinen, 2009). The combined effect of the wastewater and animal by product reduced the effect of inhibitive substances although the organic loading rate was high. Another way of improving biogas production is by pre-treatment of the substrates. This can be chemical pre-treatment, biological, thermal or physical. Thermal pre-treatment or hygienisation at a temperature of 70°C has been found to increase energy output in a study carried out by Luste and Luosterinen (2009). The use of an inoculum, for example, digested wastewater biosolid will enhance digester startup by providing the methanogenic organisms required thus improving biogas production at the onset of digestion.
8.2 EFFECTIVE SANITATION: ENVIRONMENTAL AND HEALTH IMPLICATIONS

One of the aims of decentralised wastewater treatment improvement is to provide effective sanitation. With the proposal in this work for wastewater treatment, the BOD, N, SS, fecal coliform and other contaminants are effectively removed. This is illustrated in the mass balance of some of the contaminants. The effluent when finally dispersed to groundwater, will meet discharge limits. Fecal sludge management is presently very poorly handled in the study area; most cases emptied into surface water which is very harmful environmentally with such effect as nutrient enrichment, fish contamination, reduction or extinction in aquatic production, turbidity among others. The fecal sludge handling proposed is thus a total upgrade from the existing system. There is sludge reuse for energy production and the final outfall is composted, a product that is more inert, can be more easily handled, acceptable, marketable and with no detectable pathogenic amount (Nazih and Wang, 2007). For a holistic approach to decentralised wastewater handling, greywater treatment is combined with the blackwater. The proposed treatment for greywater will also detract from the environmental impact of the present disposal method into drains or by indiscriminate ponding. The health and environmental challenge from the septic tank and soakaway pits in the study area such as pollution of groundwater with wastewater nitrate will be averted by proper design and construction of a combined intermittent sand filtration and wetland.

Another relevant implication of the combination of technologies chosen is the reduction of green house gas emission because the methane produced from the system is harnessed thus aiding in combating the global warming effect. The environmental arm of the FGN in 2003 reported that wastewater treatment contributed about 32% of the total methane emission in Nigeria. The system will therefore reduce in the overall methane emission. In addition, co-digestion of market vegetable wastes and the organic fraction of municipal solid waste in this study is aimed at reducing the amount of waste that accumulate and subsequently end up in landfills. The European Union has given a landfill directive to member states for reduction of landfill waste from the year 2005. This will lead to the eventual ban (Marshall, 2007). Landfill gases are produced from biodegradable wastes in landfills and contribute to climate change. Hence reducing the amount of wastes to landfill will help check also climate change.

8.3 FRESHWATER SAVINGS

There is about 45% freshwater savings when rainwater is harvested and utilised. There is already inadequate supply of fresh water from the governmental agency in the area. With the treatment and use of rainwater (without the first flush), the non potable water need such as toilet flushing, street washing, car washing, backyard gardening, even laundry, can be satisfied. This will reduce dependence on freshwater supply which can then be used for drinking and cooking purposes. On the long run, rainfall contribution to stormwater runoff and flooding can be greatly minimised.

However, it is necessary that mathematical models that represent the real rainwater and wastewater generation in the study area be performed. Such that before any monetary or material commitment are made, problems that may emerge are detected and solution proposed (Mah et al., 2009). The quality and quantity of greywater, blackwater and rainwater have to also be analysed. Data collection is therefore very relevant in the study area for successful implementation.
8.4 Effect of Technology on the Community

The overall choice of technology in this work for decentralised wastewater treatment is an effective decentralised waste management tool. However, in the implementation consideration has to be given to the present site situation which will necessitate new pipe connection for transport of wastewater to a more centralised location within the community. Although not so long as in centralised wastewater treatment that could serve the whole of the city of Asaba. There could be a phase by phase construction of the components. Sedimentation tanks can be constructed first, while there is gradual connection of the wastewater effluent from the homes. The anaerobic digester can then be installed. Rainwater harvesting may run concurrently with the wastewater treatment and finally the whole treatment process can then be affected.

There is also the effect of odours from the sedimentation facility that may need to be considered and this can be taken of by the provision of a retracting cover made with local fabric such as tarpaulin for the sedimentation tank.

8.5 Social Acceptance and Sustainability of System

An important aspect for consideration is social acceptance and sustainability of the decentralised system proposed in this work. For a decentralised wastewater treatment method to be sustainable, it must be able to satisfy the sanitation needs of the community for now without compromising the public health and undermining the quality of the environment now and in the future. The cluster system proposed in this work gives consideration to both environment and public health. However, in order not to generate negative attitude, suspicion and outright rejection of the system due to lack of relevant knowledge about the cluster system, it is necessary for the public to be informed and sensitised on the advantages of this new system. When the anaerobic digester is operated with electricity production, then domestic and commercial institutions will benefit, hence largely contributing to its sustainability. Communal involvement in the planning stage and actual construction of the units are necessary for good understanding of the system. Finally, training of communal operators for the anaerobic digester and maintenance of the wetland, intermittent sand filtration system and the invessel composting unit is relevant to its acceptance and sustainability.

For the sustainability of decentralised wastewater treatment, centralised management is highly necessary (Otis, 1999, Massoud et al., 2009). Government participation in form of subsidies for capital construction cost and periodical inspection of plant from a relevant ministerial department such as Ministry of Water Resources Development or Environment as the Responsible Management Entity may aid in sustainability. The strategies for management of decentralised wastewater treatments should be specific for each area or site and should take into cognisance environmental, cultural, social and economic situations. The lowest form of tariff for waste handling should be encouraged for plant maintenance. In addition, developing countries such as Nigeria should begin to incorporate environmental issues especially waste management as a priority area in the planning and implementation stages as an important aspect of both social and economic development. When wastewater is viewed as a source of income, renewable energy potential, its prioritisation is imminent. However, with the cluster system proposed in this work, it may be more advantageous to first implement in a new property or estate development. With the obvious advantages of the system, it can be replicated in older communities or settlement and eventually the whole of Asaba.
9. CONCLUSION

This study reviews and discusses the state of decentralised wastewater treatment in many developing countries and the challenges they present in terms of environmental and public health. For the choice of appropriate treatment technology that will be an improvement on the present handling of wastewater, anaerobic digestion of wastewater sludge is considered. First, the combined black and greywater is settled in sedimentation tanks. The sludge collected is then digested in a 50 m$^3$ anaerobic digester in the optimum ratio of 75% wastewater sludge and 25% organic fraction of municipal solid waste. The daily volume of methane production projected is 80 m$^3$. This can be used for electricity production that will meet the yearly requirement of about 112000 KWh for a portion of Ezenei community at Asaba, Nigeria with a population of 1656 persons. About 20% or more of the daily cooking energy need of the study area can be met, hence partly replacing the need for kerosene. The biosolid from the anaerobic digester is further composted in a 30 m$^3$ invessel composting unit. The product can then be sold to farmers for crop production or utilised for soil amendment. The effluent from the sedimentation tanks then receives further treatment in an intermittent sand filter and vegetated subsurface wetland connected in series. This treatment produces effluent with less than the recommend 5 mg/l BOD, SS and N and can be safely dispersed into the groundwater. Harvested rainwater is also considered for treatment in the study area and can meet the 40 m$^3$ per day required for toilet flushing and some other non potable water needs with freshwater savings of about 45%.

Amongst other benefits, the proposal for wastewater treatment in this study will reduce the area needed for landfill and the associated landfill, air, land and water pollution, alleviate energy needs, provide environmental and public health protection, mitigate against flooding and provide adequate wastewater disposal. However, the success of the system will depend on communal information and participation and centralised management from the state to local level.
10. AREAS FOR FURTHER RESEARCH

The contribution of onsite wastewater disposal on the groundwater resources is a salient issue in this work due to the density of onsite disposal system in the study area. Presently, it is not taken into cognisance the point source contaminant load that enters the groundwater resource in the environment through the soakaway pits for septic tank effluent disposal. The problems associated with them include improper design, faults, clogging, wear and leakages due to aging, collapse and nearness to boreholes. The amount of nitrate and other contaminants in groundwater in this area should be studied and mapped with the aim of remediation.

A pilot project with anaerobic digestion of wastewater sludge in co-digestion with organic fraction of municipal solid waste or other substrates such as slaughter house wastes, used cooking oils or grease trap wastes can be carried out to demonstrate the technology with the view of widespread acceptance. The possibility of small individual household digestion units can be looked into especially with the generation of biogas to meet the cooking needs.

Rainwater harvesting as studied in this work has great potential for meeting the non potable water need especially for toilet flushing. It will be worth knowing, with questionnaires and other tools why this is not being practised on the scale that this work proposes or on a larger communal scale with the aim of educating the population on the possibility. Finally, the design carried out in this work has been based on values from literature reviews. Further research can be initiated for onsite collection of data such as black and greywater quantity including seasonal variation, quality, and amount of contaminants present, soil absorption capacity etc, for about a year. This will form the basis for wastewater design in that area. The cost analysis also of the entire cluster system proposed can be undertaken including the possible payback period and the cost advantage of the energy production.
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12. APPENDIX

APPENDIX 1: Map of the world showing the location of Nigeria (www.mapsofworld.com)
APPENDIX 2: Diagrammatic presentation of the component of intermittent sand filters showing the distribution network, media and the underdrain. (USEPA, 2002)
APPENDIX 3a: Diagrammatic presentation of the subsurface wetland showing the inlet, treatment and outlet zones, media size, water level and depth (USEPA, 2000)

APPENDIX 3b: Diagrammatic presentation of the subsurface wetland showing root zone and major flow path (USEPA, 2000)
APPENDIX 4: Map of Nigeria showing spatial variation of annual rainfall. Approximate location of Asaba is shown. (FRN, 2003)
APPENDIX 5

Improvement of Decentralised Wastewater Treatment in Asaba, Nigeria
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Abstract

Decentralised wastewater treatment in Asaba, Nigeria is mainly by septic tanks and soakaway pits. The density of these onsite systems, often greater than 15/km$^2$ can result to groundwater contamination with nitrates. Also fecal sludge is disposed of untreated in surface water bodies and dumpsites which have negative environmental and health consequences. To improve on the state of decentralised wastewater treatment appropriate technology should be chosen. From the study a cluster system using sedimentation tanks 70 m$^3$ and 60 m$^3$ in series will produce wastewater sludge that can be digested anaerobically in the proportion 75% to 25% of municipal solid waste with vegetable wastes to produce electricity. Biogas generated electric energy of 112,000 KWh will meet the yearly per person consumption of the study area. 20% kerosene needed for cooking will also be replaced. The stabilised sludge can finally be treated in a 30 m$^3$ invessel composting unit to produce biosolids that can be used for agricultural and land amendment purposes. The primary effluent from the sedimentation tank is discharged to groundwater but first treated in a combined intermittent sand filter and subsurface flow constructed wetland both of which are each 500 m$^2$. The effluent meet recommended BOD, SS, and N discharge limits of less than 5 mg/l respectively. The combination of technology is envisaged to improve sanitation while minimizing health risk, reduce methane emission, provide fresh water saving and encourage reuse of waste for energy.

Keywords: Decentralised wastewater treatment, biogas, waste reuse, anaerobic digestion, invessel composting, appropriate technology

Introduction

Nigeria is located in West Africa with a population of over 120 million. The Federal Government of Nigeria reported that there was lack of comprehensive strategy for the disposal of excreta, wastewater and solid waste in Nigeria (FRN, 2000). While more emphasis has been placed on freshwater supply however, only 30% of the urban populace has access to acceptable potable water supply. The desperation to meet water need drives people to shallow hand wells, groundwater and surface water bodies that may be polluted to some degree from improperly treated wastewater from sanitation systems among other sources. The wastewater handling concept that has found wide application in Nigeria is the decentralised method. This entails the collection, treatment and disposal of wastewater close to the point of generation. The most common technology utilised presently is the septic tank which achieves only primary treatment of wastewater with effluent pollutant load ending in the groundwater through soakaway pits that have limited capacity to renovate contaminants including nutrients. The result is nutrient enrichment of surface water, nitrate contamination of ground water, high public vulnerability to disease causing agents resulting in such diseases that are now endemic to Nigeria such as diarrhea, typhoid fever, dysentery and gastro-enteritis.
The management of fecal sludge from the septic tanks is also a sensitive issue whereby it is disposed off in open water bodies, landfills or dumpsites. As the population in urban centres in Nigeria continue to grow at an annual rate of over 3% (FRN, 2003) more waste sludge will be produced. The recovery of energy and nutrients from waste such as wastewater sludge and solid wastes in Nigeria is yet to gain prominence. This situation is as a result of dependence on fossil energy resources such as crude oil, natural gas and coal which are finite. The refining of crude oil has resulted in environmental pollution which together with wastewater treatment accounts for 25% CO$_2$ and 32% CH$_4$ emission respectively in Nigeria (FRN, 2003). Both gases are greenhouse gases which contribute to global warming. In addition, the allocation of windfall from oil and gas has contributed largely to socio-economic and political unrest especially in the Niger Delta area of the country. With the above scenario, the development and choice of group of technologies appropriate for decentralised wastewater treatment in Nigeria is important. These will include a combination of wastewater treatment that promotes reuse of waste for energy production. This objective of this work is therefore to improve on the decentralised wastewater handling currently practiced in Nigeria typical of most developing world context. Wastewater treatment will be site specific in terms of the population, climate, soil conditions, socio-cultural acceptance and economics. Wastewater treatment will also be taking into cognizance health and environmental impacts with a view to minimizing or eradicating the negative contribution.

**Method**

A site is chosen for improvement of decentralised wastewater treatment. Description and analysis of the present wastewater treatment based on site information received is done. The impact of the present wastewater treatment regime is presented based on review of literature. A combination of treatment technologies will be selected based on the population, water use, waste composition, area map, rainfall data and wastewater reuse option. Contaminant mass balance for treatment options and the overall treatment will be represented in a scheme. Finally, the technologies are appraised based on local context.

**Description of Location**

The study area chosen for the improved decentralised wastewater treatment is part of the Ezenei Quarters, Asaba, Nigeria. It lies between 6° 11’ 0” N, 6° 45’ 0” E. Asaba is located at the western edge of the River Niger which links other West African countries and flows into the Atlantic Ocean. This location is chosen because it represents the population dynamics of many middle to low income urban settings in developing nations and the wastewater treatment type most prevalent. It is also communal setting where it may be relatively easy to establish a cluster system. The area delineated is about 137,000 m$^2$, with 92 houses, 18 persons to a house. The total population is about 1656 persons. The present form of wastewater treatment is the septic tank and associated soakaway pit.

**Results and Discussion**

The choice of technology for the improved decentralised wastewater treatment includes combined blackwater and greywater treatment. Figure 1 below shows the proposed wastewater treatment scheme with the components. Wastewater is treated in sedimentation tanks connected in series with the effluent dispersed to a combined intermittent sand filter and vegetated subsurface flow wetland.
The wastewater sludge is then directed to an anaerobic digester for the production of biogas in combination with organic fraction of municipal solid waste and market vegetable waste. The biogas is used for the production of electricity with the excess stored in gas canisters to meet the cooking requirement of the community. The biosolid produced from the biogas digester is composted in an invessel composting unit. The compost can be used for land amendment purposes or agricultural production.

Figure 1: Schematic Representation of Wastewater Treatment Scheme

Theoretical Mass Balance

Figure 2a below shows the mass balance of BOD for the design system. The BOD entering the system through the sedimentation tanks is about 120 kgBOD/d or 600 mg/l. After thickening of sludge in the sedimentation tank, the effluent to the intermittent sandfilter contains 40 kgBOD which is subsequently reduced by sedimentation and straining. The effluent is finally polished in the vegetated subsurface constructed wetland. The treated wastewater can be safely discharged to the groundwater having a BOD of 2 mg/l which is less
than the recommended 5 mg/l for safe disposal to groundwater (USEPA, 2002). Figure 2b is also the mass balance for the Total suspended solid with an influent of 660 mg/l and effluent which also meets the less than 5 mg/l disposal limit for groundwater.

**Figure 2a: Mass balance of BOD in the design system**

**Figure 2b: Mass balance of TSS in the design system**

Figure 2c below is the mass balance of nitrogen in the design system for decentralised wastewater treatment in the study area. The nitrogen entering the system through the sedimentation tanks is about 20 kg/d or 100 mg/l. 8 kg is removed as particulate nitrogen with the suspended solids as they settle out while 12 kg N in the form of NH$_4$-N from the septic tank effluent is first nitrified in the intermittent sand filter, leaving about 5% which is denitrified in the anaerobic vegetated subsurface wetland. The resultant nitrogen is 0.2 mg/l which is less than the 5 mg/l required for discharge to groundwater. However, 8 kgN/d enters with the sludge to the digester. The carbon to nitrogen required for anaerobic digestion is in the range of 8 – 30 (Kossmann et al., 1998, Ostrem, 2004). Hence with a 60 kgCOD in the digester, a C/N ratio of 20 is met. After digestion however the biosolid nitrogen content is about 4.2% (Epstein, 2003). For a compost C/N ratio of 30 (Nazih and Wang, 2007) to be
maintained however, manure can be added with saw dust as bulking agent to enhance the nitrogen necessary for effective composting.

Figure 2c: Mass balance of N in the design system

Approximate Design of Wastewater Treatment Components

Rectangular sedimentation tanks receive raw domestic wastewater allowing quiescence in order for particles that are suspended in the wastewater to settle. They are designed based on the surface flow or settling rate which is the average daily flow per metre area of the tank in a day given by 24 m$^3$/m$^2$. The contaminant removal in a sedimentation tank include, 35% BOD, 15% Total Solid, 12% Volatile Solids, 15% Nitrogen and 15% Phosphorus. However, the second sedimentation tank is designed as a septic tank with longer retention time, more suspended solid sedimentation and higher removal of contaminants such as 50% BOD removal, 70% TS, 30% Nitrogen and 40% Phosphorus removal (Patterson, 1985, USEPA, 2002, Lombardo, 2004, Hammer and Hammer, 2004). The tank sizes obtained are 70 m$^3$ and 60 m$^3$ respectively for the two sedimentation tanks with retention times of 3 hrs and ½ day respectively.

The intermittent sand filter is an aerobic device that can be used for the secondary treatment of septic tank effluent. The design depends on the sand media characteristics and the contaminant loading rates. Based on the hydraulic loading rate of 0.12 m$^3$/m$^2$ per day (USEPA, 2002), the size of the sand filter required is about 500 m$^2$ (25 m x 20 m) with a depth of less than 1 m. The sand media is about 0.25 mm in diameter. The dosing tank volume would be about ½ times the volume of septic tank effluent which is about 30 m$^3$, dosed between 12 – 24 times daily with 76 mm (3 inch) diameter pipes.

The subsurface flow wetlands are anaerobic in function, connected in series: the horizontal and the vertical flow constructed wetland. The vertical flow wetland is the component by which the septic tank effluent is dispersed into the ground water. The organic loading rate is 75 kgBOD/ha.d (Wood, 1995) which gives an area of 0.05 ha (500 m$^2$) for two beds of 18 m x 14 m each. The two staged wetland can obtain the following contaminant removal 97% TSS, 95% N, 97% BOD, 93% COD, 99% E coli, and 47% P removals (Shrestha et al., 2001). The effect of rainfall and evaporation in the subsurface wetland were considered by checking for surfacing. The media depth required to prevent surfacing is about 0.65 m

The digestion process is mesophilic at a temperature of about 35°C which is suitable for the study area. The substrates used is settled wastewater sludge and organic fraction of municipal solid waste with waste vegetables from the nearby market. The mix proportion of the
substrate is 75% to 25% respectively resulting to 220 kg of organic matter. The digester size requirement for the study area based on primary sludge production of 30 l/p.d (Gillberg et al., 2003) is 50 m$^3$. Rueil-Malmaison (1991) also specifies a biogas production of 500 L/organic matter input. Hence the expected biogas production will be 110 m$^3$ with methane proportion of 80 m$^3$ (70%). From energy potential of 10.4 KWh/m$^3$ for methane (Davidsson et al., 2008) the amount of electric power that can be produced from 37% of the yearly methane production is 112000 KWh. This can satisfy the yearly electric power consumption of the study area. On the other hand, the per person consumption of biogas in the study area for cooking is 0.26 m$^3$/d (Adeoti et al., 2000, De Alwis A., 2002) with a total of 400 m$^3$/d for 1656 persons. The extra biogas from the anaerobic digestion process will satisfy up to 20% of the daily cooking energy need of the study area or more if electricity generation is only in the event of power outage from the national grid. The electricity generator capacity required will be 300 KWh. In the anaerobic digestion, there is 20% solids and 40 – 70% organic matter reduction (USEPA, 2002, Gillberg et al., 2003, Hammer and Hammer, 2004). The study area needs storage or further treatment for biosolids from anaerobic digestion and this can be done in an invessel composting unit. COD and N from digester are 60 kg and 0.3 kg respectively. For a C/N ratio of 30 required for composting, additional nitrogen can be inputted as amendment through the combined use of manure and saw dust as bulking agent. The size of the composting unit required for a retention time of 21 days and saw dust (bulking agent) to biosolid ratio of 0.8:1 is about 30 m$^3$ with a total solid content of 200 kg.

Figure 3 below is the mass balance for TS

**Figure 3: Mass balance of TS in the design system**

<table>
<thead>
<tr>
<th>TS mass balance</th>
<th>120l/p.d x 1656p = 200m$^3$</th>
<th>200gTS/p.d x 1656p = 330kg/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedimentation tank 1</td>
<td>380kg TS</td>
<td>70m$^3$</td>
</tr>
<tr>
<td>15% (50kg)</td>
<td>Sludge withdrawal</td>
<td>70% (200kg)</td>
</tr>
<tr>
<td>Anaerobic Digester (50%)</td>
<td>250kg TS</td>
<td>20% TS (50kg TS)</td>
</tr>
<tr>
<td>Biogas</td>
<td>80% TS biosolid</td>
<td></td>
</tr>
<tr>
<td>200kg TS</td>
<td>Invessel composting Unit (30m$^3$)</td>
<td></td>
</tr>
</tbody>
</table>

**Rainwater harvesting potential**

Due to fresh water shortage in the study area, rainwater can be harvested from the roofs. The total roof area and yearly rainfall value are 4,500 m$^2$ (google earth) and 2200 mm respectively. The rainfall harvest estimated from the rational method is 28000 m$^3$ per year. This can satisfy up to 45% non potable water need. The required storage of harvested rainfall per building will be about 1- 2 m$^3$. This is done above ground and alum added as a coagulant.

**Conclusion**

- Improvement of decentralised wastewater treatment is feasible with the choice of appropriate technology in developing countries.
- Theoretically with the reuse of wastewater sludge and organic fraction of municipal waste in the biogas production, there can be electricity generation that would meet the
yearly per person consumption and an additional 20% daily cooking requirement in
the study area reducing dependence on kerosene.

- Mass balance shows that combined sedimentation, sand filtration and constructed
  subsurface wetland treatment can effectively reduce major contaminant level in
  domestic wastewater and enable safe groundwater discharge while the sludge can be
  composted and used for agriculture or land improvement.
- Rainwater harvesting has potential of reducing 45% demand on the limited freshwater
  supply, meeting non potable need such as toilet flushing.
- Community involvement, information and education in study area and centralised
  management are important in the successful implementation and sustainability of the
  decentralised improvement proposed.

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