Engineering and Technology Journal
Journal homepage: https://etj.uotechnology.edu.iq

# Design of waste stabilisation ponds for municipal effluent treatment in Sobi cantonment, Ilorin, Nigeria

**Francis James Ogbozige**

Department of Civil Engineering, Federal University Otuoke, Nigeria
*Corresponding author Email: ogbozigefj@fuotuoke.edu.ng

## HIGHLIGHTS

- The sludge accumulation rate at the primary anaerobic pond is 135 $m^3$ per year, requiring a desludging frequency of 2.8 years.
- The retention time of the anaerobic, facultative, and maturation ponds were 3.9 days, 10.7 days and 8 days respectively.
- The designed WSP would highly comply with both WHO and FEPA effluent discharge standards regarding BOD and faecal coliform.

## ARTICLE INFO

**Handling editor:** Qusay F. Alsalhy

**Keywords:**
Anaerobic
Facultative
Maturation
Ponds
Wastewater

## ABSTRACT

The generation of wastewater or effluent is unavoidable as long as humans exist, and it could cause potential danger if not properly managed. One way to manage it is by ensuring proper treatment before discharging into rivers, streams, and creeks. This will not just protect human health but will also protect aquatic fauna and flora. Thus, this research applied scientific and engineering principles to design a waste stabilization pond (WSP) to treat municipal effluent generated in a large military community (Sobi Cantonment), which currently lacks a wastewater treatment plant. The research revealed that the WSP will comprise two anaerobic ponds, one facultative pond, and two maturation ponds, all arranged in series with the addition of two (in series) anaerobic ponds as standby, arranged in parallel with the other set of anaerobic ponds. The research further revealed that the sludge accumulation rate at the primary anaerobic pond is 135 $m^3$ per year, requiring a desludging frequency of 2.8 years. However, the retention time of the anaerobic, facultative, and maturation ponds were 3.9 days, 10.7 days, and 8 days, respectively. Also, the diameter of the inlet and outlet pipes was designed to be 3 inches. while the required land area (including additional space for personnel accessibility during maintenance) is approximately one hectare. Both orthographic and isometric drawings of the WSP were provided in detail. It was concluded that the designed WSP would highly comply with both WHO and FEPA effluent discharge standards regarding BOD and faecal coliform.

## 1. Introduction

Proffering technical solutions to protect humans from adverse environmental effects, protect the ecosystem, and improve the quality of the environment through applied scientific and designed principles has always been the focus of environmental engineers. Untreated wastewater or sewage discharged into the environment (rivers, streams, creeks, etc.) has long been identified of causing adverse effects on humans and aquatic life [1,2]. This is because such sewage contains high concentrations of biodegradables, which could reduce the dissolved oxygen content of the receiving water bodies due to the high oxygen demand usually needed to oxidize the biodegradables either biologically or chemically. The nutrients in the wastewater could also cause algal bloom, which could lead to eutrophication of the receiving water bodies and the associated consequences on both aquatic lives and people using the water downstream. Among the different sources of wastewater, municipal wastewater, especially in developing countries, are mostly known for not being treated before discharge. This could be due to numerous factors such as inadequate or non-availability of sewerage network systems linking residential houses to central treatment plants, relevant monitoring agencies mostly focusing on industrial and institutional wastewater without bothering about municipal wastewater, unavailability of functional waste treatment plants, etc.

The treatment of wastewater can be done in a decentralized or centralized system [3]. The decentralized system involves onsite treatment, mostly applicable in poorly planned or zoned communities. In contrast, the centralized system is usually carried out on a large scale, especially for properly planned communities where a sewerage network links the various households to a central treatment plant [4–7]. Thus, the centralized system is more appropriate for Sobi cantonment in Ilorin town of Kwara state (Nigeria) since it is properly planned. Waste stabilization ponds (WSP) are one of such centralized



http://doi.org/10.30684/etj.2024.146992.1698
Received 18 February 2024; Received in revised form 08 March 2024; Accepted 21 March 2024; Available online 05 April 2024
2412-0758/University of Technology-Iraq, Baghdad, Iraq

treatment plants. It is a secondary or biological mode of treating wastewater meant to be discharged into flowing water bodies (rivers, streams, creeks) just as trickling filters (TF) and activated sludge process (ASP).

Generally, waste stabilization ponds are constructed in earthwork such that the depths are relatively small compared to their large surface areas, with continuous inflow and outflow of wastewater. Runoff entry into the ponds is excluded by building embankments to certain heights around the ponds. The ponds are usually classified as anaerobic, facultative, and maturation ponds (depending on the depth and mechanism of purification) and are arranged in series. It is well known that WSP is more appropriate than TF and ASP in developing countries due to the ease of maintenance by unskilled labour, lower initial and maintenance costs, and higher treatment efficiency and reliability. In addition, WSP functions favourably in hot climates associated with high wind speed and sunlight intensity, which are very common in most communities in developing countries, including Ilorin town, where the Sobi Cantonment is located. Hence, this research focused on designing waste stabilization ponds to treat municipal wastewater generated in Sobi Cantonment in North-Central Nigeria since there is no existing treatment plant at the moment.

## 2. Materials and methods

### 2.1 Study area

Sobi cantonment being the study area is located in Ilorin East Local Government Area of Kwara state in North-Central Nigeria, as seen in Figure 1. It lies between Latitude 8°32'56.01"N to 8°34'37.12"N and Longitude 4°32'27.36"E to 4°33'58.50"E, covering an area of 6.2 km$^2$ with surface elevation ranging from 279 – 364 m above mean sea level.

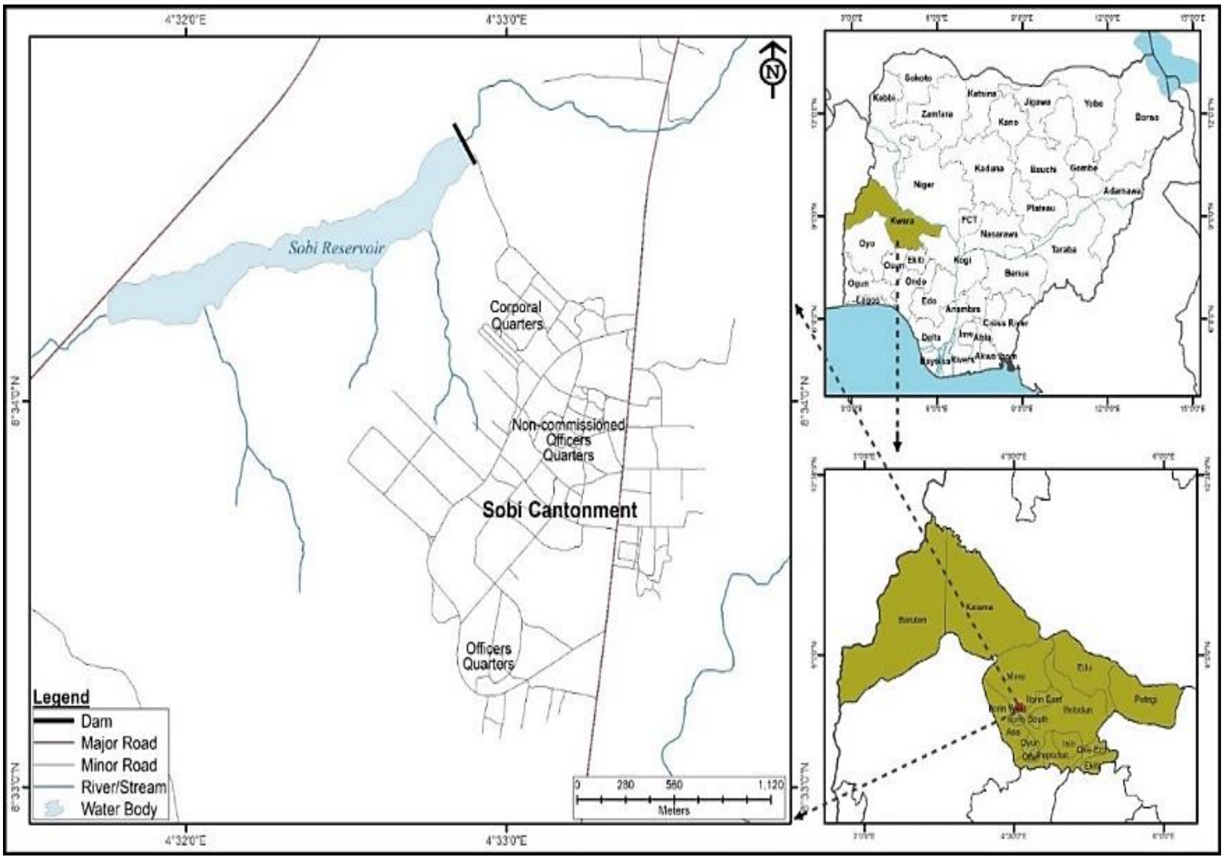


**Figure 1:** Map of the study area

The temperature varies from a minimum value of 21 ℃ during December to a maximum value of 32 ℃ during March, while the monthly rainfall varies from 1.5 mm during January to 227.3 mm during September (although climate change has made it inconsistent). However, the average relative humidity is 60% [8]. Some notable independent military units located inside the study area include the 22 Armoured Brigade, the 222 Battalion, the Nigerian Army College of Education (NACOE), and the Nigerian Army Institute of Science Education and Technology (NAISET) [9].

### 2.2 Design of WSP components

In cities, towns, and villages, the population growth is usually projected based on the design period of the facility, using different forecasting methods (especially the geometric growth model). However, a military cantonment is different because the housing units were designed for a certain number of persons. Hence, the population of persons (including civilians) expected to live in the cantonment based on the housing units' design was gotten, and the value was multiplied by a safety factor for future expansion. Thereafter, a standard per capita water demand was adopted to estimate the effluents (wastewater) generated. The various effluent generation sources in the cantonment were identified, and the source with the highest organic load (BOD) was selected for the design. main components of WSP being anaerobic pond, facultative pond, and maturation pond were designed based on the criteria shown in Table 1 as reported in previous related researches [10–12]. However, the geometry of the ponds was designed based on the cross-section shown in Figure 2, which includes a freeboard space embankment above the ground surface to exclude runoff entry.

**Table 1:** Design criteria for WSP in tropical climates

| Parameter | Type of pond | | |
|---|---|---|---|
| | **Anaerobic** | **Facultative** | **Maturation** |
| Depth (m) | 2 – 5.0 | 1.0 – 2.0 | 1.0 – 1.5 |
| Detention time (day) | 3 – 6 | 5 – 30 | 5 – 18 |

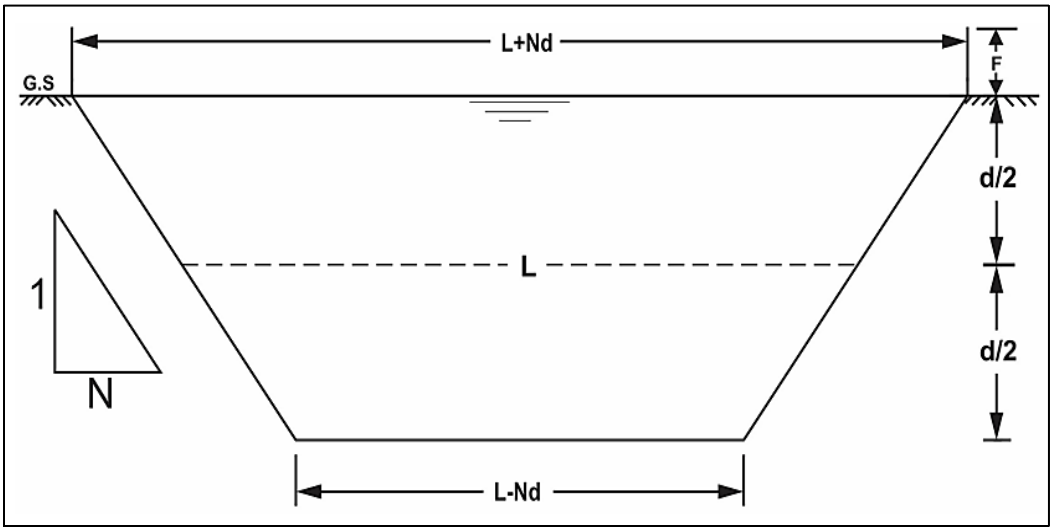


**Figure 2:** Cross-section of ponds with freeboard space above ground surface

In Figure 2, $d$ represents the depth of the pond, $L$ is the length of the pond at mid-depth, $N$ is the horizontal distance corresponding to a unit depth of the side slope, and $F$ is the freeboard space above the ground surface. Usually, the freeboard space is 0.5 m for ponds less than 1.0 hectare and between 0.5 to 1.0m for ponds ranging from 1.0 and 3.0 hectares; however, for larger ponds, Equation (1) is used in determining the freeboard space [12]. It should also be known that as $L$ in Figure 2 represents the length of the pond at mid-depth, it implies the associated breadth *(B*) and area ($A$) for such shape are also meant for mid-depth. Hence, the top and bottom dimensions for the selected shape were gotten using the information in Figure 2 with respect to top and bottom lengths, expressed as $L + Nd$ and $L - Nd$ respectively.

$$F = \sqrt{\log A} - 1 \tag{1}$$

where $F$ is the freeboard space in meters (m) for ponds larger than 3.0 hectares while $A$ is the pond top surface area in square meters (m$^2$).

### 2.2.1 Anaerobic pond

The volume of the anaerobic pond was designed through Equations (2) and (3), while the detention time was obtained through Equation (4). Thereafter, the required land area was determined by dividing the designed volume by a convenient depth based on the design criteria shown in Table 1 and considering the geometry of Figure 2 earlier explained. The pond's dimensions were then determined after adopting the plane shape of the pond.

$$V_a = \frac{\lambda_{oa}}{\lambda_{va}} \tag{2}$$

$$\lambda_{oa} = (L_{ia})Q \tag{3}$$

$$t_a = \frac{V_a}{Q} \tag{4}$$

where: $V_a$ is the volume of the anaerobic pond in cubic meters (m$^3$), $\lambda_{oa}$ is the organic loading rate of BOD in an anaerobic pond expressed in kg/day, $\lambda_{va}$ is the volumetric loading rate of BOD in an anaerobic pond measured in kg.m$^{-3}$ day$^{-1}$ expressed in Table 2 [11], $L_{ia}$ is the influent BOD to the anaerobic pond (i.e., BOD of raw wastewater), while $Q$ and $t_a$ are the daily flow rate of wastewater and retention time, respectively. The efficiency of BOD removal from the pond was determined using the information given in Table 3, as reported in previous related research [11].

The desludging of the anaerobic pond was designed based on the time required for the sludge layer to reach one-third of the pond's depth [13]. This was achieved by assuming a standard sludge accumulation rate.

**Table 2:** Recommended volumetric loadings for anaerobic pond design

| Mean ambient temperature of the coldest month, $T$ (°C) | Permissible volumetric loading rate of BOD, $\lambda_{va}$ (kg.m$^{-3}$day$^{-1}$) |
|---|---|
| <10 | 0.10 |
| 10 – 20 | 0.020$T$ – 0.10 |
| 20 – 25 | 0.010$T$ + 0.10 |
| >25 | 0.35 |

**Table 3:** Efficiencies of BOD removal in the anaerobic pond

| Mean ambient temperature of the coldest month, $T$ (°C) | BOD removal efficiency, $\eta_a$(%) |
|---|---|
| 10 – 25 | 2$T$ + 20 |
| >25 | 70 |

### 2.2.2 Facultative pond

The facultative pond was designed based on the surface organic loading rate criterion, expressed in Equation (5). However, the area of land required for excavation was determined through Equations (6) and (7) in accordance with the geometry of Figure 2 while the detention time was determined through Equations (8) based on the report of a previous related reserch [14].

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25} \tag{5}$$

$$A_f = \frac{\lambda_{of}}{\lambda_{sf}} \tag{6}$$

$$\lambda_{of} = (L_{if})Q \tag{7}$$

$$t_f = \frac{A_{f(d_f)}}{Q} \tag{8}$$

where $\lambda_{sf}$ is the surface loading rate of BOD in facultative pond measured in kg.ha$^{-1}$day$^{-1}$, $T$ is the mean ambient temperature of the coldest month in $^{o}$C, $A_f$ is the area of facultative pond in m$^2$, $\lambda_{of}$ is the organic loading rate of BOD in the facultative pond in kg day$^{-1}$. However, $L_{if}$ and $d_f$ represent influent BOD to the facultative pond and detention period in the facultative pond respectively, while $Q$ remains the daily wastewater flow rate.

### 2.2.3 Maturation pond

The maturation ponds were specifically designed to reduce the number of faecal coliforms (fc) to comply with effluent discharge standards, using Equations (9) and (10).

$$k_T = 2.6(1.19)^{T-20} \tag{9}$$

$$N_e = \frac{N_i}{\{[1+k_T(t_a)][1+k_T(t_f)][1+k_T(t_m)]^n\}} \tag{10}$$

where: $k_T$ is the first-order rate constant of faecal coliform removal (day$^{-1}$), $T$ remains the mean temperature of the coldest month ($^{o}$C), $N_e$ is the number of faecal coliforms per 100 mL of the effluent leaving the maturation pond (usually taken as effluent discharge standard), $N_i$ is the number of faecal coliforms per 100 mL of influent (raw water), $n$ is the number of maturation ponds while $t_a$, $t_f$ and $t_m$ are the detention times in anaerobic, facultative, and maturation ponds, respectively.

### 2.2.4 Size of inlet and outlet pipes

The size of the sewer pipes was designed such that self-cleansing of the pipes is maintained while preventing scouring of th The size (diameter) of the pipes was determined by applying a continuity equation that relates discharge, cross-sectional area, and flow velocity. After that, it was ensured that the calculated size or diameter of the sewer pipe was approximated to the nearest value that is obtainable in the market.

## 2.3 Land requirement and location of WSP

The land required for the construction of the fascility was calculated so that adequate space was made available for the movement of personnel within the vicinity during maintenance. However, the site was selected in line with standard practice which includes geotechnical, accessibility and topographic considerations as well as prevailing wind direction and distance from residential areas.

## 3. Results and discussion

In this section, the dimensions of various components of the facility (WSP), the designed quality of the final effluents as well as the selection of site were determined in accordance with existing standards and compared with past literatures.

## 3.1 Design of anaerobic pond

### 3.1.1 Geometry of anaerobic pond

$Expected\ pupulation = 2000$ [9]. By adopting a safety factor of 1.5 to cater for future expansion, it implies;
$Design\ population = 1.5(2000) = 3000$
$Per\ capita\ water\ demand = 120\ Lday^{-1}$ [15]
$Total\ water\ demand\ by\ designed\ population = 3000 \times 120Lday^{-1} = 360000L\ day^{-1}$
Usually, 80 – 90% of water demand appear as effluent [16] hence by adopting 80%,
$Flowrate\ of\ wastewater\ (Q) = 0.8(360000L.day^{-1}) = 288000Lday^{-1} = 288\ m^3day^{-1}$

The identified possible sources of effluent generation were kitchens, bathrooms, toilet rooms, slaughterhouses (abattoirs), and clinics. Among these effluent sources, abattoir effluents are known to be more oxygen-demanding. Hence, the average BOD of abattoir wastewater 1209 mg/L [17, 18] was adopted for the design. i.e.
$\mathrm{BOD\ of\ raw\ wastewater\ (L_{ia}) = 1209\ mgL^{-1} = 1.209\ kg\ m^{-3}}$
$\mathrm{Total\ organic\ loading\ rate\ of\ BOD\ (\lambda_{oa}) = (L_{ia})Q = \frac{1.209\ kg}{m^3} \times \frac{288\ m^3}{day} = 348.192\ kg\ day^{-1}}$

The mean temperature of the coldest month, January, was found to be 21 °C [8]. Hence, the volumetric loading rate $(\lambda_{va})$ was calculated based on the information in Table 2 as follows;
$(\lambda_{va}) = 0.010(21) + 0.10 = 0.31\ kg.m^{-3}day^{-1}$
$\therefore Volume\ of\ anaerobic\ pond\ (V_a) = \frac{\lambda_{oa}}{\lambda_v} = \frac{348.192\ kg.day^{-1}}{0.31\ kg.m^{-3}day^{-1}} = 1123.2\ m^3$

By adopting a depth $(d_a)$ of 3.5 m for the anaerobic pond since it ranged from 2 – 5 m, it implies the required area of land to be excavated at mid-depth is:
$\mathrm{Area\ of\ anaerobic\ pond\ (A_a) = \frac{volume\ of\ pond}{depth} = \frac{V_a}{d_a} = \frac{1123.2\ m^3}{3.5\ m} \cong 321\ m^2}$

A rectangular shape was adopted for the pond's surface area such that the ratio of the length ($L_a$) to breadth ($B_a$) at mid-depth is 3:1. In other words, $L_a = 3B_a$.
$\mathrm{=> Area\ of\ anaerobic\ pond\ at\ mid\ depth\ (A_a) = L_a(B_a) = 3B_a(B_a) = 3B_a{}^2}$
$\mathrm{=> B_a = \sqrt{\frac{A_a}{3}} = \sqrt{\frac{3201\ m^2}{3}} = 10.34\ m}$
$\mathrm{L_a = 3B_a = 3(10.34\ M) = 31.02\ m}$

A side slope of 1 vertical to 1 horizontal (i.e., $N = 1$) was adopted for all the different ponds hence, the top surface and bottom or base dimensions are given below;

1) Top surface dimensions for anaerobic ponds
$\mathrm{(Length_{top})_a = L_a + N(d_a) = 31.02 + 1(3.5) = 34.52\ m}$
$\mathrm{(Breadth_{top})_a = B_a + N(d_a) = 10.34 + 1(3.5) = 13.84\ m}$
2) Bottom surface dimensions for anaerobic ponds
$\mathrm{(Length_{bottom})_a = L_a - N(d_a) = 31.02 - 1(3.5) = 27.52\ m}$
$\mathrm{(Breadth_{bottom})_a = B_a - N(d_a) = 10.34 - 1(3.5) = 6.84\ m}$

Since the top surface area of 477.77 $m^2$ (i.e., 34.52 m × 13.84 m) is less than 1.0 hectares (10,000 $m^2$), it implies a freeboard space of 0.5 m high earthwork embankment should be built around the pond above the ground level while the depth of the pond below the ground surface should be 3.5 m. In addition, the top surface area is less than 0.5 hectares (5000 $m^2$), which implies the designed area need not be split into two or more ponds. However, another anaerobic pond of the same dimension has to be provided in parallel as standby during maintenance, such as desludging.

### 3.1.2 Detention time in the anaerobic pond

$\mathrm{Detention\ time\ in\ anaerobic\ pond\ (t_a) = \frac{V_a}{Q} = \frac{1123.2\ m^3}{288\ m^3day^{-1}} = 3.9\ days}$, and the designed detention time of 3.9 days in the anaerobic pond is within the 3 – 6 days recommended by a previous research [12].

### 3.1.3 BOD of effluent leaving the anaerobic pond

Since the mean ambient temperature of the coldest month is 21 °C, the efficiency of BOD removal from the anaerobic pond $(\eta_a)$ was estimated based on the information in Table 3 as follows:
$\mathrm{\eta_a = 2T + 20 = 2(21) + 20 = 62\%}$.

However, a designed value of 60% (i.e., 0.6) was adopted for safety purposes in case of temperature variation in future years. Hence, the BOD of the effluent leaving the anaerobic pond will be (1 – 0.6) of raw wastewater BOD (i.e. 0.4 of 1209 mg $L^{-1}$) = 483.6 mg $L^{-1}$ which is greater than 300 mg $L^{-1}$, hence too high to be treated in a Facultative pond. Therefore, a second anaerobic pond of the same dimension should be constructed in a series arrangement so that the effluent leaving the second anaerobic pond will be 40% of 483.6 mg $L^{-1}$ which is 193.44 mg $L^{-1}$ or 0.19344 kg $m^{-3}$. Notwithstanding, a duplicate of the two anaerobic ponds has to be arranged in parallel as standby for maintenance, such as desludging, as earlier stated.

### 3.1.4 Desludging frequency in the anaerobic pond

The annual accumulation rate of sludge in anaerobic ponds ranged from 0.03 – 0.10 $m^3$ per inhabitant per year, with the lower range common in hot climates or tropical regions [13]. Adopting an accumulation rate of 0.045 $m^3$ per inhabitant per person implies:

$$\text{Total annual accumulation of sludge} = \left(\frac{0.045\ \text{m}^3}{\text{inhabitant.year}}\right) 3000\ \text{inhabitants} = \frac{135\ \text{m}^3}{\text{year}}$$

$$\text{Annual thickness of sludge} = \frac{\text{total anual accumulation rate}}{\text{area of pond}} = \frac{135\ \text{m}^3\ \text{year}^{-1}}{321\ \text{m}^2} = 0.42\ \text{m. year}^{-1}$$

$$\text{Time for sludge to reach } 1/3 \text{ of pond's depth} = \frac{\frac{\text{depth of pond}}{3}}{\text{yearly thickness of sludge}} = \frac{3.5\ \text{m}/3}{0.42\ \text{m year}^{-1}} \cong 2.8\ \text{years}$$

The volume of accumulated sludge during this period of 2.8 years is 378 $m^3$ (i.e. 135 $m^3 year^{-1}$ × 2.8 years). Hence, 378 $m^3$ of sludge should be removed every 2.8 years (approximately 2 years, 10 months), or 135 $m^3$ should be removed annually.

## 3.2 Design of facultative pond

### 3.2.1 Geometry of facultative pond

$$\lambda_{sf} = 350(1.107 - 0.002T)^{T-25} = 350(1.107 - 0.002(21))^{21-25} = 272\ \text{kg ha}^{-1}\text{day}^{-1}$$

The influent BOD to the facultative pond will be the same as the BOD of effluent leaving the last (second) anaerobic pond, which is 193.44 mg $L^{-1}$ or 0.19344 kg $m^{-3}$; hence,, the organic loading rate of BOD in the facultative pond $(\lambda_{of})$ becomes:

$$\lambda_{of} = (L_{if})Q = (0.19344\ \text{kg m}^{-3})288\ \text{m}^3\ \text{day}^{-1} = 55.71\ \text{kg day}^{-1}$$

$$\text{Area of facultative pond } (A_f) = \frac{\lambda_{of}}{\lambda_{sf}} = \frac{55.71\ \text{kg day}^{-1}}{272\ \text{kg ha}^{-1}\ \text{day}^{-1}} = 0.2048\ \text{ha} = 2048\ \text{m}^2$$

By adopting a rectangular shape and ensuring that the length ($L_f$) and breadth ($B_f$) at mid-depth of the facultative pond is in a ratio of 3:1, then it becomes:

$$B_f = \sqrt{\frac{A_f}{3}} = \sqrt{\frac{2048\ \text{m}^2}{3}} = 26.13\ \text{m}$$

$$L_f = 3(B_f) = 3(26.13\ \text{m}) = 78.39\ \text{m}$$

A design depth of 1.5 m was adopted since the depth of facultative pond $(d_f)$ ranged from 1 – 2 m [11]. Hence, the top and bottom dimensions are given below:

1) Top surface dimensions for facultative pond

$$\left(\text{Length}_{top}\right)_f = L_f + N(d_f) = 78.39 + 1(1.5) = 79.89\ \text{m}$$

$$\left(\text{Breadth}_{top}\right)_f = B_f + N(d_f) = 26.13 + 1(1.5) = 27.63\ \text{m}$$

2) Bottom surface dimensions for facultative pond

$$(\text{Length}_{bottom})_f = L_f - N(d_f) = 78.39 - 1(1.5) = 76.89\ \text{m}$$

$$(\text{Breadth}_{bottom})_f = B_f - N(d_f) = 26.13 - 1(1.5) = 24.63\ \text{m}$$

Since the top surface area of 2207.36 $m^2$ (i.e., 79.89 m × 27.63 m) is smaller than a hectare of land (10,000 $m^2$), it implies a freeboard space of 0.5 m high embankment should be built above the ground surface around the pond. However, the entire depth of the pond below the ground surface should be 1.5 m. In addition, a single facultative pond will be required since the surface area is less than 0.5 hectares.

### 3.2.2 Detention time in the facultative pond

The detention time in the pond $(t_f)$ becomes;

$$t_f = \frac{A_{f(d_f)}}{Q} = \frac{2048\ \text{m}^2(1.5\ \text{m})}{288\ \text{m}^3\ \text{day}^{-1}} \cong 10.7\ \text{days},$$ within the range (5 – 30 days) recommended in past literature [10].

### 3.2.3 BOD of effluent leaving the facultative pond

The efficiency of facultative ponds in removing BOD in tropical climates ranged between 70 – 80% [19 – 21]; hence, 70% (i.e., 0.7) was adopted for this design. i.e., $\text{BOD of effluent leaving facultative pond} = (1 - 0.7)L_{if} = 0.3(193.44\ \text{mg L}^{-1}) = 58.03\ \text{mg L}^{-1}$

## 3.3 Design of maturation pond

### 3.3.1 Detention time and number of maturation ponds in series

The first order rate constant $(k_T)$ for faecal coliform removal in the maturation pond was determined as:

$$k_T = 2.6(1.19)^{T-20} = 2.6(1.19)^{21-20} = 3.09\ \text{day}^{-1}$$

For the maturation pond to function satisfactorily in reducing bacteria, the detention time in the pond $(t_m)$ must be less than that of the facultative pond $(t_f)$ but greater than the minimum retention time in the maturation pond being 3 days [12]. Since the detention time in the facultative pond is 10.7 days, it implies 3 days < $t_m$ < 10.7 days thus, 8 days was adopted for the retention time $(t_m)$. The required effluent discharge standard for faecal coliform (fc) by the Nigerian Federal Environmental Protection Agency (FEPA) is 400 fc/100 mL. However, the concentration of faecal coliform in raw wastewater for the purpose of designing WSP is usually taken as $1 \times 10^8$ fc/100 mL [12]. Since the detention times in the anaerobic and facultative ponds are respectively 3.9 days and 10.7 days, the number of maturation ponds $(n)$ was determined as follows:

$$N_e = \frac{N_i}{\{[1+k_T(t_a)][1+k_T(t_f)][1+k_T(t_m)]^n\}} = \frac{1\times10^8}{\{[1+3.09(3.9)][1+3.09(10.7)][1+3.09(8)]^n\}} = \frac{1\times10^8}{444.556[25.72]^n}$$

$$=> 400 = \frac{1\times10^8}{444.556[25.72]^n} = \frac{224943.54}{25.72^n}$$

$$\therefore 25.72^n = \frac{224943.54}{400} = 562.36$$

$$=> n = \log_{25.72} 562.36 = \frac{\log 562.36}{\log 25.72} = 1.95 \cong 2$$

Thus, using two (2) maturation ponds in series will yield an effluent concentration of 340fc/100 mL, which is better as it is even less than 400 fc/100 mL when $n$ is 1.95.

### 3.3.2 Geometry of maturation pond

The depth of maturation ponds $(d_m)$ usually range from 1 – 1.5 m [11]; hence, a 1.4 m depth was adopted. Thus, the area of land for each maturation pond $(A_m)$ was determined as follows:

$$\mathrm{A_m} = \frac{\mathrm{(Q)t_m}}{\mathrm{d_m}} = \frac{(288\ \mathrm{m^3 day^{-1}})8\mathrm{day}}{1.4\ \mathrm{m}} = 1645\ \mathrm{m^2}$$

A rectangular shape was adopted for the surface area, and the ratio of length ($L_m$) to breadth ($B_m$) at the mid-depth of the pond was made 3:1. Hence, the dimensions of the length and breadth at mid-depth become;

$$\mathrm{B_m} = \sqrt{\frac{\mathrm{A_m}}{3}} = \sqrt{\frac{1645\ \mathrm{m^2}}{3}} = 23.42\ \mathrm{m}$$

$$\mathrm{L_m} = 3(\mathrm{B_m}) = 3(23.42\ \mathrm{m}) = 70.26\ \mathrm{m}$$

1) Top surface dimensions for maturation ponds

$$(\mathrm{Length_{top}})_\mathrm{m} = \mathrm{L_m} + \mathrm{N(d_m)} = 70.26 + 1(1.4) = 71.66\ \mathrm{m}$$

$$(\mathrm{Breadth_{top}})_\mathrm{m} = \mathrm{B_m} + \mathrm{N(d_m)} = 23.42 + 1(1.4) = 24.82\ \mathrm{m}$$

2) Bottom surface dimensions for maturation ponds

$$(\mathrm{Length_{bottom}})_\mathrm{m} = \mathrm{L_m} - \mathrm{N(d_m)} = 70.26 - 1(1.4) = 68.86\ \mathrm{m}$$

$$(\mathrm{Breadth_{bottom}})_\mathrm{m} = \mathrm{B_m} - \mathrm{N(d_m)} = 23.42 - 1(1.4) = 22.02\ \mathrm{m}$$

The top surface area (i.e., 71.66 m × 24.82 m = 1778.60 $m^2$) is not up to a hectare of land (10,000 $m^2$); thus, a freeboard space of 0.5 m high above the ground surface should be provided by building a barrier around the ponds with earth materials. A parallel set of maturation ponds will not be required since the surface area for each pond is less than 0.5 hectares. Besides, maintenance such as desluding is hardly noticed in maturation ponds.

### 3.3.3 BOD of effluent leaving maturation pond

Although maturation ponds are mainly designed to remove faecal coliform, notwithstanding, some level of BOD is beign removed. There is no precise design equation for BOD removal in maturation pond. However, a maturation pond designed to produce an effluent of less than 1000 fc/100 mL will produce an effluent containing a BOD of less than 25 mg $L^{-1}$ [12]. Since the effluent leaving the last maturation pond is designed to have faecal coliform (fc) concentration of 340fc/100 mL, which is much lesser than 1000 fc/100 mL, it implies the designed facility (WSP) will surely produce effluent that will comply with both FEPA and WHO BOD discharge standard of 30 mg $L^{-1}$. Besides, the designed influent BOD in the first maturation pond was small.

### 3.3.4 Size of inlet and outlet pipes

The self-cleaning velocity for sewer pipes ranges from 0.6 – 0.9 m $s^{-1}$; hence, the flow velocity of the raw wastewater was assumed to be 0.70 m $s^{-1}$ to permit self-cleansing of pipes and avoid scoring of the pipes. Since the facility (WSP) is designed to treat municipal effluent, it was assumed that the daily flow would occur within the 24 hours of the day. From the continuity equation, Discharge = Area × velocity. Hence:

$$\text{Daily discharge in pipes} = \frac{\text{daily volume of raw wastewater}}{\text{24hours duration of flow}} = \frac{288\ \mathrm{m^3}}{24\times60\times60\ \text{seconds}} = 0.0033\ \mathrm{m^3\ s^{-1}}$$

$$\text{Cross section area of pipes } (\mathrm{A_p}) = \frac{\text{Discharge in pipes}}{\text{flow velocity in pipes}} = \frac{0.0033\ \mathrm{m^3\ s^{-1}}}{0.70\ \mathrm{ms^{-1}}} = 4.7143\times10^{-3}\ \mathrm{m^2}$$

$$\text{Since Area of circular pipe} = \frac{\pi D^2}{4}\ \text{[where D is diameter, } \pi \text{ is 3.142]}$$

$$=> \text{Diameter of pipes } (\mathrm{D_p}) = \sqrt{\frac{4(\mathrm{A_p})}{\pi}} = \sqrt{\frac{4(4.7143\times10^{-3}\ \mathrm{m^2})}{3.142}} = 0.077\ \mathrm{m} = 77\ \mathrm{mm} \cong 3\text{inches}$$

The inlet pipes to the ponds should be laid to discharge below the liquid surface at full level (possibly at mid-depth) to reduce short-circuiting, while the outlet pipes should be at the level of the designed liquid surface at full level to ensure the complete detention time.

The designed dimensions were converted to millimeters (mm) and used to produce the orthographic and isometric drawings of the facility (WSP) through 2023 AutoCAD software, as shown in Figure 3 and Figure 4 respectively.

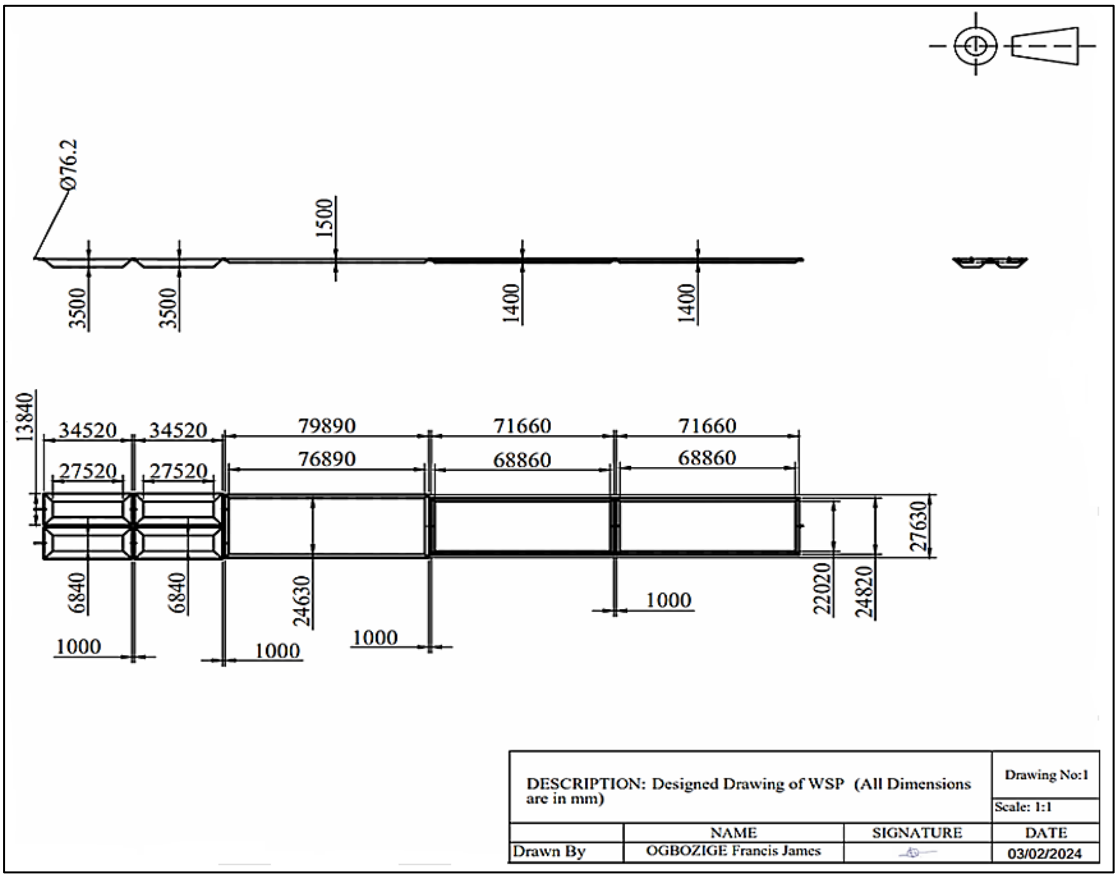


**Figure 3:** Orthographic drawing (3rd angle) of designed WSP

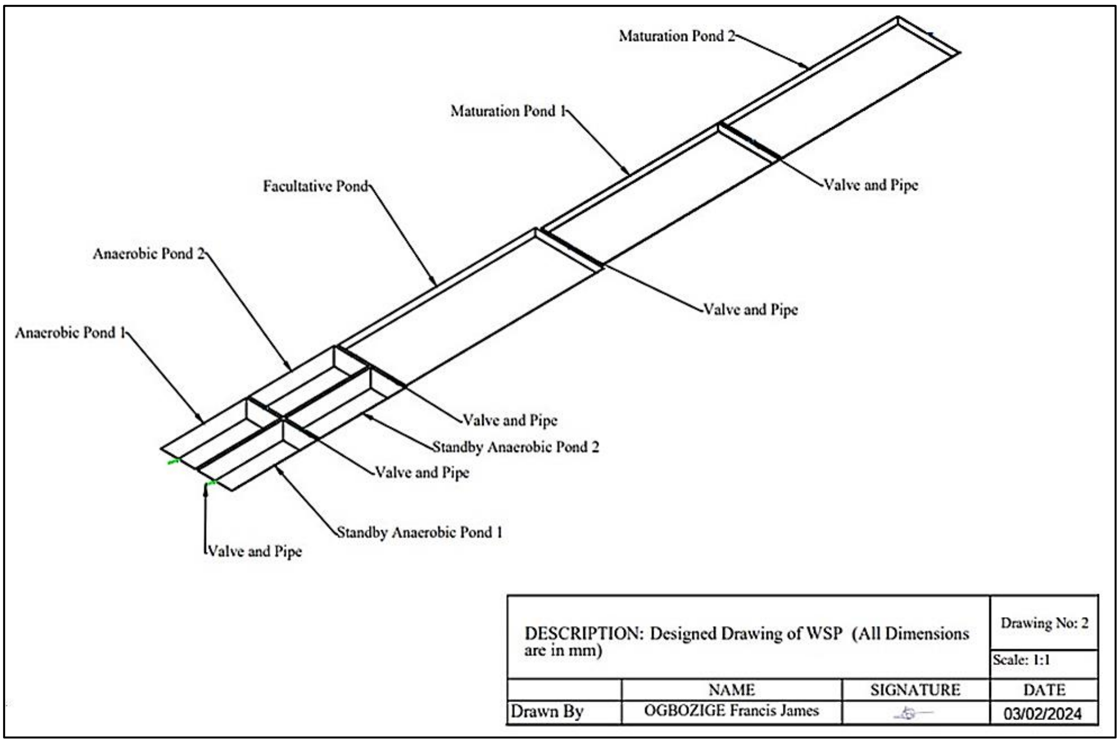


**Figure 4:** Isometric drawing of designed WSP

## 3.4 Total land requirement and site selection

The total area of land needed for the WSP is a summation of all the top surface areas of the different number of designed ponds (i.e., 4 anaerobic, 1 facultative, and 2 maturation ponds) shown in Figure 5, with an additional space usually 25% for accessibility or movement of personnel during maintenance.

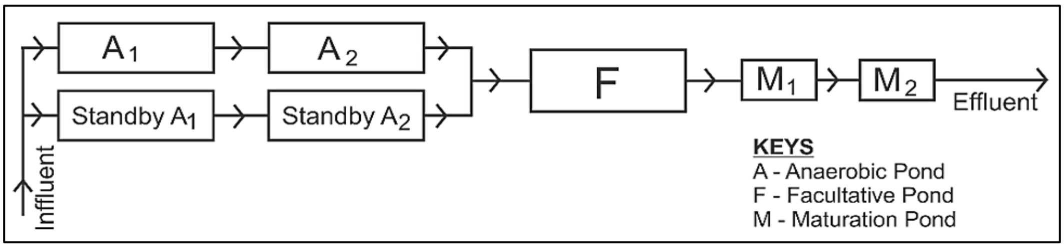


**Figure 5:** Schematic diagram of ponds arrangement

$$\text{Tottal top surface area of ponds} = 4(477.77\ \text{m}^2) + 1(2207.36\ \text{m}^2) + 2(1778.60\ \text{m}^2)$$
$$\therefore \text{total top surface area of ponds} = 7675.64\ \text{m}^2$$
Since 25% of the area required for excavation is usually added, it implies:
$$\text{Total area of land required for WSP} = 1.25 \times 7675.64\ \text{m}^2 = 9594.55\ \text{m}^2 \cong 1.0\ \text{hectare}$$

The waste stabilization pond (WSP) was proposed to be sited at the northern part of the community, somewhere around latitude 8°34'40.00" N and longitude 4°33'35.69"E. In selecting this appropriate site, it was ensured that the location is downwind direction and considerably distant from residential areas so that the pungent smell produced, especially in the anaerobic ponds would not disturb the lives of people living within the community (military cantonment). The prevailing wind direction of Ilorin is from the southwest cardinal point, followed by the southern cardinal point, while wind from the northern cardinal point is rare [22]. Consequently, siting the WSP in the northern part of the community as proposed will prevent the unpleasant smell in the ponds from reaching residential areas. The Corporal Quarters is the nearest residential area to the proposed site of the WSP, as can be seen in Figure 6; it is at a distance of 1.2 km away from the ponds, which is much higher than the recommended minimum permissible distance of 200 m from residential areas [12]. Since a dam is built across the stream identified to receive the treated effluent, the proposed WSP was positioned downstream of the dam so that the discharged effluent would not contaminate the water in the reservoir, which may be treated for domestic use. The proposed location of the WSP is 236.8 m away from the major road. This will ease the accessibility of personnel to the site during maintenance. The site's topography was also considered. Hence, the proposed location has a low elevation (based on contour lines) so the flow of influent into the facility could be supported by gravity.

The average permeability of Ilorin soil is $6.03 \times 10^{-9}$ m s$^{-1}$ [23]. This is quite close to the recommended permeability value of $1.0 \times 10^{-9}$ m s$^{-1}$ for the construction site of WSP. Nevertheless, the ponds will be lined with ordinary Portland Cement (8 kg/m$^2$) to prevent the possibility of contamination of groundwater through seepage. Besides, the distance between the Dam and the proposed location of the WSP is 1.3 km. Such distance will checkmate the base flow of effluent into the reservoir peradventure of failure of the lining.

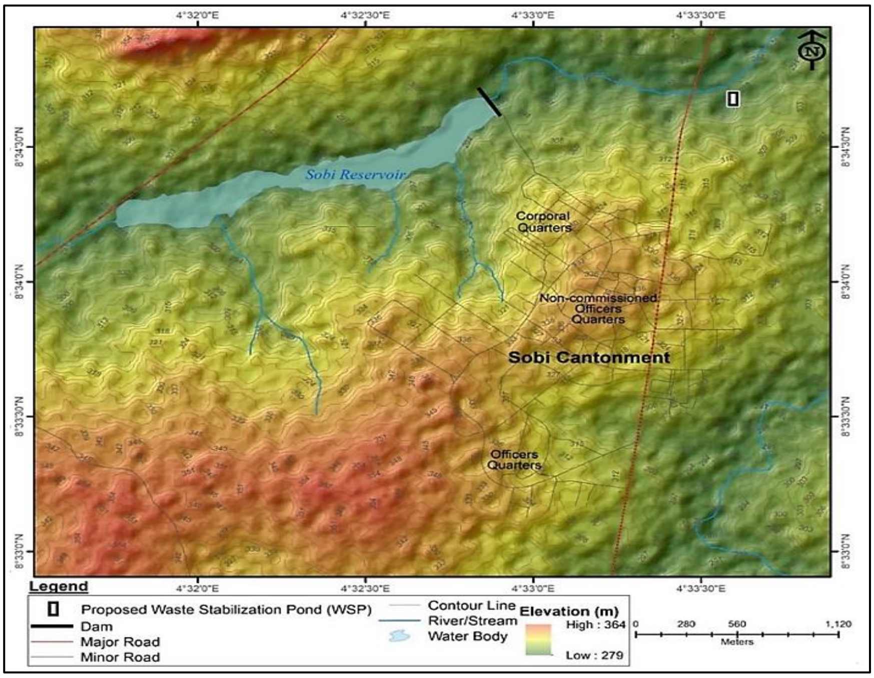


**Figure 6:** Location of WSP based on digital elevation model and contour map

## 4. Conclusion and recommendations

The research has successfully applied engineering principles to design detailed waste stabilization ponds to treat wastewater generated in a community (Sobi cantonment) within a tropical climate. The design of the proposed WSP will consist of two anaerobic ponds, one facultative pond, and two maturation ponds arranged in series with a duplicate set of anaerobic ponds as standby. The retention periods in these ponds are 3.9 days, 10.7 days, and 8 days, respectively. Also, an approximate sludge accumulation rate of 135 $m^3$ per year will occur at the primary anaerobic pond, requiring a desludging frequency of 2.8 years. The design calculations, drawings, and site selection were simplified to give prospective researchers and students a thorough understanding of the subject matter. It is recommended that the construction of the facility (WSP) in the said location adhere to the designed calculations while similar cantonments or planned large estates, campuses, etc., could implement the design with appropriate modifications.

## Funding

This research received no specific grant from any funding agency in the public, commercial, or not-for-profit sectors.

## Data availability statement

The data that support the findings of this study are available on request from the author.

## Conflicts of interest

The authors declare that there is no conflict of interest.

## References

**[1]** M. A. Onu, O.O. Ayeleru, B. Oboireien and P.A. Olubambi, Challenges of Wastewater Generation and Management in Sub-Saharan Africa: A Review, Environ. Challenges, 11 (2023) 1 – 21. https://doi.org/10.1016/j.envc.2023.100686

**[2]** F.J. Ogbozige and H.U. Nwobu, Biosorption of $Ni^{2+}$ and $Cr^{3+}$ in Synthetic Sewage: Adsorption Capacities of Water Hyacinth (Eichhornia crassipes), Environ. Res. Technol., 4 (2021) 342 – 351. https://doi.org/10.35208/ert.980490

**[3]** Y.T. Jung, N.C. Narayanan and Y.L. Cheng, Cost Comparison of Centralized and Decentralized Wastewater Management Systems Using Optimization Model, J. Environ. Manage., 213 (2018) 90 – 97. https://doi.org/10.1016/j.jenvman.2018.01.081

**[4]** M.I. Alfa, H.I. Owamah, A. Ishaq, P.A. Enokela, D. Oluwaseun and A.O. Chikezie, Performance Evaluation of Waste Stabilization Pond for Treatment of Wastewater from a Tertiary Institution Campus Located in Jos North Local Government Area, Plateau State, Nigeria, Int. J. Appl. Sci. Environ. Manage., 26 (2022) 1523 – 1528. https://dx.doi.org/10.4314/jasem.v26i9.10

**[5]** F. Pasciucco, I. Pecorini, and R. Iannelli, Planning the Centralization level in Wastewater Collection and Treatment: A Review of Assessment Methods, J. Cleaner Prod., 375 (2022) 1 – 22. https://doi.org/10.1016/j.jclepro.2022.134092

**[6]** K.K. Kuok, P.C. Chiu, M.R. Rahman, M.K. Bin-Bakri and M.Y. Chin, Effectiveness of Centralized Wastewater Treatment Plant in Removing Emerging Contaminants: A Case Study at Kuching, Malaysia, J. Water Resour. Prot., 14 (2022) 650 – 663. https://doi.org/10.4236/jwarp.2022.149034

**[7]** F.J. Ogbozige and M.A. Toko, Adsorption Isotherms and Kinetics of Lead and Cadmium Ions: Comparative Studies Using Modified Melon (Citrullus colocynthis) Husk, Iran. Int. J. Energy Environ., 11 (2022) 157 – 162. https://doi.org/10.5829/ijee.2020.11.02.10

**[8]** Seasonal Climate Prediction, An Online Information Released by the Directorate of Weather Forecasting Service of the Nigerian Meteorological Agency (NIMET). https://nimet.gov.ng/scp

**[9]** B.E. Daukere, Crop Production and Soldiers' Economic Development in Sobi Cantonment, Ilorin, Nigeria. A Book Chapter in Contemporary Issues in Agriculture, Rural Development and Population Dynamics in Nigeria [A Publication of the Department of Geography, Usmanu Danfodiyo University, 2024.

**[10]** Karia, G.L. and Christian, R.A., Wastewater Treatment Concepts and Design Approach (2nd Edition), PHI Learning Private Limited, Delhi, 2013.

**[11]** Chimwemwe B. G., Modern Design of Waste Stabilization Ponds in Warm Climates: Comparison with Traditional Design Methods, A Dissertation Submitted to the School of Engineering, University of Leeds, 2003. https://www.sleigh-munoz.co.uk/wash/Mara/WSP/Chimwemwe_MSc.pdf

**[12]** D.D. Mara and H.W. Pearson, Waste Stabilization Ponds: Design Manual for Mediterranean Europe, World Health Organization, Copenhagen, 1987

**[13]** M. von Sperling, Waste Stabolization Ponds, International Water Association (IWA), London, 2007.

**[14]** Mara, D.D., Waste Stabilization Ponds: A Highly Appropriate Wastewater Treatment Technology for Mediterranean Countries, Efficient Management of Wastewater: Its Treatment and Reuse in Water Scarce Countries, Springer, Heidelberg, 113 – 123, 2008. http://dx.doi.org/10.1007/978-3-540-74492-4_10

**[15]** Water Sanitation and Hygiene: National Outcome Routine Mapping, A Report of the Federal Ministry of Water Resources (FMWR), Nigeria. https://www.unicef.org/nigeria/media/5951/file/2021%20WASHNORM%20Report%20.pdf

**[16]** D.D. Mara, H.W. Pearson, J. Oragui, H. Arridge and S.A. Silva, Development of a New Approach to Waste Stabilization Pond Design, Research Monograph No. 5, 2001. https://www.sleigh-munoz.co.uk/wash/Mara/Books/TPHEResMon5.pdf

**[17]** A. Derakhshan, R.R. Kalantari, M. Farzadkia, A. Tiyuri and A. Esrafili, The Effect of Biological Treatment Methods on the Concentration of Carbonaceous Pollutants in the Slaughterhouse Wastewater: A Systematic Review, Case Stud. Chem. Environ. Eng., 8 (2023) 1 – 9. https://doi.org/10.1016/j.cscee.2023.100451

**[18]** C. Bustillo-Lecompte, M. Mehrvar and E. Quinones-Bolanos, Slaughterhouse Wastewater Characterization and Treatment: An Economic and Public Health Necessity of the Meat Processing Industry in Ontario, Canada, J. Geosci. Environ. Prot., 4 (2016) 175–186. http://dx.doi.org/10.4236/gep.2016.44021

**[19]** J.P. Arthur, The Development of Design Equations for Facultative Waste Stabilization Ponds in Semi-Arid Areas, Proc. Inst. Civ. Eng., 71 (1981) 197 – 213. https://doi.org/10.1680/iicep.1981.2148

**[20]** D.D. Mara, and S.A. Silva, Sewage Treatment in Waste Stabilization Ponds: Recent Research in Northeast Brazil, Prog. Water Technol., 11 (1979) 341-344.

**[21]** M.G. McGarry and M.B. Pescod, Stabilization Pond Design Criteria for Tropical Asia, Proceedings of the Second International Symposium for Waste Treatment Lagoons, Kansas City, 1970,114 – 132.

**[22]** T. Yahaya, H.A. Ajimotokan, J.A. Adebisi, I.I. Ahmed, T.K. Ajiboye, S. Abdulkareem, K.R. Ajao, Wind Resource of Ilorin City for Vortex-Induced Wind Turbine Power Generation and Off-grid Electrification, Niger. J. Technol. Dev., 20 (2023) 227 – 238. https://doi.org/10.4314/njtd.v20i3.1795

**[23]** O.O. Ige, O. Ogunsanwo and H.I. Inyang, Characterization of Terrain and Biotite Gneiss-Derived Lateritic Soils of Ilorin, Nigeria, for Use in Landfill Barriers, Global J. Geol. Sci., 9 (2011) 1 – 9.