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WATER REUSE AND ENVIRONMENTAL CONSERVATION PROJECT

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Aqaba Natural Wastewater Treatment Plant

Concept Design Report for Upgrades to Improve Effluent Quality for Water Reuse

June 2015

IMPLEMENTED BY AECOM

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AQABA NATURAL WASTEWATER TREATMENT PLANT CONCEPT DESIGN REPORT FOR UPGRADES TO IMPROVE EFFLUENT QUALITY FOR WATER REUSE, JUNE 2015

Prepared by:

AECOM

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ABBREVIATIONS AND ACRONYMS

AOR	Actual Oxygen Required
ASB	Aerated Stabilization Basin
AWC	Aqaba Water Company
BOD_5	5-day Biochemical Oxygen Demand
CAPEX	Capital expenditures
CDR	Concept Design Report
COD	Chemical Oxygen Demand
FOG	Fats, Oil and Grease
HRT	Hydraulic Residence Time
IR	Internal Recycle
lbs	Pounds
m^3/d	Cubic meters per day
NH_4-N	Ammonia as Nitrogen
NO_3-N	Nitrate as Nitrogen
NOx-N	Nitrate as Nitrogen (NO_3-N) plus Nitrite as Nitrogen (NO_2-N)
NTU	Nephelometric Turbidity Units
O&M	Operations and Maintenance
OPEX	Operating expenditures
PDN	Post-Denitrification
PS&E	Plans, Specifications, and Estimates
rbCOD	Readily Biodegradable COD
SOR	Standard Oxygen Requirement
TDS	Total Dissolved Solids
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TSS	Total Suspended Solids
UV	Ultraviolet
WAJ	Water Authority of Jordan
WAS	Waste Activated Sludge
WRECP	Water Reuse and Environmental Conservation Project
WWTP	Wastewater Treatment Plant

1 INTRODUCTION

The USAID Water Reuse and Environmental Conservation Project (WRECP) works throughout Jordan in institutional capacity building, pollution prevention for industries, solid waste and wastewater management, and water reuse. The project goal is to protect and conserve scarce resources through regulation, education, and coordination with industry, local communities and the private sector. The project is implemented by AECOM and a team of international and Jordanian partner firms. This five-year project has four primary tasks:

- Task 1 – Institutional and Regulatory Strengthening
- Task 2 – Pollution Prevention and Industrial Water Management
- Task 3 – Disposal sites Rehabilitation and Feasibility Studies
- Task 4 – Water Reuse for Community Livelihood Enhancement, including biosolids

Figure 1-1. Project Location

As part of Task 2, the project has prepared this report, to evaluate improvements that can be made at the pond system at the Aqaba Wastewater Treatment Plant (WWTP) to maximize beneficial reuse of treated sewage effluent by improving the effluent quality making it suitable for a variety of end uses in compliance with Jordanian Standard (JS) 893:2206, Level A.

1.1 Existing and Future Conditions

The Aqaba Water Company (AWC) owns and maintains a 21,000-m^3/day Wastewater Treatment Plant (WWTP) in the northwest portion of Aqaba, Jordan. The location of the WWTP is shown in Figure 1-1.

The WWTP operates with two parallel treatment trains. One, referred to as the “Mechanical Plant,” consists of an activated sludge process utilizing oxidation channels. The other, referred to as the “Natural Plant,” uses facultative ponds followed by maturation ponds. The layout of the WWTP is shown in Figure 1-2.

Aqaba is a fast-growing area, and this trend is expected to continue. Projected future population growth and associated wastewater flows have been documented in the “Aqaba Wastewater Collection System Master Plan,” dated July 2010. That report examined two future flow scenarios: one with major new development in the Aqaba area (Scenario 1) and one without major new development in the Aqaba area (Scenario 2). These scenarios are summarized in Table 1-1.

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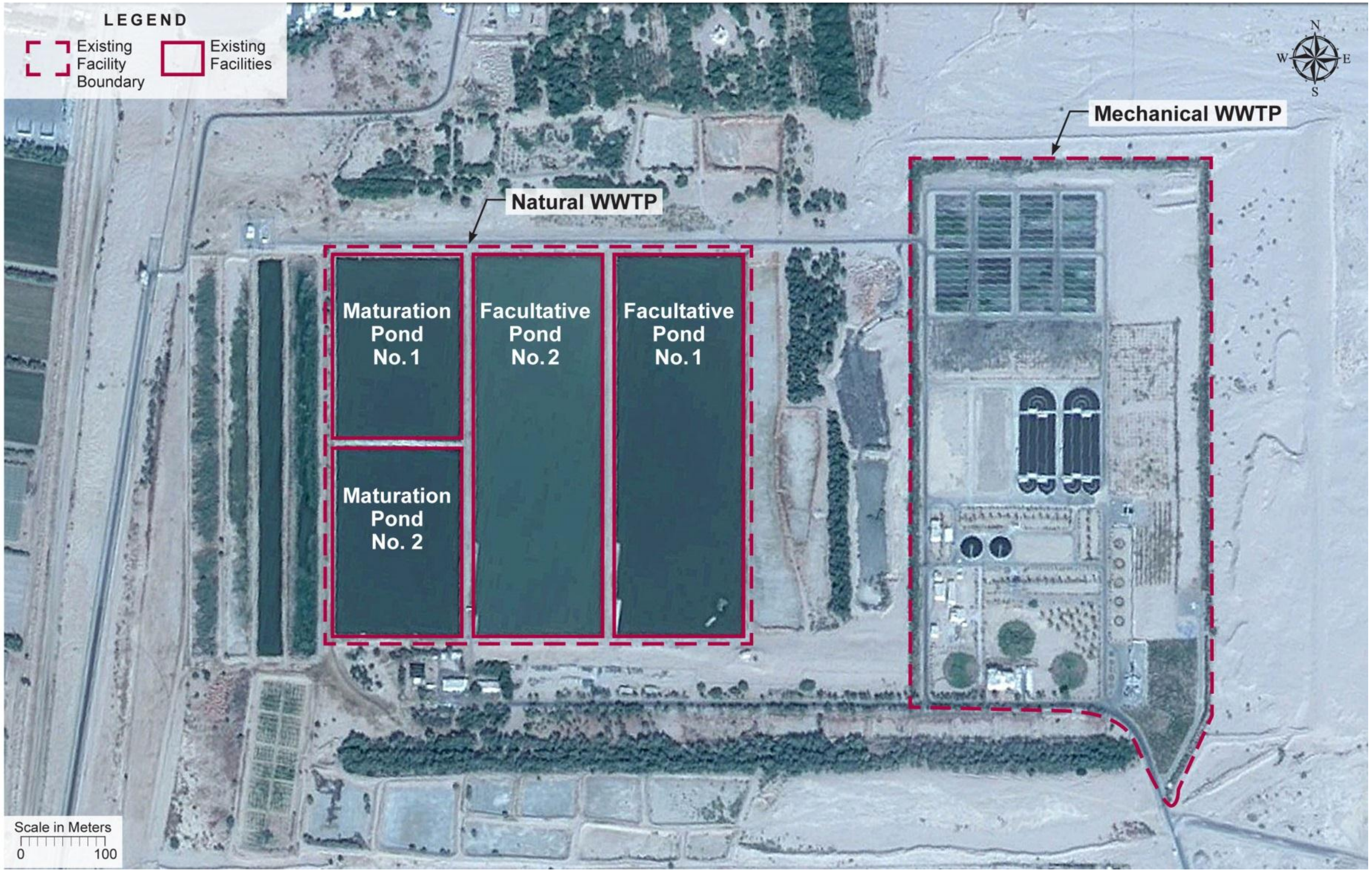
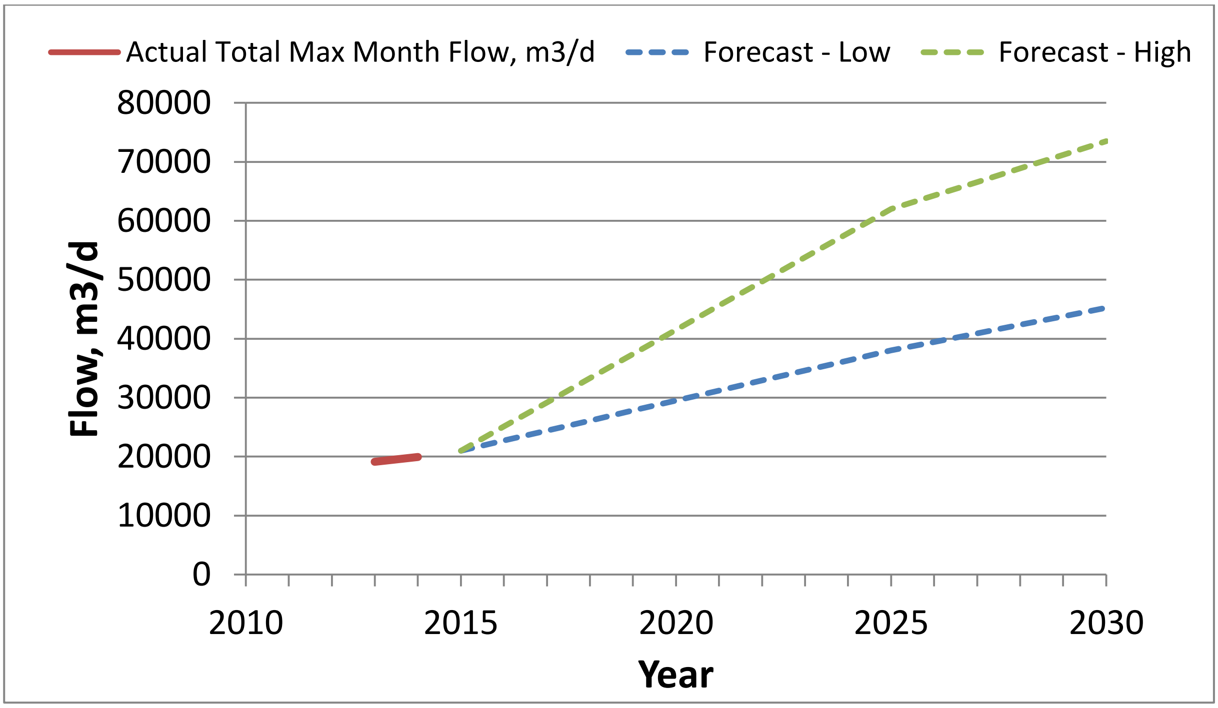


Figure 1-2. Aqaba WWTP - Existing Layout

Table 1.1. Future Growth Scenarios

Description	2010	2020 Additional Flow/Units	2030 Additional Flow/Units	Total Future Flow/Units
Scenario 1 Average Daily Flows (m^3/d)	21,000	40,992	22,978	75,970
Scenario 2 Average Daily Flow (m^3/d)	21,000	17,000	14,475	43,475

The growth projections in the 2010 Master Plan have not been realized because of the global economic conditions during the last 5 years. For the purposes of this report, it is assumed that the growth projections predicted in the 2010 Master to begin in Year 2010 will be deferred five years, starting in 2015 as shown in **1**. Maximum month flows for 2013 and 2014 are also shown based on historical data provided by AWC.



1. Aqaba WWTP Wastewater Influent Flow Projections

The original facultative pond system of the Natural WWTP was constructed in 1986. The ponds were modified as part of the 2005 Mechanical Plant construction project. The ponds are designed based on the design parameters:

Design flow:	9,000 m^3/d
BOD_5:	390 mg/l
Facultative Ponds:	Two (in parallel), each at 450 m x 150 m x 2 m SWD
Maturation Ponds:	Two (in series), each at 225 m x 150 m x 1.3 m SWD

Septage is received at the Aqaba WWTP delivered by tanker trucks. Tankers currently discharge liquid waste into a manhole that discharges directly into Facultative Pond 2. The plant operations staff elected to discontinue using the existing septage receiving station located upstream of the Headworks (screening and grit removal) because of several concerns including problems with odors, difficultly cleaning debris captured by the coarse screens at the discharge structure, and general lack of control over the contents and a potential negative impact on the downstream biological treatment process in the Mechanical Plant.

Underdrainage from the sludge drying beds is currently routed to the Maturation Ponds, which are a part of the Natural Plant. AWC has prepared plans to add a new sludge thickener with the ability to return decant water to the Headworks, which would significantly reduce or eliminate the need to continue discharging drying bed underdrainage to the Natural Plant.

Waste filter backwash water is currently routed to a plant drain pumping station that is understood to also service the sanitary wastewater from the Administration Building. This wastewater is pumped to the Headworks for treatment. However, for the purpose of this study, it is assumed that the waste filter backwash water is routed directly to the Natural Plant for treatment.

A process flow diagram of the existing Natural WWTP is shown below in Figure 1-4.

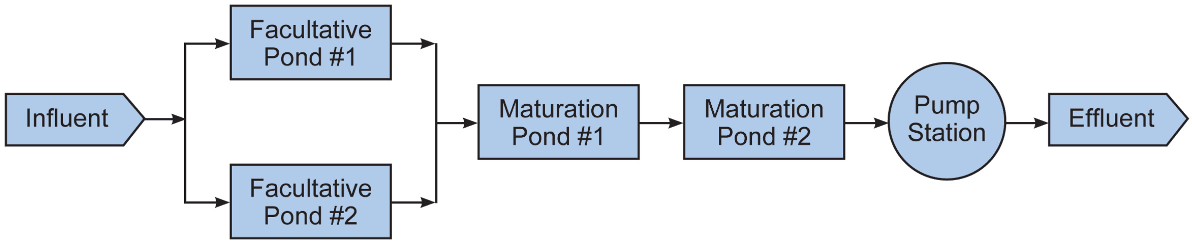


Figure 1-3. Natural WWTP Existing Process Flow Diagram

In recent years, the flow and BOD to the ponds have been at or slightly below design, and process performance has been reasonable for a system such as this; however, it has not met the Jordanian Standards for water reuse (JS893:2006). It is the desire of AWC to improve the quality of the natural plant effluent so that it meets JS893:2006, Level A.

A review of Table-1-2 shows that an improvement in treatment efficiency is needed for BOD, Chemical Oxygen Demand (COD), Total Suspended Solids (TSS), and nitrogen species. While there are no data for Total Nitrogen (TN) or Nitrate as N (NO_3-N), the historical Ammonia as N (NH_4-N) values already exceed the effluent standard for TN, without any other N species considered. Similarly, while there are no data for E. Coli or Helminth Eggs, the elevated values for both Total Coliform and Fecal Coliform would suggest that the E. Coli/Helminth requirements are not being achieved by the current system. While there are also no available data on fats, oil and grease (FOG) in the wastewater, it is reasonable to assume that the relatively long detention time afforded by the Mechanical Plant process will be able to biologically remove FOG to comply with the waster reuse standard of 8 mg/l.

Accordingly, the scope of this Concept Design Report (CDR) is to develop and evaluate technically feasible options to improve the Natural Plant effluent quality to the levels required by JS893:2006, Level A. Options include upgrades to the existing ponds and the addition of new treatment processes.

It is assumed that all improvements can be made within the footprint of the existing site such that no additional land acquisition is required. The proposed scope of work consists of developing a conceptual plan that can meet the intended result for pre-planning-cost estimating purposes. A more in-depth feasibility study to evaluate all the potentially available options is beyond the scope of this work.

It is understood that AWC is planning to expand the existing Mechanical Treatment Plant from its existing capacity of 12,000 m^3/d. The concepts developed in this proposed scope of work could be implemented as part of the planned plant expansion. If both are to be built at the same time, economies of scale could be realized and costs reduced by combining common treatment processes, such as disinfection by chlorination, into one common facility instead of two separate and independent systems.

This CDR identifies a conceptual plan for technically feasible approaches to improving the stabilization pond effluent quality and block diagrams and order-of-magnitude cost estimates suitable for planning purposes.

Table 1-2. Comparison of Historical Natural Plant Effluent Quality with JS 893:2006 Requirements

Standards and characteristics	Natural Plant Effluent Quality (Historical)			Permitted limits according to aspects of uses	
				Landscape within City Limits	Landscape outside City Limits
	Ave	Min	Max	A	B
BOD_5 (mg/l)	34	27	40	30	200
COD (mg/l)	389	56	464	100	500
DO (mg/l)	6.8	4.0	9.0	>2	-
TDS (mg/l)	704	565	933	1,500	1,500
TSS (mg/l)	276	247	313	50	200
pH (S.U.)	7.9	7.6	8.0	6-9	6-9
Turbidity (NTU)	-	No Data	-	10	-
Nitrate (mg/l) [1]	-	No Data	-	30	45
TN (mg/l)	-	No Data	-	45	70
Ammonia (mg/l) [2]	59	39	94	N/A	N/A
E. Coli MPN or cfu/100ml	-	No Data	-	100	1000
Total Coliform (cfu/100 ml)	>1,600	>1,600	>1,600	N/A	N/A
Fecal Coliform (cfu/100ml)	>1,600	>1,600	>1,600	N/A	N/A
Intestinal Helminth Eggs (egg/l)	-	No Data	-	< or = 1	< or = 1
FOG (mg/l)	-	No Data	-	8.0	8.0

[1] JS 893:2006 lists the parameter as “nitrate” and do not differentiate between this being measured as Nitrate-as-Ion or Nitrate-as-Nitrogen. Given that Nitrate-as-N in aerobically-treated sewage effluent can easily represent more than two-thirds of the Total Nitrogen, then for the purpose of this study, the values specified as limits for nitrates are assumed to be for Nitrate-as-N. For the opposite interpretation to be true, the Nitrate-as-N would need to be calculated from the Nitrate-as-ion by dividing by 4.43 (e.g. 30 m/l Nitrate-as-Ion / 4.43 = 6.8 mg/l Nitrate-as-N) which seems unrealistic given a Total Nitrogen limit of 45 mg/l. Also, a low Nitrate-as-N concentration in reclaimed water intended for irrigation doesn’t make sense because vegetation irrigated with water rich in soluble nitrates will benefit from this essential growth nutrient present in the irrigation water.

[2] JS 893:2006 lists the parameter as “ammonia” and do not differentiate between this being measured as Ammonia-as-Ion or Ammonia-as-Nitrogen. Ammonia-as-N = Ammonia-as-Ion / 1.29

1.2 Major Items of Consideration

The following is a summary of the sections of this CDR and the information they present.

1.2.1 Design Basis

Section 2 presents a general process description of the Natural WWTP, tabulates existing WWTP data and describes the present and future loadings to the Natural WWTP. Aeration requirements for the Natural WWTP are also discussed, as well as general alternatives for pond upgrades.

1.2.2 Presentation of Upgrade Options

Section 3 provides more detailed descriptions of the upgrade options for the Natural WWTP, including upgrading the existing ponds with surface aspirators or adding a new aerated pond with surface mechanical floating aerators. Process flow diagrams of these options are also presented.

1.2.3 Description of Proposed Unit Processes

Section 4 presents a more detailed description of each unit process for the upgrade option discussed in Section 3, including pond configurations, new downstream processes including denitrification filters and chlorine contact tanks. General layouts of each unit process are also presented.

1.2.4 Schedule

Section 5 presents a general schedule to implement upgrades and improvements at the Natural WWTP. The schedule considerations include additional planning, preliminary and final design, regulatory approvals, tendering, award, and construction assuming a conventional design-bid-build delivery method.

1.2.5 Project and Life Cycle Costs

Section 6 presents planning level construction cost estimates for major process equipment and develops preliminary construction costs of the options that are evaluated to upgrade the Aqaba Natural WWTP.

In evaluating alternatives, construction costs and operations and maintenance (O&M) costs are evaluated. The annual O&M costs include estimates of power and chemical costs. Escalation of these costs over time must also be considered.

1.2.6 Conclusions and Recommendations

Section 7 presents conceptual arrangements of the options presented and steps required to implement a selected option.

2 DESIGN BASIS

2.1 General Process Description

The plant features both a mechanical treatment process with oxidation channels, clarifiers, filters and ultraviolet (UV) disinfection and a natural treatment process featuring facultative and maturation ponds. The mechanical treatment process is rated for 12,000 m^3/day and the natural process is rated for 9,000 m^3/day. Influent arrives at the headworks by a single 1,200-mm gravity sewer. However, the majority of the flow to this gravity sewer is contributed by pump stations. No effluent is discharged to the nearby Red Sea. The effluent from the Natural WWTP is sent to evaporation ponds or agriculture (irrigation). The effluent from the mechanical process is sent to evaporation ponds, industrial users (for cooling water) and agriculture (including irrigation).

2.2 Design Basis

For the purpose of this study, the design basis assumes that raw influent and septage are blended upstream of the headworks facility servicing both the mechanical and natural plants. Using available data, the mechanical plant design team developed headworks effluent characteristics as indicated below. For consistency, these same characteristics were used in the development of the Aerated Stabilization Basins (ASB) design.

Max Month BOD, mg/L:	514
Max Month TSS, mg/L:	437
Max Month TKN, mg/L:	137

The design temperature established ranges from a low of 18 degrees C to a high of 28 degrees C.

The total design flow to the Natural Plant is assumed to be 11,260 m^3/d for the purpose of this study. The split of combined influent/septage allocated to the Natural WWTP is capped at 9,000 m^3/d. In addition to the 9,000 m^3/d of Headworks (HW) effluent, the design basis assumes that sidestreams from the Mechanical Plant are to be sent to the Natural Plant. While the flow from the HW to the natural plant is to be maintained at 9,000 m^3/d, an additional flow of 2,260 m^3/day to the Natural Plant is based on the estimated quantities of sidestream flows under a future condition when the Mechanical Plant is processing 29,000 m^3/d (assuming the low-flow projection beginning Year 2015 after 10 years).

Sludge drying bed under-drainage and tertiary filter backwash from the mechanical plant will vary with forward flow. As a result, these future scenarios needed to be evaluated in terms of their impact to the ASB design. Certain assumptions were derived from the result of the process modeling done on the mechanical plant. Wastewater characteristics of the sidestreams were assumed based on textbook reference values and engineering judgment. Based on estimated flows from each sidestream, a mass balance calculation was used to redefine the characteristics of the combined sewerage from the Headworks and the sidestreams that would become the Natural Plant influent. A summary of the resultant influent concentrations and loads to the ASB system is shown in Table 2-1.

Table 2-1. Proposed Design Basis for Flow and Loadings to Natural Plant

Parameter	At Design Flow 11,260 m^3/d	
	Concentration, mg/l	Loading, kg/d
TSS	768	8,647
BOD_5	488	5,494
TKN	175	1,971

The two existing facultative ponds are the same size, and each has a volume of 132,000 m^3 (450 m x 150 m x 2 m).

Process Design to Convert Existing Facultative Ponds to ASB

For BOD removal, the required detention time is defined by the following decay equation.

$$BOD_{out} = BOD_{in}\left(\frac{1}{1+k_d\Theta}\right)$$

Where:

BOD_{out} =	Effluent BOD, mg/l or lbs/d
BOD_{in} =	Influent BOD, mg/l or lbs/d
k_d =	Decay coefficient at operating temperature, d^{-1}
Θ =	Reactor hydraulic detention time, d

Textbook[1] values for the decay coefficient at 20 degrees C range from 0.5 to 1.5 d^{-1}. Based on the lack of any kinetic data from this site, the conservative end of the range was used. When the decay factor for 20 degrees C is adjusted lower to reflect lower water temperatures during cold weather, and using a target effluent BOD of 30 mg/l, a total aerobic volume of 369,000 m^3 was estimated as required. Given that the existing lagoons have volumes of approximately 132,000 m^3 each, an alternate approach was sought.

Aerated pond volume requirements can be reduced by providing two baffled zones in series. This effectively provides kinetic benefits by taking advantage of higher substrate concentrations in the first zone. Providing two equally sized zones in series results in a total aerobic volume required of 173,000 m^3. Guidelines for settling indicate a required Hydraulic Retention Time (HRT) of 1 to 2 days, or 22,500 m^3 for 2 days at the design flow of 11,260 m^3/day. For the existing facultative ponds, this leaves a balance of 67,630 m^3 in the available volume of the two lagoons combined.

Process modeling of the above configuration indicates that, in ideal conditions, the design could be adequate for full BOD removal and full nitrification, resulting in extremely high NO_3-N levels (as high as 150 mg/l). In order to reduce NO_3-N levels, the remaining available lagoon volume was allocated for anoxic operation. Process modeling indicated that this could reduce effluent NO_x-N levels down to approximately 80 mg/l by using a 100% Internal Recycle (IR) rate. Effluent NO_x-N removals did not improve appreciably at higher IR rates because of the relatively low carbon-to-nitrogen ratio in the lagoon influent. All of this resulted in the following lagoon configuration, as summarized in Table 2-2 below:

---

[1] Wastewater Engineering Treatment and Reuse, Metcalf & Eddy, 4th edition, page 843.

Table 2-2. Final Design at 11,260 m^3/day (2 Trains in Parallel, 2 Aerobic Zones in Series/each)

	HRT Design Criteria, days	Pond 1 Volume, m^3	Pond 2 Volume, m^3
Anoxic Zone	6.0	33,815	33,815
Aerobic Zone A	7.7	43,253	43,253
Aerobic Zone B	7.7	43,253	43,253
Settling Zone	2.0	11,260	11,260
Volume, m^3	23.4	131,581	131,581

For the existing facultative ponds, the modeled volumes were adjusted down by 15 percent to account for solids deposition in the existing lagoons and AWC's desire to avoid dredging at this time.

It should be cautioned that BioWin™ modeling is based on ideal reactors operated in series. Because the existing basin is shallow with irregular bottom contours, and the proposed mechanical mixers and aspirators will result in non-uniform mixing, the reliability of process modeling results are less certain and may not be a true representation of actual conditions in the field.

Another option was developed to convert the existing facultative pond a partially-mixed aerated lagoon; however, this option was eliminated from further consideration early during the study. That option included dividing each of the existing facultative ponds in half, so that one half could be used as a partially-mixed aerated lagoon and the other half could be converted to effluent storage after dredging and cleaning. This option was determined to be non-viable because of insufficient aerobic volume necessary to provide adequate retention based on a conservatively assumed decay rate.

Alternative (Deeper) ASB Design
Another option is to use a new, deeper lagoon, which might facilitate the use of more energy efficient aerators. It is intended that that shallower lagoons use surface aspirators to minimize risk of potential damage to the existing synthetic pond bottom liner. The options to reconfigure the existing lagoons and to construct a deeper lagoon are presented in more detail in Section 3.

Aeration Requirements
Actual oxygen requirements (AORs) were derived from the process model for both options. The BioWin™ model predicted AORs about 15 percent lower than an AOR calculated using the conventional manual method of multiplying BOD and TKN by standard ratios of 1.3 and 4.6 kg/kg, respectively. This model result is reasonable because it reflects some degree of readily biodegradable COD (rbCOD) consumption occurring in the anoxic zone.

These AORs were then converted to Standard Oxygen Requirements (SORs) using the standard textbook[2] corrections. Table 2-3 presents a summary of AOR and SOR values for the aerobic zones based on the design flow of 11,260 m^3/d.

[2] Wastewater Engineering Treatment and Reuse, Metcalf & Eddy, 4th edition, pages 429-430

Table 2-3. AOR and SOR Values for Design Flow 11,260 m^3/d

Pond Configuration	Existing Ponds with Surface Aspirators		New Deeper Pond with Aerators
	Per Train	Total	Total
Aerobic Zone A			
Volume, m^3	43,253	86,506	86,506
AOR, kg/hr	173	346	346
SOR, kg/hr	310	620	708
Aerobic Zone B			
Volume, m^3	43,253	86,506	86,506
AOR, kg/hr	85	170	170
SOR, kg/hr	153	306	348

Downstream Systems

It is assumed that effluent from the settling zones of the Aerated Stabilization Basins will continue to flow to the existing maturation ponds for polishing. However the requirements for Level A reuse dictate treatment levels (particularly for TSS, TN, NO_3, and bacteriological constituents) beyond what can be reliably expected from an ASB system. Modeling indicates full nitrification with the proposed design, which is reasonable given the relatively elevated temperatures of this system compared to natural systems in North America. However, NO_3-N levels are expected to still be elevated due to poor carbon to nitrogen levels in the ASB influent. Combined with the need to reliably meet the TSS and turbidity requirements, this resulted in the selection of a post-denitrification (PDN) filter as a tertiary process step.

For the purpose of this study, the project team assumed that a sensible interpretation of the Nitrate limit in the Jordanian Standards for water reuse is 30 mg/l NO_3-N. Further, to account for times when the actual aerated stabilization basins cannot consistently provide ideal operating conditions necessary to achieve the calculated denitrification effect within the ASB, an influent NOx-N concentration higher than the calculated value is assumed.

Modeling suggests the ASBs may be capable of reducing NO_3-N to effluent NO_3-N of approximately 80 mg/l based on ideal conditions that can be difficult to consistently achieve for large aerated ponds under actual operating conditions. As a result, the influent concentration of NO_3-N to the filters was increased to 100 mg/l to account for some degradation in actual ASB denitrification efficiency relative to the modeled ideal reactor. The general criteria for denitrification are presented in Table 2-4.

Table 2-4. General Design Criteria for Denitrifying Filter System at Maximum Month Conditions

Parameter	Influent	Effluent
Flow, m^3/d	11,260	11,260
TSS, mg/l	75	50
NOx-N, mg/l	100	30
Turbidity, NTU	---	10
Min temp, ‘C	18	---

Further details of a proposed denitrification system are presented in Section 4.

Similarly, the requirements for E. Coli and helminth eggs demand final chlorination for disinfection. A standard 30-minute contact time at the future flow is the basis of design, using gaseous chlorine as the disinfectant. A target dose of 15 mg/l as Chlorine has been selected. Further details of a proposed chlorination system are presented in Section 4.

3 PRESENTATION OF UPGRADE OPTIONS

3.1 General

This section presents general descriptions of the upgrade options for the Natural WWTP and provides site plans and flow diagrams for these options. Based on the design data presented in Section 2, two main options were developed. The first is to convert the two existing facultative ponds into ASBs with surface aeration systems. The second is to add a new ASB at the WWTP site to treat all of the flow to the Natural WWTP. For each of these options, additional facilities would be added which include denitrification filters, an intermediate pumping to lift the flow to the filters, and disinfection. These additional systems are described in more detail in Section 4.

3.2 Convert Existing Facultative Ponds to ASBs – Option 1

Option 1 would involve the upgrade of the two existing facultative ponds with the installation of surface aeration systems and floating baffles to segregate each pond into four distinct zones. Each of the two existing ponds is approximately 450 m long, 150 m wide and 2 m deep. The four zones in each of the ponds would consist of an Anoxic Zone, Aerobic Zone A, Aerobic Zone B and a Settling Zone, each separated by floating baffles. A general layout of the proposed facilities is shown in Figure 3-1. As shown in this figure, additional facilities proposed would include denitrification filters and an intermediate pumping station to pump flow up to the filters, a chlorine contact tank for disinfection and a chlorination building to house the chlorination equipment including chlorine cylinders. A process flow diagram of the proposed facilities is shown in Figure 3-2. Additional details of the proposed facilities and pond upgrades are included in Section 4.

3.3 Construct New ASB – Option 2

Option 2 would consist of construction of a new pond which would be located between Facultative Pond No. 1 and the Mechanical WWTP. There is available land which could accommodate the footprint of 450 m x 150 m, which would be adequate for a new ASB. Using the volume requirements derived in the design of the existing lagoons as described in Section 2, an evaluation of what could be done within the available space at a deeper depth was conducted. In the absence of site soil conditions, an initial target depth of 4.6 m was selected. Using the same length to width ratio and slope as that of the existing lagoons, the footprint of 450 m x 150 m was derived. This results in a more efficient use of land area. For the same treatment capacity, for new ASB at 4.5 meters deep requires only half of the total area that would be required if the two existing 2-meter deep facultative ponds were converted to ASBs.

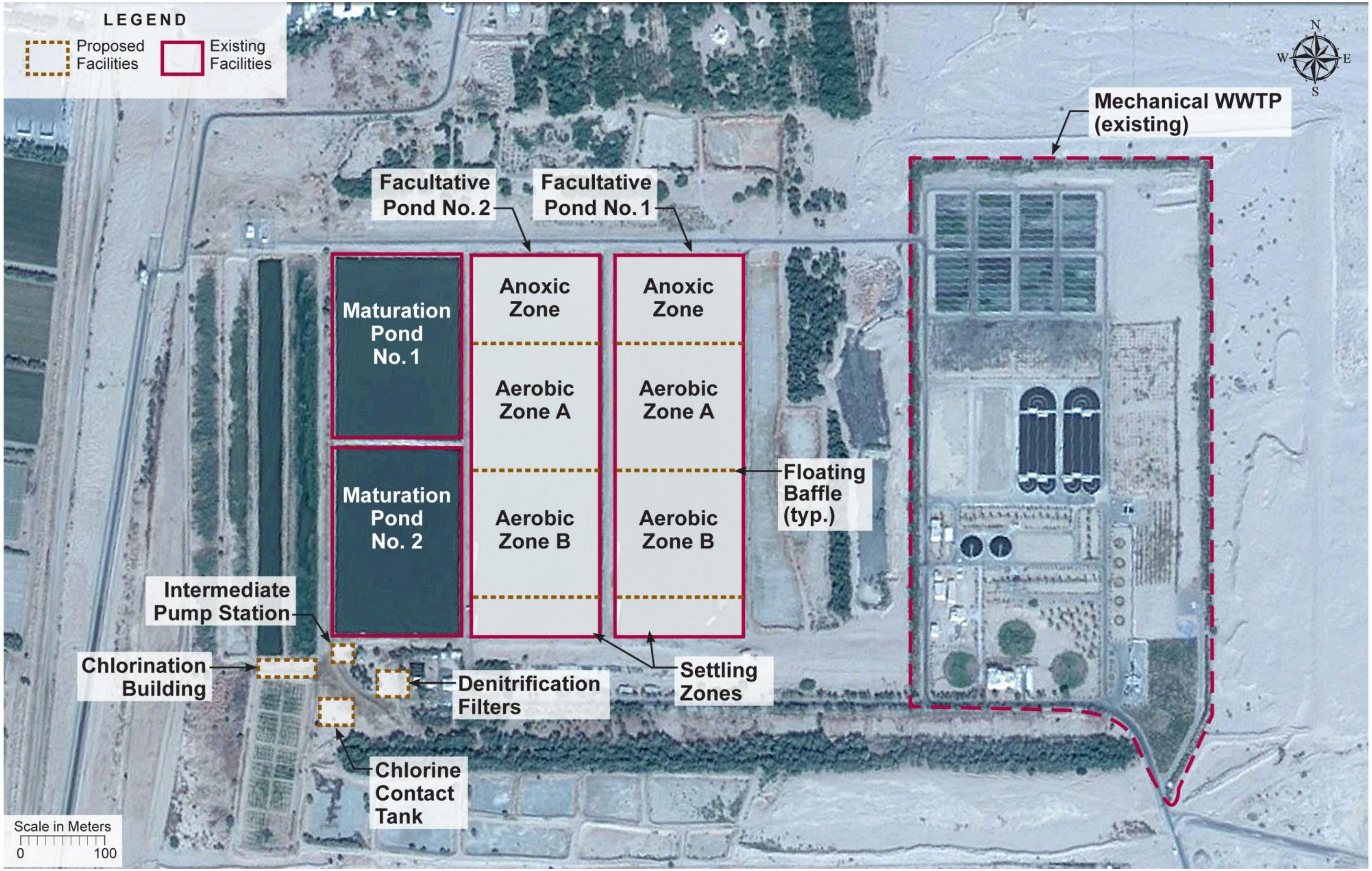


Figure 3-1 Natural WWTP Option 1 Proposed Layout Modify Existing Facultative Ponds

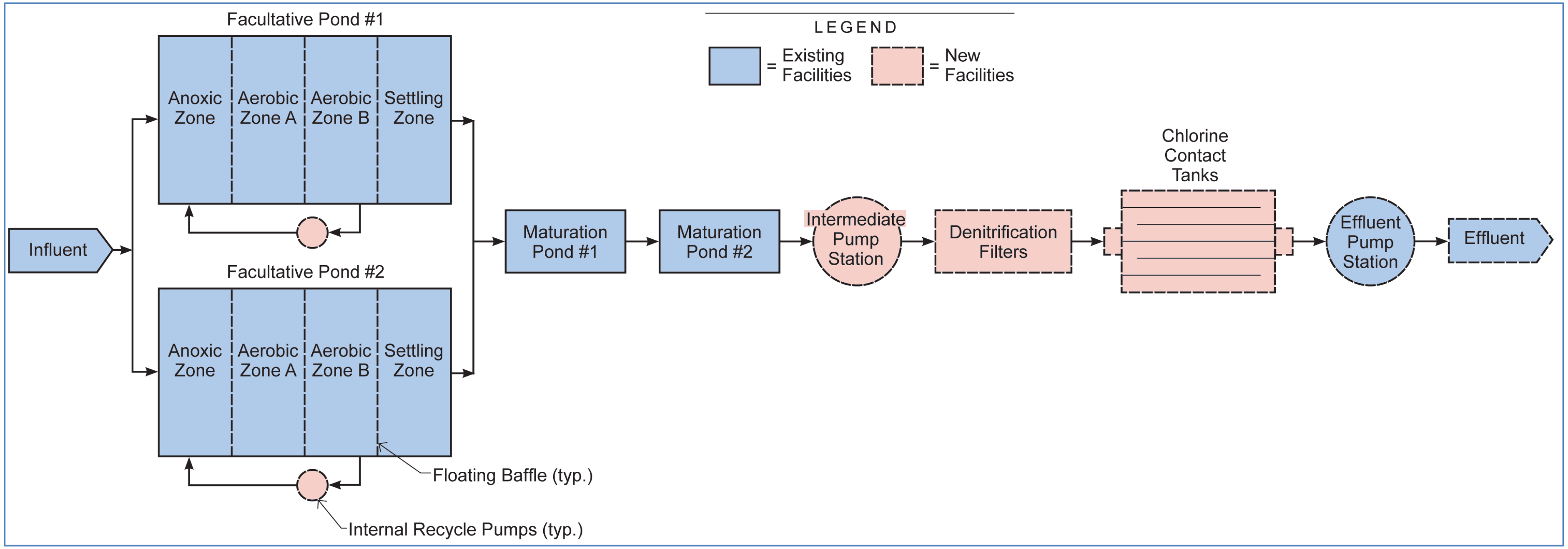


Figure 3-2. Natural WWTP Option 1 Process Flow Diagram of Existing Facultative Ponds Converted to ASBs

Surface aerators work more efficiently in depths greater than 3 meters and are generally more energy efficient at transferring oxygen than aspirating-type aeration. A layout of the proposed facilities is shown in Figure 3-3. It should be noted that for this option, the other proposed facilities would also be required similar to that shown for Option 1. A process flow diagram of the proposed facilities is shown in Figure 3-4.

3.4 General Assumptions

In developing the general layout of the proposed facilities as presented in the figures, no site survey was performed and no subsurface (geotechnical) investigations were conducted as part of this CDR. Also, it was assumed that no existing facilities were to be upgraded or rehabilitated for the options presented in this CDR. Construction of new facilities such as a new chlorination building and contact tank is proposed. In order to confirm that the options presented in this CDR are feasible, additional study is required, including a detailed site survey, geotechnical investigations and coordination with any planned Mechanical WWTP upgrades or additions.

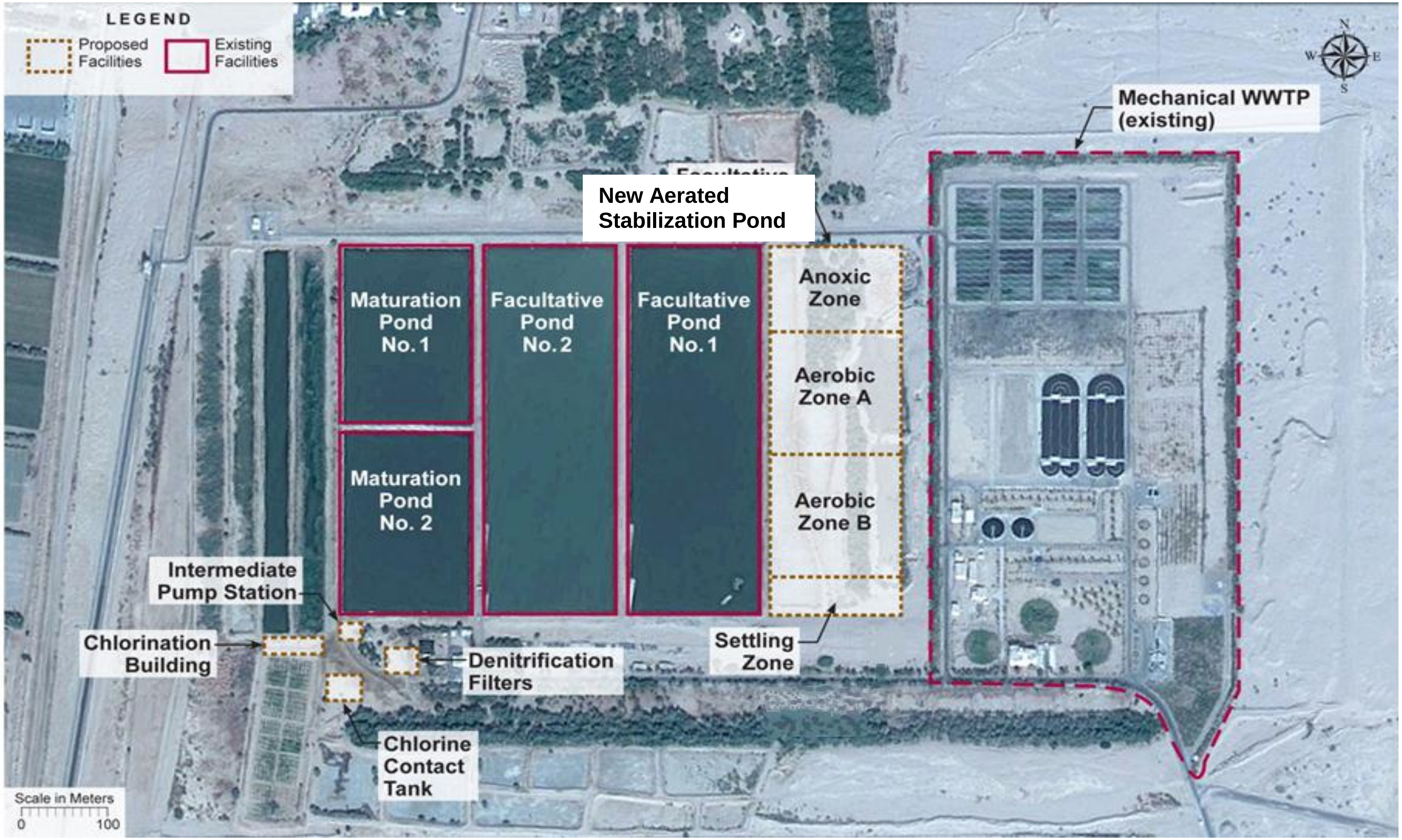


Figure 3-3 Natural WWTP Option 2 Proposed Layout of New Aerated Stabilization Basin

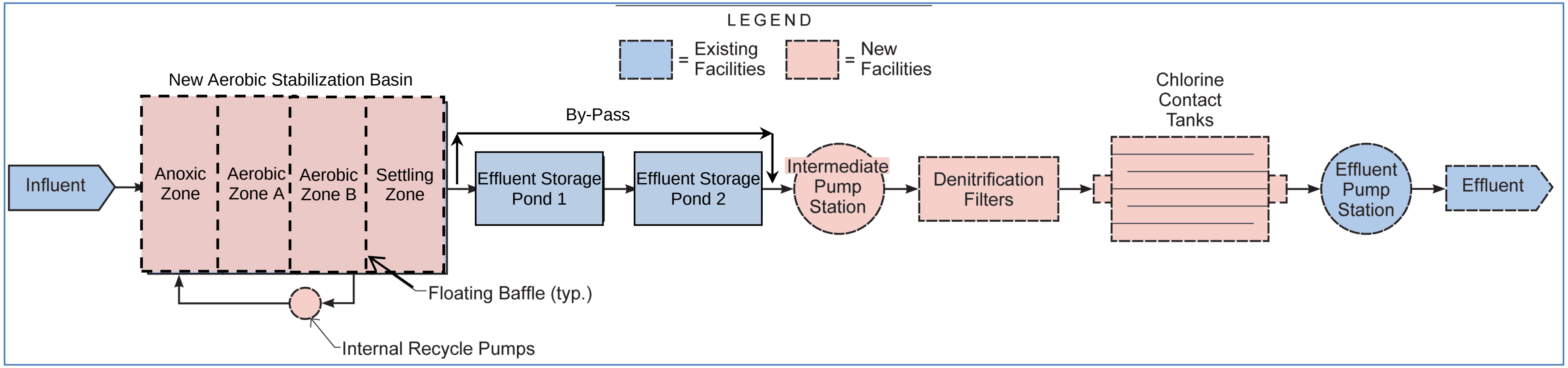


Figure 3-4 Natural WWTP Option 2 Process Flow Diagram of New Aerobic Stabilization Basin

4 DESCRIPTION OF PROPOSED UNIT PROCESSES

4.1 General

This section presents further information on the upgrades to the existing facultative ponds, construction of a new pond, as well as additional unit processes proposed for Options 1 and 2 as discussed in Section 3. Elements common to both Options 1 and 2 are anoxic zone surface mixers, new unit processes such as intermediate pumping and denitrifying filters, and a chlorination system for disinfecting pond effluent, including chlorine contact tanks and a chlorination building.

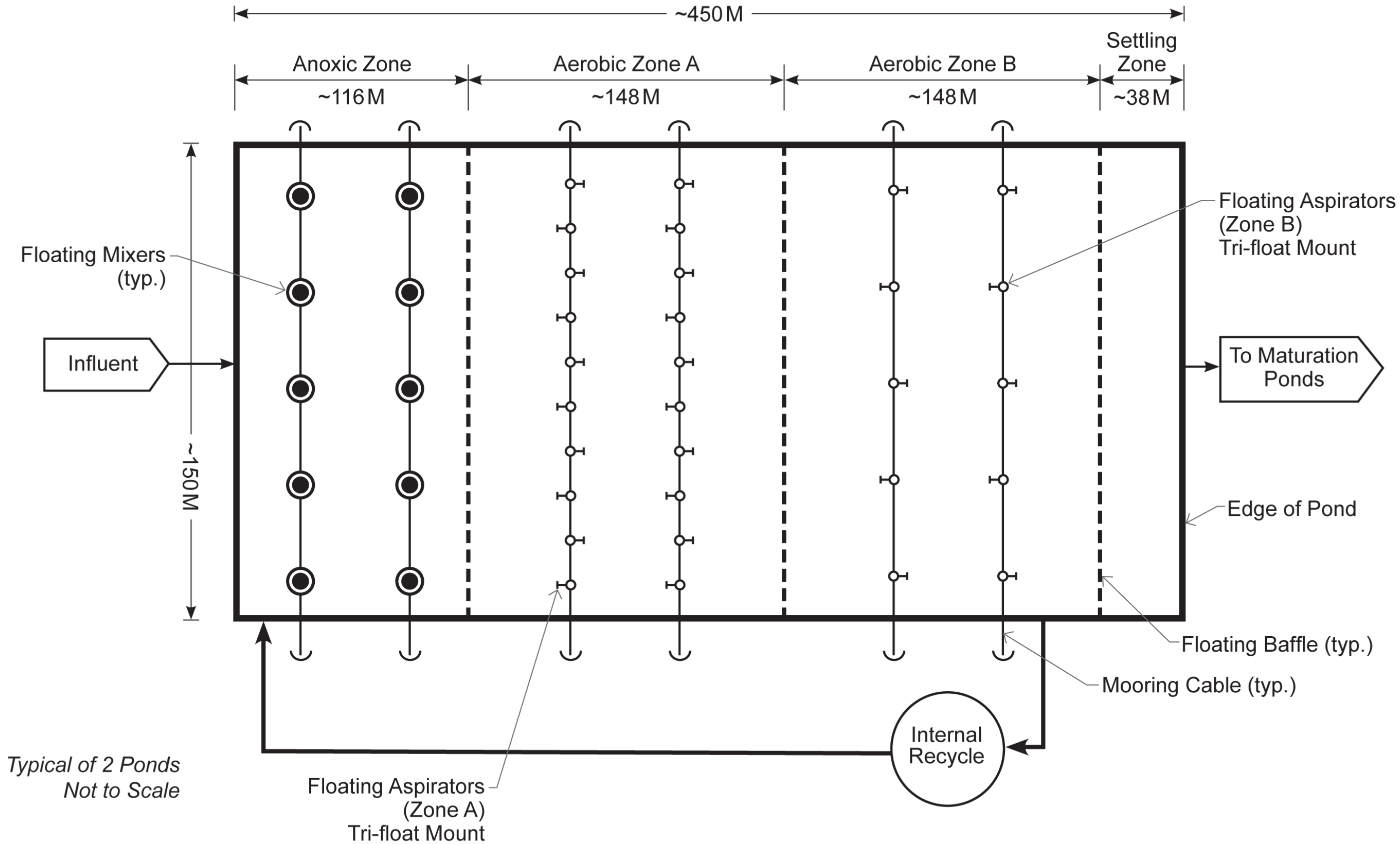
The power requirements for electrical motors at this location are assumed to be 380-volt, 50 Hz to meet local power characteristics and would apply to surface aerators, aspirators, pumping units and other electro-mechanical equipment.

4.2 Convert Existing Ponds Using Floating Surface Aspirators

As discussed in Section 3, upgrades to the existing facultative ponds will include floating surface aspirators and is designated as Option 1. The floating surface aspirators include a motor, shaft and propeller assembly installed on a framework mounted on pontoons. One vendor proposed aspirators installed at a 30-degree incline to accommodate the relatively shallow lagoon depth, each mounted on three pontoons. Aeration from this device is accomplished by an atmospheric air intake at the upper end of the diffuser shaft and injecting air through the shaft into the wastewater, with agitation provided by a propeller at the base of the shaft. The aspirators would be secured by mooring cables spanning the width of each lagoon. The aspirators would be installed to direct the stream of air bubbles in an alternating pattern.

In this option, the existing ponds would be divided up into four zones, including an Anoxic Zone, Aerobic Zones A and B, and a Settling Zone. Each zone would be separated by a floating baffle system of a flow-through design and anchored to the bottom of the lagoon. Three baffles would be required for each pond and would be approximately 150 m wide by 2 m deep. For each of the ponds, two internal recycle pumps would be provided (one duty, one stand-by) for recycling of flows from Aerobic Zone B to the Anoxic Zone. Each pumping system would be sized to be capable of returning flows up to 200 percent of the influent flow from the end of the Aerobic Zone B to the start of the Anoxic Zone.

A general layout of the configuration described above is shown in Figure 4-1.



FLOATING MIXER / ASPIRATOR SUMMARY*			
	Anoxic Zone	Aerobic Zone A	Aerobic Zone B
Number of Mixers	28	—	—
Number of Aspirators	—	30	14

* Totals shown are for two ponds. Diagram may not show all mixers and aspirators for clarity.

Figure 4-1 Option 1 Layout – Convert Existing Facultative Ponds to ASBs (Mixers and Floating Aspirators at 20 HP each)

Note that due to differing SORs for Aerobic Zones A and B, fewer aspirators would be required for Zone B. A general diagram of an aspirator is shown on Figure 4-2.

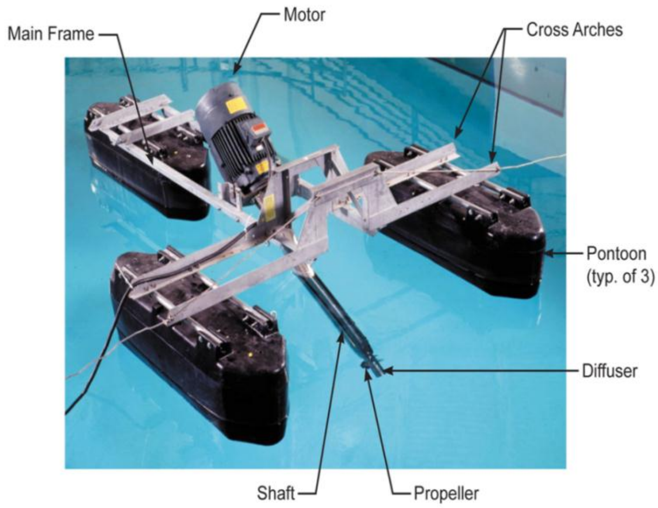


Figure 4-2 Typical Floating Aspirator

4.3 New ASB with Floating Aerators

For this alternative, designated as Option 2, one new lagoon would be constructed between the existing Facultative Pond No. 1 and the Mechanical WWTP as previously described in Section 3. This pond would be approximately 450 m long, 150 m wide and 4.6 m deep, the same footprint area as one of the existing facultative ponds. This pond would be similar in construction to the existing ponds, of earthen construction with a membrane lining and hardened side slopes. Similar to the options described above, this pond would also be divided into four zones: one anoxic, two aerobic, and one settling. Each zone would also be segregated by floating baffle curtains designed to accommodate the deeper lagoon depth of 4.6 m.

Aeration would be accomplished using surface mechanical floating aerators similar to the aerator shown in Figure 4-3. As with Option 1, two internal recycle pumps, one active and one stand-by, are included. A general layout of this option is shown in Figure 4-4.

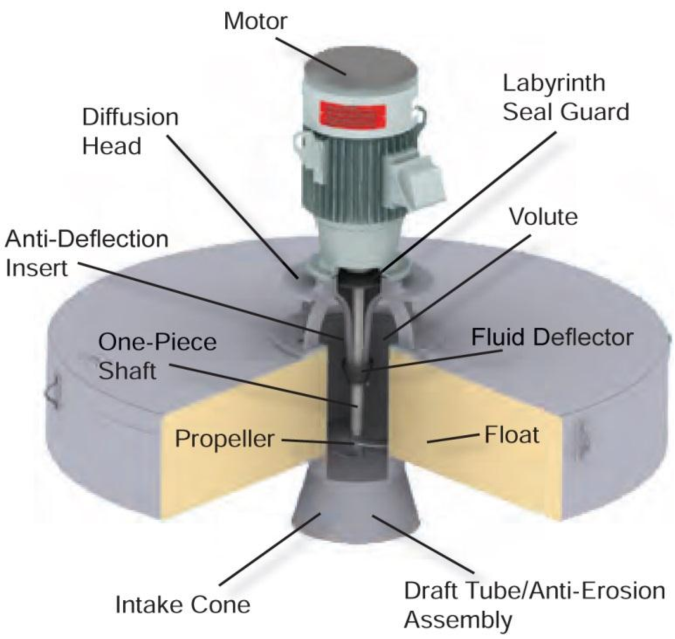
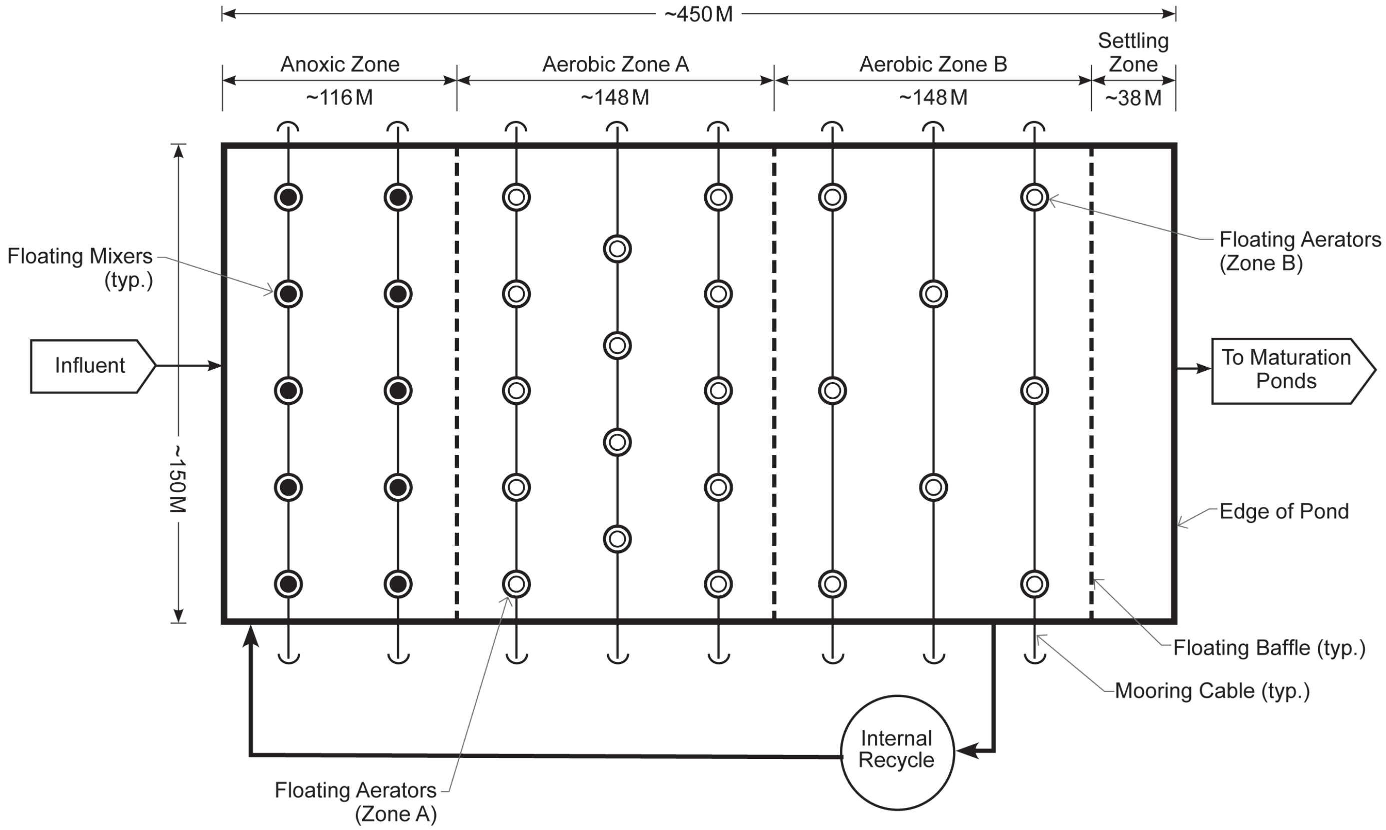


Figure 4-3 Typical Floating Aerator

4.5 Intermediate Pumping

Additional lift will likely be required to feed flow to the denitrifying filters. During the next stage of detailed design, the hydraulic profile needs to be revisited in greater detail to determine if the need for this intermediate pumping can be eliminated. If done properly, eliminating the intermediate pumping can substantially reduce operating and capital cost.

The location of the intermediate pumping station is shown in the process flow diagrams presented as Figures 3-2 and 3-4 (above) for Options 1 and 2, respectively. The general location of the intermediate pumping station on the site is shown in Figures 3-1 and 3-3 (above). For both Options 1 and 2, this intermediate pumping station would be sized to handle the maximum day flow of 11,260 m^3/d.



FLOATING AERATOR / MIXER SUMMARY*			
	Anoxic Zone	**Aerobic Zone A**	**Aerobic Zone B**
Number of Mixers	15	—	—
Number of Aerators	—	14	8

Figure 4-4 Option 2 Layout – New Aerated Stabilization Basin (Mixers and Floating Aerators at 25 HP each)

Two submersible pumps could be provided, one duty and one stand-by, each sized for the ASB design capacity. These pumps could be installed in a precast concrete wet well and adjacent valve chamber, with an above-ground control panel. Both the pump station wet well and valve chamber would be installed below ground and equipped with access hatches at the surface to protect the equipment from the weather.

4.6 Denitrification System

The denitrifying filter is selected as the most applicable process to remove nitrogen from ASB effluent.

As mentioned previously, process modeling predicted that the ASB effluent NO_x-N levels may be 80 mg/l; however, to account for times when field conditions are not favorable to ideal operating conditions, it is recommended that the denitrifying system be designed to treat ASB effluent with NO_x-N concentrations of 100 mg/l. Because of the relatively high concentration of nitrogen in the filter influent, Nitrate-N becomes the parameter that controls the design and sizing for the denitrifying filters for the design flow condition. Meeting the Nitrate-N effluent limit will also result in TSS concentrations substantially below the TSS effluent limit, which will be important to being able to meet the effluent turbidity limit.

For filter influent NO_3-N concentrations greater than 80 mg/l, two-stage denitrifying filters are necessary. The sizing of a 2-stage denitrifying filter system is shown in Table 4-1.

Table 4-1. Sizing of 2-Stage Denitrifying Filters

	Stage 1	Stage 2	Total
Number of Units	5	4	9
Filter Media Volume, ft^3	25,580	7,368	32,948
Depth of Filter Media, ft	10	6	n/a
Direction of Flow	Upflow	Downflow	n/a
Filter Surface Area, ft^2	2,558	1,228	3,785
Reactor Width, ft	9.5	9.5	n/a
Reactor Length per Unit, ft	54	32	n/a

Stage 1 consists of five up-flow filters with media depth of 10 feet. Stage 2 consists of four down-flow filters, each with media depth of 6 feet. The Stage 1 internal equipment includes sump cover plates, air headers and laterals, underdrain block gravel, stainless steel weir plates and electrically actuated butterfly valves. The Stage 2 internal equipment is similar to that for Stage 1 and also includes backwash air blowers, backwash water pumps, mudwell pumps and manual valves. The system also includes field instruments, a methanol chemical storage and feed system to provide a source of carbon essential to the biological process, and electrical control panels, all housed inside a building designed to protect the equipment from harsh weather conditions.

In order to avoid formation of undesirable nitrites (NO_2-N) in the Natural Plant effluent, the denitrification process is recommended to be “performed to completion,” which is based on an endpoint NO_3-N concentration of 2 mg/l. With a target effluent of 30 mg/l NO_3-N, a portion of the ASB effluent is allowed to bypass the filters and is then blended with the fully-denitrified filter effluent downstream of the filters.

The expected performance of the 2-stage system is shown in Table 4-2 at two different influent NO_3-N concentrations: 80 mg/l and 100 mg/l.

Table 4-2. Estimated Performance of 2-Stage Denitrifying Filters at Influent NO_3-N Concentrations of 80 mg/l and 100 mg/l at Flow of 11,260 m^3/d

	Influent NO_3-N = 80 mg/l			Influent NO_3-N = 100 mg/l		
	Stage 1	Stage 2	Total	Stage 1	Stage 2	Total
Filter Influent Flow, m^3/d	10,103	10,103	10,103	8,041	8,041	8,041
Filter Bypass Flow, m^3/d	1,157	1,157	1,157	3,219	3,219	3,219
Influent NOx-N, mg/l	80	24	80	100	30	100
Effluent NOx-N, mg/l	24	2	2	30	2	2
NOx-N Removed, kg/d	566	222	788	566	222	788
Blended Effluent NOx-N, mg/l	30	4	10	---	---	30

A methanol storage and pumping facility is proposed to be located within a new filter building. For economy, the chlorine storage and chlorinators are proposed to be located in the filter building separate rooms. A waste filter backwash tank and pumping system is required to equalize the flow before returning the waste backwash to the head of the ASB. The waste backwash tank is sized assuming two filters are backwashing at the same time as summarized in Table 4-3.

Table 4-3. Sizing of Waste Filter Backwash Tank

Backwash Rate	6	gpm/ft^2
Area of one unit (9.5 ft x 52 ft)	494	ft^2
Number of units backwashing	2	
Waste Backwash Flow rate	5,928	gpm
Backwash duration	30	minutes
Total Volume of WBW Tank	177,840	gallons
At depth = 10 feet, Area	2,378	ft^2

4.7 Chlorination System

Currently there is a gaseous chlorine system at the Aqaba WWTP. In order to provide disinfection for effluent from the Natural WWTP, gaseous chlorine is to be provided. A typical system of this sort includes compressed chlorine gas storage cylinders, chlorine gas vacuum lines, chlorinators, chlorine residual analyzers, and process instrumentation and control systems including a flow meter. A typical schematic of a chlorine gas disinfection system is shown in Figure 4-5.

Reference can also be made to Figures 3-3 and 3-4 in Section 3 above for the general layout and process flow diagram for the aeration alternatives described for Option 2.

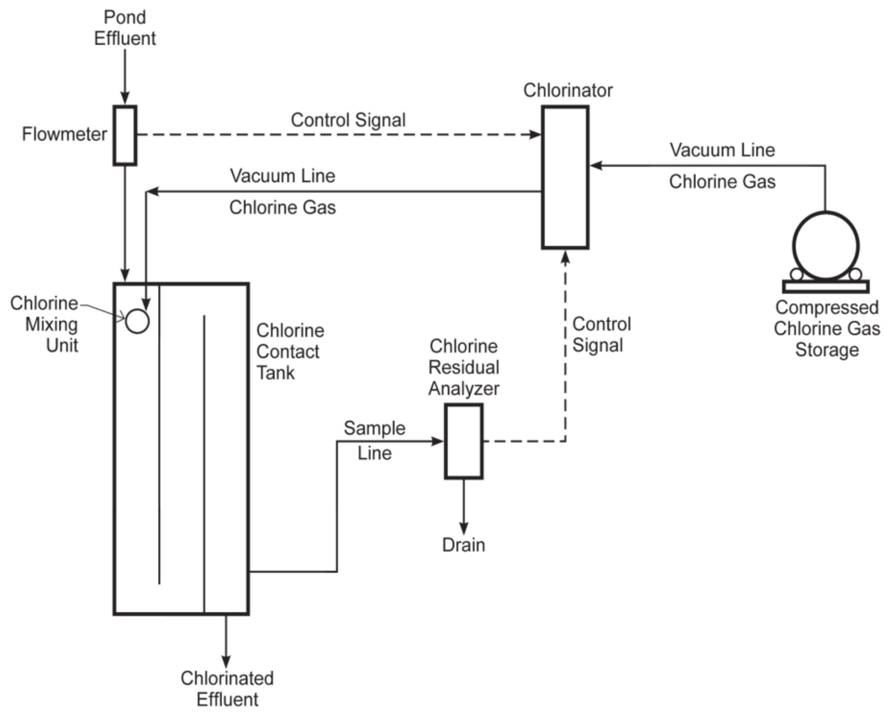


Figure 4-5 Typical Schematic of Chlorine Gas Disinfection System

4.7.1 Chlorine Contact Tank Sizing

The primary purpose of the contact chamber is to provide the detention time necessary for the chlorine compounds to reduce the bacteria to acceptable levels, specifically, the requirements for E. Coli and helminth eggs dictate final chlorination. For the Natural WWTP, new chlorine contact tanks are proposed to be provided and designed for a standard minimum detention time of 30 minutes and to accommodate the ASB design flow of 11,260 m^3/d. A length-to-width ratio (L/W) of at least 40 to 1 (length of contact channel to width of channel) is required to help minimize short circuiting in the tank. Also to minimize excessive solids deposition on the contact channels, the ideal horizontal flow velocities within the channels should be in the range of 2.0 to 4.5 m/min (0.033 m/sec to 0.075 m/sec.). Table 4-1 presents the sizing criteria used for the contact tank.

Table 4-4. Chlorine Contact Tank Sizing

Flow Rate (m^3/d)	Flow Rate (m^3/min)	Volume @ Spec'd Det. Time (m^3)	Check L/W Ratio and Velocity Calc.				
			Channel Width (m)	Channel Depth (m)	Calculated Length of Channel (m)	Length-to-Width Ratio (L/W)	Channel Velocity (m/sec)
10,146	7.05	211.4	1.50	2.3	61.3	40.8	0.034
11,260	7.82	234.6	1.50	2.3	68.0	45.3	0.038

Based on the information provided in Table 4-1, the total channel length calculated based on a flow rate of 11,260 m^3/d is 68 m at 1.5 m wide and 2.3 m deep. Figure 4-6 shows a general diagram of this tank configuration. For redundancy, it should be noted that two contact tanks of identical configuration, both sized for the design flow condition, are to be provided as shown.

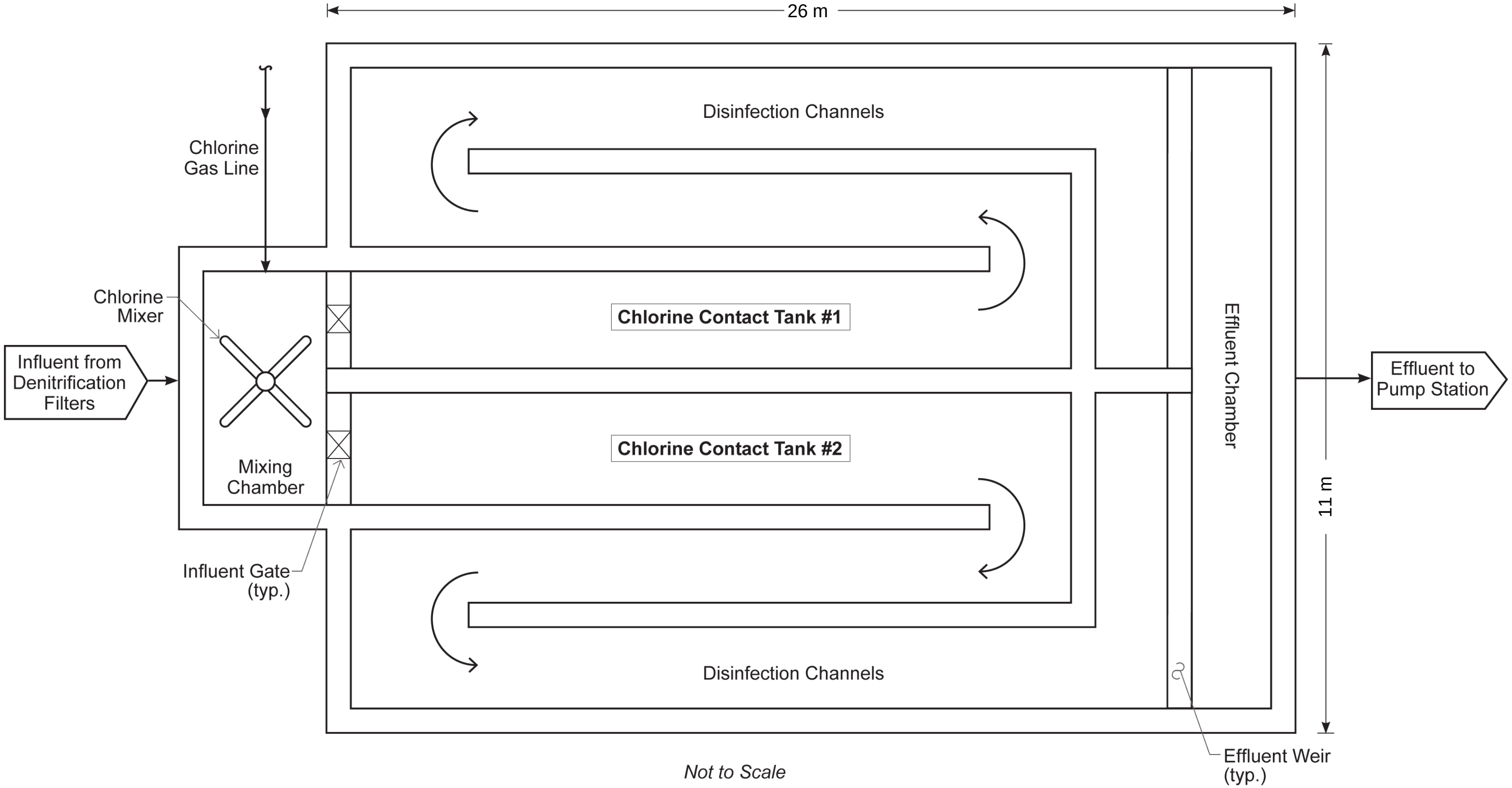


Figure 4-6 Chorine Contact Tanks – General layout with approximate overall dimensions

5 SCHEDULE

An approximate schedule for Natural WWTP upgrades has been estimated for preliminary planning purposes. A more detailed study and preliminary design phase is recommended following review of the options presented in this CDR. Once an upgrade option is selected, detailed planning and design can commence.

Depending on the option selected, design of the upgrades will take approximately six months to one year. This process would require survey and/or soil borings to update detailed information on site conditions. Progress submittals should be provided at 30%, 60%, and 100% stages, with anticipated two- to four-week review periods following each submittal. After the 100% design document review has been completed, production-ready Plans, Specifications, and Estimates (PS&E) should be provided for use securing construction bids and for construction. Bidding, contract award, and contract negotiation typically can take four to six months, followed by one to two years for construction of upgrades.

6 PLANNING LEVEL PROJECT COSTS

6.1 Development of Project Costs

In preparing this document, order-of-magnitude estimates were developed for the various options of the proposed upgrades to the Aqaba Natural WWTP. The costs for the alternatives obviously vary with the specific design considerations and layout configuration ultimately selected. Nonetheless, it is possible to put together an estimate that can be used for Planning Level Costs to determine the most cost effective alternative.

The costs presented in Table 6-1 are planning level costs and should be refined as additional informational details are identified and/or determined. The project scope would have to be expanded to include further study of the specific types of process equipment, type of mechanical systems, redundancy and site security, and types of control systems. In addition, project constraints, project schedule, and overall project complexity will impact project costs. It is recommended that planning level project costs be updated just prior to any appropriation of funding for design and construction.

The estimated planning level project costs and annual operation and maintenance costs for Options 1 and 2 presented in Table 6-1 and Table 6-2 were developed with the following assumptions:

- Project costs shown include estimated labor, materials and construction costs
- A 25% contingency was added to project costs.
- Project costs do not include engineering fees for preliminary, final design, field work, soil borings, field survey, or any other fees associated with developing a detailed design.
- General vendor quotes were received for the floating baffles, surface aspirators and aerators, and denitrification filters to assist in developing the planning level project costs.
- Power costs estimated the floating mixers, aspirators and aerators assumed a 24/7 operational schedule. For planning purposes, other O&M costs were assumed to be equivalent for all the options, and were not used for comparison purposes.
- Electrical power costs were calculated for the years 2015 and 2017, since it is estimated that the power costs in Jordan will increase in 2017. To calculate power costs, the costs per KWH in Jordan were assumed to be $0.133 USD for the year 2015 and increasing to $0.187 USD starting in 2017.

Table 6-1. Summary of Estimated Capital and Annual Costs to Convert Existing facultative Ponds into New Aerated Stabilization Basins (Option 1)

Description	Cost	Annual Electrical Costs, USD	Annual Chemical Costs, USD	Annual Maintenance Costs, USD
Initial Dredging/Cleaning	350,000			
Floating Baffles/Curtains	198,600			16,700
Anoxic Mixers, floating 20HP	353,800	436,700		16,700
Recycle Pumping System 200% of flow, TDH = 8 ft, 2 pumps	190,000	14,500		12,500
Floating Aspirators, 20 HP	274,600	767,600		8,200
Permanent portable dredging equipment	75,000			2,300
Denitrifying Filters	3,179,735	25,800	345,800	79,200
Waste Filter Backwash Tank	254,000			
Chlorine Disinfection System	50,000	61,500	35,600	12,000
New Reclaimed Water Storage Pond (225m x 150m x 4.5m)	1,115,088			
Civil/Site Work	604,082			
Sub-Total	6,644,905			
Contractor OH&P	1,395,430			
Engineering and Administrative	332,245			
Contingency - Planning-level	1,608,067			
Total	**9,980,648**			

At a conversion rate of 1.41, the total capital cost rounded-up for Option 1 is JOD 7,080,000.

Capital expenditures (CAPEX) are based on design for 11,260 m^3/d and reducing NOx-N from 100 to 30 mg/l (maximum month condition). Operating expenses (OPEX) are based on design for 11,260 m^3/d and reducing NO_x-N from 80 to 30 mg/l (average annual condition).

Table 6-2. Summary of Estimated Capital and Annual Costs for New Aerated Stabilization Basin (Option 2)

Description	Capital Cost, USD	Annual Electrical Costs, USD	Annual Chemical Costs, USD	Annual Maintenance Costs, USD
Floating Baffles/Curtains	155,520			
Anoxic Mixers, floating 25HP	225,400	436,700		16,700
Recycle Pumping System 200% of flow, TDH = 8 ft, 2 pumps	100,000	14,500		8,100
New Lined Basin (450m x 150m x 4.5m)	2,175,175			
Floating Aerators	365,079	671,600		15,700
Permanent portable dredging equipment	75,000			2,300
Denitrifying Filters	3,179,735	25,800	345,800	79,200
Waste Filter Backwash Tank	254,000			
Chlorine Disinfection System	50,000	61,500	35,600	12,000
Rehab existing Maturation Ponds for Effluent Storage	350,000			
Civil/Site Work	692,991			
Sub-Total	7,622,900			
Contractor OH&P	1,600,809			
Engineering and Administrative	381,145			
Contingency - Planning-level	1,844,742			
Total	**11,449,596**			

At a conversion rate of 1.41, the total capital cost rounded-up for Option 2 is JOD 8,120,000.

Estimated electrical power costs are based on the tariff currently proposed for Year 2017.

The estimated cost associated with Option 1 includes construction of an effluent storage pond. This pond could be sited in the same location as a proposed new Aerated Stabilization Basin. This pond was assumed to be of the same dimensions as the existing ponds, that is, 450 m long by 150 m wide by 2 m deep. It is assumed that this storage pond would not be constructed if implementing Option 2, because the existing Maturation Ponds can potentially be re-configured for this service.

The project cost associated with implementing Option 2 with a new ASB is slightly higher than the cost for Option 1. The electrical power costs for each option are estimated to be within 10 percent of the other, with Option 1 costing slightly more than Option 2.

WRECP has no control over costs of labor, materials, competitive bidding environments and procedures, unidentified field conditions, financial and/or market conditions or other factors likely to affect the opinion of probable project costs, all of which are and will unavoidably remain in a state of change. It is further understood that the probable project costs are a snapshot in time, and that the reliability of this opinion of costs will inherently degrade over time.

7 CONCLUSIONS AND RECOMMENDATIONS

The capital cost for all of the options is considerable and is largely exacerbated by the nitrogen and disinfection requirements of JS893:2006, Level A, as outlined in Table 1-2 above. Using Option 1, the costs associated with nitrogen treatment and disinfection amounted to approximately 80% of the total project capital cost.

Ultimately, reliability of the process to meet the required effluent targets must be the determining factor in selecting a treatment process. Of the two options, AECOM would recommend proceeding with Option 2, a new ASB with surface aerators. The recommendation for Option 2 is based on the expectation that the new ASB will reliably and consistently meet the effluent quality objectives whereas, Option 1 cannot be expected to reliably and consistently meet the same objectives. Option 2 offers many benefits to AWC including:

- It would be properly designed using standard design parameters to more reliably meet the limits as a new facility, as opposed to re-purposing a shallow facultative pond as a high-rate aerobic process.

- Construction can be made without taking the existing system out of service, and the resultant difficult operation with half the system in service while each existing pond is taken off-line for cleaning and modifications.

- Existing facultative ponds can be used for long-term sludge storage and existing maturation ponds can be used for effluent storage.

Should AWC desire to proceed with the recommended concept, the following items should be given further consideration.

- Project Phasing/Concept Demonstration: Given the relatively high costs associated with a tertiary Nitrogen removal process, a one denitrification filter demonstration pilot study should be conducted before final design. AECOM has sized the tertiary denitrification systems with a degree of conservatism.
- Sampling: An expanded sampling program should be conducted to include parameters required by JS893:2006, Level A that are not currently analyzed (e.g. better Nitrogen speciation and bacteriological testing). This should be done both on the effluent from the Headworks as well as the lagoon effluent. The design influent strength, particularly as it pertains to N species is extraordinarily high. This is a major driver to the project cost and more extensive testing - both in terms of frequency and speciation - can help build better confidence on what the design parameters should be. If the elevated Nitrogen levels are determined to be consistent, the denitrification systems can be designed with more confidence.
- Consolidation of Tertiary Systems with Mechanical Plant: Given that much of the project costs associated with any of these options have to do with the tertiary treatment step downstream of the ASBs, including the denitrifying filters, further study of opportunities to combine these systems with those required by the mechanical plant should be considered.
- Consolidation with Mechanical Plant: While outside the scope of this Concept Design Report, a final item to consider would be performing a cost/benefit analysis of decommissioning the natural plant and making the upgrades necessary to the mechanical plant to handle total influent loads. Under separate cover, various flow

scenarios for the mechanical plant have been evaluated which would provide some ability to perform this analysis. If feasible, the mechanical plant might provide process control advantages over the natural plant which might reduce the cost of downstream systems.

AECOM is pleased to have had the opportunity to develop the concepts described herein and hopes that this CDR provides the AWC with a basis for moving forward with an effluent treatment system that will provide the AWC and its customers with reliable effluent management services for years to come.