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# Nitrogen Removal Strategy: Design of a Modified Ludzack-Ettinger Process for Clark Water Corporation's Wastewater Treatment Plant

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***Abstract***

Clark Water Corporation (CWC) is the private sector concessionaire of the Clark Development Corporation (CDC) for the Clark Freeport Zone (CFZ). The company provides water and wastewater services to various industrial, commercial and residential customers inside the Freeport. One of the conditions specified in the concession contract is (Under Clause 9.1) [1]; "the Company shall ensure that from the Commencement Date until the expiry of the Concession Period the Facilities are at all times operated and maintained in accordance with: (a) Applicable Laws;" Among the applicable laws that CWC needs to comply is the DENR DAO 2016-08 – Water Quality Guidelines and General Effluent Standards (GES) superseding the previous DAO 1990-35 imposing stringent values on the removal of Nutrients in the effluent discharge. CWC's existing wastewater treatment plant (WWTP) is not adequately designed for this. The need to reduce ammonia is a new requirement and require a more sophisticated process. The paper focused on the process conversion of the current WWTP into a modified Ludzack-Ettinger (MLE) process to meet the standards set forth in the revised DAO. Activated Sludge (ASPonds) will be converted into MLE which consist of an anoxic tank followed by an aeration tank and an internal sludge recycling system. The complete nitrogen cycle shall happen in the retrofitted ASponds. Nitrification will occur in the aeration tank while the nitrate produced will be recycled back to the anoxic tank. The design will involve the two (2) major components: process design configurations (volume projections, influent characterization & computation for air requirements) and the design of mechanical systems. Formulas & computations were encoded in Microsoft excel and its goal seek function was utilized to calculate the plant's optimum design. Though the paper made use of scientific set of calculations, many assumptions were made and formed the basis of the design.

***Keywords***- *BNR, Clark Water, CMAS, DENR, DAO2016-08, GES*

## 1. INTRODUCTION

Nitrogen and phosphorus are the primary causes of eutrophication in surface waters. Over-enrichment of these nutrients manifest as algal blooms which can cover entire water bodies, diminish its dissolved oxygen content ergo the fish kills and the depletion of desirable flora and fauna. Moreover, excessive amounts can stimulate the activity of microbes which may be harmful to human health [2]. Nutrient removal in wastewater is an essential element of cleaning the water bodies in the country.

In an effort to reduce the nutrient levels in our water bodies, the Department of Environment and Natural Resources (DENR) promulgated a new order imposing the removal of nutrients in the effluent discharge.

### *1.1. Study area*

CWC operates a wastewater Lagoon system with a design capacity of 30 million liters per day (MLD). The system is being operated as a "separate" system that only caters for domestic and non-domestic (after undergoing preliminary treatment at point source) sewage. It makes use of biological process wherein the raw influent is treated by aeration through a series of oxidation ponds with the aid of fine bubble aerators and no chemicals are used for treatment.

The design of CWC's wastewater treatment plant is a conventional multiple pond based (stabilization ponds or lagoon) system. The lagoon treatment facility makes use of biological process wherein the raw Wastewater is treated by aeration through a series of oxidation ponds with the aid of fine bubble aerators before it is released to Dolores Creek – a nearby receiving water body. No chemicals are used for treatment.

The treatment facility has a very sufficient land area of approx. 27 hectares. Lagoon has six ponds – two (2) of each type namely Activated Sludge ponds, Partial Mixed ponds and Maturation Ponds.

The treatment system can be broken down into four (4) major processes:

- Preliminary Treatment (pumping station and screens
- Secondary Treatment (Two Activated Sludge Ponds in parallel and Two Partially Mixed Ponds in series
- Tertiary Treatment (Two Maturation Ponds in series and a Chlorination Contact Tank) and
- Sludge management/Drying bed (Two Sludge ponds)

The system provides BOD removal, solids elimination, and pathogen destruction for CFZ. Currently, the existing treatment does not have the capability to comply with the new DAO 2016-08 standards but is designed to comply with the DAO 1990-35 [3] parameters. Figure 1 shows the schematic of the Lagoon system.

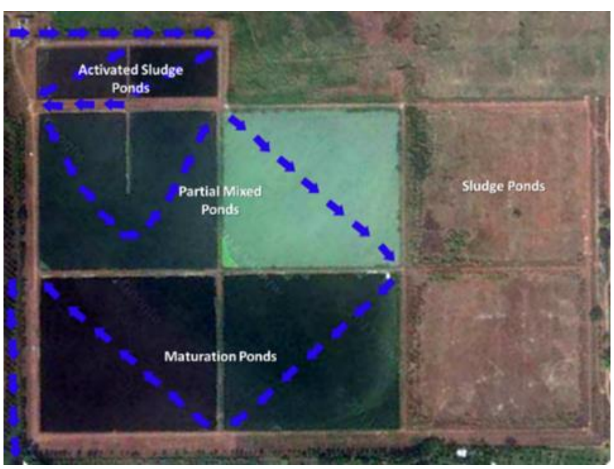


*Figure 1: Schematic Diagram of CWC-WWTP showing flow through plant operations*

### *1.2. New Water Quality Guidelines (DAO2016-08)*

In June 2016, DENR promulgated a new Administrative Order (DAO 2016-08 – Water Quality Guidelines and General Effluent Standards) superseding the previous DAO 1990-35 imposing stringent values on the removal of nutrients in the effluent discharge [4]. Clark Water's Wastewater Treatment Plant (WWTP) would need to be upgraded to comply with the new DAO 2016-08 and likewise cope with the increasing volume of wastewater discharge due to the increasing number of locators.

The new law contains new effluent standards incorporating stringent nutrient concentration limits. The new standard identifies several industries and the significant parameters which need to be monitored and tested. CWC's WWTP falls under sector 37000 – Sewerage (operation of sewer systems or sewage treatment facilities that collect, treat and dispose of sewage) where significant parameters and their design limits are as follows:

*Table 1: Comparison of Significant Parameters (DAO 1990-35 vs. DAO 2016-08) [3][4]*

| **Parameter** | **Unit** | **DAO 35 (Old)** | **DAO 2016-08 (New)** |
|---|---|---|---|
| | | **Class C** | **Class C** |
| BOD | mg/L | 50 | 50 |
| Fecal Coliform | MPN/ 100mL | - | 400 |
| Ammonia | mg/L | - | 0.5 |
| Nitrates as $NO_3$-N | mg/L | - | 14 |
| Phosphates | mg/L | - | 1 |
| Oil and Grease | mg/L | 5 | 5 |
| Surfactants | mg/L | 5 | 15 |
| COD | mg/L | 100 | 100 |
| Total Suspended Solids | mg/L | 70 | 100 |

The existing WWTP still follows the old DENR Administrative Order 35 (DAO35) and is not designed for this. The need to reduce ammonia is a new requirement and require a more sophisticated process.

### *1.3. Growth and the Increase in Wastewater Flows*

The CFZ have great potential for business within North Luzon. The main economic driver for CFZ is its accessibility as it is located in the heart of Central Luzon, it is accessible both from NLEX and SCTEX, it is likewise connected to one of the proposed North Rail Stations and it has an existing International airport [5]. Moreover, the "Build, Build, Build" project of the current administration streamlines the motion to decongest Manila and entice investors to do business in the provinces [6]. This along with the abovementioned characteristic of CFZ make it a very good go-to business hub.

### *1.4. Statement of the Problem*

The main drivers for this paper are two-fold: 1) Clark Water's existing treatment plant does not have the capability to comply with the new DAO 2016-08. It needs to undergo retrofitting to meet the new GES limits and 2) cope with the increasing volume of incoming wastewater.

### *1.5. Objectives*

This paper will focus on the process conversion of the current WWTP into a Modified Ludzack-Ettinger (MLE) process to meet the standards set forth in the revised DAO. It aims to design a system to attain the ammonia general effluent standards (GES) limit for class C waterbody.

Specifically, the design will involve the two (2) major components: a) the process design configurations (volume projections, influent characterization & computation for air requirements) and the design of mechanical systems and b) process design configurations (computation for air requirements, and design of the aeration system)

### *1.6. Significance*

The study will serve as guide for CWC and its regulator (CDC) to better their judgement during optioneering (Technology comparison and selection stage). The paper may also serve as reference for fellow consultants and other concessionaires/wastewater operator having similar treatment setup and are looking for alternative treatment options to comply with the new law. The study may also serve as a supplement to the textbook in terms of practical applications of design and process configurations in real life situation.

### *1.7. Scope and Limitation*

The paper relies on the following data to arrive with the necessary components and timelines of the upgrade: a) future wastewater flows derived from the projected billed volumes of the CFZ. Quality on the other hand will be compiled from 2017 up until the pre-COVID-19 pandemic. The study will provide the necessary tank dimensions and define the required equipment to meet the nitrogen requirements of the new DAO. Detailed design engineering is not included. Moreover, the excel Masterfile of the design will be made available to the public.

A desktop approach utilizing mass balances for nitrogen and other design parameters such as specific denitrification rate (SDNR) are used in this study. Because nutrient removal processes are very complex with many dependent interactions, designers nowadays are relying more on paid software and other comprehensive simulation models such as BioWin. Since WW design is highly iterative, the researcher utilized Microsoft Excel worksheet and goal seek function to calculate the plant's optimum design. Both design approaches can achieve the required standard in the freezing climate of parts of the USA and should have no issue in the tropical climate.

It is also important to note that the proposed enhancements are not meant to treat high concentration of industry pollutants such as heavy metals in the influent. Status quo is assumed for industries that discharge to the sewer lines; that is, they will continue to pre-treat their waste to meet all the parameters of CWC's internal standards.

Infiltration is taken as 0.5MLD throughout although it may not be as reliable in the coming years due to climate change. Design of the retrofitting works will focus on the main mechanical parts (piping, instrumentation and equipment). Civil and electrical components will be not be detailed as it is beyond the scope of this study.

## 2. METHODOLOGY

### *2.1. Research Design*

This study made use of descriptive method of research since it will describe the current characteristics of CWC wastewater and a plan to comply with the new standards.

### *2.2. Conceptual Framework*

Drafting of CWC's BNR upgrade (MLE system) made use of a simple input-process-output model as shown in Figure 2.

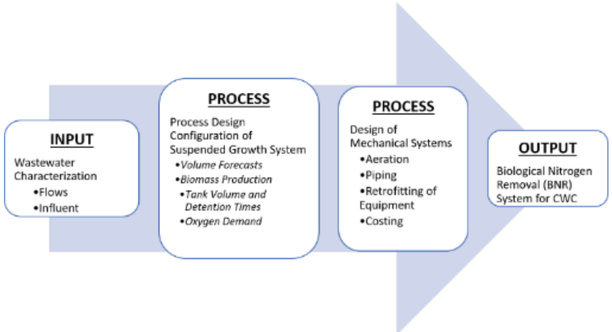


*Figure 2: Conceptual Framework*

### 2.3. *Sources of Data*

There were four (4) main participants for the fulfilment of this study. First, Business Operations group whom are in charge of the monthly and annual forecast of the billed volume. Second the laboratory services team wherein historical influent and effluent data will be obtained. Third, the Project Management and Technical Services group for the AS-Built plans of the facility and finally, pertinent data such as the daily flow, pH, and dissolved oxygen levels which were obtained from the operational records of the Used Water Operations group.

### 2.4. *Procedure*

Incoming flow computations were based on the average historical wastewater return factor (RF). Wastewater RF is defined as the proportion of water used that is returned to the sewer system. Average dry weather flows (ADWF) will be computed by multiplying RF to the billed volume (BV) or the annual water demand. Average wet weather flows (AWWF) on the other hand will be computed simply by adding 0.5MLD volume to consider rainwater infiltration. AWWF numbers will be tallied against the current WWTP capacity to determine the upgrade and retrofit schedules.

Baseline nutrient and significant parameter data of the WWTP was done by compiling the last three (3) years – 2017 up to 2019 prior the COVID-19 pandemic- of influent tests conducted by a third-party laboratory. Data gathered were tabulated to show the average monthly fluctuations and determine the additional level of treatment.

The compiled data served as the baseline for the process computations. The existing lagoon WWTP will be converted into an MLE process. Activated Sludge (ASPonds) will be converted into MLE (Figure 3) which consist of an anoxic tank followed by an aeration tank and an internal sludge recycling system. The complete nitrogen cycle shall happen in the retrofitted ASponds. Nitrification will occur in the aeration tank while the nitrate produced will be recycled back to the anoxic tank. Organic substrate in the influent wastewater provides the electron donor for the oxidation-reduction reactions using nitrate.

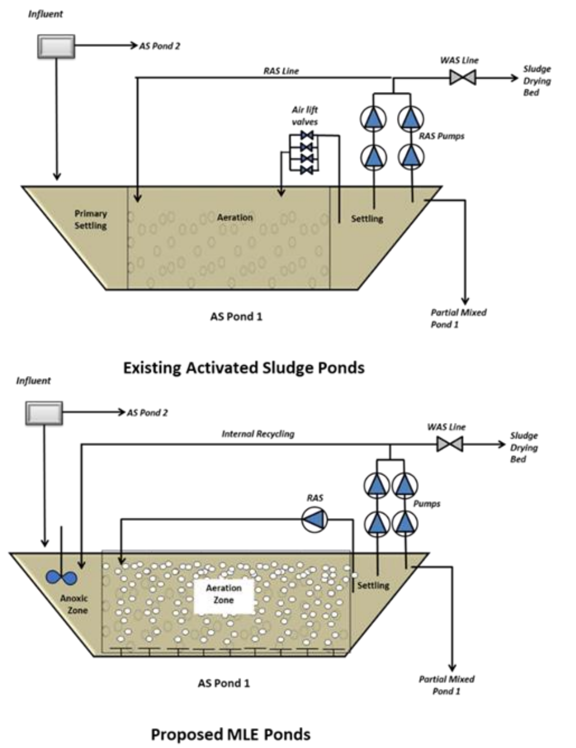


*Figure 3: Existing Activated Sludge (AS Ponds) and Proposed MLE Setup*

Computations will be divided in the following manner. Part A: Process Design Configurations and Part B: Design of mechanical systems. Major design procedures for Part A include:

1. Defining effluent requirements in terms of $NH_4$-N, TSS and BOD concentrations and selection of appropriate safety factors based on peak/ave. TKN loadings.
2. Design of complete-mix activated sludge process to treat influent to meet BOD, and $NH_4$-N concentrations.
    a. Designing for a suspended growth system for BOD and $NH_4$-N,
    b. Computation of mass of TSS and VSS in the aeration basin

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   c. MLSS concentration and computation for the aeration tank volume and detention times
3. Design of the pre-anoxic basins for denitrification.
   a. Computing the internal recycle flowrate IR
   b. Determining the amount of $NH_4$-N oxidized to $NO_3$
   c. Computation for the anoxic volume
   d. Computations for the amount of $NO_3$ that can be reduced
4. Calculation of net $O_2$ demand including air flowrates

While for Part B:

1. Blower sizing
2. Air diffusion system including the diffuser density for the aeration zone
3. Piping layout and auxiliary devices such as Mixing devices for the anoxic zone and pump recirculating capacities.
4. Determining the sludge denitrification rates

Abbreviations, formulas, constants, and standard values used in the process design computations of the new plant will be listed in Annex 1 [7][11][13].

## 3. RESULTS AND DISCUSSION

### *3.1. Influent Characterization*

Table 2 summarizes the wastewater discharge projections of the freeport at 5-year intervals [8].

*Table 2: Projected Wastewater Flows*

| Year | 2020 | 2025 | 2030 | 2035 | 2040 |
|---|---|---|---|---|---|
| Wet Weather Flows (MLD) | 27.1 | 46.8 | 58.6 | 61.6 | 65.3 |
| Current WWTP capacity (MLD) | 30 | 30 | 30 | 30 | 30 |

Based from the projected flows, the WWTP will start having capacity issues by 2021. By then, the WWTP should have been upgraded to accommodate the increase in flows. A multiple stage upgrade is vital to avoid the sudden surge in tariff prices and have the recovery in capital expense spread on a longer period throughout the concession. Figure 2 show the forecast BV, AWWF and timing of future expansion.

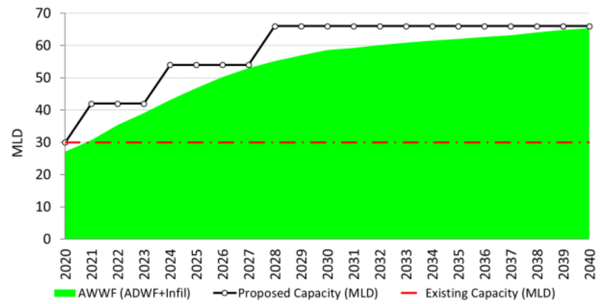


*Figure 4: Forecast Wastewater Volumes*

Table 3 shows the observed influent concentrations covering January 2017 to March 2020 in comparison to the CWC internal standards. These internal standards are imposed to ensure that the discharge from the locators are within the design capacity of the WWTP. Influent is examined every fortnight via 24-hour time weighted composite samples. WW discharged into the sewer lines is not constantly complying with the internal standards of CWC. High fluctuating values of oil and grease (O&G) and coliform levels are observed and still aggravating [9].

*Table 3: List of Influent Nutrient Parameters and their Concentrations*

| Parameter | Unit | Internal Standard | 2017 | 2018 | 2019 | 2020 |
|---|---|---|---|---|---|---|
| BOD | mg/L | 200 | 101.0 | 122.5 | 143.5 | 125.2 |
| COD | mg/L | 400 | 242.5 | 248.1 | 238.2 | 240.9 |
| TSS | mg/L | 200 | 102.4 | 101.2 | 98.2 | 96.05 |
| O&G | mg/L | 5 | 3.9 | 4.1 | 6.3 | 10.68 |
| Color | TCU | 150 | 46.9 | 40.5 | 57.8 | 42.9 |
| pH | range | 6.5-9 | 7.4 | 7.2 | 7.5 | 6.9 |
| T. Coliform | MPN/100mL | $10x10^6$ | $12x10^6$ | $37x10^6$ | $29x10^6$ | $35x10^6$ |
| TKN | mg/L | 35 | 28.1 | 26.6 | 26.4 | 32.0 |
| NH4-N | mg/L | 26 | 8.11 | 4.28 | 10.1 | 7.46 |
| $NO_3$-N | mg/L | - | 0.39 | 0.72 | 0.52 | 0.35 |
| TP | Mg/L | | 2.66 | 3.14 | 3.2 | 3.4 |

## *3.2. Design Summary*

### *3.2.1.Process Design*

The table below summarizes the process design configurations of the proposed WWTP. Values contained in this table were computed following the formulas presented in **ANNEX**. Separate columns were drawn to show comparisons between BOD removal (DAO35 standards) vs BOD and nitrification (DAO2016-08 standards) and the increase in overall operational expense between the two process. Tabletop computation was done and an excel master file was created to optimize computations. Influent flowrate used the forecasted Y2040 values. Existing pond sizes, geographic data and AS built plans were likewise used in the computations. Influent parameters such as BOD, COD, TSS, TKN, NH4-N, TP, bCOD/BOD ratio and ambient temperature were average historical values from 2017-2020. On the other hand, design conditions and assumptions were used for sBOD, sCOD, rbCOD, alkalinity. Effluent or target quality was based on the existing GES of DAO2016-08. Sanitaire was selected as diffuser due to readily available technical data on the web [10].

*Table 4: Process Design Summary*

| Design Parameter | Unit | Design for BOD removal only | Design for BOD and nitrification |
|---|---|---|---|
| Influent Parameter | | | |
| Average WW flow | m3/d | 65,300 | 65,300 |
| Ave BOD load | kg/d | 8,623 | 8,623 |
| Ave TKN load | kg/d | 2,240 | 2,240 |
| Aeration Zone | | | |
| Aerobic SRT | d | 5 | 6.80 |
| Aeration tank volume, ea | m3 | 12,757 | 16,708 |
| *Length* | m | 46.5 | 46.5 |
| *Width* | m | 34.29 | 44.91 |
| Hydraulic detention time, t | hr | 4.69 | 6.14 |
| MLSS | g/m3 | 3,000 | 3,000 |
| MLVSS | g/m3 | 1,580 | 1,543 |
| F/M | g/g.d | 0.43 | 0.33 |
| BOD loading | kg BOD/m3. d | 0.68 | 0.52 |
| Sludge Production | kg/d | 7,654 | 7,375 |
| Observed yield | kg TSS/kg bCOD | 0.89 | 0.86 |
| | kg VSS/kg BOD | 0.47 | 0.45 |
| Oxygen required | kg/h | 377.7 | 723.9 |
| Air flowrate at average WW flow | m3/min | 176.6 | 257.3 |
| Clarifier/Settling Zone | | | |
| RAS ratio | Unitless | 0.6 | 0.6 |
| Clarifier hydraulic application rate | m3/m2. d | 22 | 22 |
| Clarifier area, ea | m3 | 2,968 | 2,968 |
| *Length* | m | 46.5 | 46.5 |
| *Width* | m | | 7.98 |
| Effluent BOD | g/m3 | <30 | 8.99 |
| TSSe | g/m3 | <30 | 10 |
| Effluent NH4-N | g/m3 | 28.67 | 0.5 |
| Anoxic Zone | | | |
| Effluent NO3-N | g/m3 | n/a | 6 |
| Internal recycle ratio | Unitless | | 3.1 |
| RAS recycle ratio | Unitless | | 0.6 |
| Anoxic Volume, ea | m3 | | 4,081 |
| *Length* | m | | 46.5 |
| *Width* | m | | 10.97 |
| MLSS | g/m3 | | 3000 |
| Overall SDNR | g NO3-N/g MLSS.d | | 0.09 |
| Detention Time | hr | | 1.5 |
| Mixing Power | kW | | 41 |
| Alkalinity required | kg/d as CaCO3 | | 4,048 |

From the ASbuilt plans, the required aeration tank volume was converted into physical dimensions (given length and depth). Figure 5 depicts that the proposed MLE system can readily be incorporated in the existing ASPonds without increasing the pond's footprint. Existing ASponds have a total effective volume of 15,810$m^3$ each while MLE would only require 11,879$m^3$ each. Length to width ratio is important in computing for diffuser spacing because it establishes spatial distribution of oxygen demands and constrains how the air diffusion system can be arranged.

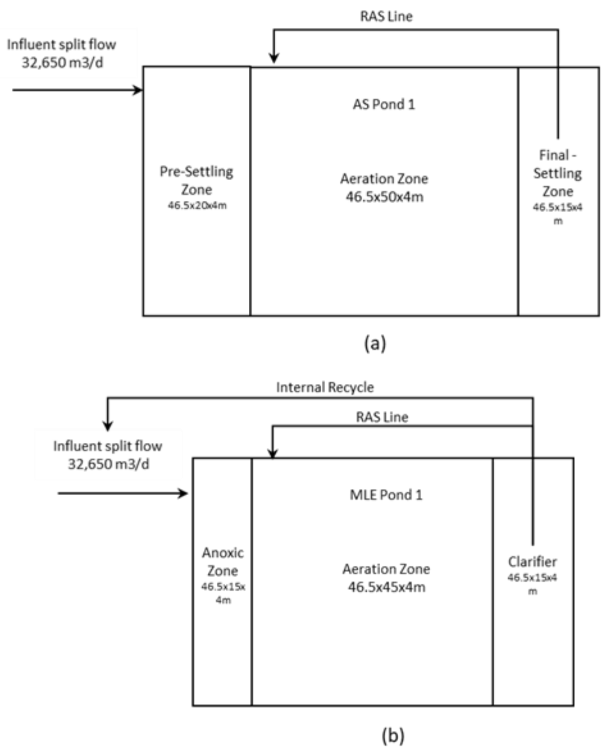


*Figure 5: Comparison in the dimensions (in meters) of existing (a) ASPond and (b) Proposed MLE setup*

### *3.2.2.Mechanical Systems*

Table 5 below show the design summary of the aeration system. The air supply system will consist of air filters, blowers, air piping, and airflow control equipment (flow meters and flow control valves). The diffusion system consists of a series of headers and lateral piping placed in the aeration basin, to which ceramic diffusers are attached.

*Table 5: Aeration System Design Summary*

| Design Parameter | Unit | Design | Remarks |
|---|---|---|---|
| Pond Dimensions | | | |
| Pre-Anoxic Zone | $m^3$ | 6,802 | |
| *Length* | m | 46.5 | AS-Built |
| *Width* | m | 36.6 | Computation |
| Aerobic Zone | $m^3$ | 16,708 | |
| *Length* | m | 46.5 | AS-Built |
| *Width* | m | 89.8 | Computation |
| IR Zone | | | |
| *Length* | m | 15 | To be used as IR for denitrification |
| *Width* | m | 141.4 | Used existing width for diffuser spacing |
| Temp | ºC | 30 | Ave. Value 2017-2020 |
| Total liquid depth when full | m | 4.00 | AS-Built |
| Point of air release for the ceramic diffusers | m | 0.50 | AS-Built |
| Freeboard | m | 0.50 | Actual Operating conditions |
| Volume flowrate | kg/h | 551.38 | Oxygen requirements was reduced by 23.4% due to internal recycling |
| Average oxygen transfer rate | kg/h | 551.38 | |
| Standard oxygen transfer rate | kg/h | 712.65 | |
| Inlet Pressure | | | |
| Patm, | lb/in$^2$ | 6.80 | |
| H | Ft | 391.96 | |
| Losses | lb/in$^2$ | 0.25 | Assumed losses at blower fittings/pipings |
| Pinlet | Kpa | 99.97 | |
| Poutlet | kPa | 148.10 | |
| Power required | kW | 33.81 | |
| Estimated Daily Power Required for Blowers | kW-hr/day | 812 | |
| Diffusers | | | |
| SOTR | Kg/hr | 712.65 | |
| SOTE | %/m | 6.5 | Technical data from Sanitaire |
| SOE | % | 22.8% | Technical data from Sanitaire |
| Effective Surface area | m2 | 0.04 | Technical data from Sanitaire |
| No. of diffusers required | No. | 9,986 | Theoretical requirement based on computation |
| Diffuser Density | | | |
| Aeration tank area | m2 | 4,176.96 | |
| Zone area (each) | | | Divide aeration area into 3 zones |
| *Length* | m | 46.5 | |
| *Width* | m | 29.94 | |

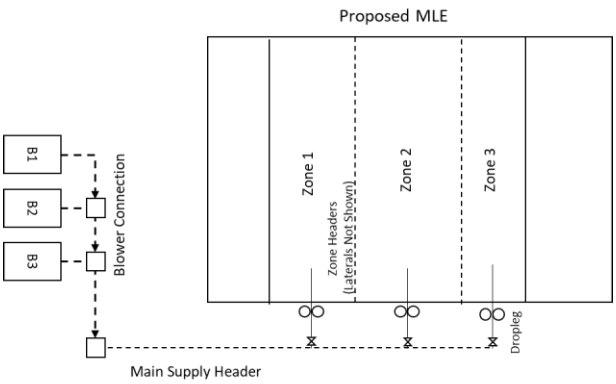


*Figure 6:Proposed alignment of process air piping*

### *3.2.3.Piping and Diffuser Layout*

Next section provides the logical approach in the system layout and airflow distribution. The system was arranged as a series of "grids/zones" to allow for proper airflow distribution and accommodate oxygen transfer rate variations within the aeration basin [11]. Zone 1 (synthesis) need to satisfy the highest oxygen demands, therefore it will have the highest diffuser densities and use the highest airflow rates. Note that sludge bulking in the initial zone may be experienced it operated at low DO levels. Losses through the various piping components were solved using Darcy-Weisbach and were also included in the .xls masterfile. The design was refined by inputting new diameter sizes accordingly.

*Table 6: Diffuser layout for each proposed MLE Pond*

| Zone | No. of laterals | Total no. of laterals on both sides | No. of diffuser per lateral | Check total number of diffusers | No. of diffusers to be plugged | No. of diffusers to be plugged per lateral | No. of active diffusers per lateral | Spacing between diffusers, (m) | Spacing between lateral, (m) |
|---|---|---|---|---|---|---|---|---|---|
| Zone 1 | 15 | 30 | 42 | 2,520 | 51 | 2 | 40 | 0.29 | 0.75 |
| Zone 2 | 12 | 24 | 35 | 1,680 | 34 | 1 | 34 | 0.35 | 0.95 |
| Zone 3 | 19 | 18 | 28 | 1,008 | 21 | 1 | 27 | 0.44 | 1.28 |

Figure 7 show the proposed layout plan of zone 1 and the general arrangement of diffusers as summarized in Table 6. Zones 2 and 3 will follow suit but will be installed with different no. and spacing between diffusers and laterals.

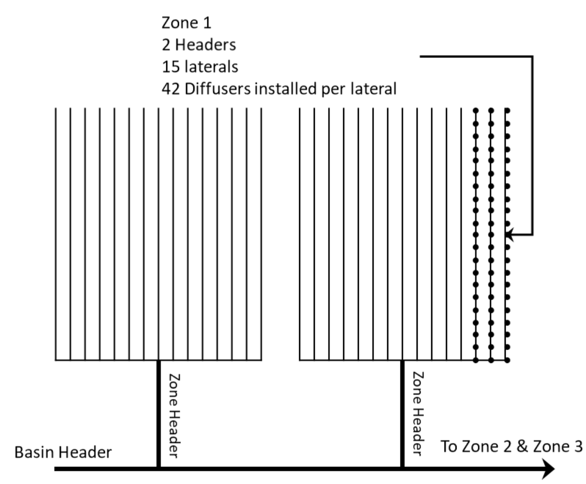


*Figure 7: Plan – Proposed piping for MLE Pond (Zone 1)*

## 4. CONCLUSIONS AND RECOMMENDATIONS

Given the assumed average flow of 65,000cum/d with influent parameter not diverging much, the proposed MLE system can readily be incorporated in the existing ASPonds without increasing the pond's footprint. To minimize the cost of retrofit, the existing final settling zone dimensions need not be changed to ensure sufficient settling time for the RAS. Existing partially mixed and maturation ponds can be used in lieu of mechanical clarifiers. PM and Maturation ponds can also act as buffer for recalcitrant and leftover BOD. Although the new design requires a smaller footprint, there will be significant refurbishments to be done in the aeration system such and the RAS system, the addition of internal recycle pumps and the type and no. of diffusers. From the existing 162 units of tubular diffusers (81 for each pond) the new MLE now require a total of 10,316 ceramic dome diffusers. It is also best to consider wastewater peak flows, diurnal patterns and loadings for the entire range of operating conditions anticipated. From these, system oxygen requirements can be computed more accurately, and the plant shall be able to handle major fluctuations in load.

The paper centered on the design and retrofit of the existing WWTP for nitrogen removal. However, nitrogen is only half of the equation as phosphorus must also be removed from the plant's effluent. Phosphorus can be removed from wastewater using a variety of biological phosphorus removal processes including the addition of an anerobic tank prior the aeration zone [12]. Although biologically possible, phosphorus removal can be a challenge for designers. When designing for BNR P removal it is best to install chemical dosing as backup [15]. Iron or aluminum salts can be added at a variety of different points in the treatment process, but because polyphosphates and organic phosphorus are less easily removed than orthophosphorous, adding aluminum or iron salts after secondary treatment typically result in the best removal.

Detailed discussion of the design and the bill of quantity (BOQ) is beyond the scope of this paper. Future work may include checking the integrity and the individual asset condition as this is an added excellent reference in doing the former. Moreover, operating expenses were reserved although estimated daily power requirements were added. Another deficiency is the methodology to be utilized during the actual retrofit of the WWTP. Current daily flows already exceed 50% of the plant's capacity, thus one AS ponds can't be put offline as the incoming raw WW will only be diverted to the PM ponds leading to an incomplete process. A better approach is to tackle combined BNR for nitrogen and phosphorus and construct a separate tank. Once the new BNR tank is complete then the company can pursue the retrofitting of the existing ponds.

The design procedure used herewith can be easily replicated by using the excel master file but is not intended to provide a standard set of design procedure to arrive at the "right" design. While component sizing of the system is based on scientific set of calculations, many assumptions were made and formed the basis of the design. Still, the best way to increase accuracy of design is to use computer aided software.

**ANNEX**

## *Process Design Configurations and Air Requirements*

Section below summarizes the steps, formulas and specific set of calculations used in the process design configurations of the new plant. Plant effluent requirements should satisfy the new DAO2016-08 shown in **Error! Reference source not found.**. Equations 1 to 22 were used for the process design configurations while equations 23 onwards were used for the design of mechanical systems

For the computation of average BOD and TKN load:

$$\mathrm{BOD} = \mathrm{Ave.WWflow} \times \mathrm{influent\ BOD} \quad (1)$$
$$\mathrm{TKN} = \mathrm{Ave.WWflow} \times \mathrm{influent\ TKN} \quad (2)$$

As for the wastewater characteristics needed for the design:

$$\mathrm{bCOD} = 1.6(\mathrm{BOD}) \quad (3)$$
$$\mathrm{COD} = \mathrm{bCOD} + \mathrm{nbCOD} \quad (4)$$
$$\mathrm{sCOD_e} = \mathrm{sCOD} - 1.6\mathrm{sBOD} \quad (5)$$
$$\mathrm{nbVSS} = \left[1 - \left(\frac{\mathrm{bpCOD}}{\mathrm{pCOD}}\right)\right]\mathrm{VSS} \quad (6)$$
$$\frac{\mathrm{bpCOD}}{\mathrm{pCOD}} = \frac{(\mathrm{bCOD/BOD})(\mathrm{BOD} - \mathrm{sBOD})}{\mathrm{COD} - \mathrm{sCOD}} \quad (7)$$
$$\mathrm{iTSS} = \mathrm{TSS_o} + \mathrm{VSS_o} \quad (8)$$

Where:

bCOD - Biodegradable COD
COD - Non-Biodegradable COD
$sCOD_e$ - Soluble COD
bpCOD - Biodegradable particulate COD
nbVSS - Nonbiodegradable VSS
iTSS – Inert TSS

Specific growth rate for Nitrification:

$$\mu_n = \left(\frac{\mu_{nm} N}{K_n + N}\right)\left(\frac{DO}{K_O + DO}\right) - k_{dn} \quad (9)$$

Applying temperature correction for the activated sludge nitrification kinetic coefficient T = 30ºC

$$\mu_{m,T} = \mu_m \theta^{t-20} \quad (10)$$

Where:

$\mu_n$= specific growth rate for nitrification, $T^{-1}$
$\mu_{nm}$= maximum specific growth rate for nitrifying bacteria, $T^{-1}$
N = nitrogen concentration, $ML^{-3}$
$K_n$=half-velocity constant, $ML^{-3}$
DO = dissolved oxygen, $ML^{3}$
$K_O$= oxygen inhibition coefficient, $ML^{-3}$
$k_{dn}$=endogenous decay coefficient for nitrifying organisms, $T^{-1}$

Theoretical and Design SRT given factor of safety of 1.5

$$\mu = 1/\mathrm{SRT} \quad (11)$$

Solving for the biomass production:

$$P_{XVSS} = \frac{QY(S_o - S)(1\mathrm{kg}/10^3\mathrm{g})}{1 + (k_d)\mathrm{SRT}} + \frac{(f_d)(k_d)QY(S_o - S)\mathrm{SRT}(1\mathrm{kg}/10^3\mathrm{g})}{1 + (k_d)\mathrm{SRT}} + \frac{QY_n(NO_x)(1\mathrm{kg}/10^3\mathrm{g})}{1 + (k_{dn})\mathrm{SRT}} \quad (12)$$

Where:

$P_{XVSS}$ = net waste activated sludge produced each day, kg VSS/d
$NO_x$ = concentration of $NH_4$-N in the influent flow that is nitrified, mg/L
$k_d$ = endogenous decay coefficient, $T^{-1}$
$k_{dn}$ = endogenous decay coefficient for nitrifying organisms, g VSS/g VSS.d

Solving for nitrogen oxidized to nitrate

$$NO_x = TKN - N_e - 0.12\frac{P_{x,bio}}{Q} \quad (13)$$

Where:

$NO_x$ = Nitrogen oxidized, mg/L
TKN = influent TKN concentration, mg/L
$N_e$ = effluent $NH_4$-N concentration, mg/L
$P_{x,bio}$ = biomass as VSS wasted, g/d

Formula used to determine the concentration and mass of VSS and TSS in the aeration basin

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$$P_{XVSS} = \frac{QY(S_o - S)(1kg/10^3g)}{1+(k_d)SRT} + \frac{(f_d)(k_d)QY(S_o - S)SRT(1kg/10^3g)}{1+(k_d)SRT} + \frac{QY_n(NO_x)(1kg/10^3g)}{1+(k_{dn})SRT} + Q(nbVSS)(1kg/10^3g) \quad (14)$$

Where:

$P_{XVSS}$ = net waste activated sludge produced each day, kg VSS/d
$NO_x$ = concentration of $NH_4$-N in the influent flow that is nitrified, mg/L
$k_d$ = endogenous decay coefficient, $T^{-1}$
$k_{dn}$ = endogenous decay coefficient for nitrifying organisms, g VSS/g VSS.d

Design of MLSS concentration and aeration tank volume, assuming MLSS is at @ 3,000mg/L

$$V = \frac{P_{XTSS}}{MLSS} \quad (15)$$

Formulas for Food to Microorganism ratio and BOD volumetric load

$$F/_M = \frac{\text{Total applied substrate rate}}{\text{Total microbial biomass}} = \frac{QS_O}{VX} = \frac{gBOD}{gMLVSS.d} \quad (16)$$

$$F/_M = \frac{S_o}{TX};\ L_{org} = \frac{QS_O}{V} = \frac{kg\ BOD}{m^3 d} \quad (17)$$

Where:

$F/_M$ = food to biomass ratio, g BOD or bsCOD/g VSS.d
Q = influent wastewater flowrate, $m^3/d$
$S_O$= influent BOD or bsCOD concentration, $m^3/d$
V = aeration tank volume, $m^3$
X = mixed liquor biomass concentration in the aeration tank, $g/m^3$
T = hydraulic retention time of aeration tank, V/Q, d

Calculating the observed solids yield g TSS/g BOD removed and g VSS/g BOD removed

$$\text{Observed yield} = gTSS/gbCOD = kgTSS/kg\ bCOD \quad (18)$$

$$bCODremoved = Q(S_o - S) \quad (19)$$
$$Y_{obs,TSS} = P_{XTSS}/bCODremoved \quad (20)$$
$$Y_{obs,VSS} = \text{fraction VSS} = VSS/TSS \quad (21)$$

Calculating for the oxygen demand

$$R_o = Q(S_o - S) - 1.42P_{xbio} + 4.33Q(NO_x) \quad (22)$$

Where:

$R_o$ = total oxygen required, g/d
$P_{xbio}$= biomass as VSS waster, g/d (parts A, B, and C of Eq. 13)

Formulas used in the Aeration Design

$$SOTR = AOTR\left[\left(\frac{C_{s,20}}{\alpha F(\beta C_{STH} - C_L)}\right)(1.024^{20-T})(\alpha)(\ \ )\right] \quad (23)$$

Where:

AOTR= actual oxygen transfer rate under field conditions, $kgO_2/h$
SOTR = standard oxygen transfer rate in tap water at 20°C, and zero dissolved oxygen, $kgO_2/h$
β = salinity-surface tension correction factor, typically 0.95 to 0.98, see Eq. (5-54)
$C_{STH}$ = average dissolved oxygen saturation concentration in clean water in aeration tank at temperature T and altitude H, mg/L

$C_{STH} = (C_{STH})\frac{1}{2}\left(\frac{P_d}{P_{atm,H}} + \frac{O_t}{21}\right)$ (Note: For surface aerators, $C_{STH} = C_{STH}$)

The term in the brackets when multiplied by one-half represents the average pressure at mid depth and accounts for the loss of oxygen to biological uptake. If the biological uptake is not considered, then the following expression can be used:

$$C_{STH} = (C_{STH})\left(\frac{P_{atm,H} + P_{w\ mid\ depth}}{P_{atm,H}}\right)$$

$C_{STH}$ = oxygen saturation concentration in clean water at temperature T and altitude H, (see Appendix D), mg/L
$P_d$ = pressure at the depth of air release, kPa
$P_{atm,H}$ = atmospheric pressure at altitude H (see Appendix B), kPa
$P_{w\ mid\ depth}$ = pressure at mid depth, above point of air release, due to water column

$O_t$ = percent oxygen concentration leaving tank, usually 18 to 20 percent
$C_L$ = operating oxygen concentration, mg/L
$C_{s,20}$ = dissolved oxygen saturation concentration in clean water at 20°C and 1 atm, mg/L
T = operating temperature, °C
α = oxygen transfer correction factor for waste [(Eq.5-53)]
F = fouling factor, typically 0.65 to 0.9

Computing for the expected BOD value

$$BOD_e = sBOD + \left(\frac{1g\ BOD}{1.42g\ VSS}\right)\left(\frac{0.85g\ VSS}{g\ TSS}\right)(TSS, mg/L) \tag{24}$$

Where:

$BOD_e$ = final BOD effluent concentration, mg/L
sBOD = soluble carbonaceous BOD, (usually <3.0mg/L for AS process with SRT ≥ 4d

Next is the computation for the required anoxic zone, first the researcher computed for the active biomass concentration in the aeration tank.

$$X = \left(\frac{SRT}{t}\right)\left[\frac{Y(S_o - S)}{1 + (k_d)SRT}\right]$$

substituting V/Q for T (25)

$$X = \left[\frac{Q(SRT)}{V}\right]\left[\frac{Y(S_o - S)}{1 + (k_d)SRT}\right]$$

Then, computing for the IR ratio

$$IR = \frac{NO_x}{N_e} - 1.0 - R \tag{26}$$

Where:

IR = internal recycle ratio (internal recycle flowrate/influent flowrate)
R= RAS recycle ratio (RAS flowrate/influent flowrate)
$NO_x$ = nitrate produced in aeration zone as a concentration relative to influent flow, mg $NO_3$-N/L
$N_e$ = effluent $NO_3$-N concentration, mg/L

Computing for the NO3-Nfed to Anoxic Tank

$$\text{Flowrate to Anoxic Tank} = IR(Q) + R(Q) \tag{27}$$

Computation for the Anoxic Volume required

$$V_{NOX} = T \times Q \tag{28}$$

Determining the BOD: F/M ratio

$$F/M_b = \frac{QS_o}{(V_{nox})X_b} \tag{29}$$

Where:

$F/M_b$ = BOD F/M ratio based on active biomass concentration, g BOD/g biomass. d
Q = influent flowrate, $m^3$/d
$S_o$ = influent BOD concentration, mg/L
$V_{nox}$ = anoxic volume, $m^3$
$X_b$ = anoxic zone biomass concentration, mg/L

Computing for specific denitrification rates using Fig 8-23 (Metcalf & Eddy) and applying temperature correction

$$SDNR_T = SDNR_{20}\theta^{T-20} \tag{30}$$

Where:

From Fig. 8-23, SDNRb, (g/g.d at 20°C)
SDNR = specific denitrification rate, g $NO_3$-N/g MLVSS.d
$\theta$ = temperature coefficient (1.026)
T = temperature, °C

Amount of $NO_3$-N that can be reduced inside the Anoxic Tank and optimizing anoxic zone detention times and new SDNR values

$$NO_r = (V_{nox})(SDNR)(MLVSS) \tag{31}$$

Where:

$NO_r$ = Nitrate removed, g/d
$V_{nox}$ = anoxic volume, $m^3$
SDNR = specific denitrification rate, g $NO_3$-N/g MLVSS.d
MLVSS = mixed liquor volatile suspended solids concentration, mg/L

New net oxygen requirement after passing thru anoxic zone, lesser oxygen will be required since the bound oxygen from nitrates will be used up.

$$R_o = Q(S_o - S) - 1.42P_{xbio} + 4.33Q(NO_x) \tag{32}$$

Where:

$R_o$ = total oxygen required, g/d
$P_{xbio}$= biomass as VSS waster, g/d (parts A, B, and C of Eq. 14)

Next is the computation for new alkaline requirements

pH = Influent alkaline - alkaline used + alkaline to be added
Alkalinity savings = alk needed for nitrification only – alk needed with denitrification (33)

Finally, the computation for anoxic zone mixing energy requirements

$$\text{Mixing Energy Req.} = 10\text{KW}/10^3\ \text{m}^3 \quad (34)$$
$$V_{NOX} = T \times Q$$

### *Design of Mechanical Systems*

### *Sizing of Blowers*

The following formulas were used for the sizing of blowers.

$$P_w = \frac{wRT_1}{29.7ne}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right] \text{ SI Units} \quad (35)$$

$$P_w = \frac{wRT_1}{550ne}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right] \text{ U.S. Customary Units}$$

Where:

$P_w$ = power requirement of each blower, kW(hp)
w = weight of flow of air, kg/s (lb/s)
R = engineering gas constant for air, 8.314 kJ/k mol K (SI units)
53.3 ft.lb/(lb air).°R (US customary units)
$T_1$ = absolute inlet temperature, K (°R)
$P_1$ = absolute inlet pressure, atm ($lb_f/in^2$)
$P_2$ = absolute outlet pressure, atm ($lb_f/in^2$)
$n = (k-1)/k - 0.283$ for air
k = 1.395 for air
29.7 = constant for SI unit conversion
550 = ft.lb/s.hp
E = efficiency (usual range for compressors is 0.70 to 0.90

### *Formulas used for computing the no. of diffusers and diffuser density*

As for the total no. of diffusers required, the standard oxygen transfer rate must be solved first to determine the total no. of diffusers along with the diffuser density for each of the three zones of the aeration tank (synthesis, endogenous and carbonaceous zones).

$$q_s = (0.04\ \text{scfm/lb O2 /d})\ \text{SOTR/SOE}$$
$$q_s = (0.0275\ \text{cum/min/lb O2 /d})\ \text{SOTR/SOE} \quad (36)$$

$$q_s = \frac{0.04(R_o)}{[(\text{SOTE})(\text{AOTR/SOTR})(F_a)(N_d)]} \quad (37)$$

$$q_s = \frac{0.00275(R_o)}{[(\text{SOTE})(\text{AOTR/SOTR})(F_a)(N_d)]}\text{; metric}$$

Where:

q = air flow rate, scfm/diffuser, $m^3$/min diffuser
$R_o$ = actual oxygen requirement, kg/d, lb/d
SOTE = standard OTE
F = average fouling factor
$N_d$ = number of diffusers in the zone
0.04 = conversion factor to obtain scfm of air from lb/d of oxygen
SOTR = oxygen transfer rate under standard conditions (20°C, 1atm)

Vol. 3, No. 1, (2022), pp., 1-15.

### *Piping Design*

Piping design will depend on the actual dimensions and lot area of the existing WWTP. Basic principles of fluid mechanics was used to determine headloss in air piping systems. At the rates of flow and velocities found in these systems, air can be treated as an incompressible fluid within the pipe and the Darcy-Weisback equation can be used to determine headloss.

$$\mathrm{N_R} = \frac{\mathrm{vD\rho}}{\mu};\ \frac{4Q}{\pi} \times \frac{\rho}{\mu} \times \frac{1}{D} \tag{38}$$

$$\mathrm{L}_E = \frac{\mathrm{KD}}{\mathrm{f}} \tag{39}$$

$$\mathrm{H_L} = \frac{\mathrm{fL}}{\mathrm{D}} \times \frac{\mathrm{v}^2}{2\mathrm{g}};\ \mathrm{H_L} = \frac{\mathrm{L}}{\mathrm{D}} \times \frac{\rho \mathrm{v}^2}{2} \tag{40}$$

Where:

$N_R$ = Reynold’s Number
ρ = density, kg/m3
D = diameter, m
V = velocity, m/s
μ = abs. viscosity, Pa. sec, kg/m.s
f = friction coefficient
$H_L$ = head loss, m
f = friction factor
L = length of pipe, m
d = inner dia. of pipe, m
v = velocity of fluid, m/s
g = acceleration due to gravity m/s$^2$

## AUTHOR PROFILE

**Manalang Mark Franklin P.** A Registered Mechanical Engineer, Registered Master Plumber, and a Registered Environmental Impact Assessment (EIA) Preparer. Mark has more than 8 years' experience in water and sanitation management with solid exposure in the operation, design, and failure analysis of manufacturing equipment. Currently, he is the Used-Water Operations Manager for Clark Water Corporation (CWC) and a concurrent Asset and CAPEX Planner for Manila Water's consortium for Saudi Arabia's state-run water agency, National Water Company. Mark is also a part-time teacher and a guest lecturer for Engineering at Holy Angel University (HAU) and Holy Cross College (HCC). He is also a freelancer - with a Research & Design background, focused on asset management, water quality and wastewater designs for individual clients up to municipal level treatment plants. Mark also has several research publications to his credit in several international journals. Mark finished both degrees for BS Mechanical Engineering and Master's in Engineering Management at HAU and his PhD in Nueva Ecija University of Science and Technology (NEUST).

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