4.5 Surficial Geology & Geological Structures

According to the geological map information (Beyrouth Leaf, by M. Louis Dubertret), the Aeolian landforms in the region under consideration are formed by Dunes (ad / qd) and red soils (q1l) of Quaternary age as seen in below Figure 4.5, and the major structural element within the region is Mount Lebanon Trust (MLT, see Figure 4.6).

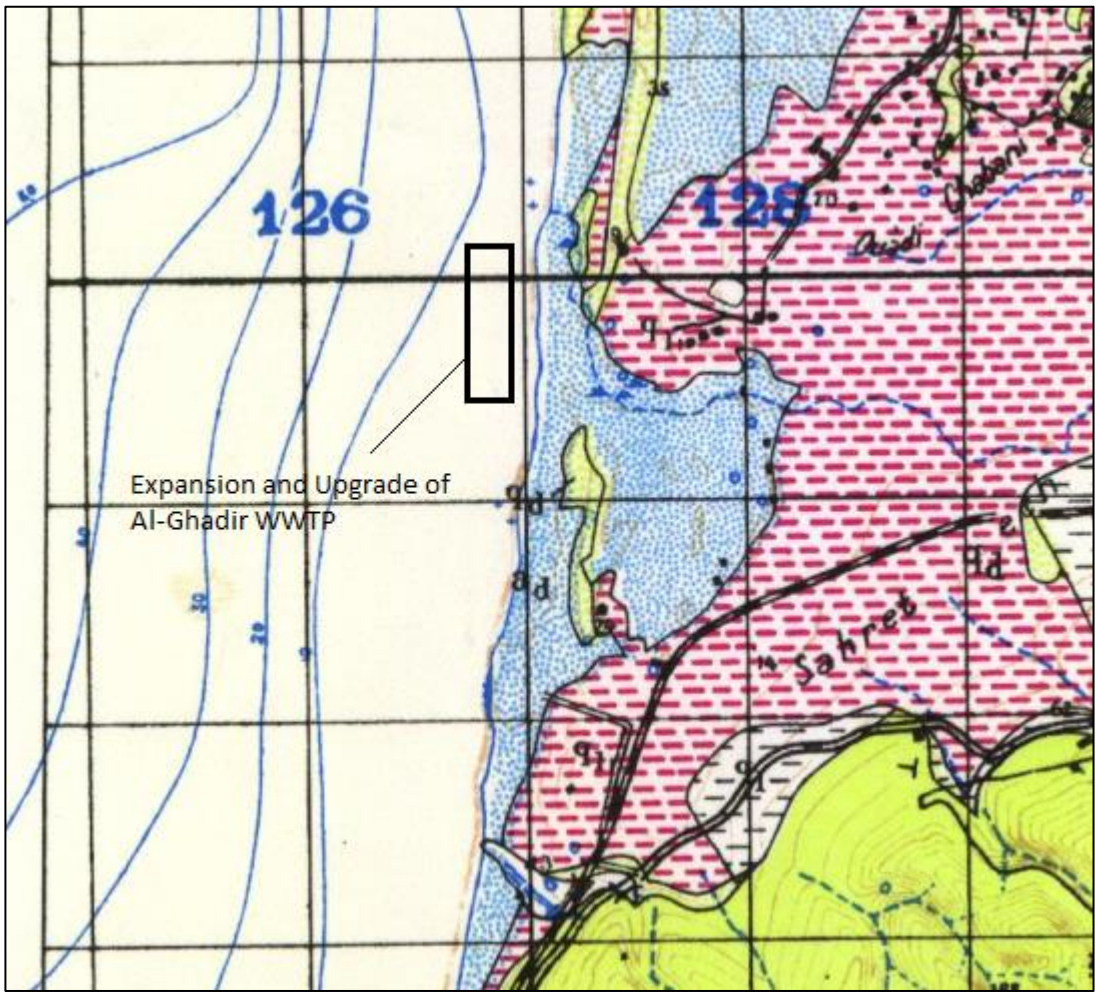


Figure 4.5 Beyrouth Leaf, by M. Louis Dubertret

4.6 Subsurface Geology & Hydrogeology

According to the log of boring (Appendix 4.2), penetration rates of coring and laboratory test results; the subsurface geology/hydrogeology of the site is as follows:

BH1/Ghadir-2024 at elevation 2.60asl

0-1.5m:

Loose to medium dense (according to penetration rate) partially saturated/submerged Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used),

1.5-9m:

Gravely very Silty medium dense to dense (according to corrected SPT values for the increase resistance due to negative excess pore water pressure set up during the testing) fully saturated/submerged poorly graded fine SAND of intermediate plasticity (Seabed Sediments),

9-18m:

Beige highly weathered crushed and closely fractured weak to moderately strong medium to coarse grained vuggy Calcareous SANDSTONE / very Sandy LIMESTONE (very poor to poor rockmass/bedrock according to RMR)

BH2/Ghadir-2024 at elevation 1.50asl

0-3m:

Loose to medium dense (according to penetration rates) partially saturated/submerged Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used),

3-10m:

Gravely very Silty medium dense to dense (according to penetration rates) fully saturated/submerged poorly graded fine SAND of intermediate plasticity (Seabed Sediments),

10-18m:

Beige highly weathered crushed and closely fractured weak to moderately strong medium to coarse grained vuggy Calcareous SANDSTONE / very Sandy LIMESTONE (very poor to poor rockmass/bedrock according to RMR)

BH3/Ghadir-2024 at elevation 1.00asl

0-4m:

Loose to medium dense (according to penetration rates) partially saturated/submerged Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used),

4-18m:

Gravely very Silty medium dense to dense (according to corrected SPT value for the increase resistance due to negative excess pore water pressure set up during the testing) fully saturated/submerged poorly graded fine SAND of intermediate plasticity (Seabed Sediments).

BH4/Ghadir-2024 at elevation 1.25asl

0-3.5m:

Medium dense to dense (according to SPT values) partially saturated/submerged Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used),

3.5-18m:

Gravely very Silty medium dense to dense (according to corrected SPT value for the increase resistance due to negative excess pore water pressure set up during the testing) fully saturated/submerged poorly graded fine SAND of intermediate plasticity (Seabed Sediments).

Neither loss of water circulation nor cavity was observed/detected in drilled boreholes.

Groundwater table (seawater-level, after airlifting/bailing out) was detected in drilled boreholes during the piezometric surveys on September 17, 2024 (see below Table 4.2).

As per the above; the subsurface geology (Gravely very Silty fine SAND and underlying Calcareous SANDSTONE / very Sandy LIMESTONE) shall be considered as groundwater/seawater bearing strata of aquiclude to semi-aquifer of low transmissivity.

Table 4.2 Piezometric survey (September 17, 2024)

Borehole Identity	Depth of Groundwater Table/Seawater-Level from Z	Z	Groundwater Table/Seawater-Level
BH1/Ghadir-2024	1.87m	2.60asl	0.73asl
BH2/Ghadir-2024	0.68m	1.50asl	0.82asl
BH3/Ghadir-2024	0.58m	1.00asl	0.42asl
BH4/Ghadir-2024	0.60m	1.25asl	0.65asl

4.7 Seismology

The expected peak ground acceleration (PGA) of 0.30g for a 50 years' time span shall be adopted and used in engineering analysis, taking Mount Lebanon Trust (MLT) into consideration (see below Figure 4.6).

As per the above and in reference to Section 4.6 (Subsurface Geology), the following seismic coefficients shall be used in engineering analysis according to UBC-1994.

Soil Profile Type = SD

Ca = 0.36

Cv = 0.54

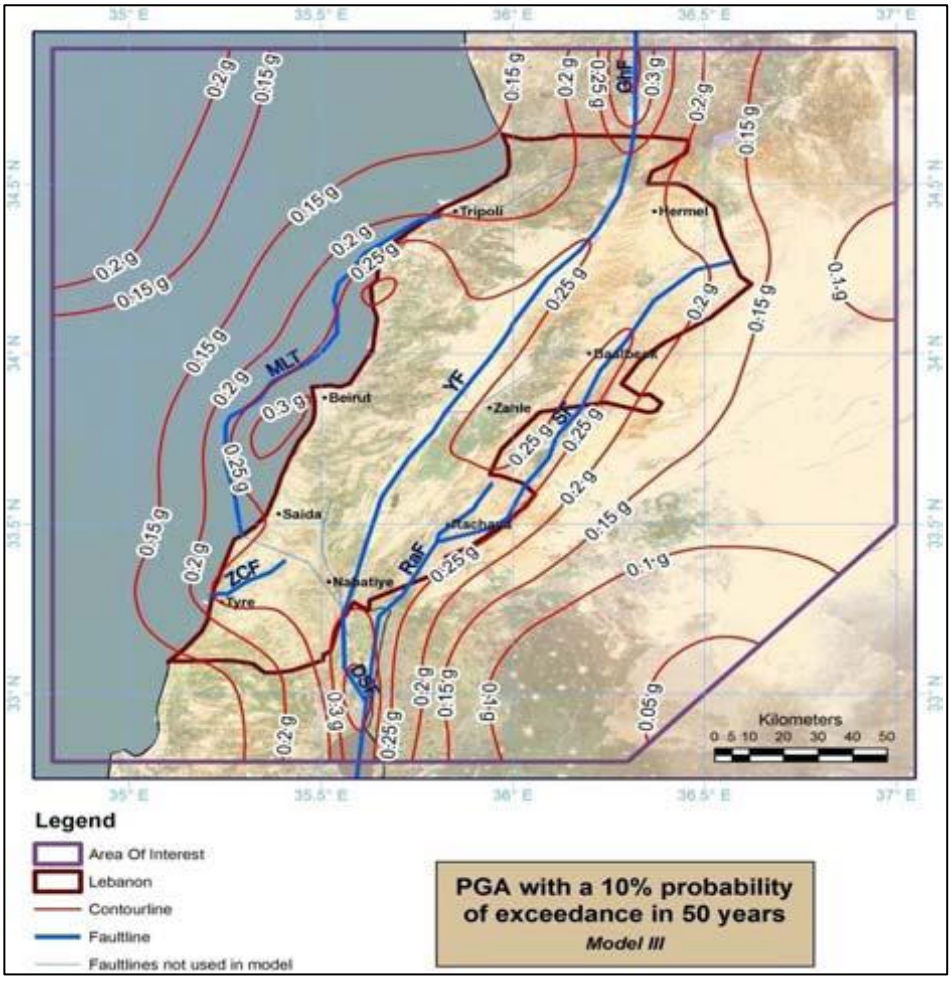


Figure 4.6 Contour map of PGA with a 10% probability of exceedance in 50 years

4.8 Landforming

The landforming will consist of cutting and filling (reclamation) the high (+2.97) and low (-3.50) level areas (respectively) to elevation +2.00 (see Figure 4.3 and refer to Appendix 4.1).

The low-level areas shall be filled by using the granular material resulted from the cut of Manmade Fill and/or imported A-2-5 to A-2-6 material with Cobbles and Boulders. The imported A-2-5 to A-2-6 material (0/304.8mm) shall have a minimum 4-days soaked CBR of 20% at 95% proctor density.

As for the cut, the sharp vertical drops shall be flattened, and the small hills or stockpiles of miscellaneous material should be smoothed out and bladed to a more nearly level surface at elevation +2.00

The surface runoff (overland flow) shall be diverted around the landform by using perimeter (or equivalent) conveyance ditches and/or swales, and shall finally be discharged into sea.

4.9 Bearing Strata

In view of the findings (refer to Sections 4.2 and 4.6), the WWTP will be founded on loose to medium dense Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used) that is underlaid by medium dense to dense Seabed Sediments (Gravely very Silty fine SAND of intermediate plasticity) with water table/seawater-level at bottom level of foundation.

4.10 Interpretations

The fully saturated/submerged multilayered foundation soil (refer to Section 4.9) having presumed bearing value of =<105kPa with modulus of subgrade reaction of =<5.5MN/m^3, cannot be considered safe bearing strata for the Expansion and Upgrade of Al-Ghadir WWTP.

Accordingly; the multilayered foundation soil shall be densified by Deep Dynamic Compaction (DDC) from the elevation +2.00 (refer to Appendix 4.1), knowing that the Manmade Fill and Underlying Seabed Sediments are favorable (Zone 1) and intermediate (Zone 2) for DDC according to FHWA-SA-95-037 (1995), and the type of foundation shall be enlarged-raft, taking the heterogeneous Manmade Fill (Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used) into consideration.

As an alternative solution to DDC, the raft foundations may be provided with bored-piles 60cm in diameter and center to center 2.1m (piled-raft). The piles shall be embedded at least 14m into the Seabed Sediments (more or less 19m deep piles from the elevation +2.00) with no end-bearing capacity, and/or the piles shall rest on bedrock with point-bearing capacity. However, it is to be noted that the solution of piled-raft will cost significantly more than the DDC for a construction area of about 7.8ha.

As per the above; densifying the multilayered foundation soil by Deep Dynamic Compaction can be considered the most cost-effective solution.

The DDC shall consist of high energy compaction by dropping octagonal heavy tampers, low energy compaction (called ironing phase) by dropping square light tampers, impact rolling by using impact roller with 3-sided non-circular steel drum, final leveling and smooth rolling.

The tampers shall be raised and dropped on a repeated basis without a significant amount of down time by using a crawler crane. After each drop, craters created by the tampers shall be backfilled and graded with quarry runs Engineering Fill before the next drop is applied.

A seismograph shall be used to measure the magnitude of ground vibrations being transmitted off-site. The Particle Velocity (at frequency of 6 to 10 hertz) shall be limited to 19mm/sec during the DDC, accordingly; the overall DDC area (Areas Phase 1, Phase 2a and Phase 2b) shall be surrounded by a 3m deep and 0.6m wide trench as a ground-vibration barrier within a setback of 6m, to minimize the effect of ground-vibration and lateral soil-mass displacement on neighboring structures (such as the side-slopes of landfill cells and airport's runway).

An open trench is the most effective in reducing vibrations, however; open trench is a concern for side-slope stability of landfill cells and airport's runway. Accordingly, the open trench shall be backfilled with loosely placed (dumped) compressible material resulted from excavation of the trench.

The verification tests (Pressiometer and Static Cone Penetration) shall be executed before and 30 days after the Deep Dynamic Compaction (including final leveling and smooth rolling), taking the multi-layered foundation soil improvement with additional time (due to dissipation of the excess porewater pressure) into consideration, to determine the depth and degree of improvement.

If the desired improvement (having dense to very dense Manmade Fill and underlying gravely very silty fine SAND that is called multi-layered foundation soil) has not been achieved upon completion of Deep Dynamic Compaction; additional energy shall be applied, to result in greater ground improvement.

The Type of Cement, Class of Concrete, W/C ratio, Cement Content and Concrete Cover to Reinforcing Steel shall be as follows, taking the Exposure Class of XS2 to XS3 into consideration.

Type of Cement: Type V (High Sulfate Resistance),

Class of Concrete >= 35MPa,

W/C =< 0.45,

Cement Content >= 340kg/m^3, and

Concrete Cover to Reinforcing Steel >= 65mm

4.11 Closure

The conclusions (including interpretations) and recommendations presented in this chapter are based on assumption that the subsurface stratums and conditions do not deviate appreciably from those disclosed in the borings. There may be conditions pertaining to the site which were not disclosed by this subsurface soil/rock survey, and thus could not be taken into account.

As per the above; the conclusions (including interpretations) and recommendations presented in this chapter shall be validated/updated upon completion of the further geotechnical investigations to be done by the Contractor prior to start DDC.

CHAPTER 5 WASTEWATER SAMPLING AND TESTING

Chapter 5 – Wastewater Sampling and Testing

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Chapter 5 Wastewater Sampling and Testing

5.1 Introduction

- This chapter provides an overview of the characteristics of wastewater in the existing collection systems, which will guide the design services. To achieve this, the consultant has appointed the "Trust for the Quality and Products (TQP)" Laboratory to monitor the wastewater at the existing Ghadir WWTP, alongside the "Litani River Authority (LRA) - Central Laboratory Department" for parameter testing, starting from December 2024 (Figure 5.1). This monitoring process includes conducting grab or composite water sampling at scheduled intervals throughout the winter and summer seasons, followed by testing for the following parameters:
- pH
- Temperature of wastewater (field)
- BOD5
- COD
- TSS
- VSS
- TN
- Ammonia NH4+
- Nitrate NO3-
- TP
- PO43-
- FOG
- Electrical Conductivity

The composite samples are taken every 2 hours for 24 consecutive hours, in such a way to cover winter and summer seasons.

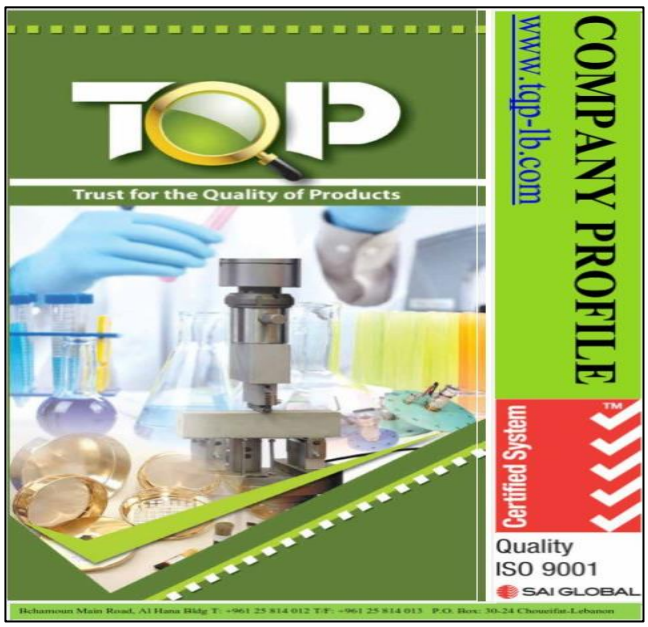


Figure 5.1 Trust for the Quality and Products (TQP) and LRA Company Profile

5.2 Sampling Schedule

A total of 25 wastewater tests are scheduled, spanning both the winter and summer seasons. The first grab sample was taken on Tuesday, 17th September 2024, at 10:00 AM at the Ghadir WWTP, in the presence of engineers from Dar Al Handasah Nazih Taleb & Partners, CDR and TQP representatives. Further sampling and testing are scheduled as detailed below.

Table 5.1 Sampling Schedule

SN	Type of Sample	Date	Parameters
1	Grab	17 Sep 2024	BOD_5, COD, TSS, VSS
2	Grab	27 Sep 2024	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
3	Grab	7 Oct 2024	BOD_5, COD, TSS, VSS
4	Composite	22 to 23 Oct 2024	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
5	Grab	5Nov 2024	BOD_5, COD, TSS, VSS
6	Grab	18 Nov 2024	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
7	Grab	5 Dec 2024	BOD_5, COD, TSS, VSS
8	Composite	19 to 20Dec 2024	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
9	Grab	02 Jan 2025	PH, BOD_5, COD, TSS, VSS
10	Grab	15 Jan 2025	PH, BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
11	Grab	29Jan 2025	PH, BOD_5, COD, TSS, VSS
12	Composite	11 to 12 Feb 2025	PH, BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
13	Grab	26 Feb 2025	PH, BOD_5, COD, TSS, VSS
14	Grab	10 Mar 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
15	Composite	24 Mar 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
16	Grab	7 Apr 2025	BOD_5, COD, TSS, VSS
17	Grab	21 Apr 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
18	Grab	5 May 2025	BOD_5, COD, TSS, VSS
19	Composite	20 May 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
20	Grab	3 Jun 2025	BOD_5, COD, TSS, VSS
21	Grab	17 Jun 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP
22	Grab	1 Jul 2025	BOD_5, COD, TSS, VSS
23	Composite	15 Jul 2025	BOD_5, COD, TSS, VSS, TN, NH_4^+, NO_3^-, PO_4^-, TP, FOG, Conductivity
24	Grab	28 Jul 2025	BOD_5, COD, TSS, VSS
25	Grab	8 Aug 2025	BOD_5, COD, TSS, VSS

The total number of samples is as follows:

Samples	Total	Executed	Progress %
Composite Samples (No.)	6	3	50%
Grab Samples (No.)	19	10	52%
Total Samples (No.)	25	13	52%

5.3 Sampling Methodology

Grab sample is collected directly from the Inlet manhole/tank using manual sampler as shown in the below photos.

As for composite sample, it is taken using automatic sampler, programmed to take equal portions of the sample every two hours up to 24 hrs. After Sampling, samples are transported via cooled container to the lab for immediate testing.

All equipment used for sampling are previously sterilized.



Figure 5.2 Site Photos of Grab Sampling on 17 September 2024

5.4 Testing Methodology

All parameters are tested using internationally recognized standard methods, as outlined in the table below. Tests are conducted by Chief Chemist, Khalil Sharkawi, and supervised and approved by Dr. Mohammad Dokmak-PhD Chemistry. However, the test results for the samples are obtained at the LRA laboratory under the supervision of Emilie Abou Hamad, starting from December 2024, with the standard test method specified for each parameter on the test result report sheets in section 5.5.1.

Table 5.2 TQP Laboratory Standard Testing Methods and Used Equipment

Parameter	Standard Test Method	Equipment	Description
pH	ASTM D1293	Digital Hach benchtop potentiometer	Electrometric measurement using the glass electrode as the sensor and using at least two of standard reference buffer solutions for instrument standardization
Temperature (ºC)	ASTM D1293	Digital thermometer with stainless probe	Direct measurement at site
Biological Oxygen Demand, BOD5 (mg/L)	APHA 5210 B	LaMotte Spectrophotometer	BOD5, or Biochemical Oxygen Demand over 5 days, is a measure of the amount of oxygen that microorganisms will consume while decomposing organic matter in a water sample over a five-day period. Method includes the following main steps: - Dilution whenever needed, - Measuring initial dissolved oxygen, - Sealing and incubation for 5 days at 20deg - Measuring the final dissolved oxygen after the incubation period - Calculation of BOD5
Chemical Oxygen Demand, COD (mg/L)	APHA 5220 B	(Open Reflux Method 1-2 hr time)	Organic matters are oxidized by a boiling mixture of chromic and sulfuric acids. A sample is refluxed in strongly acid solution with a known excess of potassium dichromate (K2Cr2O7). After digestion, the remaining unreduced K2Cr2O7 is titrated with ferrous ammonium sulfate to determine the amount of K2Cr2O7 consumed and the oxidizable matter is calculated in terms of oxygen equivalent.
Total Suspended Solids, TSS (mg/L)	ASTM D5907	Gravimetric	Measuring Total Suspended Solids (TSS) involves filtering the water sample to determine the amount of solid particles suspended in it. Regulated Vacuum apparatus is used for filtration, the previously weighed filter paper is dried with an oven, cooled in a desiccator, the weighed again using digital balance to the nearest 0.001g
Volatile Suspended Solids, VSS (mg/L)	APHA 2540 E	Gravimetric	The residue from suspended solids is ignited to constant weight at 550 ± 50°C in a previously ignited and weighed crucible to the nearest 0.0001g. The residue lost during ignition are volatile suspended solids.
Total Nitrogen as N (mg/L)	APHA 4500-N C	Persulfate oxidation Method	The method relies on oxidation and reduction steps, involving persulfate digestion of nitrogen

Parameter	Standard Test Method	Equipment	Description
			compounds into nitrate followed by spectrophotometric determination.
Ammonium as NH_4^+ (mg/L)	APHA 4500-NH3 C	Preliminary Distillation Step followed by Titration step.	The sample is buffered at pH 9.5 with a borate buffer to decrease hydrolysis of cyanates and organic nitrogen compounds. It is distilled into a solution of boric acid. The ammonia in the distillate is determined titrimetrically with standard HCl and a mixed indicator.
Nitrate as NO_3^- (mg/L)	APHA 4500-NO3 B	Ultraviolet Spectrophotometric Screening Method	Measuring UV absorption at 220 nm and at 275 nm nitrate is calculated by the difference between the two absorptions.
Total Phosphorus as P (mg/L)	APHA 4500-P C	Vanadomolybdophosphoric Acid Colorimetric Method (LaMotte Spectrophotometer)	In a dilute orthophosphate solution, ammonium molybdate reacts under acid conditions to form a heteropoly acid, molybdophosphoric acid. In the presence of vanadium, yellow vanadomolybdophosphoric acid is formed. The intensity of the yellow color is proportional to phosphate concentration which is measured by LaMotte Spectrophotometer using the appropriate LaMotte test Kit.
Phosphates as PO_4^{3-} (mg/L)	APHA 4500-P C	Vanadomolybdophosphoric Acid Colorimetric Method (LaMotte Spectrophotometer)	Same procedure as Phosphorus
FOG (mg/L)	EPA 1664-B	Quantitative Extraction using specific organic solvent	By determining the n-hexane extractable materials such as non-volatile hydrocarbons, vegetable oils, animal fats, waxes, soaps, greases, and related materials; The sample is acidified and serially extracted with n-Hexane in a separatory funnel; the extract is dried over anhydrous sodium sulfate, The solvent is distilled from the extract and the FOG is desiccated and weighed.
Electrical Conductivity (µs/cm)	EPA 2510 A	Using digital conductometer	The conductivity meter is calibrated before use with standard conductivity solution and then direct measurement is done for the waste water sample.

Below are some photos of the equipment used in the above testing:



Hack Potentiometer



LaMotte Smart Spectrophotometer



LaMotte test Kits



COD Reactor



Protherm Muffle Furnace, up to 1200deg



SHIMADZU Analytical Balance

5.5 Laboratory Test Results and Interpretation

5.5.1 Laboratory Test Results

The test results from the initial thirteen tests conducted on September 17, 2024 till end of February, 2025 are presented in the following analysis reports, dated from September 23, 2024, through the end of February 2025 (with the exception of the last sample extracted on February 26, 2025, for which the result has not yet been received).

Date: 23-Sep-24
Serial No.: 0919356-1
Page: 1/1

TEST REPORT

Tested at: Trust for the Quality of Products, TQP S.A.R.L.
P.O. Box: 30-24 Choueifat-Lebanon Bchamoun Main Road, Al Hanaa Bldg., Basement Floor
Tel: +961 25 814 012/13/77

Client: Dar Al Handasah Nazih Taleb & Partners
Bachir Kassar Street, Verdun Sector Beirut, Lebanon
Tel: 01-866665 ; Fax: 01- 863434

CHEMICAL ANALYSIS OF WATER

Project: Ghadir WWTP
Sample Type: Wastewater
Source: Wastewater Inlet

Testing Order No.: 19356
Sampled by: TQP
Sample Received on: 17-Sep-24
Sampling Time: 10:00 AM

Test	Result	Test Method
Temperature (°C)	30.2	ASTM D1293
pH at 26.2°C	6.8	ASTM D1293
Chemical Oxygen Demand, COD (mg/L)	290	APHA 5220 B
Biological Oxygen Demand, BOD5 (mg/L)	120	APHA 5210 B
Total Suspended Solids, TSS (mg/L)	600	ASTM D5907
Volatile Suspended Solids, VSS (mg/L)	460	APHA 2540 E

Tested by	Khalil Sharkawi	TQP Trust for the Quality of Products	**Received by** (Name & Sign)
Head of Chem Dep.			
Reviewed by	Mohammad Dokmak		
General Manager.			

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Date: 2-Oct-24
Serial No.: 0919377-1
Page: 1/1

TEST REPORT

Tested at: Trust for the Quality of Products, TQP S.A.R.L.
P.O. Box: 30-24 Choueifat-Lebanon Bchamoun Main Road, Al Hanaa Bldg., Basement Floor
Tel: +961 25 814 012/13/77

Client: Dar Al Handasah Nazih Taleb & Partners
Bachir Kassar Street, Verdun Sector Beirut, Lebanon
Tel: 01-866665 ; Fax: 01- 863434

CHEMICAL ANALYSIS OF WATER

Project: Ghadir WWTP
Sample Type: Wastewater
Source: Wastewater Inlet

Testing Order No.: 19377
Sampled by: TQP
Sample Received on: 27-Sep-24
Sampling Time: 10:00 AM

Test	Result	Test Method
Temperature (°C)	29.5	ASTM D1293
pH at 26.5°C	6.5	ASTM D1293
Chemical Oxygen Demand, COD (mg/L)	285	APHA 5220 B
Biological Oxygen Demand, BOD5 (mg/L)	125	APHA 5210 B
Total Suspended Solids, TSS (mg/L)	200	ASTM D5907
Volatile Suspended Solids, VSS (mg/L)	150	APHA 2540 E
Total Nitrogen as N (mg/L)	140	APHA 4500-N C
Ammonium as NH_4^+ (mg/L)	130	APHA 4500-NH3 C
Nitrate as NO_3^- (mg/L)	2	APHA 4500-NO3 B
Phosphates as PO_4^{3-} (mg/L)	87	APHA 4500-P C
Total Phosphorus as P (mg/L)	30	APHA 4500-P C

Tested by	Khalil Sharkawi	TQP Trust for the Quality of Products	**Received by** (Name & Sign)
Head of Chem Dep.			
Reviewed by	Mohammad Dokmak		
General Manager.			

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Date: 12-Oct-24
Serial No.: 1019388-1
Page: 1/1

TEST REPORT

Tested at: Trust for the Quality of Products, TQP S.A.R.L.
P.O. Box: 30-24 Choueifat-Lebanon Bchamoun
Main Road, Al Hanaa Bldg., Basement Floor
Tel: +961 25 814 012/13/77

Client: Dar Al Handasah Nazih Taleb & Partners
Bachir Kassar Street, Verdun Sector
Beirut,Lebanon
Tel: 01-866665 ; Fax: 01- 863434

CHEMICAL ANALYSIS OF WATER

Project: Ghadir WWTP
Sample Type: Wastewater
Source: Wastewater Inlet

Testing Order No.: 19388
Sampled by: TQP
Sample Received on: 7-Oct-24
Sampling Time: 10:00 AM

Test	Result	Test Method
Temperature (°C)	**28.8**	ASTM D1293
pH at 26.1°C	**6.7**	ASTM D1293
Chemical Oxygen Demand, COD (mg/L)	**150**	APHA 5220 B
Biological Oxygen Demand, BOD5 (mg/L)	**60**	APHA 5210 B
Total Suspended Solids, TSS (mg/L)	**125**	ASTM D5907
Volatile Suspended Solids, VSS (mg/L)	**100**	APHA 2540 E

Tested by	Khalil Sharkawi	TQP Trust for the Quality of Products	**Received by** (Name & Sign)
Head of Chem Dep.			
Reviewed by	Mohammad Dokmak		
General Manager.			

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Date: 28-Oct-24
Serial No.: 102305-1
Page: 1/1

TEST REPORT

Tested at: Trust for the Quality of Products, TQP S.A.R.L.
P.O. Box: 30-24 Choueifat-Lebanon Bchamoun Main Road, Al Hanaa Bldg., Basement Floor
Tel: +961 25 814 012/13/77

Client: Dar Al Handasah Nazih Taleb & Partners
Bachir Kassar Street, Verdun Sector Beirut, Lebanon
Tel: 01-866665 ; Fax: 01- 863434

CHEMICAL ANALYSIS OF WATER

Project: Ghadir WWTP
Sample Type: Wastewater
Sample Ref.: Composite sample
Source: Wastewater Inlet

Testing Order No.: 19409
Sampled by: TQP
Sampled on: 22-Oct-24
Sample Received on: 23-Oct-24
Sampling Time: 11:00 AM

Test	Result	Test Method
Temperature (°C)	24.1	ASTM D1293
pH at 23.8°C	6.9	ASTM D1293
Chemical Oxygen Demand, COD (mg/L)	145	APHA 5220 B
Biological Oxygen Demand, BOD5 (mg/L)	65	APHA 5210 B
Total Suspended Solids, TSS (mg/L)	250	ASTM D5907
Volatile Suspended Solids, VSS (mg/L)	195	APHA 2540 E
Total Nitrogen as N (mg/L)	150	APHA 4500-N C
Ammonium as NH_4^+ (mg/L)	135	APHA 4500-NH3 C
Nitrate as NO_3^- (mg/L)	<1	APHA 4500-NO3 B
Phosphates as PO_4^{3-} (mg/L)	24	APHA 4500-P C
Total Phosphorus as P (mg/L)	8	APHA 4500-P C
Oil and Grease (mg/L)	<2	EPA 1664-B
Electrical Conductivity (µs/cm)	4100	APHA 2510 B

Tested by	Khalil Sharkawi	Trust for the Quality of Products	**Received by** (Name & Sign)
Head of Chem Dep.			
Reviewed by	Mohammad Dokmak		
General Manager.			

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Date: 11-Nov-24
Serial No.: 1119422-1
Page: 1/1

TEST REPORT

Tested at: Trust for the Quality of Products, TQP S.A.R.L.
P.O. Box: 30-24 Choueifat-Lebanon Bchamoun
Main Road, Al Hanaa Bldg., Basement Floor
Tel: +961 25 814 012/13/77

Client: Dar Al Handasah Nazih Taleb & Partners
Bachir Kassar Street, Verdun Sector Beirut, Lebanon
Tel: 01-866665 ; Fax: 01- 863434

CHEMICAL ANALYSIS OF WATER

Project: Ghadir WWTP
Sample Type: **Wastewater**
Sample Ref.: **Composite sample**
Source: **Wastewater Inlet**

Testing Order No.: 19422
Sampled by: TQP
Sampled on: 4-Nov-24
Sample Received on: 5-Nov-24
Sampling Time: 9:00 AM

			***ELV regarding discharges of treated sewage water to seawater based on the Decision 8/1 of the Ministry of Environment, dated 1.03.2001**	
Test	**Result**	**Test Method**	**Existing facilities**	**New facilities**
Temperature (°C)	**20.2**	ASTM D1293	**max 35°C**	**max 35°C**
pH at 20.2°C	**6.7**	ASTM D1293	**5 – 9**	**5 – 9**
Chemical Oxygen Demand, COD (mg/L)	**105**	APHA 5220 B	**max 250 mg/L**	**max 125 mg/L**
Biological Oxygen Demand, BOD5 (mg/L)	**45**	APHA 5210 B	**max 100 mg/L**	**max 25 mg/L**
Total Suspended Solids, TSS (mg/L)	**190**	ASTM D5907	**max 200 mg/L**	**max 60 mg/L**
Volatile Suspended Solids, VSS (mg/L)	**160**	APHA 2540 E	-----	-----
Total Nitrogen as N (mg/L)	**115**	APHA 4500-N C	**max 40 mg/L**	**max 30 mg/L**
Ammonium as NH_4^+ (mg/L)	**105**	APHA 4500-NH3 C	**max 10 mg/L**	**max 10 mg/L**
Nitrate as NO_3^- (mg/L)	**3**	APHA 4500-NO3 B	**max 90 mg/L**	**max 90 mg/L**
Phosphates as PO_4^{3-} (mg/L)	**17**	APHA 4500-P C	**max 5 mg/L**	**max 5 mg/L**
Total Phosphorus as P (mg/L)	**5**	APHA 4500-P C	**max 16 mg/L**	**max 10 mg/L**
Oil and Grease (mg/L)	**<2**	EPA 1664-B	**max 30 mg/L**	**max 30 mg/L**
Electrical Conductivity (µs/cm)	**4200**	APHA 2510 B	-----	-----

***ELV: Environmental Limit Value**

Tested by	Khalil Sharkawi	TQP Trust for the Quality of Products	**Received by** (Name & Sign)
Head of Chem Dep.			
Reviewed by	Mohammad Dokmak		
General Manager.			

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AMERICAN UNIVERSITY OF BEIRUT
MAROUN SEMAAN FACULTY OF ENGINEERING & ARCHITECTURE

Department of Civil & Environmental Engineering
قسم الهندسة المدنية والبيئية

CIVIL ENGINEERING SERVICE LABORATORIES مختبرات الهندسة المدنية

CERTIFICATE OF TEST

Requested by: **Dar al Handasah Nazih Taleb and Partners**
Beirut

Date: **25/11/2024**

Your Ref.:

Our Ref.: **R-31558-W**

Nature of Test

Sample Type: Waste Water
Date Received: 18 November 2024
Project: Expansion and Upgrade of Al-Ghadir Wastewater Treatment Plant

Parameter	Result
Chemical Oxygen Demand (mg/L as O2)	620
Biochemical Oxygen Demand (mg/L BOD5)	377
Total Nitrogen (mg/L -N)	28.6
Ammonia-Nitrogen (mg/L NH3-N)	21.1

All tests are performed in accordance to the "Standard Methods for the Examination of Water and Wastewater", 22nd Edition, 2012 as approved by the American Public Health Association, the American Water Works Association, and the Water Environment Federation unless otherwise noted.

George M. Ayoub, Ph.D.
Professor of Civil & Environmental Engineering

American University of Beirut
Beirut
PO Box 11-0236, Riad El Solh,
Beirut 1107 2020, Lebanon
T +961 1 35 00 00 - Ext 3463/8/9

New York
The Debs Center, 3 Dag Hammarskjold Plaza, 8th Floor
New York, NY 10017-2303, USA

ceelabs@aub.edu.lb | aub.edu.lb/msfea

الجمهورية اللبنانية
المصلحة الوطنية
لنهر الليطاني
المختبر المركزي

تاريخ إصدار التقرير:10-12-2024

أ- الجهة المرسلة: دار الهندسة

ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات:..... 5-12-2024

تاريخ استلام العينات في المختبر:..... 5-12-2024

Sample ID	TSS (mg/L)	BOD (mg/L)	COD (mg/L)
INLET	200*	320*	698*
المعايير الحدية المسموح بها حسب القرار 1/8	60	25	125
Standard Method	Standard Method section 2450	Hach Method 10099. Respirometric Method	Standard Method 5220 D

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

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تاريخ إصدار التقرير: 26-12-2024

أ- الجهة المرسلة: دار الهندسة

ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات:..... 20-12-2024

تاريخ استلام العينات في المختبر:..... 20-12-2024

Sample ID	PH	TSS (mg/L)	Conductivity (µS/cm)
INLET	8.01	120*	8500
المعايير الحدية المسموح بها حسب القرار 1/8	6-9	60	-
Standard Method	Standard Method 4500 – H+B	Standard Method section 2450	Standard Method section 2450

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Sample ID	Total Nitrogen (mg/L)	Total Phosphorus (mg/L)	Phosphate as (PO4) (mg/L)	BOD (mg/L)	COD (mg/L)
INLET	158*	8.1	24.8	155*	517*
المعايير الحدية المسموح بها حسب القرار 1/8	30	10	-	25	125
Standard Method	Digestion	Standard Method 4500-PBE (Hach Method 10209)	By Calculation	Hach Method 10099. Respirometric Method	Standard Method 5220 D

Sample ID	Nitrate NO_3^- (mg/L)	Ammonia (mg/L)	Ammonium (mg/L)
INLET	9.5	95*	101
المعايير الحدية المسموح بها حسب القرار 1/8	90	10	-
Standard Method	Standard Method 4500 – NO3 - B		

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

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لنهر الليطاني
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تاريخ إصدار التقرير: 7-1-2025

أ- الجهة المرسلة: دار الهندسة نزيه طالب و شركاؤه

ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات:..... 2-1-2025
تاريخ استلام العينات في المختبر:..... 2-1-2025

Sample ID	pH اليدس/ تركيز شوارد الهيدروجين	Temperature (^{O}C)	TSS (mg/L)	BOD (mg/L)	COD (mg/L)
Sample	8.07	14.5	140*	172*	477*
المعايير الحدية المسموح بها حسب القرار 1/8	6-9	30	60	25	125
Standard Method	Standard Method 4500 – H+B		Standard Method section 2450	Hach Method 10099. Respirometric Method	Standard Method 5220 D

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

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المختبر المركزي

تاريخ اصدار التقرير : 20-1-2025

أ- الجهة المرسلة: دار الهندسة نزيه طالب و شركاؤه

ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات: 15-1-2025

تاريخ استلام العينات في المختبر: 15-1-2025

Sample ID	pH اليدس/ تركيز شوارد الهيدروجين	TSS (mg/L)	BOD (mg/L)	COD (mg/L)
INLET	8.07	200*	263*	745*
المعايير الحدية المسموح بها حسب القرار 1/8	6-9	60	25	125
Standard Method	Standard Method 4500 – H+B	Standard Method section 2450	Hach Method 10099, Respirometric Method	Standard Method 5220 D

الجمهورية اللبنانية
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Sample ID	Total Nitrogen (mg/L)	Total Phosphorus (mg/L)	Phosphate as (PO4) (mg/L)
INLET	158*	9.35	28.7
المعايير الحدية المسموح بها حسب القرار 1/8	30	10	-
Standard Method	Digestion	Standard Method 4500-PBE (Hach Method 10209)	By Calculation

Sample ID	Nitrate NO_3^- (mg/L)	Ammonia (mg/L)	Ammonium (mg/L)
INLET	7.3	110.5*	117
المعايير الحدية المسموح بها حسب القرار 1/8	90	10	-
Standard Method	Standard Method 4500 - NO3 - B		

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

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المختبر المركزي

تاريخ إصدار التقرير: 3-2-2025

أ- الجهة المرسلة: دار الهندسة نزيه طالب و شركاؤه

ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات:..... 29-1-2025

تاريخ استلام العينات في المختبر:..... 29-1-2025

Sample ID	pH البدس/ تركيز شوارد الهيدروجين	TSS (mg/L)	BOD (mg/L)	COD (mg/L)
Sample	7.79	333*	273*	941*
المعايير الحدية المسموح بها حسب القرار 1/8	6-9	60	25	125
Standard Method	Standard Method 4500 – H+B	Standard Method section 2450	Hach Method 10099, Respirometric Method	Standard Method 5220 D

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

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المختبر المركزي

تاريخ إصدار التقرير: 17-2-2025

أ- الجهة المرسلة: دار الهندسة نزيه طالب و شركاؤه

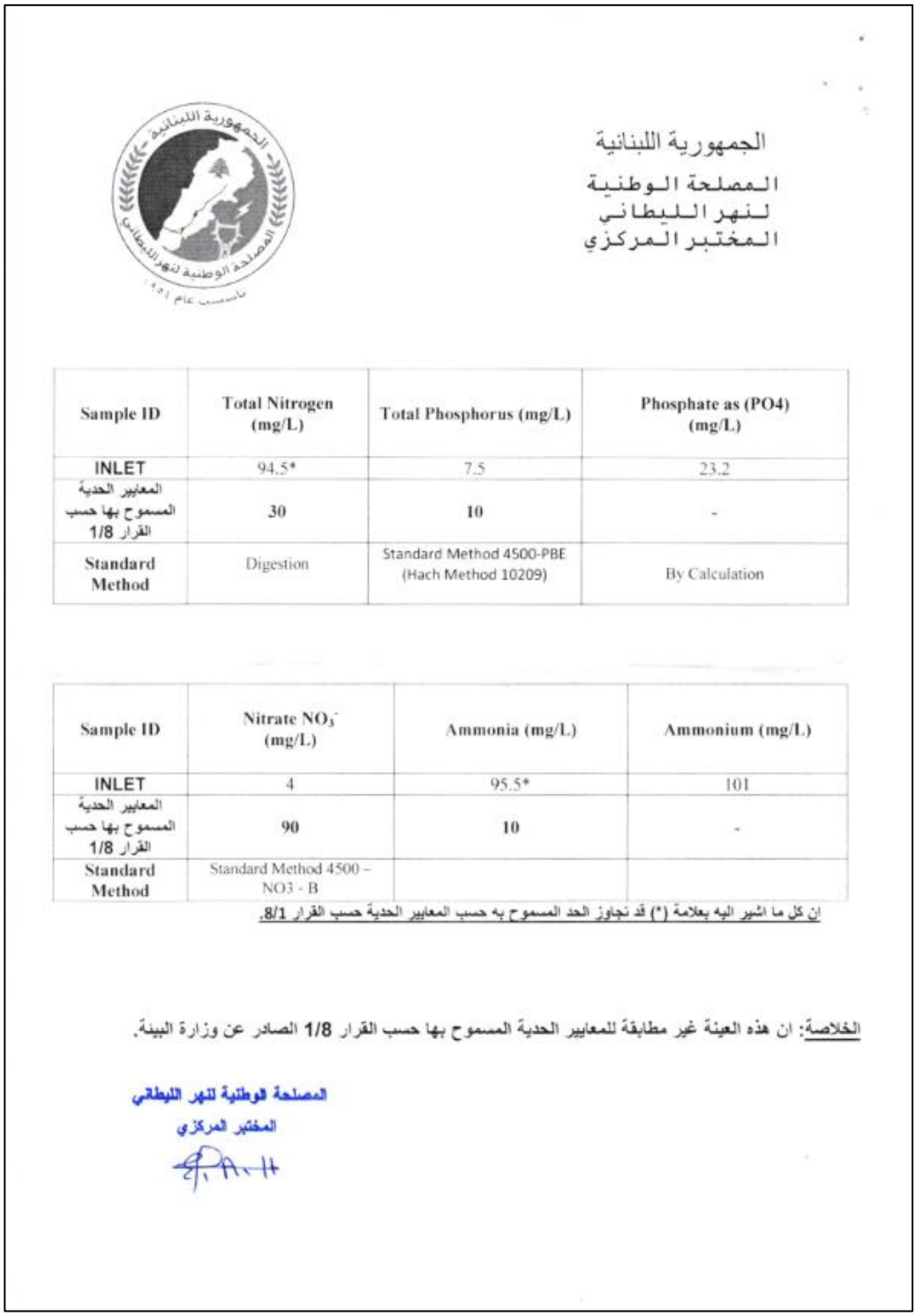
ب- المعلومات الواردة حول العينة:

اسم العينة:.....

تاريخ جمع العينات: 12-2-2025

تاريخ استلام العينات في المختبر: 12-2-2025

Sample ID	pH اليدس/ تركيز شوارد الهيدروجين	Conductivity (µS/cm)	TSS (mg/L)	BOD (mg/L)	COD (mg/L)
INLET	7.87	7090	330*	162*	560*
المعايير الحدية المسموح بها حسب القرار 1/8	6-9	-	60	25	125
Standard Method	Standard Method 4500 – H+B	Standard Method section 2450	Standard Method section 2450	Hach Method 10099. Respirometric Method	Standard Method 5220 D



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Sample ID	Total Nitrogen (mg/L)	Total Phosphorus (mg/L)	Phosphate as (PO4) (mg/L)
INLET	94.5*	7.5	23.2
المعايير الحدية المسموح بها حسب القرار 1/8	30	10	-
Standard Method	Digestion	Standard Method 4500-PBE (Hach Method 10209)	By Calculation

Sample ID	Nitrate NO_3^- (mg/L)	Ammonia (mg/L)	Ammonium (mg/L)
INLET	4	95.5*	101
المعايير الحدية المسموح بها حسب القرار 1/8	90	10	-
Standard Method	Standard Method 4500 – NO3 - B		

ان كل ما اشير اليه بعلامة (*) قد تجاوز الحد المسموح به حسب المعايير الحدية حسب القرار 8/1.

الخلاصة: ان هذه العينة غير مطابقة للمعايير الحدية المسموح بها حسب القرار 1/8 الصادر عن وزارة البيئة.

المصلحة الوطنية لنهر الليطاني
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Figure 5.3 Ghadir Wastewater Sample Test Results up to end of February 2025

5.5.2 Interpretation

Some tests results are beyond the interval of municipal wastewater characteristics as indicated in the remarks column. The low values of the main parameters are under investigation and may be caused by low loads at the time of sampling.

Table 5.3 Sampling Schedule and Testing Summary

SN	Date	Type	Temp °C	pH at 26.2°C	BOD_5 mg/L	COD mg/L	TSS mg/L	VSS mg/L	TN mg/L	NH_4^+ mg/L	NO_3^- mg/L	PO_4^{3-} mg/L	TP mg/L	FOG mg/L	Cond. µs/cm	Remarks
1	17 Sep 2024	G	30.2	6.8	120	290	600	460								
2	27 Sep 2024	G	29.5	6.5	125	285	200	150	140	130	2	87	30			High TN, NH_4^+, TP
3	7 Oct 2024	G	28.8	6.7	60	150	125	100								Low BOD_5, COD
4	22 to 23 Oct 2024	C	24.1	6.9	65	145	250	195	150	135	1	24	8	2	4,100	High TN, NH_4^+, cond., Low BOD_5, COD
5	5 Nov 2024	G	20.2	6.7	45	105	190	160	115	105	3	17	5	2	4,200	
6	18 Nov 2024	G			377	620			28.6	21.1						
7	5 Dec 2024	G			320	698	200									
8	19 to 20 Dec 2024	C		8.01	155	517	120		158	95	9.5	24.8	8.1		8,500	Too High TN, NH4+, cond., PO43-, Low BOD5, COD - Important infiltration to Sewer Net - Industrial Wastewater
9	2 Jan 2025	G		8.07	172	477	140									
10	15 Jan 2025	G		8.07	263	745	200		158	110.5	7.3	28.7	9.35			Ammonia is high, 50% higher than municipal WW. TN is high. Other parameters are OK
11	29 Jan 2025	G		7.79	273	941	333									
12	11 to 12 Feb 2025	C		7.87	162	560	330		94.5	95.5	4.0	23.2	7.5		7,090	
13	26 Feb 2025	G														
14	10 Mar 2025	G														
15	24 Mar 2025	G														
16	7 Apr 2025	C														
17	21 Apr 2025	G														
18	5 May 2025	G														
19	20 May 2025	G														
20	3 Jun 2025	C														
21	17 Jun 2025	G														
22	1 Jul 2025	G														
23	15 Jul 2025	C														
24	28 Jul 2025	G														
25	8 Aug 2025	G														

CHAPTER 6 REVIEW OF THE FEASIBILITY STUDY

Chapter 6 - Review of the Feasibility Study

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Chapter 6 Review of the Feasibility Study

6.1 Main Criteria Set in the Feasibility Study

The feasibility study report issued on 2013 included comprehensive information on the Upgrade of the Ghadir Wastewater Treatment Plant (GWWTP) as well as the development of the networks of the GWWTP Drainage area. The review of the feasibility shows that the Conventional Activated Sludge (CAS) was recommended to be used and proposed 2 phases for implementation of the GWWTP as following:

- Flows data:

Table 6-1 GWWTP - Basic Population Data

	Unit	Year 2050
Population	Capita	594,734
Population Equivalent	Capita	1,913,680
Degree of connection to network		0.90
PE Connected	Capita	1,722,312
Peak Factor		1.44

Table 6-2 GWWTP - Design Flows calculation

Dry Season Flows	Unit Flow (L/cap/d)	Flow (m^3/d)	Flow (m^3/s)	Remarks
Domestic	112.0	192,899	2.23	
Domestic and Industry	32.0	55,114	0.64	
Infiltration and Inflow	20.0	34,446	0.40	
Total Average	**164.0**	**282,459**	**3.27**	Average flow used in the design
Peak Flow	227.4	391,585	4.53	Peak factor is not applied to infiltration and inflow
Wet Season Flows				
Domestic	112.0	192,899	2.23	
Domestic and Industry	32.0	55,114	0.64	
Infiltration and Inflow	50.0	86,116	1.00	
Total Average Flow	194.0	334,129	3.87	
Total Peak Flow	**257.4**	**443,254**	**5.13**	Peak Flow used in the design. Peak factor is not applied to infiltration and inflow

- Phase I: The existing facility at AI-Ghadir which provides preliminary treatment will be extended to accommodate the increased incoming flow. Assuming that the connectivity ratio to the network and population growth increases in line with study of the network related to the concerned drainage area, the GWWTP works were proposed to split into 2 stages:
  - Stage 1a: Extending the capacity of the existing preliminary works
  - Stage 1b: Upgrade the wastewater treatment plant to primary treatment
- Phase II: Upgrade the GWWTP to secondary treatment level
- Design Data:

- The suggested design concentrations are based on SPD 9, incorporating a factor of safety increase of 20%, with the exception of Total P for which the suggested value is based on the average monitoring measurements:
  - BOD5: 360 mg/L;
  - TSS: 450 mg/L;
  - TKN: 72 mg/L; (Total Kjedhal Nitrogen); and
  - Total P: 10 mg/L; (Total Phosphorous TP).
- Regarding the effluent quality: based on the national (MoE 8/1 - 2001) limits for discharge to sea and relevant EU limits (Urban Wastewater Treatment Directive 91/271/EEC), the following effluent requirements will apply:
  - BOD5: 25 mg/L;
  - TSS: 35 mg/L;
  - Total N: 30 mg/L; (Total Nitrogen)
  - NH4+: 10 mg/L (equivalent to N-NH_4^+ 7.8 mg/L);
  - NO3-: 90 mg/L (equivalent to N-NO_3^-: 20.3 mg/L);
  - Total P:10 mg/L (of which P-PO_4^{3-} : 5 mg/L); and
  - Total coliforms: 2,000 MPN/100 mL.

In the Draft Final design, the requirements as defined in the Feasibility Study are mostly met with some minor modifications mainly:

- Combining Phase Ia and Phase Ib into one single phase (i.e Phase I) as following:
  - Upgrade and extension of the existing preliminary treatment plant as described above to a capacity of 282,459 m^3/d dry weather flow. The upgrade consists of additions of fine screens 6 mm clear opening
  - Upgrade the GWWTP works to primary treatment level additions of primary sedimentation tanks, lamella type, capacity 282,000 m^3/d dry weather flow
  - Construction of Sludge Treatment System including anaerobic digestion and gas cogeneration
  - Construction of Power Station (Transformers and Generators)
  - Construction of Odor Control system
  - Installation of SCADA system
- Phase II: Upgrade the GWWTP to secondary treatment level for safe discharge to sea through the existing gravity sea outfall

It is to note that the dimensions of the primary sedimentation tanks, biological tanks and secondary sedimentation tanks mentioned in the feasibility study are oversized. The proposed new dimensions are justified in chapters 7 (Design Criteria and Process Description) and 8 (Process Calculations).

The sludge digestion system as described in the feasibility study is based on Conventional Mesophilic Anaerobic Digester with 10 days solids retention time at 35 °C. While 10 days is a theoretical acceptable value, the common practice is to use 20 days SRT for the conventional mesophilic anaerobic digester. It is recommended to provided Thermal Hydrolysis Process upstream of the anaerobic digester to improve efficiency and benefit of more energy and better biosolids. By incorporating thermal hydrolysis into your project, you can achieve significant benefits, including:

- 50%-75% smaller digesters
- 25%-40% increase in biogas production
- 40%-60% reduction in sludge volume

Fully hygienized and odorless final sludge, ready for land application, following practices adopted by municipalities and utilities worldwide.

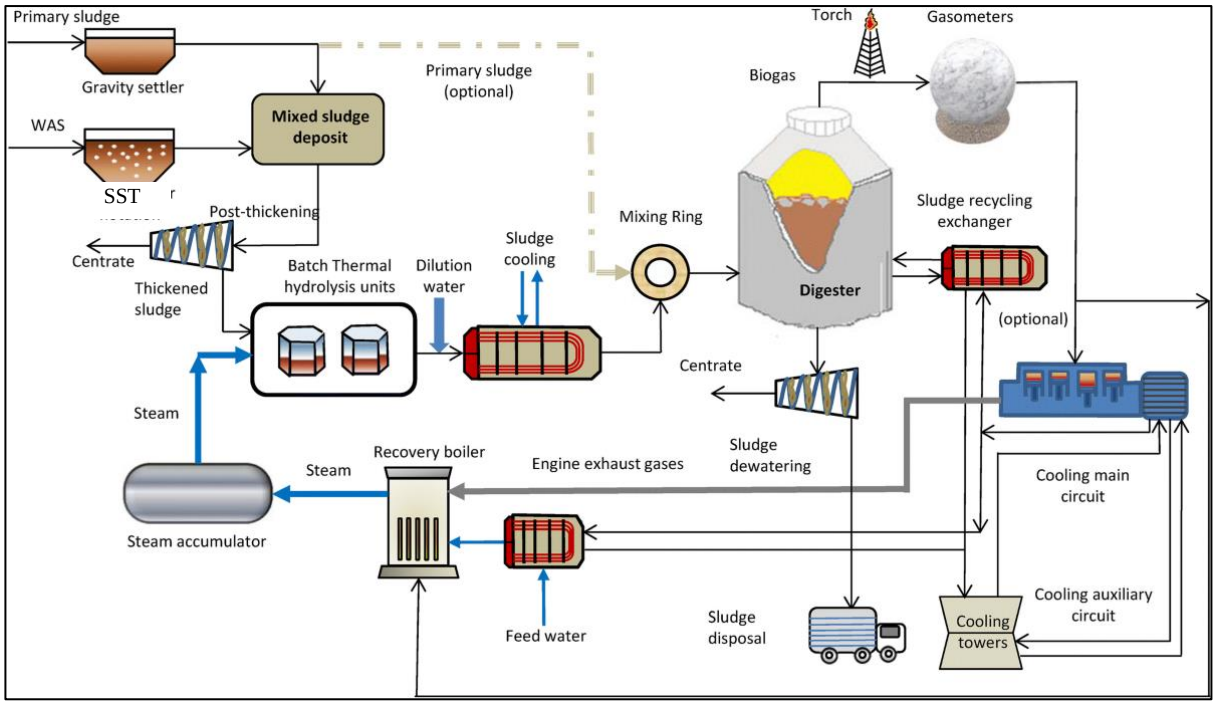


Figure 6-1 Anaerobic Digestion with Thermal Hydrolysis Process (THP)

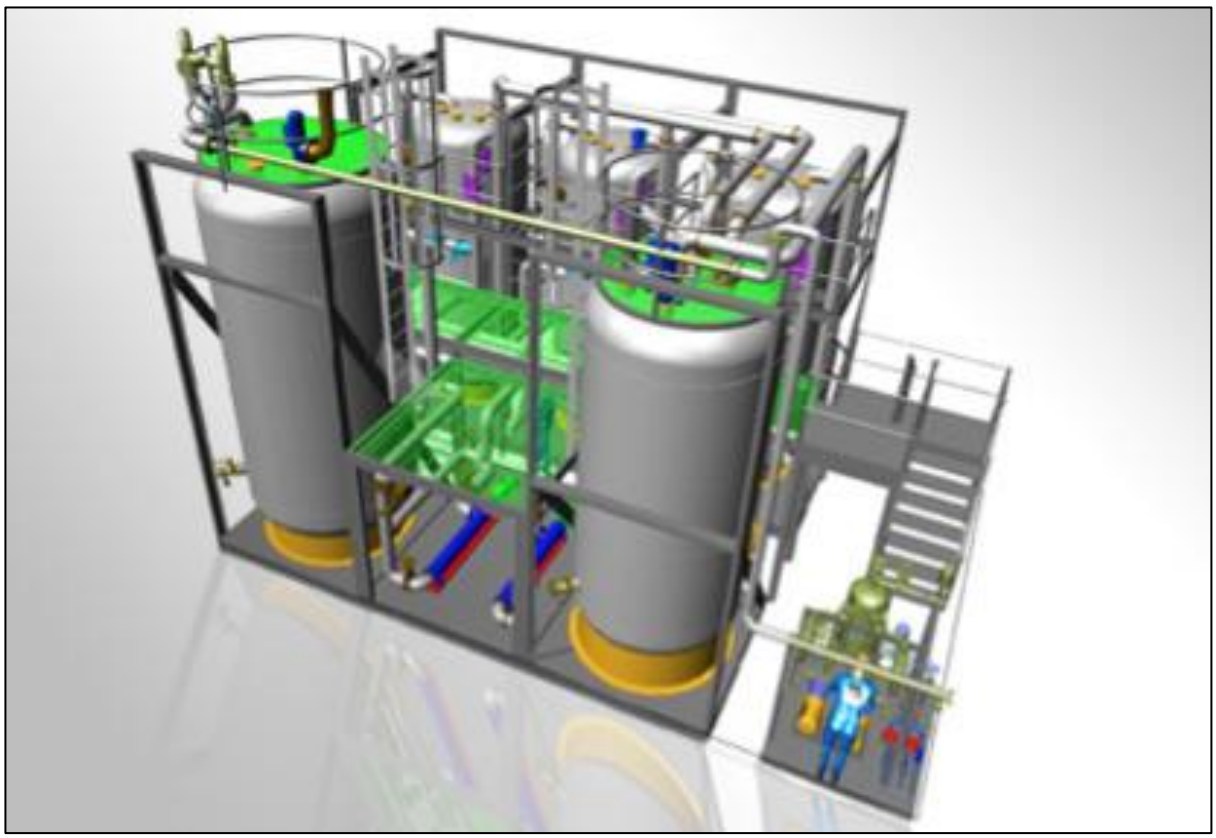


Figure 6-2 Typical Thermal Hydrolysis Process

6.2 Notes on existing Flow conditions

The latest flows data collection reported by Beirut and Mount Lebanon Water Establishment (BMLWE) was conducted in the year 2021 and shows the following:

- **Maximum recorded daily flow:** 116,580 m^3/d (January 17, 2021)
- **Minimum recorded daily flow:** 49,996 m^3/d (August 12, 2021)
- **Maximum monthly flow:** 2,588,134 m^3 (December 2021) ➔83,488 m^3/d
- **Minimum monthly flow:** 1,692,254 m^3 (August 2021)) ➔54,589 m^3/d
- **Yearly flow:** 24,447,545 m^3 ➔66,980 m^3/d

Table 6-3 Summary of collected flows in 2021

Year	Max daily flow		Min daily flow		Max Monthly flow		Min Monthly flow		Yearly Flow
	m^3/d	date	m^3/d	date	m^3	date	m^3	date	m^3
2017	88,079	21/11	39,057	02/09	3,668,303	Oct.	1,652,385	Apr.	23,816,030
2018	118,633	27/12	118,633	16/06	2,539,797	Dec.	1,877,750	Feb.	25,272,885
2019	134,328	28/12	56,794	11/08	2,828,730	Dec.	2,171,126	Aug.	28,375,479
2020	157,772	04/11	47,119	01/08	3,047,018	Jan.	1,797,478	Aug.	22,293,970
2021	116,580	17/01	49,996	12/08	2,588,134	Dec.	1,692,254	Aug.	24,447,545

The recorded figures shows that the current flows are less than calculated in the feasibility study. This is mainly due to the fact that the network and house connections were not completed as planned. However, the design of the plant shall remain for the calculated flows at the year 2050, expecting that the network and house connections shall be completed any time. The plant shall have multiple trains for flexibility of incoming flows.

CHAPTER 7 DESIGN CRITERIA AND PROCESS DESCRIPTION

Chapter 7 – Design Criteria and Process Description

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Chapter 7 Design Criteria and Process Description

7.1 Geographical Data

The Ghadir Wastewater Treatment Plant (GWWTP) is strategically located approximately 1.75 km southwest of the Beirut International Airport Building. This plant is designed to serve a large drainage area including part of Beirut and the districts of Baabda, Aley and Chouf. From the south side, it receives water from Damour pumping station which discharges into Naameh pumping station. This latter discharges into Khaldeh pumping station which discharge into Ghadir WWTP. From the north side, Carlton pumping station (PS1) discharges into Sultan Ibrahim pumping station (PS2) which discharges into GWWTP.

The total number of municipalities that will be served by Ghadir WWTP is 47 municipalities with a total surface area of approximately 175 km^2. Currently, the plant consists of preliminary treatment with a sea outfall.

The plant coordinates are: 33° 48' 48.50" N - 35° 29' 0.37" E

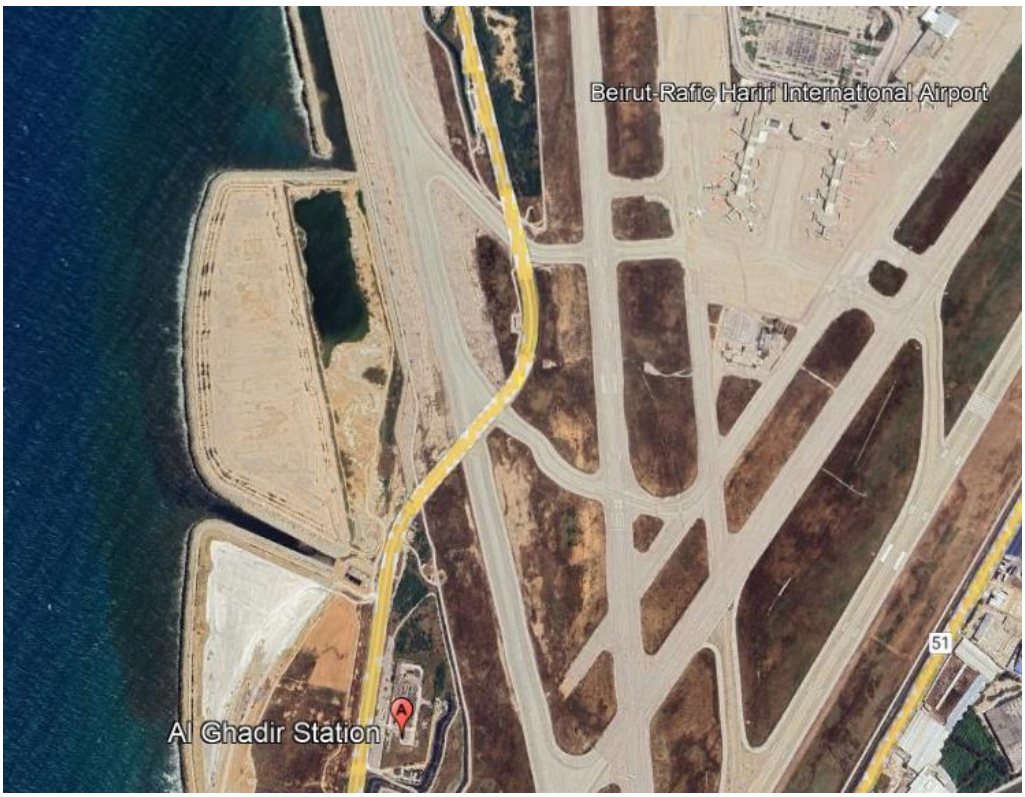


Figure 7.1 Al Ghadir WWTP Site Location

7.2 Environmental data

- Minimum Water Temperature (Winter) : 15°C
- Maximum Water Temperature (Summer) : 31°C
- Minimum Air Temperature (Winter) : 7°C
- Maximum Air Temperature (Summer) : 32°C
- Site Elevation (amsl) : 4 m
- Relative Humidity : 65% - 75%
- Annual Rainfall : 830 mm

7.3 Design Standards & Codes

A- Sewage Treatment & Process:

- SPD (Standard Practice Documents) by CDR, mainly SPD 9
- Ministry of Environment Directive mainly directive 8/1 dated 2001 (latest update)
- EU Directives, mainly EU Directive 91/271/EEC (latest update)

B- Electro-Mechanical Equipment and Pipelines:

Electro-Mechanical Equipment: IEC (International Electrotechnical Commission) and DIN (Deutsches Institute fuer Normung) Standards

Pipeline Standards: ISO, EN, BS, NF standards.

C- Soil Treatment, Roads and Trenches:

Deep Dynamic Compaction: FHWA-SA-95-037, 1995

Roads: AASHTO Geometric Design and Flexible Pavement Design

Trench: ASTM D 2321, CAN/CSA B182.11 and AASHTO Section 30

D- Structure:

ACI 318-14	Building Code requirements for Structural Concrete and Commentary
ACI 350-06	Code Requirements for Environmental Engineering Concrete Structures
ACI 350.3-05	Seismic Design of Liquid-Containing Concrete Structure and Commentary.
ASCE 7-05	American Society of Civil Engineers
UBC 1997	Uniform Building Code
AINSI/AISC 360-16	American national standard for Structural Steel Buildings.

7.4 Design Data

- Flows data:
  - Design year: 2050
  - Population Equivalent connected to the network by 2050: 1,722,312 capita
  - Dry season Unit Flows : 144 L/cap/day
  - Infiltration and inflow – Dry weather : 20 L/cap/day
  - Design Dry Season Average Flows : 3.27 m^3/s
  - Dry Season Peak Flows : 4.53 m^3/s
  - Wet Season Unit Flows : 144 L/cap/day
  - Infiltration and inflow – Wet weather : 50 L/cap/day
  - Wet Season Average Flows : 3.87 m^3/s
  - Wet Season Peak Flows : 5.13 m^3/s
- Influent data:
  - BOD_5: 360 mg/L;
  - TSS: 450 mg/L;
  - TKN: 72 mg/L; (Total Kjeldahl Nitrogen); and
  - Total P: 10 mg/L; (Total Phosphorous TP)

- Primary Effluent data;
  - BOD_5: ≤ 252 mg/L
  - TSS: ≤ 225 mg/L
- Secondary Effluent data:
  - BOD_5: 25 mg/L;
  - TSS: 35 mg/L;
  - Total N: 30 mg/L; (Total Nitrogen)
  - NH_4^+: 10 mg/L (equivalent to N-NH_4^+ 7.8 mg/L);
  - NO_3^-: 90 mg/L (equivalent to N-NO_3^-: 20.3 mg/L);
  - Total P:10 mg/L (of which P-PO_4^{3-} : 5 mg/L); and
  - Total coliforms: 2,000 MPN/100 mL
- Type of Process: Conventional Activated Sludge with lamella type primary clarifiers

7.5 Description of the Treatment Scheme

In order to comply with the feasibility study requirements and the discharge requirements, set by Lebanese legislation and EU standards, the treatment scheme of the Conventional Activated Sludge plant will include the following components:

- Preliminary Treatment Unit, to include:
  - Initial Lifting Pumping Station;
  - Screening (coarse and fine);
  - Aerated De-gritting Chambers.
- Primary Treatment Unit (rectangular lamella settling tanks);
- Secondary Treatment Unit, including:
  - Anoxic tanks;
  - Aeration tanks;
  - Secondary Settling Tanks;
  - Chlorination/de-chlorination tanks.
- Sludge Treatment Facilities, including:
  - Gravity Thickeners, for primary sludge (PS);
  - Sludge Screening for primary sludge;
  - Mechanical Thickening, for surplus activated sludge (SAS);
  - Buffering-Mixing tanks, for PS and SAS;
  - Anaerobic Digestion Treatment, for mixed sludge;
  - Mechanical Dewatering of digested sludge;
  - Storage of dewatered sludge.
- Biogas Utilization Unit, including:
  - Biogas Collection and Storage;
  - Combined Heat - Power (CHP) Engines, for power production;

- Emergency Flaring Station.

A flow chart of the treatment scheme is presented in Figure 7.2.

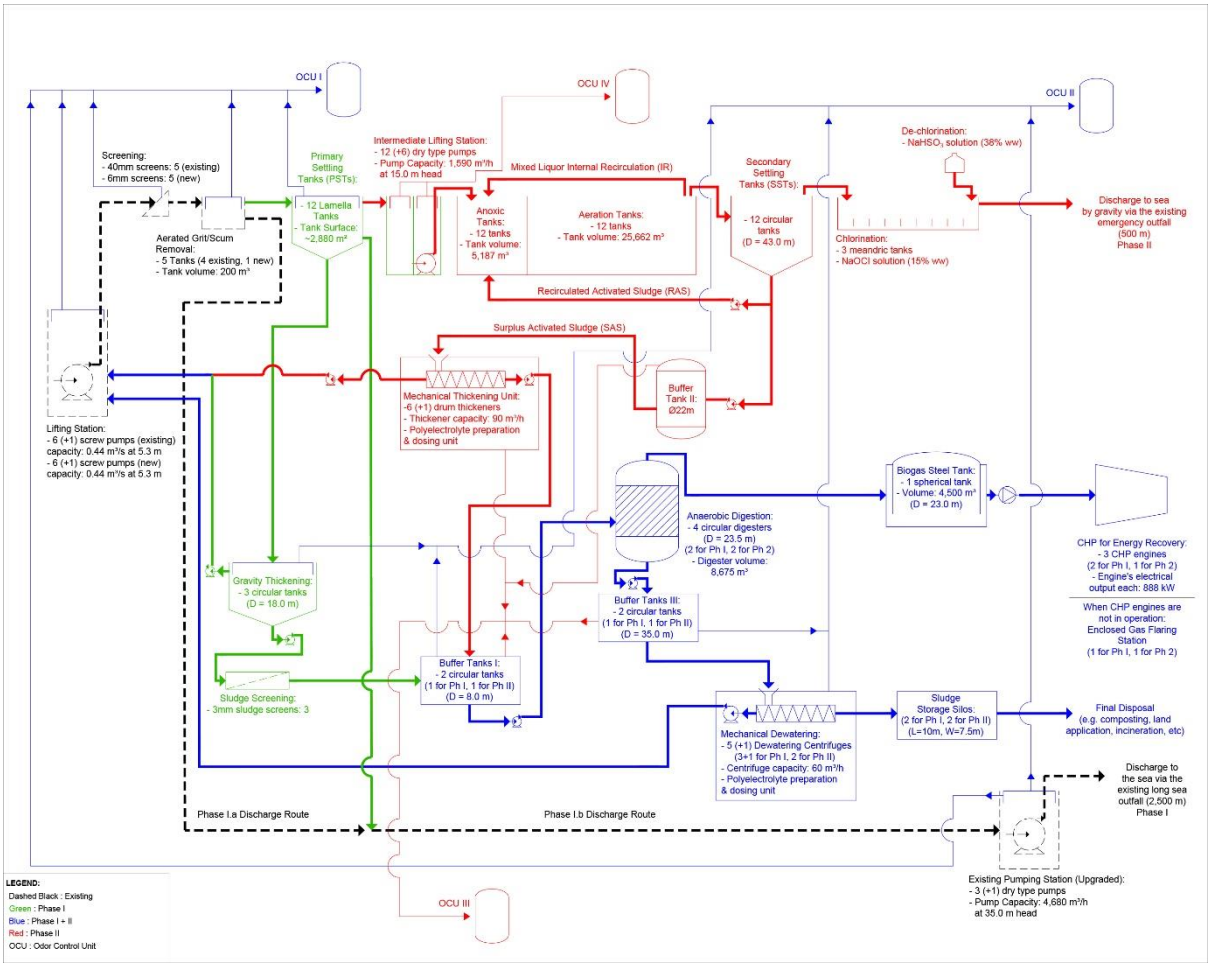


Figure 7.2 Process Flow Chart

7.6 Site conditions – Equipment Assessment

The assessment of the existing equipment is an essential part of the project design and constitute an important factor in using the design data. It is important to reiterate that the plant was subject in 2023 to minor refurbishment works called "Critical repairs" in order to put the plant in operation. Our assessment shall consider those repairs as partial since no regular one-year guarantee is given for the operation of the equipment.

7.6.1 Initial Lifting Station

The existing Archimedean screw pumps, despite their limited repair are in bad conditions. It is up to the bidder to propose either complete replacement of the existing pumps or overhauling the existing pumps based on his assessment. In case of overhauling, the contractor shall guarantee the equipment for one year against any manufacturing defect. A comprehensive assessment report should be submitted for approval prior to overhauling works.



7 Screw pumps geared motor



Screw pumps (D=140cm, 440 L/s each)



Pump Lower Bearing under repair



Basin access stairs



Screw pump



Lifting crane need structural repair

7.6.2 Screening

There are Five (5) existing coarse screens which frames are still in excellent conditions. The screens were recently subject to critical repair works by UNICEF. The contractor shall overhaul the existing screen and provide One year warranty against any manufacturing defect.

New stop logs in the screens channels shall be provided.

New screenings compactors shall be provided. The existing ones shall be dismantled, cleaned and delivered to the client warehouse as will be instructed by the Engineer.

The electrical control panel internal parts are repaired by UNICEF. The outside enclosure is in bad conditions since they are subject to wastewater fumes. The contractor shall replace them by new ones and protect them from direct exposure to wastewater fumes.



Face View for coarse screen



Face View for Coarse screen and relative panel boards in bad status




Mechanical bar screen not in service (mechanical defect) – subject to priority works repair



Mechanical bar screen



Channels



Mechanical bar screen



Screen channel



Electrical panel boards




Electrical Control Panel boards

7.6.3 Grit removal tanks and grit classifier

The grit classifiers were subject to critical repair works and are in operating conditions. The existing equipment can be used with some maintenance and new grit pumps. The grit classifier to be completely replaced by new ones.

The electrical control panel internal parts are repaired by UNICEF. The outside enclosure is in bad conditions since they are subject to wastewater fumes. The contractor shall replace them by new ones and protect them from direct exposure to wastewater fumes.

The air is supplied to the grit tanks via 3 blowers. Two (2) of the blowers are old but operating in good conditions. They are equipped with sound proof enclosure. One of the blower is new, without soundproof enclosure.

The contractor shall refurbish the existing blowers, provide new one similar to the existing old blowers complete with soundproof enclosure. The existing third blower shall be maintained, cleaned and delivered to client's warehouse as instructed by the Engineer.







Aerated degritting tanks

Mobile travelling bridge

Grit pit and Classifier







Sand pit and classifier

Electrical Panel Boards





Grit Classifier

Blower Room For Grit Removal Tanks





3 blowers pumping system-in service





Blower pumps

Valve





Pump Technical data sheet plates

MH MF-E1 & upstream



Outlet of Pipe DN 1500 toward Mh MF-E1



3 penstocks on 3 pipes DN 600 in Manhole MF-E1

7.6.4 Discharge Pumping Station

The pumping station is under good condition. It needs finishing works. The pumps are in good operating conditions. They can be used with proper maintenance and refurbishment. They are operated manually. The following works shall be done by the contractor:

- Overhaul of the existing pumps and supply/install new pump with same capacity to be fitted in the space already allocated for that purpose
- Overhaul of the existing piping, valves, supports, penstocks, lifting crane
- Replace the instruments by new ones to include: Float switches, hydrostatic level sensor, high/low pressure switches, flowmeter.
- Update the operation and control of the pumps so that variable speed can be operated automatically and the full protection of the pumps will be functional

It is to mention that the existing DN1200 sea outfall is designed to receive a peak flow of 2.6 m^3/s. The ultimate flow that it can receive is 3.6 m^3/s. It was initially designed that flows exceeding those values will be diverted to the gravity sea outfall. In the current design for phase 1, both sea outfall will be used simultaneously during the ultimate peak flow of 5.13 m^3/s;

Pumping Station Main Building



3 Existing pumps



3 pumps Q=4900m^3/h at 580 RPM



2 wet well 8mx6m (1 well for 2 pumps)



Circulation pump to wet well (purge pipe)



Existing Pumps & wet wells



Travelling crane



Interior View of pumping station building



Pump



1 Wet Well with 2 pumps



1 Wet Well & pump




Main pumping sea outfall DN1200



Thermometer in VFD panel boards room



Valves electrical panel boards



Flowmeter for pumping sea outfall pipe (in service)



Underground access



VFD Panel Boards



VFD Panel Boards



VFD panel boards



Inside VFD panel boards (installed in 2018)



Schneider Brand (2nd VFD panel boards)

7.7 Phasing of the implementation and proposed works

The complete GWWTP project shall be constructed in Two (2) phases, based on the budget availability and assumptions for completion of all necessary networks serving the Ghadir drainage area. The networks completion being expected to be complete any time, the capacity of the plant shall be then considered that of year 2050.

The phasing of the construction of the GWWTP shall be as follows:

a. Phase I which is part of this contract (operational in 2028-2029): For a capacity of 282,459 m^3/day and includes the extension of the preliminary treatment (to accommodate the increased flow beyond the plant's current capacity and upgrade to primary treatment level by adding primary sedimentation tanks. In addition, construction of sludge treatment, energy recovery, odor control and SCADA are envisaged. Effluent will be discharged through the existing Long Sea Outfall and/or existing emergency sea outfall. This process step shall remove at least 30% of the BOD_5 and at least 55% of the Total Suspended Solids.

b. Phase II which is not part of this contract (operational when budge is available,): Upgrading the plant to Secondary Treatment and extending sludge treatment and related ancillaries to accommodate secondary sludge.

7.8 Description of the facilities

Extension of the preliminary and upgrade to the primary treatment is assumed to take place onshore, within the boundary of the existing plant including all required ancillaries. The construction of sludge treatment and energy production in addition to future secondary treatment shall be located on the reclaimed land which have a total area of 131,850 m^2; only 112,255 m^2 can be used for construction due to setback from airport runway axis and the existing Ancient Wall. Moreover, due to height restriction set by the airport authorities, there shall be restriction on the height of the structure and tanks.

The Layout and Cross sections of the upgraded WWTP are presented in Drawings section.

7.9 Preliminary Treatment Unit

7.9.1 Initial Lifting Station

The existing lifting station contains 6 (+ 1 standby) Archimedean screw pumps, each with a capacity of 440 1/s at 5.30 m head. These pumps were subject to critical repair by UNICEF to put them into operation. However, a comprehensive repair is still needed in case those pumps will be refurbished and used in this contract. It is up to the bidder to make his evaluation and decide if he will replace the

existing pumps or proceed with the comprehensive repair. In both cases, a One year warranty shall be provided.

The supplementary pumping station will be identical to the existing and extend on both side of the existing one as per site conditions. It will then cover the needs of the increased capacity expected during the wet season flow i.e 5.13 m^3/s from the first year of the operation of the extended plant (2050 peak flows).

Wastewater from the pumping stations is gravity-fed to the screens. The pumps shall have the following specifications:

Type of Pumps	: Archimedean Screw pumps
Quantity	: Fourteen (14)
Number of Duty Pumps	: Twelve (12)
Number of Standby Pumps	: Two (2)
Pumps Capacity	: 440 l/s each
Pump head	: 5.3 m
Screw diameter	: 1,400 mm
Flight length	: 10.37 m
Tube diameter	: 762 mm
Angle of Inclination	: 35 °
Material of Screw	: S235JR or equal
Material of Tube	: S235JRH or equal
Side rail material	: S235JR or equal
Cast-in steel trough material	: S235JR or equal
Motor Power	: 37 kW
Type of lower bearing	: Eco-friendly (maintenance free) – For new pumps only
Upper bearing type	: floor mounted
Design life time upper bearing	: >100.000 hrs
Material lower bearing housing	: cast iron GG 25 (DIN 1691)
Shaft material	: cast iron GGG 60/70 (DIN 1693)
Material bushing	: bronze

Surface protection:
Surface preparation : shot blasted SA 2,5
Coating system immersed parts : 2 layers coal tar free epoxy or equal dft 300 microns
Coating system not immersed parts: 1 layer Amercoat 235, fast dry epoxy or equal dft 80 microns

7.9.2 Screening and Treatment of Screening Waste

There are 5 existing rake type screens, of 40 mm bar spacing, in operation at the Ghadir plant.

They appear to be in very good condition; and will be maintained in operation until Phase II, mainly for the driving mechanism. It is estimated that their capacity will cover the wastewater peak wet weather flow up to 2050.

Additionally, 5 fine screens with 6 mm clear spacing shall be installed after the 40 mm screens, which will serve the flows of the present situation as well as the flows up to 2050. During Phase II, the existing 40 mm screens are to be replaced with new ones, as it is assumed that they will be obsolete by that time. The screenings conveyor and screenings compactor are in bad conditions; they shall be replaced by new ones.

After screening, the wastewater gravitates to the aerated grit chambers. Screenings are to be collected via belt conveyors to respective compactors, and will be disposed of as municipal wastes.

For the fine screens:

Total average flow	: 3.87 m^3/s
Total peak flow	: 5.3 m^3/s
Number of channels	: Five (5) (existing, can be adjusted as needed)
Number of screens	: Five (5)
Clear opening	: 6 mm
Type	: Vertical or inclined (75°) bar screen
Material of construction	: SS316
Minimum Approach velocity	: 0.3 m/s
Maximum velocity through screen	: 1.2 m/s (existing channels)
Power	: 1.1 kW

For Belt Conveyor:

Quantity	: One (1)
Length	: 13,000 mm
Width	: 800 mm
Belt material	: coated in PVC
Power	: 0.18 kW

For Screenings Compactor

Quantity	: One (1)
Capacity	: 3 m^3/h
Nominal diameter	: 400 mm
Power	: 5.5 kW
Material of construction	: SS316

7.9.3 Aerated Grid Chambers

There are 4 existing chambers for the removal of grit and FOG (Fat-Oil-Grease) from the incoming screened wastewater. Each of the existing chambers has effective volume of 240 m^3 (L x W x H is 20.0 m x 3.0 m x 4.0 m).

The existing grit chambers are aerated by 2 (+ 1 standby) common blowers with a capacity of 960 m^3/h at 0.4 bar head each. Two blowers are old blowers complete with soundproof enclosure and one is a new replacement without soundproof enclosure. All the blowers are operating in good condition. However, for ease of maintenance, all blowers shall be unified i.e the 2 blowers with soundproof shall be overhauled. The other one shall be replaced with blowers of the same make and model complete

with soundproof enclosure. The dismantled blower shall be maintained, greased, packed and delivered to the client's warehouse as per Engineer's instructions.

In order to treat the flows up to 2050 within the available space, One additional grit chamber is required, with identical dimensions as the existing ones.

The existing blowers have the capacity to provide the aeration needs up to 2050.

The grit collected at the chamber will be removed by grit pumps, the capacity of each being 2.0 m^3/h at 6.0 m head. The pumps will deliver the grit to the respective grit classifiers which will finally be disposed of in containers as municipal waste. The existing grit classifier is in bad conditions and shall be replaced. All existing grit pumps will be replaced and one additional pump will be installed at the new de-gritting chambers.

Type of grit chamber	: Aerated Grit Chamber
Quantity	: Four (4) existing + One (1) New
Total peak flow	: 5.3 m^3/s
Total average flow	: 3.87 m^3/s
Detention time at peak flow	: 3 minutes
Air supply	: m^3/min.m

7.9.4 Discharge Pumping Station

The pumps in the existing outfall pumping station consists of 3 pumps (2 duty, 1 standby) with wall insert for one additional pump. Examining the name plate of the pump, the following are noted:

- Flow 1.32 m^3/s at 11.12 m head
- Motor: 6 poles, 580 rpm, 250 kW

The existing DN1200 long outfall pipe is made from black steel and has approximately 2,500m long. The outfall pipe was designed for a peak flow of 2.6 m^3/s in a new smooth steel pipe.

In other words, the existing outfall pipe cannot transfer the ultimate peak wet weather flow of 5.13m^3/s. Considering those facts, the following works shall be carried out for the Discharge Pumping station:

- Refurbish the existing 3 pumps so that they can be guaranteed for the DLP period
- Supply new pump of similar capacity or less for more flexibility
- Refurbishment works mainly the automatic control of the pumps and their VFDs operation

The pumps will be used to discharge the maximum possible flow through the existing sea outfall. The balance of primary treated effluent will be discharge through the existing emergency gravity sea outfall (DN1500, 500 m long). The characteristics of the pumps are:

Type of pumping station	: Dry type
Number of duty pumps	: Two (2) existing + One (1) New
Number of standby pumps	: One (1)
Capacity each	: 1.32 m^3/s at 11.12 m
Motor Power	: 250 kW
Number of poles	: Six (6)

7.9.5 Primary Treatment

Primary treatment of the wastewater shall consist of 12 rectangular primary settling tanks for solids removal. The tanks will be equipped with lamella plates to reduce their footprint.

Each tank is designed to have a hydraulic load of minimum 6.6 $m^3/m^2/h$ and a hydraulic retention time (HRT) of around 1.0 hour resulting in an effective surface area of 7.5 x 32 = 240 m^2 and effective volume of 1,080 m^3. The primary effluent shall flow by gravity into the wet well of the Interim Lifting Station (Phase II) or to the Discharge Pumping Station / Emergency by-pass (Phase I).

Primary sludge will be collected into bottom hoppers and will be removed by Twelve (+ 4 standby) sludge pumps, each with capacity of 30 m^3/h at 14.0 m head. Those pumps will be installed on top slab between the clarifiers and transfer the primary sludge to the primary sludge thickeners buffer tanks which are located in the reclaimed area. Road crossing is taken into consideration for this purpose.

Also, the pumps in the Interim Lifting Station shall transfer the primary effluent to the biological tanks' distribution located in the reclaimed area.

Type clarifier	: Rectangular, lamella type
Number of clarifiers	: Twelve (12)
Hydraulic Retention time	: ~60 minutes
Tank volume, each	: 1,080 m^3
Tank Surface area	: 240 m^2 (32m x 7.5m)
Side water depth	: 4.5 m
Type of lamella	: Tube settlers
Lamella inclination	: 60°
Lamella tube clear spacing	: 10 cm
Average Dry Weather Flow	: 282,459 m^3/d
Peak Wet Weather Flow	: 5.13 m^3/s

The lamella tube settlers are included as optional item in the BoQ; based on the incoming flow expected at completion of the plant, the client shall decide whether to order the lamella or keep them till later stage. In all cases, the design of the primary clarifiers shall take into consideration that lamella tube settlers shall be installed.

The removal efficiency of the primary clarifier, lamella type, at full capacity shall be:

Removal of BOD_5: ≥ 30%

Removal of TSS: ≥ 55%

7.9.6 Secondary Treatment

Biological treatment consists of de-nitrification tanks, carbon and ammonia reduction tanks and final sedimentation tanks, with a recycle for de-nitrification. The biological treatment is to be divided in 12 operational lanes, as described below. The secondary treatment shall be constructed at Phase II for ultimate flow (year 2050 flow).

7.9.7 Interim Lifting Station

For Phase I, on the civil works shall be carried out including wall inserts. The electromechanical equipment shall be part of Phase II.

Wastewater from the lamella primary settlers will overflow into the interim lifting station wet well, which will pump it to the biological treatment basins distribution unit located in the reclaimed offshore area.

The interim lifting station shall consist of a set of 12 dry type pumps in operation (+6 standby installed), each with capacity of 1,590 m^3/h at 10 m head. The pumping station shall consist of wet well, dry well as pumps house and a ground floor level building, with dimensions L x W x H = 33.0 m x 10.0 m x 4.0m.

Two (2) pipes DN1400 shall be used for this purpose. Road crossing shall be considered. Each pipe will have its own flowmeter and isolation gate valve.

Type of pumps	: Submersible sewage pumps, dry installation
Quantity of duty pumps	: Twelve (12)
Quantity of standby pumps	: Six (6)
Capacity	: 1,590 m^3/h at 10 m head
Power	: 60 kW
Number of poles	: Six (6)

7.9.8 Biological Treatment Basins

A distribution box shall receive primary effluent from the Interim pumping station and distribute it to Twelve (12) biological treatment basins which have a total liquid volume capacity of 316,800 m^3. Each biological treatment basin will be subdivided into different zones, each having different process functions comprising:

- Anoxic Zones; and
- Aerobic Zones.

The wastewater organic pollution and ammoniacal nitrogen will be removed by biological oxidation using freely suspended biomass (mixed liquor) in the aerobic zones. Some of the nitrate produced will be converted to nitrogen gas by denitrification via the biomass in the anoxic zones.

According to the preliminary design, the biological tanks shall consist of Twelve lanes. Each treatment lane will have two anoxic pockets. The two pockets will have effective volume of ~3,840m^3 (L x W x H is (2x16.0) m x 20.0 m x 6.0 m and will be equipped with 2 submersible mixers of minimum power input of ~18.5 kW (each mixer in each pocket) to keep the biomass suspended without introducing air. A total of 24 installed mixers and an additional 4 non installed spare mixers will be provided.

From the anoxic zone the wastewater and mixed liquor will gravitate through three aerobic pockets per lane. The aerobic pockets will biologically oxidize the wastewater's organic load and nitrify the ammoniacal nitrogen. The aerobic pockets are designed to allow for 12 days sludge retention time (SRT), thus the total effective volume of the aerobic zone per lane will be ~18,720 m^3 (L x W x H = (3x52.0) m x 20.0 m x 6.0 m and will be designed to provide plug flow (see relevant layout drawing) to prevent filamentous biomass growth.

All 12 aeration zones will be aerated through 16 blowers (plus 2 blowers on standby) which will be installed in a building with tentative dimensions L x W x H = 30.0 m x 8.5 m x 5.0 m. Each blower will have a capacity of 14,732 m^3/h at 7.0 m head. The air will be supplied in the aerobic zones of each lane via fine-bubble disc diffusers. The total number of disc diffusers required for the aeration is 235,712, which are divided across the 12 lanes (19,642 diffusers per lane).

Furthermore, the mixed liquor at the end of each aeration zone will be recycled using wall propeller pumps to the respective anoxic zone at the start of the lane. Each lane basin will have 3 recycle pumps, 2 duty and 1 installed standby, each rated for 910 m^3/hr at 0.8 m head. A recycle ratio of approximately 1.5 times the sewage feed will be used to achieve the nitrogen related discharge limits, by providing nitrate to the anoxic zones for denitrification.

After passing through the aerobic zone, the mixed liquor will then gravitate to the mass/liquid separation unit, the Final Settling Tanks (FSTs).

<u>For anoxic zones</u>

Number of trains	: Twelve (12)
Compartment in each train	: Two (2)
Hydraulic Retention Time	: 4 h

Total Anoxic Volume	: 47,460 m^3
Volume per train	: 3,840 m^3
Compartment width	: 20.0 m
Compartment Length	: 16.0 m
Train length	: 32 m
Water Depth	: 6 m
Number of mixers per compartment	: - One (1)
Mixer Power	: 18 kW

For aerobic Tanks

Type of Biological Process	: Conventional Activated Sludge Process
Number of trains	; Twelve (12)
Design MLSS	: 3,000 mg/L
Dissolved Oxygen in bioreactor	: 2 mg/L
Solids Retention Time	: 12 d
Type of diffusers to be used	: Fine bubbles diffusers
Diffuser Oxygen transfer efficiency	: 32%
Aeration factor Alpha (α) – Nitrification	: 0.65
Relative DO saturation to clean water (β):	0.95
Hydraulic retention time at 30° C	: 17 h
Total tanks volume	: 224,640 m^3
Water Depth	: 6 m
Train Width	: 20.0 m
Number of compartments per train	: Three (3)
Compartment Length	: 52 m
Train Length	: 156 m

7.9.9 Final Settling Tanks (FSTs)

The final settling tanks are used to complete the biological treatment of sewage, by separating the produced biomass from the liquid.

It is recommended that Twelve (12) circular FSTs are constructed for clarification of the treated effluent. Each FST is designed to have hydraulic load of 1.2 $m^3/m^2/h$, resulting in an effective surface area of 1,256 m^2 (diameter: 40.0 m) at ~4.0-4.5 m side water depth. Each tank is designed as sloped bottom (1/12) and will be equipped with a respective rotating bridge to allow the collection and removal of scum and settled sludge.

Recirculation of the sludge back to the anoxic pockets of each lane will be performed by 12 pumps (+6 standby) with each pump having a capacity of 610 m^3/hr at 6.0 m head, while for the removal of the surplus activated sludge (SAS), 6 pumps (+6 standby) rated for 90 m^3/hr at 10 m head will be used.

Quantity of Clarifiers	: Twelve (12)
Type of clarifier	: Circular, half bridge rotating scraper
Drive type	: Peripheral drive

Overflow rate at average flow	: 20 $m^3/m^2.d$
Overflow rate at peak flow	: 30 $m^3/m^2.d$
Solids loading rate at average flow	: 5 $kg/m^2.d$
Solids loading rate at peak flow	: 8 $kg/m^2.d$
Weir loading rate	: 250 $m^3/m.d$
Hydraulic retention time	: > 4 hours
Clarifier diameter	: 40 m
Water depth	: 4.5 m
Drive power	: 0.75 kW
Peripheral speed	; ~2 m/min

7.9.10 Chlorination/De-chlorination

The clarified effluent from the FSTs will overflow to the chlorination tanks where Sodium Hypochlorite (NaOCI) 15% solution will be added for the disinfection of the effluent prior to the sea discharge.

It is recommended that Three (3) tanks are constructed for this purpose, each with dimensions L x W x H = 50.0 m x 15.0 m x 3.0 m. Each tank is designed to have three lanes of 5.0 m width (serpentine flow), in order to obtain a L/W ratio of 30:1 and HRT of more than 20 minutes. Each tank will be equipped with one dosing pump (+1 standby) with a capacity of 0-150 liters/hr at 5.0 m head. The NaOCI solution will be stored in a $7m^3$ storage tank within the Chemical Storage Building (dimensions L x W x H = 15.0 m x 9.0 m x 6.0 m).

Provision will be made for the last 1.0 m of each meander tank to be used for de-chlorination of the treated effluent prior to its discharge to the sea. For this purpose, one dosing pump (+ 1 standby) is used per tank to add $NaHSO_3$ 38% solution to the chlorinated effluent. Each pump will have capacity of 0-90 liters/hr at 5.0 m head and the $NaHSO_3$ 38% solution will be stored in a 42 m^3 tank located in the Chemical Storage Building.

After the de-chlorination tank, the effluent will overflow to the existing emergency outfall (500m long) from where it will discharge by gravity to the sea.

7.10 Sludge Treatment

7.10.1 Gravity Thickening of Primary Sludge

The primary sludge from the lamella Primary Settling Tanks (PSTs) will be pumped to the gravity thickeners, where the sludge will be settled to thicken to a dry solids content of "'6%. Three (3) gravity thickeners are designed based on a solids loading rate (SLR) of up to 100 kg $DS/m^2/day$, which results in an effective surface area of ~255 m^2 (at a diameter of 18.0 m) for each.

Each thickener will be equipped with a rotating bridge to allow the collection and removal of scum and thickened sludge from its bottom. Additionally, it will be equipped with 2 thickened sludge pumps (one as standby) each with capacity of 30 m^3/hr at 6.0 m head and with 2 more pumps (one as standby) for the pumping of supernatant liquor back to the entrance of the PSTs, each rated for $30m^3/hr$ at 5.0 m head.

Quantity of Thickeners	: Three (3)
Type of thickener	: Picket type, full fixed bridge
Solids loading rate	: 100 $kg/m^2.d$
Thickener diameter	: 18 m
Water depth	: 4.5 m
Drive power	: 0.18 kW

7.10.2 Sludge Screening

In order for any large material that has passed through the wastewater screens (and settled with the primary sludge) to be removed from the primary sludge, the thickened primary sludge will be pumped through Three (3) sludge screens. These screens will have openings up to 3.0 mm. After the screens the sludge ends up to the buffer tanks (Buffer Tanks I) prior to anaerobic digestion.

7.10.3 Mechanical Thickening of Surplus Secondary Sludge

The surplus secondary sludge will be pumped to a buffer tank (Buffer Tank 2) prior to the mechanical thickeners. The buffer tank will have an effective volume of 2,281 m^3 (diameter: 22.0 m, height: 6.0 m) which allows for more than 6 hours hydraulic retention of the secondary sludge prior to the feeding of the thickeners. Six (6) closed drum type thickeners (and one standby) will be used for this purpose, each with capacity of 90 m^3/hr.

Each drum thickener will be equipped with the respective feeding pump (86 m^3/hr at 5.0 m head) the respective polyelectrolyte dosing pump (0-10 litres/hr at 5.0 m head) and the respective thickened sludge pump (12 m^3/hr at 30.0 m head). Also, 2 pumps (+1 standby) each rated for 149 m^3/hr at 5.0 m head, will be used to return supernatant leachate back to the denitrification basins.

The polyelectrolyte solution will be prepared in a system consisting of preparation tanks and mixers and will be located within an adjacent building.

After thickening the secondary sludge will have a solids-content of ~6% and will be pumped to the thickened sludge buffer tanks (Buffer Tanks 1) prior to digestion.

7.10.4 Anaerobic Digestion

Both thickened primary and secondary sludge will be mixed in the buffer tanks prior to anaerobic digestion. Two buffer tanks will be constructed, each with an effective volume of 302 m^3 (diameter: 8.0 m, height: 6.0 m) to allow a sludge retention time of 6 hours prior to the anaerobic digestion.

The mixed sludge will then be introduced to the anaerobic digesters. The digesters will operate with a sludge retention time of 15 days at a temperature of 35 °C, in order to stabilize the sludge and transform any volatile compounds into biogas. Accordingly, Four (4) anaerobic digesters will be constructed each with an effective volume of 8,675 m^3 (diameter: 23.5 m, height: 20.0 m).

Each digester will be equipped with 1 feeding pump (+1 standby) rated for 24 m^3/hr at 22.0 m head, with a mixing system with minimum power input of 30 kW and with 1 recirculation pump (+ 1 standby) rated for 48 m^3/hr at 20.0 m head.

After the digestion tanks the sludge will be collected in the buffer tanks (Buffer Tanks III) prior to the dewatering process.

During Phase I, the biogas produced at the digesters will be collected by the use of 4 (+2 standby) biogas blowers rated for 250 m^3/hr at 5.0 m head and stored at a steel spherical tank of 4,500 m^3 effective volume.

This tank is designed to allow biogas storage for ~2 hours. In Phase II, three additional blowers (1 as standby) will be installed to cover the increase in biogas produced.

All pumps and blowers will be located within a building with dimensions L x W x H = 15.0 m x 10.0 m x 6.0 m.

7.10.5 Digested Sludge Buffer Tanks prior to Dewatering

Buffer tanks will be used to provide sufficient time for the digested sludge to cool and be degassed (from residual gasses from the digestion process). Two (2) tanks will be constructed, which are to designed based on a HRT of 5 days. Each tank will be equipped with an aeration system to facilitate degassing of the digested sludge. The aeration system will consist of 2 blowers (+1 standby) rated for 5,638 m^3/hr at 6.0 m head and coarse bubble diffusers.

7.10.6 Sludge Dewatering

After the buffer tanks, the digested sludge is to be fed to the mechanical dewatering unit. In Phase I, the mechanical dewatering unit will consist of 3 (+ 1 standby) centrifugal dewatering machines (centrifuges) each with 60 m^3/hr capacity, feeding pumps (each rated for 50 m^3/hr at 5.0 m head), polyelectrolyte dosing pumps (each rated for 0-20 liters/hr at 5.0 m head) and dewatered sludge pumps (each rated for 8 m^3/hr at 12.0 m head). The supernatant liquor is to be pumped into the primary settling tanks by 2 (+2 standby) pumps (each rated for 57 m^3/hr at 6.0 m head). In Phase II, Two (2) additional sets of centrifuges (centrifuge- feeding pump- dosing pump- sludge cake pump) will be installed.

The dewatered sludge will be pumped and stored for a week to the dewatered sludge silos. Four rectangular silos (2 for phase I, 2 for Phase II) have been designed for this purpose, each with dimensions LxW x H = 10x7.5 m x 12 m.

Sludge will be removed from the site by truck.

7.10.7 Biogas Utilization

The biogas produced from the anaerobic process will be utilized by Combined Heat-Power (CHP) engines, in order to produce electricity, which will cover significant part of the plant's power and heating needs. For this purpose, biogas from the steel spherical storage tank will be fed to 3 CHP engines (2 in Phase I and one in Phase II), each rated at an electrical output power of 888 kW.

In addition to the CHP engines, a gas flaring station will be installed for safety reasons (Phase I), in order to burn the biogas in situations when the CHP engines are out of operation (malfunction, maintenance, etc.). During Phase II, an additional gas flaring station will be installed.

Moreover, for the initiation of the anaerobic digestion process, auxiliary boilers will be used to heat the sludge.

7.10.8 Odor Control Unit

Due to the possibility of odor incidents occurring on site, and the proximity of the WWTP to Beirut Airport, the plant will be equipped with an odor control system, which will abate odor emissions.

The odor control system will use biological trickling filter with structural media for the oxidation of volatile odorous substances in the off-gases from the following units:

- Preliminary Treatment Unit: all existing and new works (lifting pumps, screens, grit chambers and auxiliaries) will be enclosed and connected to the odour control system;
- Primary Settling Tanks: the tanks will be covered and connected to the system;
- Gravity Thickeners: the tanks will be covered and connected to the system;
- Anaerobic Digesters;
- Sludge Screens;
- Buffer tanks (prior to mechanical thickening, to digestion and to dewatering);
- Mechanical drum thickeners;
- Mechanical Dewatering Unit;
- Dewatered sludge silos.

The design criteria and characteristics of the odor control systems are shown is Table 7.1.

Table 7.1 Odor Control Systems sizing criteria

									Phase I		Phase II	
SN	**Description**	**QTY**	**Air changes per hour**	**Length or dia (m)**	**Width (m)**	**Freeboard (m)**	**Volume (m3)**	**Total Air Flow (m^3/hr)**	**System I m^3/hr**	**ystem II m^3/hr**	**System III m^3/hr**	**System IV m^3/hr**
1	Outfall Pumping Station Wet Well	2	12	12.2	7	4	683.2	8,198.4	8,198.4			
2	Interim Pumping Station Wet Well	2	12	6	15.5	4	744.0	8,928.0				8,928.0
3	Lifting Station (Screw pumps)	12	8	11.5	1.75	1	241.5	1,932.0	1,932.0			
4	Screens channel (Overall dimensions)	1	8	10	12	0.8	96.0	768.0	768.0			
5	Grit Removal Tanks (overall dimensions)	1	8	20	36	0.8	576.0	4,608.0	4,608.0			
6	Primary Clarifiers	12	8	33.4	7.5	0.8	2,404.8	19,238.4	19,238.4			
7	Gravity Thickener - Primary Sludge	3	8	18		1	763.4	6,107.3		6,107.3		
8	Thickened Sludge Pumps and Screens Building	1	12	14	7	4	392.0	4,704.0		4,704.0		
9	Buffer Tank I	2	8	8		1	100.5	804.2		402.1	402.1	
10	Buffer Tank II	1	8	22		1	380.1	3,041.1			3,041.1	
11	Buffer Tank III	2	8	35		1	1,924.2	15,393.8		7,696.9	7,696.9	
12	Sludge Dewatering Building	1	12	25	15	4	1,500.0	18,000.0		18,000.0		
13	Secondary Sludge Thickening Building	1	12	21	12	4	1,008.0	12,096.0			12,096.0	
								Total	**34,744.8**	**36,910.3**	**23,236.1**	**8,928.0**

CHAPTER 8 PROCESS CALCULATION

Chapter 8 – Process Calculations

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Appendix 8.7 Ghadir WWTP – Electrical Panel Sizing (Estimation)

Appendix 8.8 Ghadir WWTP – Fuel Consumption (Estimation)

Chapter 8 Process Calculations

8.1 Main Pumping Station (Verification of Existing)

The purpose of the main pumping station (MPS) is to transfer untreated sewage through a sea outfall to certain distance (2.6 km) into the sea for pollution effect dilution. Once the treatment plant is fully functional at phase II secondary treatment level, this pumping station will not be needed (only for emergency bypass). At phase I (primary treatment level), the primary effluent can be discharge either through the main pumping station (i.e 2.6 km in sea) or through the existing gravity outfall (at 550 m in sea).

Considering the facts that the flow being at this stage less than the capacity of the existing main pumping station and that the existing sea outfall (which is not part of this project) is not capable to handle the peak design flow, it is recommended no to increase the capacity of the MPS to future peak flow and to keep it within a value that the existing sea outfall can handle (2.6 m^3/s). This can be done by maintaining the existing pumps and adding new similar pumps (or lower capacity) so that it can operate on 3 duty + 1 standby pumps.

The calculation scenario hereafter at future peak flow shows the following:

- Increase the pumps capacity to 1.71 m^3/s at 20 m head, the same wet well size can be kept provided that pumps will have 20 starts per hours. This could be acceptable since the pumps will be operated in emergency cases only
- The outfall pipe size should be DN1600

Average Daily Flow	282,000	m^3/d	Station Type	Wet Well	
Peak Flow	18,468	m^3/hr	Wet Well Shape	Rectangular	
Invert Level	+0.22	m	Number of Duty Pumps	3	Standby 1
Finished Ground Level	+4.70	m	Maximum Number of starts per hour (Manufacturer)		20
Discharge Level	+2.90	m	Number of starts per hour (Design)		20
Wastewater Temperature	15.0	°C	Ambient Air Temperature	40 °C	

$$t = \frac{V}{Q_{av}} + \frac{V}{Q_{peak} - Q_{av}}$$

t = Time between start and stop of the pump, s = 180 s

V = Effective volume of the wet well, L

Q_{av} = Average Flow, L/s = 3,263.89 L/s

Q_{peak} = Peak Flow, L/s = 5,130.00 L/s (Pumping capacity)

$$\Rightarrow 180 = \frac{V}{3263.89} + \frac{V}{5{,}130.00 - 3{,}263.89} \quad \Rightarrow \quad V = 213{,}712 \text{ L} = 213.71 \text{ m}^3$$

Select Wet well Width = 7.8 m and Length = 24.8 m ==> Effective height = 1.105 m

Choose effective height = 1.10 m ==> Effective volume = 212,784 L = 212.78 m^3

LEVELS SUMMARY

Minimum Water Level (LLVL - Low Level Alarm) =	**1.60**	m	From Manufacturer's Data Sheet
Low Stop Level (LVL - All Pumps Stop) above LVLL =	**0.10**	m	If the pumping station has more
Effective Water Level (HLVL-Last Pump ON) - Above LVL =	**1.10**	m	than one duty level, the start and
High Level Alarm (HHLVL) - Above HLVL till Invert Level =	**0.10**	m	stop level of each pump shall be
Distance between High Alarm and Invert Level =	**0.10**	m	mentioned later
Distance from impeller centerline to base	1.50	m	From Manufacturer's Data Sheet
Available Submergence of pump impeller	0.20	m	Minimum Water Level above impeller
Total water depth in wet well below invert level >=	**3.00**	**m**	

Head Loss and System curve calculation

Head Loss ΔH is calculated by the following formulas: $\Delta H = J.L$ and $J = \frac{\lambda . V^2}{2gD}$

J = Head loss per unit length, m/m

L =Total equivalent length of pipe, m

D = Pipe inside diameter, m

V = Velocity through the pipe, m/s

g = Gravitational acceleration, 9.81 m/s^2

λ = Darcy Friction factor, dimensionless, calculated by the following Colebrook-White formula

$$\frac{1}{\sqrt{\lambda}} = -2\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right) \quad \Longrightarrow \quad \lambda = \frac{1}{4}\left[\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right)\right]^{-2}$$

ε = Pipe rugosity, m

$Re = \frac{\rho V D}{\mu}$ (Reynolds number)

ρ = Density of fluid, kg/m^3 999.2 kg/m^3

μ = Dynamic viscosity of fluid, kg/(m.s) 0.00114 kg/(m.s)

TSS = Solids Concentration = 0.05% = 0.50 Kg/m^3

==> Corrected Dary Friction Factor: $\lambda = \frac{1}{4}\left[\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right)\right]^{-2} x(26.\frac{TSS}{1000} + 1)$

Head Loss in Riser Pipe Flow 6,156.0 m^3/hr

Diameter	Material	Rugosity ε, mm	V m/s	Length (m)	Reynolds Number	λ	Unit Head Loss J
DN800	DI-New	0.5000	3.4	5.00	2,384,576.4	0.0179733	0.01325 m/m
		0.0000	-		-	-	0.00000 m/m
			-		-	-	0.00000 m/m
							ΔH = 0.07 m

Head Loss in Pumping Main Flow 18,468.0 m^3/hr

Diameter	Material	Rugosity ε, mm	V m/s	Length (m)	Reynolds Number	λ	Unit Head Loss J
DN1500	DI-Worn	1.2000	2.9	2,600.00	3,815,322.2	0.0189390	0.00542 m/m
		0.0000	-		-	-	0.00000 m/m
			-		-	-	0.00000 m/m
							ΔH = 14.10 m

Particular Head Loss in Fittings of Pumping Main

Item	Size	QTY	K	ΔH = $KV^2/2g$	Item	Size	QTY	K	ΔH = $KV^2/2g$
Increaser	800 1200	1	0.33	0.20 m	Magnetic flow meter	DN1500	1	0.12	0.05 m
90° elbow, regular	DN800	3	0.30	0.54 m	Gate Valve-Full Open	DN1500	1	0.19	0.08 m
Gate Valve-Full Open	DN800	2	0.19	0.22 m	tee standard 90° turn	DN1500	1	1.80	0.77 m
Check Valve	DN800	1	3.50	2.06 m		DN1500		0.00	0.00 m
	DN800		0.00	0.00 m		DN1500		0.00	0.00 m
				0.00 m					0.00 m
				0.00 m					0.00 m
				3.02 m					**0.91 m**

TOTAL HEAD LOSSES in Discharge Piping (m) = 0.07 + 14.10 + 3.02+0.91 = 18.10 m

= STATIC HEAD (m) = 1.00 m

SAFETY HEAD (m) = 1.00 m

TOTAL DYNAMIC HEAD (m) = 18.10 + 1.00 + 1.00 = 20.10 m

NPHSa calculation

$NPSH_a = h_{atm} - h_{vapor} - \Delta H - h_{Lift} - V^2/2g$

h_{atm} = 10.3 - 0.0015z = 10.3 - 0.0015 x 4.70 = 10.29 m

h_{vapor} = Vapor pressure head = $0.0623e^{[17.27T/(T+237.3)]}$ = $0.0623e^{17.27*15.0/(15.0+237.3)}$ = 0.17 m

ΔH = Friction Loss at suction piping = 0.00 m

h_{lift} = Suction Lift = -0.20 m

$V^2/2g$ = Velocity Head at suction piping = 0.00 m

$NPSH_a$ = 10.29 - 0.17 - 0.00 - (-0.20) - 0.00 = 10.32 m

8.2 Inlet Lifting Station

Design Wet Weather Peak Flow : 5.13 m^3/s

Type of pumps : Archimedean Screw Pump

Number of operating pumps : 12

Number of standby pumps : 2

Capacity of each pump : 0.43 m^3/s at 5.3 m head

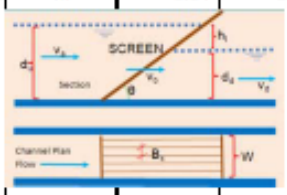
Power each : 37 kW

It is recommended to provide complete 14 new Archimedean screw pumps with eco-friendly lower bearing (greased for life)

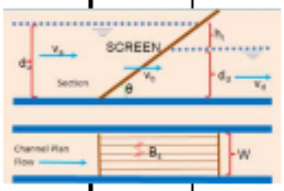
		Current Flows			Design Flows (Ultimate)		
Description	**Unit**	**Average**	**Summer**	**Winter**	**Average**	**Summer**	**Winter**
1. Design Basis							
1.1 Level of Treatment			Primary			Secondary	
1.2 Wastewater Influent Quantities and Quality							
Watewater Temperature	°C	23.00	30.30	15.30	23.00	30.30	15.30
Total Population	PE		846,969			1,913,680	1,913,680
Daily Flow Q_{av}	m^3/d	140,000	140,000	206,500	282,528	282,528	334,368
	m^3/s	1.62	1.62	2.39	3.27	3.27	3.87
Peak Hourly Flow (Dry weather) Q_{pk}	m^3/h	9,000	9,000	11,772	16,308	16,308	18,468
	m^3/s	2.50	2.50	3.27	4.53	4.53	5.13
Peak Flow Factor		1.55	1.55	1.62	1.39	1.39	1.33
2 Preliminary Treatment							
2.1 Initial Lifting Station							
Type: Screw Pumps							
No of existing duty pumps	ea	6	6	6	6	6	6
No of standby existing pumps	ea	1	1	1	1	1	1
Capacity each	m^3/s	0.44	0.44	0.44	0.44	0.44	0.44
Existing pumping capacity	m^3/s	2.64	2.64	2.64	2.64	2.64	2.64
No of new duty pumps	ea	0	0	0	6	6	6
No of standby new pumps	ea	0	0	0	1	1	1
Capacity each (New Pumps)	m^3/s	0.44	0.44	0.44	0.44	0.44	0.44
New pumps capacity	m^3/s	0	0	0	2.64	2.64	2.64
Total pumping capacity	m^3/s	2.64	2.64	2.64	5.28	5.28	5.28
Safety factor (pumping)	-	1.06	1.06	0.81	1.17	1.17	1.03

8.3 Screenings

8.3.1 Coarse Screens

		Current Flows			Design Flows (Ultimate)		
Description	**Unit**	**Average**	**Summer**	**Winter**	**Average**	**Summer**	**Winter**
2.2 Coarse Screening							
Type:							
No of Existing duty screens	ea	5	5	5	5	5	5
No of Existing standby screens	ea	0	0	0	0	0	0
Screen channel width	m	1.40	1.40	1.40	1.40	1.40	1.40
Clear opening	mm	40	40	40	40	40	40
Bars width	mm	10	10	10	10	10	10
Bars depth	mm	50	50	50	50	50	50
Weir Plate length (after grit removal unit), L	m	3.50	3.50	3.50	3.50	3.50	3.50
Number or weirs	ea	4	4	4	4	4	4
Total Length of Weirs	m	14.00	14.00	14.00	14.00	14.00	14.00
Weir Plate type: Rectangular, Sharp-crested							
Formula: $Q = C_w L.H^{1.5}$ Q in m^3/s							
C_w (for sharp crested weir)		1.84	1.84	1.84	1.84	1.84	1.84
Head over weir at average flow	m	0.16	0.16	0.20	0.25	0.25	0.28
Head over weir at peak flow	m	0.21	0.21	0.25	0.25	0.25	0.28
Highest level of weir plate	m				3.83	3.83	3.83
Lowest level of weir plate	m	3.97	3.97	3.97	3.97	3.97	3.97
Screens channel floor level	m	3.54	3.54	3.54	3.54	3.54	3.54
Minimum Water level downstream the scree	m	4.13	4.13	4.17	4.22	4.22	4.25
Minimum Water depth downstream the scre	m	0.59	0.59	0.63	0.68	0.68	0.71
Maximum Water level downstream the scree	m	4.18	4.18	4.22	4.22	4.22	4.25
Maximum Water depth downstream the scre	m	0.64	0.64	0.68	0.68	0.68	0.71
$h_f = \frac{(V_b^2 - V_a^2)}{2 \times g} \times \frac{1}{C}$ Where: h_f = head loss, m V_a = approach velocity in upstream channel, m/s V_b = velocity through the opening, m/s g = acceleration due to gravity, 9.81 m/s² C = empirical discharge coefficient 0.7 for a clean screen 0.6 for a clogged screen							
Assume water depth upstream screen	m	0.66	0.66	0.71	0.75	0.75	0.78
Velocity upstream screens at average flow, V_a	m/s	0.35	0.35	0.48	0.62	0.62	0.71
Velocity through screens at average flow, V_b	m/s	0.68	0.68	0.82	1.08	1.08	1.17
Head lost through clean screens at av. flow, h	m	0.024	0.024	0.032	0.056	0.056	0.064
Head lost through clogged screens at av. flow	m	0.028	0.028	0.038	0.066	0.066	0.074
Verify WD downstream the screens		*0.63*	*0.63*	*0.67*	*0.68*	*0.68*	*0.71*
At Peak hydraulic flow							
Head over weir at peah hydraulic flow	m	0.21	0.21	0.25	0.31	0.31	0.34
Water level downstream the screens	m	4.18	4.18	4.22	4.28	4.28	4.31

8.3.2 Fine Screens

Description	Unit	Current Flows			Design Flows (Ultimate)		
		Average	Summer	Winter	Average	Summer	Winter
2.3 Fine Screening							
Type:							
No of New duty screens	ea	5	5	5	5	5	5
No of New standby screens	ea	0	0	0	0	0	0
Screen channel width	m	1.40	1.40	1.40	1.40	1.40	1.40
Clear opening	mm	6	6	6	6	6	6
Bars width	mm	6	6	6	6	6	6
Bars depth	mm	40	40	40	40	40	40
Weir Plate length (after grit removal unit), L	m	3.50	3.50	3.50	3.50	3.50	3.50
Number or weirs	ea	4	4	4	4	4	4
Total Length of Weirs	m	14.00	14.00	14.00	14.00	14.00	14.00
Weir Plate type: Rectangular, Sharp-crested							
Formula: $Q = C_w L.H^{1.5}$ Q in m^3/s							
C_w (for sharp crested weir)		1.84	1.84	1.84	1.84	1.84	1.84
Head over weir at average flow	m	0.16	0.16	0.20	0.25	0.25	0.28
Head over weir at peak flow	m	0.21	0.21	0.25	0.31	0.31	0.34
Highest level of weir plate	m				3.83	3.83	3.83
Lowest level of weir plate	m	3.97	3.97	3.97	3.97	3.97	3.97
Screens channel floor level	m	3.54	3.54	3.54	3.54	3.54	3.54
Minimum Water level downstream the scree	m	4.13	4.13	4.17	4.22	4.22	4.25
Minimum Water depth downstream the scre	m	0.59	0.59	0.63	0.68	0.68	0.71
Maximum Water level downstream the scree	m	4.18	4.18	4.22	4.28	4.28	4.31
Maximum Water depth downstream the scre	m	0.64	0.64	0.68	0.74	0.74	0.77
$h_f = \frac{(V_b^2 - V_a^2)}{2 \times g} \times \frac{1}{C}$ Where: h_f = head loss, m V_a = approach velocity in upstream channel, m/s V_b = velocity through the opening, m/s g = acceleration due to gravity, 9.81 m/s² C = empirical discharge coefficient 0.7 for a clean screen 0.6 for a clogged screen.							
Assume water depth upstream screen	m	0.72	0.72	0.77	0.89	0.89	0.95
Velocity upstream screens at average flow, V_a	m/s	0.35	0.69	0.65	0.62	0.56	0.53
Velocity through screens at average flow, V_b	m/s	0.99	0.99	1.21	1.45	1.45	1.54
Head lost through clean screens at av. flow, h	m	0.063	0.037	0.076	0.126	0.131	0.153
Head lost through clogged screens at av. flow	m	0.073	0.043	0.089	0.147	0.153	0.179
Verify WD downstream the screens		*0.65*	*0.68*	*0.68*	*0.74*	*0.74*	*0.77*
At Peak hydraulic flow							
Head over weir at peah hydraulic flow	m	0.21	0.21	0.25	0.31	0.31	0.34
Water level downstream the screens	m	4.18	4.18	4.22	4.28	4.28	4.31

8.4 Grit Removal Units

Description	Unit	Current Flows			Design Flows (Ultimate)		
		Average	Summer	Winter	Average	Summer	Winter
2.4 Grit Removal Tanks							
Type: Aerated Grit Chamber							
Number of Existing Grit Chambers		4	4	4			
Number of New Grit Chambers					1	1	1
Total Number of rit Chambers		4	4	4	5	5	5
Design Peak flowrate	m^3/s	2.50	2.50	3.27	4.53	4.53	5.13
Design flowrate per chamber	m^3/s	0.63	0.63	0.82	0.91	0.91	1.03
Required retention time at peak flow	min	3.00	3.00	3.00	3.00	3.00	3.00
Required minimum volume, each	m^3	112.50	112.50	147.15	163.08	163.08	184.68
Width to Depth Ratio		1.10	1.10	1.10	1.10	1.10	1.10
Assume Width	m	3.50	3.50	3.50	3.50	3.50	3.50
Calculated Depth	m	3.18	3.18	3.18	3.18	3.18	3.18
Calculated Length	m	10.10	10.10	13.21	14.64	14.64	16.58
Average detention time in each chamber	min	4.63	4.63	4.10	4.16	4.16	3.98
Air supply requirement criteria	$m^3/min.m$	0.30	0.30	0.30	0.30	0.30	0.30
Existing Length	m	20.00	20.00	20.00	20.00	20.00	20.00
Air required for each grit chamber	m^3/min	6.00	6.00	6.00	6.00	6.00	6.00
Total air supply required	m^3/min	24.00	24.00	24.00	30.00	30.00	30.00
	ft^3/min	846.72	846.72	846.72	1,058.40	1,058.40	1,058.40
Number of duty blowers	ea	2.00	2.00	2.00	2.00	2.00	2.00
Number of standby blowers	ea	1.00	1.00	1.00	1.00	1.00	1.00
Air flow required per blower	m^3/h	720.00	720.00	720.00	900.00	900.00	900.00
Existing air blower capacity (each)	m^3/h	960.00	960.00	960.00	960.00	960.00	960.00
Assume unit grit removal	$m^3/1000\ m^3$	0.05	0.05	0.05	0.05	0.05	0.05
Volume of grit removed	m^3/min	7.00	7.00	10.33	14.13	14.13	16.72
Weir Plate length, L	m	3.50	3.50	3.50	3.50	3.50	3.50
Number or weirs	ea	4	4	4	5	5	5
Total Length of Weirs	m	14.00	14.00	14.00	17.50	17.50	17.50
Weir Plate type: Rectangular, Sharp-crested							
Formula: $Q = C_w L.H^{1.5}$ Q in m^3/s							
C_w (for sharp crested weir)		1.84	1.84	1.84	1.84	1.84	1.84
Head over weir at average flow	m	0.16	0.16	0.20	0.22	0.22	0.24
Head over weir at peak flow	m	0.21	0.21	0.25	0.27	0.27	0.29
Lowest level of weir plate	m	3.86	3.86	3.86	3.86	3.86	3.86
Highest level of weir plate	m	3.97	3.97	3.97	3.97	3.97	3.97
Maximum Water Level	m	4.13	4.13	4.17	4.13	4.13	4.15

8.5 Primary Sedimentation Tanks

The detailed calculations of the PST show that using 12 rectangular clarifiers lamella type having each a surface area of 240 m^2 (instead of 270 m^2 indicated in the feasibility study) can produce a very good quality of primary effluent. The quantity of sludge produced (2,829 m^3/d at 3%) is almost similar to that included in the feasibility study. While the calculations show that the clarifiers can reach 67% TSS removal and 45% BOD_5 removal, we have considered for the biological calculations: 50% for TSS and 35% BOD_5 removal.

Average Daily Flow	282,000 m^3/d	Maximum Water temperature	30 °C
Peak Flow	19,008 m^3/hr	Minimum Water temperature	15 °C
Tank Height	6.00 m	Summer Ambient temperature	35 °C
Clarifier type	Rectangular	Winter Ambient temperature	2 °C
BOD_5	360 mg/L	Total Suspended Solids TSS	450 mg/L
Number of Clarifiers	12	TKN 70 mg/L	

Typical Design Information for Lamella Primary Sedimentation Tanks (PST) and Chemically Enhanced Primary Treatment (CEPT)

Parameter	Unit	Range / Value	
		PST	CEPT
Hydraulic Retention Time in clarifier tank HRT	min	10 - 30	5 - 8
Hydraulic Retention Time on confined volume (settlers) HRT_S	min	3 - 8	1 - 3
Surface Overflow Rate in the Clarifier at peak flow SOR	$m^3/m^2.h$	10 - 15	30 - 40
Flow Velocity between plates/tubes (Confined Flow Velocity)	m/min	0.25 - 0.50	0.75 - 1.25
Weir Loading Rate	$m^3/m.d$	100 - 360	100 - 360
Angle of inclination of plates/tubes	°	55 - 60	55 - 60
Normal spacing between the plates	cm	5 - 10	5 - 10
Length of the plates	m	1 - 2	1 - 2
Reynolds number (R_e)		≤ 2000	≤ 2000
Fraction of clarifier covered by the inclined surface settlers f_S	%	75 - 100	75 - 100
Side Water depth	m	3 - 4	3 - 4
Polymer dosage in CEPT	mg/L		60
Coagulant dosage in CEPT	mg/L		1.5
Dry Solids content of sludge	%	2.5 - 3	2.5 - 3

Calculate the required tank surface area at peak flow conditions with Overflow rate SOR = 6.6 $m^3/m^2.h$

A_{TOTAL} = Q / OR = (19,008.0 m3 / h)/(7 m3 / m2.h) = 2,880.0 m^2 ==> A_{UNIT} = 240.00 m^2

Select Tank Length L = 32.0 m ==> Wdith W = 7.50 m Select Width W = 7.5 m

Select Side Water Depth = 4.5 m ==> Tank Volume = 1,080.0 m^3 ==> $HRT_{CLARIFIER}$ = 66.2 min

Minimum weir length per tank 65.28 m Weir side length = 8 m ==> # of weirs > 8.2

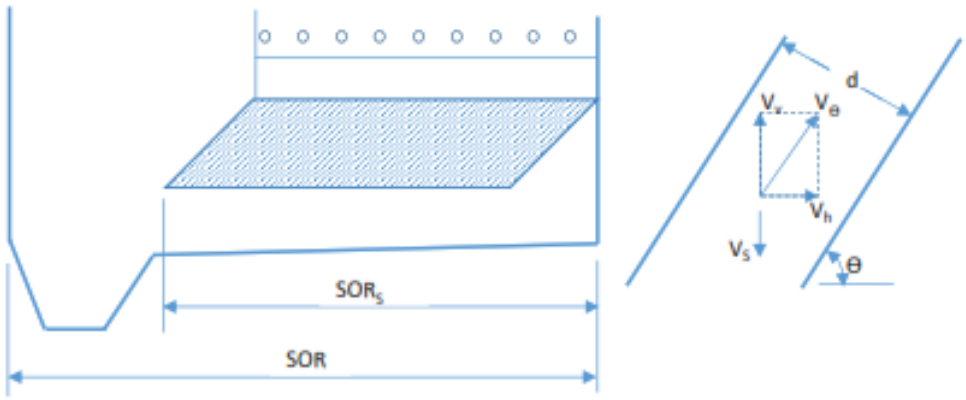
Select Number of Weirs per Tank = 12 ==> Calculated Weir Loading Rate = 244.79 m^3/m.d OK

Specify the selected tube settlers

Angle of inclination, Θ (°)	60 °
Clear spacing (d)	10 cm
Length of the plate	2.0 m
Fraction of the basin area covered by the inclined surface settlers f_S	75%
Area loss due to settler material thickness, support frames, inclination, etc. f_b	0.95
Equivalent coverage area give by the manufacturer A_{tp}	6.5 m^2/m^2
Equivalent coverage area for flat plate settlers (for comparison only)	6.2 m^2/m^2

Calculate the flow velocity

Calculate the surface overflow rate (SOR_S) in the area covered by the inclinded plate settlers



$SOR_S = SOR/f_S$ = (6.6 m3/m2.h) / 0.75 = 8.8 $m^3/m^2.h$

Calculate the vertical vector of the flow velociy between the plates

$v_v = SOR_S/f_b$ = (8.8 m3/m2.h) / 0.95/(3,600 s/h) = 0.002573 m/s

Calculate the flow velocity in the direction parallel to the plates

$v_\theta = v_v / \sin(\Theta)$ =(0.002573 m/s)/sin(60.0) = (0.002573 m/s) / (0.866) = 0.00297 m/s

Calculate Reynolds number

$$R_e = \rho . v . d_h / \mu$$

ρ	= Raw wastewater density (kg/m^3)	=	1,020.0
v	= Flow velocity (m/s)	=	0.00297
d_h	= Hydraulic diameter of lamella (m)	=	0.10
μ	= Dynamic viscosity (kg/m.s)	=	0.001016

==> R_e = 298

Evaluate the performance of the clarifier

Calculate the terminal velocity v_S (i.e Required settling velocity of the particle)

$$v_S \geq \frac{v_\theta . d}{L_p . \cos(\theta) + d . \sin(\theta)}$$

v_S = (0.002971 m/s x 0.10 m) / [2.00 m x cos(60.0) + 0.10 m x sin(60.0)] = 0.000273 m/s

= 23.62 $m^3/m^2.d$

Calculate the particles settling time in the settlers

$t_p = w/(v_s.\cos(\Theta))$ = (0.10 m) / [0.000273 m x cos(60.0)] = 731 s = 12.2 min

The Length of the settlers surface must be enough to allow particles to settles $L_p \geq \frac{d . (v_\theta - v_S . \sin(\theta))}{v_S . \cos(\theta)}$

Verify the length of the surface in the settlers

L_p = [0.10 m x (0.002971 - 0.000273 x sin(60.0) m/s] / [0.000273 m/s x cos(60.0)] = 2.0 m

Determine the terminal velocity in a plain clarifier (For comparison only)

In the plain clarifier, the terminal velocity is theoretically equal to the design surface overflow rate.

Calculate the theoretical velocity at v_S = SOR = 6.6 $m^3/m^2.h$ = 15 m/h = 0.00417 m/s

Design of Activated Sludge Process

Primary Clarifier Design - Lamella type

Terminal velocity in the plain clarifier: 0.00417 m/s

Terminal velocity in the plate/tube settlers: 0.00027 m/s

Efficiency (surface wise) of tube settlers is then: 0.00417 / 0.00027 = 15.24 times

	In Tank	In confined volume
Hydraulic retention time at average flow:	66.18 min	49.63 min
Hydraulic retention time at peak flow:	40.91 min	30.68 min

To estimate the removal efficiency, we shall use the formula established for the conventional PST. The detention time to be used is that of conventional PST having equivalent removal efficiency i.e multiply the actual retention time by 15.24

Calculate BOD and TSS removal efficiency at average flow

For conventional primary clarifier: $R = \frac{t}{a + bt}$

Where, R = Expected removal efficiency

t = nominal detention time, h

a, b = empirical constants

Empirical constants at 20°C		
Item	b	a
BOD	0.020	0.018
TSS	0.014	0.0075

For water temperature of 15 °C , Detention time multiplier = 1.16

==> Nominal Equivalent detention time = 14.48 h

BOD removal = $\frac{14.48}{0.018 + (0.020)(14.48)}$ = 47.07%

TSS removal = $\frac{14.48}{0.008 + (0.014)(14.48)}$ = 68.88%

Calculate BOD and TSS removal efficiency at peak flow

Nominal Equivalent detention time = 8.95 h

BOD removal = $\frac{8.95}{0.018 + (0.020)(8.95)}$ = 45.43%

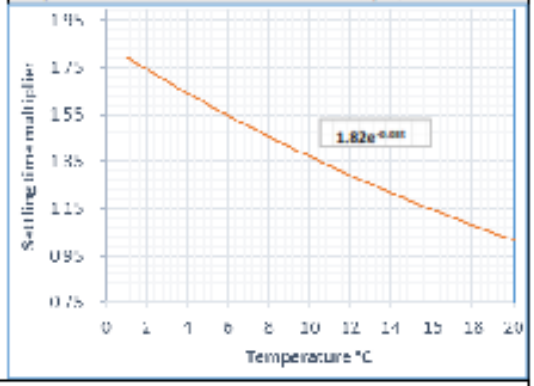
TSS removal = $\frac{8.95}{0.0075 + (0.014)(8.95)}$ = 67.40%

For calculation, use:

BOD removal efficiency = 30%

TSS removal efficiency = 50%

Curve of the increase in detention time required at cooler temperature to achieve the same sedimentation performance at 20°C



Primary Effluent TSS 225.00 mg/L

Primary Effluent BOD_5 252.00 mg/L

Organic Nitrogen (ON) removal 25%

Influent ON = TKN-NH_4 = 70 mg/L

Primary Effluent ON = 52.5 mg/L

Primary Effluent TKN = 52.5 mg/L

From the table, select the settled solids

* Concentration 3% by weight
* Sludge Specific gravity 1.03

Typical values of specific gravity and solids concentration of solids and scum removed from primary sedimentation tanks (M&E Table 5-21)

Type of solids (Sludge)	Specific Gravity	Solids concentration % Range	Solids concentration % Typical
Primary Only			
Medium strength Wastewater	1.03	4 - 12	6
From combined sewer system	1.05	4 - 12	6.5
Primary and waste activated sludge	1.03	2 - 6	3
Primary and trickling filter humus tank	1.03	4 - 10	5
Scum	0.95	Variable	-

Total settled dry solids weight = (282,000 m3/d)(450 g/m3)(1 kg/1,000 g)(0.69) = 87,409.7 kg TSS/d

Total sludge mass = (87,409.670 kg/d)/(0.030 g/g) = 2,913,655.65 kg/d

Total sludge volume = (2,913,655.7 kg/d)/(1.03)/(1,000 kg/1 m3) = 2,828.79 m^3/d

8.6 Interim Pumping Station

The interim pumping station shall transfer the primary effluent to the aeration tanks. It shall consist of wet well, pumps room suitable for 12 duty + 6 standby pumps each 1,590 m^3/hr at 14 m. The detailed calculation below shows the dimensions of the wet well (2 wet wells each 8x16M) and the head of the pumps (14 m).

Average Daily Flow	141,000	m^3/d	Station Type	Wet Well	
Peak Flow	9,540	m^3/hr	Wet Well Shape	Rectangular	
Invert Level	+2.70	m	Number of Duty Pumps	6	Standby 2
Finished Ground Level	+4.70	m	Maximum Number of starts per hour (Manufacturer)		10
Discharge Level	+7.70	m	Number of starts per hour (Design)		10
Wastewater Temperature	15.0	°C	Ambient Air Temperature	40 °C	

$$t = \frac{V}{Q_{av}} + \frac{V}{Q_{peak} - Q_{av}}$$

t = Time between start and stop of the pump, s = 360 s

V = Effective volume of the wet well, L

Q_{av} = Average Flow, L/s = 1,631.94 L/s

Q_{peak} = Peak Flow, L/s = 2,650.00 L/s (Pumping capacity)

$$\Rightarrow 360 = \frac{V}{1631.94} + \frac{V}{2{,}650.00 - 1{,}631.94} \quad \Rightarrow \quad V = 225{,}701 \text{ L} = 225.70 \text{ m}^3$$

Select Wet well Width = 8.0 m and Length = 16.0 m ==> Effective height = 1.763 m

Choose effective height = 1.80 m ==> Effective volume = 230,400 L = 230.40 m^3

LEVELS SUMMARY

Minimum Water Level (LLVL - Low Level Alarm) =	**1.05**	m	From Manufacturer's Data Sheet
Low Stop Level (LVL - All Pumps Stop) above LVLL =	**0.15**	m	If the pumping station has more than one duty level, the start and stop level of each pump shall be mentioned later
Effective Water Level (HLVL-Last Pump ON) - Above LVL =	**1.80**	m	
High Level Alarm (HHLVL) - Above HLVL till Invert Level =	**0.30**	m	
Distance between High Alarm and Invert Level =	**0.20**	m	
Distance from impeller centerline to base	0.50	m	From Manufacturer's Data Sheet
Available Submergence of pump impeller	0.70	m	Minimum Water Level above impeller
Total water depth in wet well below invert level >=	**3.50**	**m**	

Head Loss and System curve calculation

Head Loss ΔH is calculated by the following formulas: $\Delta H = J.L$ and $J = \frac{\lambda . V^2}{2gD}$

J = Head loss per unit length, m/m

L =Total equivalent length of pipe, m

D = Pipe inside diameter, m

V = Velocity through the pipe, m/s

g = Gravitational acceleration, 9.81 m/s^2

λ = Darcy Friction factor, dimensionless, calculated by the following Colebrook-White formula

$$\frac{1}{\sqrt{\lambda}} = -2\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right) \quad \Longrightarrow \quad \lambda = \frac{1}{4}\left[\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right)\right]^{-2}$$

ε = Pipe rugosity, m

$$Re = \frac{\rho V D}{\mu}$$ (Reynolds number)

ρ = Density of fluid, kg/m^3 999.2 kg/m^3

μ = Dynamic viscosity of fluid, kg/(m.s) 0.00114 kg/(m.s)

TSS = Solids Concentration = 0.05% = 0.50 Kg/m^3

==> Corrected Dary Friction Factor: $\lambda = \frac{1}{4}\left[\log_{10}\left(\frac{2.51}{Re.\sqrt{\lambda}} + \frac{1}{3.71}\frac{\varepsilon}{D}\right)\right]^{-2} x(26.\frac{TSS}{1000} + 1)$

Head Loss in Riser Pipe			Flow 1,590.0 m^3/hr				
Diameter	**Material**	**Rugosity ε, mm**	**V m/s**	**Length (m)**	**Reynolds Number**	**λ**	**Unit Head Loss J**
DN700	DI-New	0.5000	1.1	5.00	703,885.0	0.0188992	0.00181 m/m
		0.0000	-		-	-	0.00000 m/m
			-		-	-	0.00000 m/m
							ΔH = 0.01 m

Head Loss in Pumping Main			Flow 9,540.0 m^3/hr				
Diameter	**Material**	**Rugosity ε, mm**	**V m/s**	**Length (m)**	**Reynolds Number**	**λ**	**Unit Head Loss J**
DN1400	DI-New	0.5000	1.7	500.00	2,111,655.0	0.0160189	0.00173 m/m
		0.0000	-		-	-	0.00000 m/m
			-		-	-	0.00000 m/m
							ΔH = 0.86 m

Particular Head Loss in Fittings of Pumping Main

Item	**Size**		**QTY**	**K**	**ΔH = $KV^2/2g$**	**Item**	**Size**	**QTY**	**K**	**ΔH = $KV^2/2g$**
Increaser	250	400	1	0.33	0.02 m	Magnetic flow meter	DN1400	1	0.12	0.02 m
90° elbow, regular	DN700		2	0.30	0.04 m	Gate Valve-Full Open	DN1400	1	0.19	0.03 m
Gate Valve-Full Open	DN700		1	0.19	0.01 m	tee standard 90° turn	DN1400	1	1.80	0.27 m
Check Valve	DN700		1	3.50	0.23 m		DN1400		0.00	0.00 m
	DN700			0.00	0.00 m		DN1400		0.00	0.00 m
					0.00 m					0.00 m
					0.00 m					0.00 m
					0.31 m					**0.32 m**

TOTAL HEAD LOSSES in Discharge Piping (m) = 0.01 + 0.86 + 0.31+0.32 = 1.50 m

STATIC HEAD (m) = 7.70 - 2.70 + 3.50 - 1.05 = 7.30 m

SAFETY HEAD (m) = 1.00 m

TOTAL DYNAMIC HEAD (m) = 1.50 + 7.30 + 1.00 = 9.80 m

8.7 Biological Treatment

The detailed calculations below show the following:

- SRT of 12 d is a suitable value
- Aerobic tanks volume mentioned in the Feasibility study (284,000 m^3) is a conservative value v/s the calculated volume 229,457 m^3 (selected dimensions give 224,640 m^3) at MLSS of 3,000 mg/L.
- Anoxic tanks volume ~58,000 m^3 as per FS gives 4.9 h HRT which is oversized. The selected dimensions according to design give 46,080 m^3 volume (3.92 hours detention time), which is OK
- Anoxic tanks mixers total 24 x 18.5 kW are OK
- Recirculation ratio of 1.5 is OK

Waste Water Influent Data and ambient conditions

Average Daily Flow	282,000	m^3/d		Hourly Peak Flow		18,468	m3/hr	UBOD/BOD		
5-days BOD	252	mg/L		sBOD	113.4	mg/L	bCOD/BOD ratio		1.60	$\frac{(UBOD/BOD)}{(1-1.42f_d Y_H)}$
COD	504	mg/L		sCOD	201.6	mg/L		rbCOD	110.88	mg/L
TSS	225	mg/L		VSS	191.25	mg/L		Alkalinity	350	mg $CaCO_3$/L
TKN	70	mg/L		NH_4-N	42.0	mg/L		TN	70	mg/L
Water temperature, Winter		15 °C		Summer	30 °C			TP	17	mg/L
Ambient temperature, Winter		2 °C		Summer	35 °C		Site Elevation		4	m
Secondary Effluent			TP	17.0	mg/L	TN	30.0	mg/L	COD	125 mg/L
5-day BOD	25	mg/L	TSS_e	35	mg/L	NH_4-N	10.0	mg/L		
Tertiary Effluent			TP	4.0	mg/L	TN	2.0	mg/L	COD	50.0 mg/L
5-day BOD	10	mg/L	TSS_e	10	mg/L	NH_4-N	0.5	mg/L		

Design Conditions and Assumptions

1. Fine bubble membrane diffusers with an aeration clean water O_2 transfer efficiency 32%
2. Liquid depth for aeration basin (m) 5.00
3. Point of air release for the diffusers above tank bottom (m) 0.5
4. DO in aeration basin (mg/L) 2.0
5. Aeration Factor (α) 0.65
6. Relative DO saturation to clean water (β) 0.95
7. Diffuser Fouling Factor F 0.90
8. Total SRT (d) 12.0
9. Design MLSS X_{TSS} Concentration (g/m^3) 3,000
10. Peak to Average TKN loading rate ratio (SF) - Nitrification Safety Factor 1.20
11. Number of Aeration Tanks 2
12. Sludge density 1.02

Kinetic Coefficients for Activated Sludge - BOD removal and Nitrification at 20°C

	Coefficient	Unit	COD Oxidation	NH_4 Oxidation	NO_2 Oxidation
Max specific growth rate	μ_{max}	g VSS / g VSS.d	6.0	0.9	1.00
Half velocity constant	K_s, K_{NH4}, K_{NO2}	mg/L	8.0	0.5	0.20
Biomass Yield	Y_H	g VSS/g substrate oxidized	0.45	0.15	0.05
Specific endog. decay coeff	b_H	g VSS / g VSS.d	0.12	0.17	0.17
Endogeneous decay coeff	f_d	unitless	0.15	0.15	0.15
DO Half velocity constant	$K_{O,2}$	mg/L	0.2	0.5	0.90
θ Value	μ_{max}	unitless	1.07	1.072	1.063
	b_H	unitless	1.04	1.029	1.029
	K_s, K_{NH4}, K_{NO2}	unitless	1.0	1.0	1.0

Conventional Activated Sludge Process with Nitrification & Denitrification

Organic Matter Removal & Nitrification - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Develop the wastewater characteristics needed for the design

bCOD = 1.60(BOD) = 1.60(252 mg/L) = 403 mg/L (S_S and X_S)

nbCOD = COD - bCOD = (504-403) mg/L = 101 mg/L ($nbCOD=S_I+X_I$)

nbsCODe = sCOD - 1.6 sBOD = (202mg/L)-(1.6)(113mg/L) = 20 mg/L (S_I)

nbpCOD = TCOD - bCOD - nbsCODe = (504 - 403 - 20) mg / L = 81 mg/L (X_I)

$$VSS_{COD} = \frac{TCOD - sCOD}{VSS} = \frac{(504 - 202)\ mg/L}{191\ mg/L} = 1.58\ g\ COD/g\ VSS \quad (VSS)$$

$$nbVSS = \frac{nbpCOD}{VSS_{COD}} = \frac{80.64\ g\ COD/m3}{1.58\ g\ COD/g\ VSS} = 51.00\ g\ nbVSS/m3 \quad (X_{O,I})$$

iTSS = TSS - VSS = (225 - 191) mg/L = 33.8

Design Suspended Growth System for BOD removal and nitrification

Determine the specific growth rate μ_n for the ammonia oxidizing bacteria

$$\mu_{AOB} = \mu_{max,AOB}\left[\frac{S_{NH_4}}{S_{NH_4} + K_{NH_4}}\right]\left[\frac{S_{DO}}{S_{DO} + K_{O,AOB}}\right] - b_{AOB}$$

Where μ_{AOB} = specific growth rate of ammonia-oxidizing bacteria, g VSS/g VSS.d

$\mu_{max,AOB}$ = maximum specific growth rate of ammonia-oxidizing bacteria, g VSS/g VSS.d

b_{AOB} = specific endogeneous decay rate of ammonia-oxidizing bacteria, g VSS lost/g VSS.d

S_{NH4} = Effluent NH_4-N concentration, mg/L = 0.50 mg/L

K_{NH4} = half velocity coefficient for NH_4-N, mg/L

S_{DO} = DO concentration, mg/L = 2 mg/L

$K_{O,AOB}$ = half velocity coefficient for DO for AOB, mg/L = 0.50 mg/L

Select values for $\mu_{max,AOB}$, b_{AOB}, K_{NH4} and $K_{O,AOB}$ from the kinetic coefficients table at 20 °C

At 20°C:	μ_{max} = 0.9	θ = 1.072	b_H = 0.17	θ = 1.029
At 15.0 °C	$\mu_{m,T} = \mu_m . \theta^{(T-20)}$ =	0.6 g/g.d	$b_{H,T} = b_H . \theta^{(T-20)}$:	0.147 g/g.d
At 30.0 °C	$\mu_{m,T} = \mu_m . \theta^{(T-20)}$ =	1.8 g/g.d	$b_{H,T} = b_H . \theta^{(T-20)}$:	0.226 g/g.d

At 15 °C

$$\mu_{AOB} = \left\{\frac{(0.636\ g/g.d)(0.50\ g/m3)}{(0.50 + 0.50)\ g/m3}\right\}\left\{\frac{(2.0\ g/m3)}{(2.0 + 0.50)\ g/m3}\right\} - (0.147\ g/g.d)$$

$$= 0.1069\ g/g.d$$

At 30 °C

$$\mu_{AOB} = \left\{\frac{(1.804\ g/g.d)(0.50\ g/m3)}{(0.50 + 0.50)\ g/m3}\right\}\left\{\frac{(2.0\ g/m3)}{(2.0 + 0.50)\ g/m3}\right\} - (0.226\ g/g.d)$$

$$= 0.4953\ g/g.d$$

Determine the theoritical and design SRT

$$Theoritical\ SRT = \frac{1}{\mu_{AOB}}$$

Design SRT = (SF)(Theoritical SRT)

	At 15.0 °C	At 30.0 °C
SRT_T (d)	9.35	2.02
SRT_D (d)	11.22	2.42

==> SRT= 11.22 d

Chose SRT = 12.00 d

Conventional Activated Sludge Process with Nitrification & Denitrification

Organic Matter Removal & Nitrification - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Determine the biomass production

$$S = \frac{K_s[1 + b_H(SRT)]}{SRT(\mu_m - b_H) - 1}$$

At 20°C: μ_{max} = 6.0 g/g.d b_H = 0.12 g/g.d

At 15.0 °C

$\mu_{m,T} = \mu_m . \theta^{(T-20)}$ = 4.3 g/g.d (θ = 1.07)

$b_{H,T} = b_H . \theta^{(T-20)}$ = 0.099 g/g.d (θ = 1.04)

$$S = \frac{(8.0\ mg/L)[1 + (0.099\ g/g.d)(12\ d)]}{(12.00\ d)(4.3 - 0.099\ g/g.d) - 1} = 0.36\ g\ bCOD/m^3$$

S_e
33.96 g COD / m3

$$P_{X,bio} = \frac{QY_H(S_O - S)}{1 + b_H(SRT)} + \frac{(f_d)(b_H)QY_H(S_O - S)SRT}{1 + b_H(SRT)} + \frac{QY_a(NO_x)}{1 + b_{AOB}(SRT)}$$

$NO_x = TKN - N_e - 0.12P_{X,bio}/Q$

An iterative solution is required to find out NO_x and $P_{X,bio}$

Assume as a start NOx = **80%** of TKN = 0.80 * 70.00 = 56.0 mg/L

Calculated NOx = **57.4** mg/L

$$P_{X,bio,VSS} = \frac{(282,000\ m3/d)(0.45\ g/g.d)(403 - 0.36)\ g/m3](1\ kg/1,000\ g)}{[1 + (0.099\ g/g.d)(12.00\ d)]} +$$

$$\frac{(0.15\ g/g)(0.099\ g/g.d)(282,000\ m3/d)(0.45 g/g.d)(403 - 0.36)\ g/m3](12\ d)(1\ kg/1,000g)}{[1 + (0.099\ g/g.d)(12\ d)]}$$

$$+ \frac{(282,000\ m3/d)(0.15\ g/g)(57.4\ g/m3)(1kg/1,000\ g)}{[1 + (0.147\ g/g.d)(12\ d)]}$$

= 23,411.6 kg/d + 4,156.4 kg/d + 877.0 kg/d = 28,445.02 kg/d

At 30.0 °C

$\mu_{m,T} = \mu_m . \theta^{(T-20)}$ = 11.8 g/g.d $b_{H,T} = b_H . \theta^{(T-20)}$ = 0.178 g/g.d

Assume as a start NOx = **80%** of TKN = 0.80 * 70.00 = 56.0 mg/L

Calculated NOx = 60.0 mg/L

S = 0.18 g bCOD/m^3 $P_{X,bio,VSS}$ = 22,236.88 kg/d

S_e
33.78 g COD / m3

Determine the mass in term of VSS and TSS in the aeration basin

	At 15.0 °C	At 30.0 °C
Mass = $P_X(SRT)$		
$P_{X,VSS} = P_{X,bio}$ + Q(nbVSS)	= 28,445.02 kg/d + Q(nbVSS)	
	= 28,445.02 kg/d + 282000.m3/d)(51g/m3)(1 kg/1,000 g)	
	= 28,445.02 kg/d + 14382.0.00 kg/d) = **42,827.02 kg/d**	$P_{X,VSS}$ = **36,618.88 kg/d**
$P_{X,TSS}$ =	(28,445.02 kg/d)/0.85 + 14382. kg/d) + Q(TSS0 - VSS0)	
	= (33,464.73 kg/d) + 14382. kg/d) +(282,000m3/d)(34 mg/L)(1 kg/1,000g)	
	= **57,364.2 kg/d**	$P_{X,TSS}$ = **50,060.5 kg/d**
Q_w =	**9,561 m^3/d**	Q_w = **8,343 m^3/d**
Wasting rate	3.39%	2.96%

Calculate the mass of VSS and TSS in the aeration Basin

		At 15.0 °C	At 30.0 °C
$(X_{VSS})(V) = (P_{X,VSS})SRT$ =	(42,827.0 kg/d)(12 d) =	513,924.3 kg	439,426.6 kg
$(X_{TSS})(V) = (P_{T,VSS})SRT$ =	(57,364.2 kg/d)(12 d) =	688,370.8 kg	600,726.5 kg

Conventional Activated Sludge Process with Nitrification & Denitrification

Organic Matter Removal & Nitrification - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Based on selected design MLSS, determine the aeration tank volume and detention time

Design MLSS = 3,000 g/m^3

At 15.0 °C $(X_{TSS})(V)$ = 688,371 kg ==> $V = \frac{(688,370.79\ kg)(1000\ g/1\ kg)}{3,000\ g/m3}$ = 229,456.93 m3

No of aeration basins = 2 ==> V_{each} = 114,728.46 m3

Detention time $\tau = \frac{V}{Q} = \frac{(229,456.9 m3)(24\ h/d)}{(282,000\ m3/d)}$ = 19.5 h

At 30.0 °C V= 200,242.15 m3 τ = 17.0 h

V= 229,456.93 m3 ==> X= 2618 g/m3

Determine MLVSS

At 15.0 °C Fraction VSS = $\frac{513,924\ kg\ VSS}{688,371\ kg\ TSS}$ = 0.75 MLVSS =0.75(3,000)= 2,250 g/m3

At 30.0 °C Fraction VSS = ... 0.73 MLVSS = ... 1,915 g/m3

Determine F/M and BOD volumetric loading

At 15.0 °C $F/M = \frac{QS_0}{XV} = \frac{(282,000\ m3/d)(252\ g/m3)}{(2,250\ g/m3)(229,457\ m3)}$ = 0.1 g/g.d = 0.14 kg/kg.d

At 30.0 °C F/M = 0.2 g/g.d = 0.16 kg/kg.d

BOD Loading $= \frac{QS_0}{V} = \frac{(282,000\ m3/d)(252\ g/m3)}{(229,457\ m3)(1,000\ g/1\ kg)}$ = 0.31 kg/m^3.d

Determine the observed yield based on TSS and VSS

Observed Yield = g TSS / g bCOD = kg TSS / kg bCOD

At 15.0 °C $P_{X,TSS}$ = 57,364.23 kg/d

bCOD removed = $Q(S_0 - S)$ = (282,000 m3/d)[(403 - 0.4) g/m3](1 kg/1,000 g) = 113,602.2 kg/d

$Y_{obs,TSS} = \frac{(57,364.2\ kg/d)}{(113,602.2\ kg/d)} = \frac{0.50\ kg\ TSS}{kg\ bCOD} = \frac{0.50\ g\ TSS}{g\ bCOD}$

$= (\frac{0.50\ g\ TSS}{g\ bCOD})(\frac{1.60\ g\ bCOD}{g\ BOD})$ = 0.81 g TSS/g BOD

$Y_{obs,VSS} = (\frac{0.50\ g\ TSS}{g\ bCOD})(\frac{0.75\ g\ VSS}{g\ TSS})$ = 0.38 g VSS/g bCOD

$= (\frac{0.38\ g\ VSS}{g\ bCOD})(\frac{1.60\ g\ VSS}{g\ TSS})$ = 0.61 g VSS/g COD

At 30.0 °C

$P_{X,TSS}$ = 50,061 kg/d bCOD removed = ... 113,651.4 kg/d

$Y_{obs,TSS}$ = 0.70 g TSS/g BOD $Y_{obs,VSS}$ = 0.49 g VSS/g COD

Check Alkalinity

Alkalinity to maintain pH ~ 7 = Influent Alk - Alk used + Alk to be added

Influent Alkalinity: 350 g/m^3 as $CaCO_3$

Amount of nitrogen converted to nitrate: NO_x = 57.4 mg/L

Alkalinity used for nitrification = (7.14 g $CaCO_3$ / g NH_4-N)(57.4 mg/L)

= 411.53 g/m^3 used as $CaCO_3$

Conventional Activated Sludge Process with Nitrification & Denitrification

Organic Matter Removal & Nitrification - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Substitue known values and solve for alkalinity needed.

Residual Alkalinity concentration needed to maintain pH in the range 6.8-7.0: 70 g/m3 as CaCO3

70.00 = Influent Alkalinity - Alkalinity used + Alkalinity to be added

70.00 = 350 g/m3 - 412 g/m3 + alk to be added

Alkalinity to be added = 131.53 g/m3 as $CaCO_3$

= (282,000 m3/d)(131.5 g/m3)(1 kg/1,000g)

= 37,091 kg/d as $CaCO_3$

Determine the alkalinity needed as sodium bicarbonate

Sodium bicarbonate may be preferred over lime for alkalinity addition due to ease of handling and fewer scaling problems as compared to lime. The amount $NaHCO_3$ needed is as follows:

Equivalent weight of $NaHCO_3$ = 84 g/equivalent

$$NaHCO_3 \text{ added} = \frac{(37{,}091 \text{kg/d as CaCO3})(84 \text{g NaHCO3/eq})}{(50 \text{ g CaCO3/equivalent})}$$

= 62,312.43 kg/d NaHCO3

Estimate effluent BOD

$$BOD = sBOD_e + \left(\frac{0.85\ g\ BOD}{g\ VSS}\right)\left(\frac{0.85\ g\ VSS}{g\ TSS}\right)(TSS.\ g/m^3)$$

Assume $sBOD_e$ = 3.0 g/m³

TSS_e = 35 g/m³ (After secondary Treatment)

BOD = 3.0 g/m³ + (0.85)(0.85)(35 g/m3)

= 28.3 g/m³

Calculate the O_2 demand

At 15.0 °C $R_O = Q(S_O - S) - 1.42\,P_{X,bio} + 4.57(Q)\,NO_x$

$P_{X,bio,VSS}$ = 23,411.6 kg/d + 4,156.4 kg/d = 27,568.00 kg VSS/d

R_O =(282,000 m3/d)[(403.2 - 0.36) g/m3](1 kg/1,000 g) - 1.42(27,568.0 kg/d)

+ 4.57(282,000 m3/d)(57.4 g/m3)(1kg/1,000 g) = 148,423.79 kg O2/d **= 6,184.32 kg O2/h**

At 30.0 °C

$P_{X,bio,VSS}$ = = 21,553 kg VSS/d R_O = 160,418.44 kg O2/d **= 6,684.10 kg O2/h**

Conventional Activated Sludge Process with Nitrification & Denitrification

Denitrification Details – Calculation details

Denitrified Effluent Requirements

NH_4-N = 0.5 mg/L TN = 30.0 mg/L Required NO_3-N_e ≤ 20.0 mg/L

Design Assumptions

Nitrate concentration is RAS 6.0 g/m^3

RAS ratio = 0.67 Mixing Energy for Anoxic Reactor 7.5 $kW/10^3 m^3$

Determine the active Biomass concentration

$$X_b = \left[\frac{Q(SRT)}{V}\right]\left[\frac{Y_H(S_0 - S)}{1 + b_H(SRT)}\right]$$

Where $S_0 - S \approx S_0$

At 15.0 °C

$$X_b = \frac{(282{,}000\ m3/d)(12.00\ d)(0.45\ g\ VSS/g\ COD)(403 - 0.36\ g\ bCOD/m3)}{[1 + (0.099\ g/g.d)(12.00\ d)](229{,}457\ m3)} = 1{,}224.37\ g/m3$$

At 30.0 °C X_b = ... = 854.10 g/m3

Determine the IR ratio

	At 15.0 °C	At 30.0 °C
$IR = \frac{NO_X}{N_e}$ - 1.0 - R = $\frac{57.4\ g/m3}{20.0\ g/m3}$ - 1.0 -0.67 =	**1.20**	**1.34**

Choose IR = 1.50

Determine the amount of NO_3-N fed to the anoxic tank

Flowrate to anoxic tank = $IR\,(Q) + RQ$

= 1.20(282,000 m3/d) + 0.7(282,000 m3/d) = 527,280 | 611,000 m^3/d

NO_X feed = (527,280 m3/d)(6.0 g/m3) = 3,163,679 | 3,666,000 g/d

Determine the anoxic tank volume

At 20% of aerobic retention time, Anoxic τ = 0.20(19.5 h) = 3.9 h

As a first approximation, use a detention time of 4.00 h

==> τ = (4.00 h) / (24 h/d) = 0.167 d

V_{NOx} = τ x Q = 0.167 d(282,000 m3/d) = **47,000 m^3**

Determine F/M_b

At 15.0 °C

$$F/M_b = \frac{QS_0}{V_{NOx}(X_b)} = \frac{(282{,}000\ m3/d)(252\ g\ BOD/m3)}{(47{,}000\ m3)(1{,}224\ g/m3)} = 1.23\ g/g.d$$

At 30.0 °C F/M_b = ... = 1.77 g/g.d

Conventional Activated Sludge Process with Nitrification & Denitrification

Denitrification Details - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Determine the SDNR

Fraction of rbCOD = rbCOD/bCOD = (111 g/m3)/(403 g/m3)

= 0.28 = 28%

From the Table:

Select 30% rbCOD ==> b_0= 0.235 b_1 = 0.141

For $F/M_b > 0.50$, $SDNR_b = b_0 + b_1 [\ln(F/M_b)]$

For $F/M_b \leq 0.50$, $SDNR_b = 0.24(F/M_b)$

==> $SDNR_b$ = 0.235 + 0.141[ln(1.01)] = 0.2361 g NO_3-N/g MLVSS biomass.d

Percent rbCOS	SDNR Equation coefficients b_0	b_1
10%	0.186	0.078
20%	0.213	0.118
30%	0.235	0.141
40%	0.242	0.152
50%	0.270	0.162

Apply temperature correction factor θ = 1.026

$SDNR_{15}$ =0.236(1.026) (15 - 20)

At 15.0 °C	At 30.0 °C
0.208 g/g.d	0.305 g/g.d

Apply Internal Recycle Correction

For IR = 2 $SDNR_{adj} = SDNR_{IR1} - 0.0166\ln(F/M_b) - 0.078$

For IR = 3 - 4 $SDNR_{adj} = SDNR_{IR1} - 0.029\ln(F/M_b) - 0.012$

Where $SDNR_{adj}$ = SDNR adjusted for the effect pf internal recycle

$SDNR_{IR1}$ = SDNR value at internal recycle ratio = 1

F/M_b = BOD F/M ratio based on anoxic zone volume and active biomass concenttration g/g.d

IR =1.20 ==> $SDNR_{adj}$= 0.208 - 0.0166ln(1.01) - 0.078 = 0.130 g/g.d **At 15.0 °C**

IR =1.50 ==> $SDNR_{adj}$= 0.208 - 0.0166ln(1.01) - 0.078 = 0.221 g/g.d **At 30.0 °C**

Determine the Overall SDNR based on MLVSS

$SDNR = SDNR_b(MLVSS_b)/MLVSS)$

At 15 °C	At 30.0 °C	
0.07	0.10	g NO_3-N/g MLVSS.d

SDNR = 0.130 g/g.d[(1,224.4 g/m3)/(2,250 g/m3) =

Determine the amount of NO_3-N that can be reduced

Check NO_r based on τ = 4.90 h

$NO_r = (V_{NOx})(SDNR)(MLVSS, biomass)$ =

	At 15 °C	At 30 °C
= (57,575 m3)(0.130 g/g.d)(1,224.4 g/m3) =	9,133,218 g/d	10,873,816 g/d

Compared to Feed NO_x of 3,163,679 g/d < NO_r ==> OK **At 15 °C**

3,666,000 g/d < NO_r ==> OK **At 30 °C**

(If excess is greater than 20%, a smaller tank can be evaluated)

Conventional Activated Sludge Process with Nitrification & Denitrification

Denitrification Details - Calculation details

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)

Go to nitrification step and determine the net Oxygen required

At 15.0 °C R_o (without denitrification) = **6,184 kg O_2/h**

$$\text{Oxygen Credit} = \left(\frac{2.86\ g\ O_2}{g\ NO_3\text{-}N}\right)[(57.4 - 20.0\ g/m3)](282{,}000 m3/d)(1\ kg/1{,}000 g)$$

= 30,160 kg/d = 1,257 kg/h

Net O_2 required = R_o = (6,184.3 - 1,256.7) kg/h = **4,927.6 kg O_2/h**

At 30.0 °C R_o (without denitrification) = **6,684 kg O2/hr**

Oxygen Credit = 32,291 kg/d = 1,345.5 kg/h

Net O_2 required = R_o = **5,339 kg O_2/h**

Check alkalinity

Alkalininty to be added to maintain pH ~ 7 = Inlfuent Alk - Alk used + Alk produced

Influent alkalinity = 350 g/m3 as CaCO3

Alkalinity used = 7.14(57.4 g NO3-N/m3) = 409.81 g/m^3

Alkalinity produced = 3.57[(57.4 - 20.0 g/m3] = 133.50 g/m^3

Alkalinity needed to maintain neutral pH = 70.00 g/m^3 as $CaCO_3$

Alkalinity to be added = (70 - 350 + 409.8 - 133.5) g/m3 = -3.7 g/m^3 as $CaCO_3$

Mass of Alkalinity needed = (-3.7 g/m3)(282,000 m3/d)*(1 kg/1,000 g) = -1,042.6 kg/d as CaCO3

Compare to alkalinity need for nitrification only

Alkalinity needed for nitirification only = 37,091 kg/d as CaCO3

Alkalinity needed with denitrification = -1,043 kg/d as CaCO3

Alkalinity savings = 38,133 kg/d as CaCO3

Determine anoxic zone mixing energy

Mixing energy = 7.5 $kW/10^3\ m^3$

Volume = 57,575 m^3

Power = (57,575 m3) (7.5 kW/1,000 m3) = 431.8 kW

Number of installed mixer = 24

Power per mixer = 17.99 kW

Select Power = **18.5 kW**

8.8 Aeration Air Blowers

Detailed calculation for oxygen and air requirements shows 12 blowers each 14,732 m^3/hr can provide the required quantity of air. Considering a safety factor, we can use the numbers specified in the FS, i.e 16 duty + 2 standby of same capacity.

Standard Oxygen Requirement Calculation - Summer

Project	Upgrade of Ghadir Wastewater Treatment Plant 282,000 m3/d (year 2050)		
Client	Council for Development & Reconstruction (CDR)	**Date**	23-Sep-2024

Standard Temperature: 20°C (68 °F)		Standard Pressure: 1 atm [101.33 kPa (14.7 lb/in^2)]			
Field Water Temperature	30 °C	Site Elevation	4 m	Salinity	500 ppm
Dissolved Oxygen in Basin	2.0 mg/L	Diffusers Depth D_f	4.50 m	Air temp	35 °C
Oxygen Required *OTR*$_f$	**5,338.64** kg O_2/h				

$C^*_{s,t}$	$C^*_{s,20}$	τ	θ	α	β	P_b	P_s	Ω	d_e	F	$C^*_{\infty,20}$
7.54	9.06	0.83	1.024	0.65	0.95	1.00	1.00	1.00	0.4	0.90	10.69

$$OTR_f = (SOTR)\left[\frac{(\tau\beta\Omega C^*_{\infty,20} - C)}{C^*_{\infty,20}}\right][(\theta)^{t-20}](\alpha)(F)$$

OTR_f =Field oxygen transfer rate estimated for the system operationg under process conditions at an average DO concentration C, and temperature T, kg O_2/hr

$SOTR$ = Oxygen transfer rate under standard conditions (20°C, 1 atm, C=0 mg/L), kg O_2/hr

τ = Temperature correction factor = $C^*_{s,t} / C^*_{s,20}$

$C^*_{s,t}$ =Dissolved oxygen surface saturation concentration at operating temperature, mg/L

$C^*_{s,20}$ =Dissolved oxygen surface saturation concentration at standard temperature (20 °C), mg/L

β = Relative DO saturation to clean water, typically 0.95 to 0.98 = $C^*_{\infty(wastewater)} / C^*_{\infty(tap\ water)}$

C^*_{∞} = Steady-state DO saturation concentration obtained from nonlinear regression analysis of clean water test results (also available from aeration equipment manufacturers)

Ω = Pressure correction factor = P_b / P_s.

P_b =Barometric pressure at test site, m, kPa

P_s =Standard barometric pressure (1.0 atm, 10.33 m, 101.325 kPa)

$C^*_{\infty,20}$ =Saturated DO value at sea level and standard temperature (20°C) for diffused aeration, mg/L. It is higher than C_{st} as it is affected by oxygen transfer from bubbles under pressure in water column. The value if $C^*_{\infty,20}$ can be estimated using the following equation

$C^*_{\infty,20} = C^*_{s,20}[1+d_e(D_f/P_s)]$

d_e =Mid-depth correction factor; may vary from 0.25 - 0.45

D_f =Depth of diffusers in basin, m

C = Average dissolved oxygen concentation within the entire process water volume, mg/L

θ = Empirical temperature correction factor, typically 1.024

T = Field temperature, °C

T_s =Standard temperature, °C

α = Relative oxygen transfer rate in process water versus clean water = $K_La_{20(wastewater)} / K_La_{20(tap\ water)}$ (0.4 to 0.8 for diffused aeration, 0.6 to 1.2 for mechanical aeration)

F = Fouling factor, typically 0.65 to 0.9 (For surface aerators, F=1)

Standard Oxygen Requirements $SOTR = OTR_f / \left\{\left[\frac{(\tau\beta\Omega C^*_{\infty,20} - C)}{C^*_{\infty,20}}\right][(\theta)^{t-20}](\alpha)(F)\right\}$ = 11,933 **kg O_2/hr**

Standard Temperature: 20°C (68 °F)		Standard Pressure: 1 atm [101.33 kPa (14.7 lb/in^2)]	
% of O_2 in air by volume	20.95%	% of O_2 in air by weight	23.20%
Field Air Temperature	35 °C	**Site Elevation** 4 m	**Site Humidity** 50%
Standard Air Temperature	20 °C	Normal Air Temp 0 °C	Std Humidity 36%
Site Pressure	**101,285** Pa	Standard Oxygen Requirements *SOTR*	11,933 kg O_2/h
Diffusers Efficiency	32%	**Blower S.Factor** 1.20	**Diffusers Depth** 4.50 m

$$\rho_{air} = \frac{p}{R.T}$$

ρ_{air} = Density of dry air, kg/m^3
p = Absolute pressure, Pa
T = Absolute temperature, °K
R = Specific gas constant for dry air, 287.058 J/(kg·°K)

	Normal Conditions (20°C) ISO 1217 PD Blowers	Normal Conditions (0°C) ISO5389 Turbo compressor	Site Conditions	
ρ_{air} =	1.2041 kg/m^3	1.2923 kg/m^3	1.1450 kg/m^3	
V_{air} =	160,176 Nm^3/hr	149,248 Nm^3/hr	**168,446** m^3/h	99,046 ACFM
Number of duty blowers	**12**	Capacity of each	**14,037** m^3/h =	8,254 ACFM
Header velocities(m/s) V_{min} / V_{max}:	18 / 33	**Select Ø** 1500 mm	V(m/s)=	**26.5**
Blowers outlet (m/s) V_{min} / V_{max}:	13 / 20	**Select Ø** 500 mm	V(m/s)=	19.9
Expected discharge temperature	76.12 °C	**Absorbed Power per Blower**		**kW**

Pipe Ø (m)		*0.500*	1.500
	C	**QTY**	**Qty**
Angle Valve	0.9	0	0
Globe Valve	2	0	0
Gate Valve	0.25	1	0
Tee, Side outlet	1.33	1	1
Standard Elbow	0.67	0	3
Run Reduced Tee	0.67	0	0
Long Radius Elb	0.33	0	0
Run of Std Tee	0.33	0	0
Straight pipe run (m)		0.5	50
Total Equivalent pipe (m)		38.60	351
Head Loss (m)		0.856	279.4

Special Losses	**(mm/m)**	**QTY**
Air Filter	50	1
Silencer		
Centrifugal Blower	30	0
PD Blower	180	1
Check Valve	170	1
Special Head Loss (m)		0.40

Total head Loss (m)	**280.67**
Total blower head (m)	**313.24**
1 Blower Capacity (m^3/hr)	**14,037**
Number of duty Blower	**12**
Expected motor, each(kw)	323.72

8.9 Secondary Sedimentation Tanks (SST)

Wastewater Daily Flow	282,000	m^3/d	Design MLSS X_{TSS} Concentration	3,000	mg/L
Peak Hourly Flow	18,468.0	m^3/h	Assume Return Sludge Recycle Ratio X_R =	7,500	mg/L
Overflow Rate at Average Flow OFR_{avg}	20	$m^3/m^2.d$	Solids Concentation	0.74%	
Overflow Rate at Peak Flow OFR_p	30	$m^3/m^2.d$	Solids Loading Rate at Average	5.00	$kg/m^2.d$
Weir Loading Rate	250	$m^3/m.d$	Solids Loading Rate at Peak	8.00	$kg/m^2.d$
Side Water Depth	4.50	m	Number of Clarifiers	12.00	
			Average Flow per clarifier	23,500.0	m^3/d

Define return sludge recycle ratio

$Q_R X_R = (Q + Q_R)X$ = (assume waste sludge mass is insignificant)

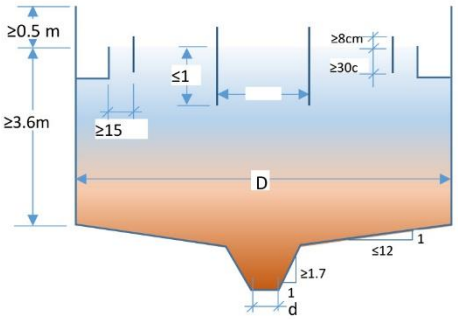
Q_R = *RAS* flowrate, m^3/d

X_R = Return sludge mass concentration, g/m^3

RAS recycle ratio = $Q_R/Q = R$

$RX_R = (1 + R)X$

$$R = \frac{X}{X_R} = \frac{(3{,}000\ g/m3)}{[(7{,}500 - 3{,}000)\ g/m3]} = 0.67$$



Area of each clarifier = Flow/OFR	At Average Flow	At Peak Flow	Required Area
	1175.00 m2	1231.20 m2	1231.20 m2

Clarifier's diameter >=	39.60	m	Select	40.00	m ==> Area=	1,256	m^2 each
Clarifier's Volume =	5,652.00	m^3 each	==>	67,824	m^3 total		
Detention Time =	5.77	hr at average flow	or	3.67	hr at peak flow		

Check Solids Loading Rate

$$\text{Solids Loading at Average Flow} = \frac{(Q + QR)(MLSS)}{A} = \frac{(1 + R)Q(MLSS)}{A}$$

$$= \frac{(1 + 0.67)(23{,}500.0m3/d)(3{,}000g/m3)(1\ kg/1{,}000\ g)}{(1{,}231.20\ m2)(24\ h/d)} = 3.90\ kg\ MLSS/m^2.h$$

$$\text{Solids Loading at Average Flow} = \frac{(1 + 0.67)(18{,}468.0m3/h)(3{,}000g/m3)(1\ kg/1{,}000\ g)}{(12*1{,}231.20\ m2)} = 6.25\ kg\ MLSS/m^2.h$$

Check Weir Loading Rate

Weir Type
V-Notch 90 deg.

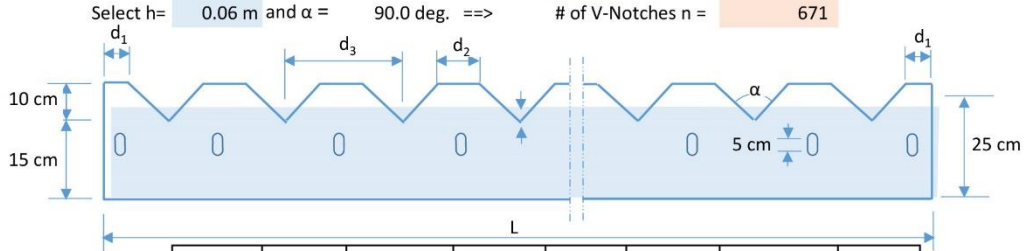
Assume Weir Diameter = 39.00 m ==>

Weir Length = 122.46 m for each clarifier

Weir Loading at Average Daily Flow = 191.90 $m^3/m.d$

Weir Loading at Peak Flow = 150.81 $m^3/m.h$ = 1,206.5 $m^3/m.d$

Select h= 0.06 m and α = 90.0 deg. ==> # of V-Notches n = 671



Final Selection:

L (m)	n	α (°)	d_1 (m)	d_2 (m)	d_3 (m)	h_{av} (m)	h_{peak} (m)
122.46	312	90.00	14.430	0.100	0.300	0.05	0.06

8.10 Primary Sludge Gravity Thickener and screening

8.10.1 Primary Sludge Thickener

The primary sludge from the lamella Primary Settling Tanks (PSTs) will be pumped to the gravity thickeners, where the sludge will be settled to thicken to a dry solids content of ~6%. Three (3) gravity thickeners are designed based on a solids loading rate (SLR) of up to 100 kg DS/m^2/day, which results in an effective surface area of ~255 m^2 (at a diameter of 18.0 m) for each.

Each thickener will be equipped with a rotating bridge to allow the collection and removal of scum and thickened sludge from its bottom. Additionally, it will be equipped with 2 thickened sludge pumps (one as standby) each with capacity of 30 m^3/hr at 6.0 m head and with 2 more pumps (one as standby) for the pumping of supernatant liquor back to the entrance of the PSTs, each rated for 30m^3/hr at 5.0 m head.

The duration pumps operation is around 7 hrs/day.

		Phase 1, Current			Phase 3, Ultimate		
Description	**Unit**	**Average**	**Summer**	**Winter**	**Average**	**Summer**	**Winter**
4.1 Primary Sludge transfer pumps to thickeners							
Number of duty pumps during cleaning		11	11	11	11	11	11
Number of duty pumps - All PST running		12	12	12	12	12	12
Number of standby pumps		6	6	6	6	6	6
Select pump capacity	m^3/h	30.00	30.00	30.00	30.00	30.00	30.00
Pump Head	m	6.00	6.00	6.00	6.00	6.00	6.00
Running hours per day for the pumps	h	3.89	3.89	5.49	8.01	8.18	9.10
4.2 Primary Sludge Gravity Thickener Design							
Number of PS gravity thickeners		2	2	2	3	3	3
Primary Sludge Flow per thickener	m^3/d	641.51	641.51	905.98	881.57	899.27	1,000.58
Primary Sludge Peak Flow	m^3/h	30.00	30.00	30.00	30.00	30.00	30.00
Primary Sludge Solids Concentration	%	3.00%	3.00%	3.00%	3.00%	3.00%	3.00%
Primary Sludge Solids Specific Garvity		1.03	1.03	1.03	1.03	1.03	1.03
Primary Sludge Peak Solids Concentration	%	3.40%	3.40%	3.40%	3.40%	3.40%	3.40%
Secondary Sludge Flow	m^3/d	-	-	-	-	-	-
Secondary Sludge Peak Flow	m^3/d	-	-	-	-	-	-
Secondary Sludge Solids Concentration	%	0.20%	0.20%	0.20%	0.20%	0.20%	0.20%
Secondary Sludge Peak Solids Concentration	%	0.23%	0.23%	0.23%	0.23%	0.23%	0.23%
Secondary Sludge Solids Specific Garvity		1.005	1.005	1.005	1.005	1.005	1.005
Primary Sludge Dry Solids	kg/d	4,085	4,085	5,769	8,420	8,589	9,556
Secondary Sludge Dry Solids	kg/d	-	-	-	-	-	-
Combined Sludge mass	kg/d	4,085	4,085	5,769	8,420	8,589	9,556
Combined Sludge flowrate	m^3/d	117	117	165	240	245	273
Combined Sludge specific gravity		1.02	1.02	1.02	1.02	1.02	1.02
Combined Sludge Solids Concentration	%	3.43%	3.43%	3.43%	3.43%	3.43%	3.43%
Select Design Solids Loading Rate	$kg/m^2.d$	100.0	100.0	100.0	100.0	100.0	100.0
Surface Area based on Solids Loading Rate	m^2	980.3	980.3	1,384.5	2,020.7	2,061.3	2,293.5
Hydraulic Loading Rate	$m^3/m^2.d$	2.86	2.86	2.86	2.86	2.86	2.86
Number of Thickeners		1	1	1	1	1	1
Diameter of each Thickener	m	35.33	35.33	41.99	50.72	51.23	54.04
Select Diameter	m	18.00	18.00	18.00	18.00	18.00	18.00

8.10.2 Sludge Screening

Number of screens : Three (3) duty

Screenings opening : 3mm

Capacity : 60 m^3/hr (based on 7 hr/day operation)

8.11 Mechanical Thickening of Secondary Sludge

Type : Closed drum thickener

Quantity : Six (6)

Capacity : 90 m^3/hr

8.12 Anaerobic Digestion

Both thickened primary and secondary sludge will be mixed in the buffer tanks prior to anaerobic digestion. Two buffer tanks will be constructed, each with an effective volume of 302 m^3 (diameter: 8.0 m, height: 6.0 m) to allow a sludge retention time of 6 hours prior to the anaerobic digestion.

The mixed sludge will then be introduced to the anaerobic digesters. The digesters will operate with a sludge retention time of 15 days at a temperature of 35 °C, in order to stabilize the sludge and transform any volatile compounds into biogas. Accordingly, Four (4) anaerobic digesters will be constructed each with an effective volume of 8,675 m^3 (diameter: 23.5 m, height: 20.0 m).

Each digester will be equipped with 1 feeding pump (+1 standby) rated for 24 m^3/hr at 22.0 m head, with a mixing system with minimum power input of 30 kW and with 1 recirculation pump (+ 1 standby) rated for 48 m^3 /hr at 20.0 m head.

After the digestion tanks the sludge will be collected in the buffer tanks prior to the dewatering process.

During Phase I, the biogas produced at the digesters will be collected by the use of 4 (+2 standby) biogas blowers rated for 250 m^3/hr at 5.0 m head and stored at a spherical tank of 4,500 m^3 effective volume. This tank is designed to allow biogas storage for ~2 hours. In Phase II, three additional blowers (1 as standby) will be installed to cover the increase in biogas produced.

pumps and blowers will be located within a building with dimensions LxWxH = 15.0 m x 10.0 m x 6.0 m.

		Phase 1, Current			Phase 3, Ultimate		
Description	**Unit**	**Average**	**Summer**	**Winter**	**Average**	**Summer**	**Winter**
Anaerobic Sludge Digestion							
Type: Singel stage, high-rate							
Reactor Hydraulic Regime: Complete Mix							
Number of Digesters		2	2	2	4	4	4
Primary Sludge daily mass	kg/d	8,169.3	8,169.3	11,537.3	25,259.3	25,766.6	28,669.2
Primary Sludge daily Flow	m^3/d	1,283.0	1,283.0	1,812.0	2,644.7	2,697.8	3,001.7
Primary Sludge BOD_5	mg/L	213	208	223	21	203	218
Primary Sludge TSS	mg/L	167	160	179	161	155	173
Primary Sludge Solids Content	%	5.0%	5.0%	5.0%	5.0%	5.0%	5.0%
Primmary Sludge Specific Gravity		1.02	1.02	1.02	1.02	1.02	1.02
Operating Temperature	°C	35.0	35.0	35.0	35.0	35.0	35.0
Solids Retention Time (SRT)	d	15	15	15	15	15	15
Efficiency of waste utilization (solids conversion)		0.70	0.70	0.70	0.70	0.70	0.70
Theoretical Yield Y at operating temperature	kg VSS/kg bCOD	0.08	0.08	0.08	0.08	0.08	0.08
b at operating temperature	d^{-1}	0.03	0.03	0.03	0.03	0.03	0.03
Methane content in digester gas	%	65.0%	65.0%	65.0%	65.0%	65.0%	65.0%
Sludge Mass	kg/d	23,355	22,405	36,935	45,416	43,775	57,712
Sludge Volume	m^3/d	457.9	439.3	724.2	890.5	858.3	1,131.6
bCOD loading	kg bCOD/d	29,861	29,085	46,001	5,886	57,494	72,857
Digesters Volume	m^3	6,869	6,590	10,863	13,358	12,875	16,974
Digester Volume (each)	m^3	3,435	3,295	5,432	3,339	3,219	4,244
Volumetric Loading	$kg/m^3.d$	4.35	4.41	4.23	0.44	4.47	4.29
VSS produced per day	kg VSS/d	1,153.24	1,123.30	1,776.59	227.33	2,220.45	2,813.77
Methane conversion factor at design temp		0.40	0.40	0.40	0.40	0.40	0.40
Volume of Methane produced per day	m^3/d	7,706	7,506	11,871	1,519	14,837	18,802
Total gas production	m^3/d	11,855	11,548	18,263	2,337	22,826	28,926

8.13 Sludge Dewatering

Type : Centrifuge

Phase I:

Quantity : 3 duty, 1 standby for Phase I,

Capacity : 60 m^3/hr

Sludge feed pumps:

Quantity : 1 duty, 1 standby per centrifuge

Capacity : 50 m^3/hr at 5 m head

Polyelectrolyte dosing pumps:

Quantity : 1 duty, 1 standby per centrifuge

Capacity : 0 -20 L/hr at 5 m head

Dewatered Sludge pumps:

Quantity : 1 duty, 1 standby per centrifuge

Capacity : 8 m^3/hr at 12 m head

Phase II:

Quantity :2duty (with complete ancillaries as above)

Supernatant Liquor Pumps

Quantity : 1 duty, 1 standby

Type : submersible, centrifugal

Capacity : 57 m^3/hr at 6 m head

Sludge Storage Silos:

Quantity : Two (2)

Diameter : 12 m

Height : 8 m

Volume each: ~800 m^3

8.14 Biogas Utilization

The biogas produced from the anaerobic process will be utilized by Combined Heat-Power (CHP) engines, in order to produce electricity, which will cover significant part of the plant's power and heating needs. For this purpose, biogas from the spherical storage tank will be fed to 3 CHP engines (2 in Phase I and one in Phase II), each rated at an electrical output power of 888 kW.

In addition to the CHP engines, a gas flaring station will be installed for safety reasons (Phase I), in order to burn the biogas in situations when the CHP engines are out of operation (malfunction, maintenance, etc.). During Phase II, an additional gas flaring station will be installed. Moreover, for the initiation of the anaerobic digestion process, auxiliary boilers will be used to heat the sludge.

8.15 Electricity supply

8.15.1 Existing WWTP electricity supply

The wastewater treatment plant is supplied from the EDL power grid via a loop connection on a 20kV medium-voltage line.

A transformer substation equipped with medium-voltage cells and two 20kV/400V transformers, each with a power rating of 1,000kVA.

Each transformer supplies a Main Distribution Board (MDB) installed in the transformer substation building.

Transformer TR1 supply the MDB/S1 and transformer TR2 supply the MDB/S2.

Each MDB is also powered in normal/emergency mode by three diesel generators connected in parallel on a common bus bar panel (TBB).

From these Main Distribution Boards, various MCC panels and sub-panel board (SPB) are supplied, divided into the following main process phases (Refer to Appendices 8.2 & 8.7):

- MDB-S1: Main Distribution Board feeding the following:
  - MCC-LS (1) and MCC-LS (2): Lift Station (Screw Pumps).
  - MCC-BS: Bar Screens, Belt Conveyor and Screw Compactor
  - MCC-GS: Degritting Area. (Degritting Bridges, Grit Pumps, Cable Reels)
  - MCC-BR: Blower Room. (Air Blowers)
  - MCC-MPS: Main Pumping Station.
  - MCC-FP: Fuel Pumps.
  - SPB LS: For Lift Station.
  - SPB E
  - SPB BR: For Blower Room.
  - SPB GH: For Guard House.
  - SPB MP: For Pumping Station.
  - SPB LS: For Lift Station.
- MDB-S2: Main Distribution Board feeding the following:
  - VSD-01, VSD-02, VSD-03: Variable Speed Drives for Main Outfall Pumps.

All MCCs are in good condition and will be kept, except for the MCC-GS (Degritting Area), which will have to be replaced (Refer to Appendix 8.2).

8.15.2 Phase I Estimated power balances

8.15.2.1 Power balance calculation for the existing WWTP (Refer to Appendix 8.1)

Calculating the power balance of the existing station gives a useful power of around 820kW (1,025kVA).

8.15.2.2 Power balance estimation for the extension of the existing WWTP (Refer to Appendix 8.1)

Electrical power requirements for the extension of the existing plant are estimated at 1,560kW (1,950kVA).

As the transformers installed in the transformer building have a rated unit power of 1,000kVA, they will not be able to meet the needs of the extension.

8.15.2.3 Power balance estimation for the expansion site of the WWTP (Refer to Appendix 8.1)

Electrical power requirements for the expansion of the plant (new site) are estimated at 2,640kW (3,300kVA).

Some equipment planned for Phase II is included in this Phase I power balance. These are the future facilities planned for the common area between Phase I and Phase II.

8.15.3 Phase I Proposed electrical distribution architecture

8.15.3.1 Existing WWTP Extension

As the current electrical installation is unable to meet the new requirements, we propose the following modifications:

- Retention of most existing electrical distribution equipment.
- Creation of a new transformer substation to supply the extension-related equipment, comprising two transformers installed in normal/standby mode and two diesel generators.
- Replacement of the 20kV medium-voltage switchgear to enable it to be extended to supply the two new transformers.
- Installation of a new Main Distribution Board (MDB/S3).
- Installation of a reactive energy compensation cabinet.
- Installation of a new MCCs panels (Lifting Station, Grit Removal, etc.).
- Installation of a new Sub-panel Board (SPB) in each new building (Administration building, relocated Blower building, new transformer building, Workshop & storage building, etc.)

8.15.3.2 WWTP expansion site

As the plant expansion site is a new one, we propose the creation of a transformer substation comprising the following main equipment:

- Installation of a 20kV medium-voltage distribution cabinet for loop connection to the 20kV network.
- Installation of three transformers installed in normal/standby mode (2 in duty and 1 in standby).
- Installation of a Main Distribution Board.
- Installation of a reactive energy compensation cabinet.
- Installation of MCCs panels (Sludge Dewatering, Pumps blowers for digesters, CHP Unit, etc.).
- Installation of two diesel generators.

As the sludge treatment process includes cogeneration, the plant will be able to produce biogas, which will be used to generate electricity that will be injected into the plant's low-voltage network at the low-voltage switchboard.

The following main equipment is required:

- Installation of Gaz generators (2 CHP engines will be used, each rated at 888kW).
- Installation of synchronization cabinets.

8.15.4 Phase I Proposed distribution of electrical power

8.15.4.1 Existing WWTP Extension

The power supply to the process equipment and its sizing determined according to the following proposed breakdown (Refer to Appendices 8.3 & 8.7):

- MCC-LS2 (Lifting Station)
  - Lifting Screw Pumps (No 7).
  - Grease Pumps (No 7).
- MCC-BS2 (Bar Screens)
  - Fine Screens (No 10).
  - Screenings Conveyor (No 1).
- MCC-GS (Degritting Area)
  - Grit Classifier (No 1).
  - Screw Conveyor (No 1).
  - Degritting Bridge (No 1).
  - Grit Pumps (No 2).
- MCC-BR (Blower Room)
  - Air Blowers (No 3).
- MCC-MPS2 (Main Pumping Station)
  - Pumps Suction Motorized Gate Valve (No 1).
  - Pumps Discharge Motorized Gate Valve (No 1).
  - Main Outfall Pump (No 1).
  - Grease Pump (No 1).
- MCC-FP2 (Fuel Pump)
  - Fuel Pumps (No 3).
- MCC-PT1 (Primary Treatment)
  - PST Scrapers (No 6).
  - Primary Sludge Pumps (No 8).
  - PST Effluent Pumps (No 9).
- MCC-PT2 (Primary Treatment)
  - PST Scrapers (No 6).
  - Primary Sludge Pumps (No 8).
  - PST Effluent Pumps (No 9).
- MCC-OW (Odor Control Unit & Wash water Booster Set)
  - Odor Control Units (No 2).
  - Wash water Booster Sets (No 2).

8.15.4.2 WWTP expansion site

The power supply to the process equipment and its sizing determined according to the following proposed breakdown (Refer to Appendices 8.4 & 8.7):

- MCC-OW (Odor Control Unit & Wash water Booster Set)
  - Odor Control Unit (No 2).
  - Wash water Booster Set (No 2).
- MCC-SD1 (Sludge Dewatering)
  - Centrifuges (No 2).
  - Feed Pumps (No 2).
  - Polyelectrolyte Dosing Pumps (No 2).
  - Dewatering Sludge Pumps (No 2).
  - Supernatant Liquor Pumps (No 2).
  - Crane (No 1).
- MCC-SD2 (Sludge Dewatering)
  - Centrifuges (No 2).
  - Feed Pumps (No 2).
  - Polyelectrolyte Dosing Pumps (No 2).
  - Dewatering Sludge Pumps (No 2).
  - Supernatant Liquor Pumps (No 2).
- MCC-SD3 (Sludge Dewatering)
  - Centrifuges (No 2).
  - Feed Pumps (No 2).
  - Polyelectrolyte Dosing Pumps (No 2).
  - Dewatering Sludge Pumps (No 2).
  - Supernatant Liquor Pumps (No 2).
- MCC-DS1 (Digester Sludge Buffer Tank)
  - Aeration Blowers (No 3).
- MCC-DS2 (Digester Sludge Buffer Tank)
  - Aeration Blowers (No 3).
- MCC-GT (Gravity Thickener Pumping Station)
  - Sludge Scrapers (No 3).
  - Thickened Sludge Pumps (No 6).
  - Supernatant Pumps (No 6).
  - Sludge Screen (No 3).
- MCC-AD1 (Anaerobic Digestion)
  - Feeding Pumps (No 4).
  - Mixing Systems (No 2).
  - Recirculation Pumps (No 4).

- Biogas Blowers (No 6).

- MCC-AD2 (Anaerobic Digestion)
  - Feeding Pumps (No 4).
  - Mixing Systems (No 2).
  - Recirculation Pumps (No 4).
  - Biogas Blowers (No 3).
- MCC-MT (Mechanical Thickening Secondary)
  - Feed Pumps (No 6).
  - Polyelectrolyte Dosing Pumps (No 6).
  - Supernatant Return Pumps (No 3).
  - Polyelectrolyte Preparation System Mixers (No 1).

8.15.5 Phase I Electrical work description

8.15.5.1 Existing WWTP Extension

The following electrical work is to be carried out as part of the project (Refer to Appendix 8.3):

- Supply and installation of a new 20kV medium-voltage panels in replacement of the existing 20kV medium-voltage panels, comprising two incoming cells and four transformer protection cells, including rework of existing cable connections.
- Supply and installation of 20kV medium-voltage cables for the connection between the new 20kV medium-voltage panels and each new transformer.
- Supply and installation of two ONAN-type transformers, each with a power rating of 2,500kVA.
- Supply and installation of two prime power (PRP) diesel generator a power rating of 1,000kVA and canopy type.
- Supply and installation of a new Main Distribution Board.
- Supply and installation of low-voltage cables for the connection between the new transformers, the diesel generators, and the new Main Distribution Board, including connections.
- Supply and installation of a reactive energy compensation cabinet connected to the new Main Distribution Board.
- Supply and installation of a new MCC-GS panel for the Degritting area in replacement of the existing Degritting area MCC-GS panel, comprising the protections for the existing and the news associated equipment.
- Supply and installation of a new MCC-LS(1) and MCC-LS(2) panels for the lifting station.
- Supply and installation of low-voltage cables for the connection between the new Main Distribution Board and the news MCC Panels, including connections.
- Removal and relocation of Blower room MCC-BR, including rewiring and reconnections.
- Supply and installation of a new Sub-Panel Board (SPB) in each new building (Administration building, relocated Blower building, new transformer building, Workshop & storage building, etc.)
- Supply and installation of low-voltage cables for the connection between the new Main Distribution Board and the news SPB Panels, including connections.

8.15.5.2 Phase I Expansion WWTP (New site)

The following electrical work is to be carried out as part of the project:

- Supply and installation of a 20kV medium-voltage panels, comprising two incoming cells and three transformer protection cells.
- Supply and installation of 20kV medium-voltage cables for the connection between the EDL room and the 20kV medium-voltage panels.
- Supply and installation of 20kV medium-voltage cables for the connection between the new 20kV medium-voltage panels and each transformer.
- Supply and installation of three ONAN-type transformers, each with a power rating of 2,000kVA.
- Supply and installation of two prime power (PRP) diesel generators with a unit power of 1,250kVA and canopy type.
- Supply and installation of Main Distribution Boards.
- Supply and installation of low-voltage cables for the connection between the transformers, the diesel generators, the gas generators, and the Main Distribution Boards, including connections.
- Supply and installation of a reactive energy compensation cabinet connected to the Main Distribution Boards.
- Supply and installation of a MCC panels at minima for Sludge Dewatering, Pumps blowers for digesters and CHP Unit.
- Supply and installation of low-voltage cables for the connection between the Main Distribution Boards and the MCC Panels, including connections.
- Supply and installation of a synchronization cabinet for supply of the Main Distribution Boards from the parallel diesel generators, including cables, cabling, and connections.
- Supply and installation of a synchronization cabinet for parallel supply of the Main Distribution Boards from the electrical network and the gas generators, including cables, cabling, and connections.
- Supply and installation of low-voltage cables for the connection between the MCC Panels and equipment, including connections.
- Supply and installation of a Sub-Panel Board (SPB) for each building.
- Supply and installation of low-voltage cables for the connection between the Sub-Panel Boards and equipment, including connections.
- Supply and installation of outdoor lighting, including cables, cabling, and connections.
- Supply and installation of lightning protection system.
- Supply and installation of earthing system.

8.15.6 Phase II WWTP expansion site Estimated power balances

8.15.6.1 Power balance estimation for the phase II of expansion site of the WWTP (Refer to Appendix 8.1)

Electrical power requirements for the phase II expansion of the plant (new site) are estimated at 5,085kW (6,355kVA).

8.15.7 Phase II Proposed electrical distribution architecture

As the plant expansion site, we propose for the phase II the creation of a second transformer substation located near the blowers building comprising the following main equipment:

- Installation of a 20kV medium-voltage distribution cabinet for loop connection to the 20kV network.
- Installation of six transformers installed in normal/standby mode (two time 2 in duty and 1 in standby).
- Installation of Main Distribution Boards.
- Installation of a reactive energy compensation cabinet.
- Installation of MCCs panels (RAS/SAS pumping station, Secondary sludge thickening building, etc.).
- Installation of four diesel generators.

As the sludge treatment process includes cogeneration, the plant will be able to produce biogas, which will be used to generate electricity that will be injected into the plant's low-voltage network at the Main Distribution Boards.

The following main equipment is required:

- Installation of Gaz generators (1 CHP engine will be used, each rated at 888kW).
- Installation of synchronization cabinets.
- The gas generator must be connected to the MDB/S1 installed in phase I.

8.15.8 Phase II Proposed distribution of electrical power

The power supply to the process equipment and its sizing determined according to the following proposed breakdown (Refer to Appendices 8.5, 8.6 & 8.7):

- MCC-ATM (Anoxic Tank Mixers)
  - Anoxic Tank Mixers (No 24).
- MCC-IRP1 (IR Pumps)
  - IR Pumps (No 18).
- MCC-IRP2 (IR Pumps)
  - IR Pumps (No 18).
- MCC-RS1 (RAS/SAS Pumps)
  - FST Scraper (No 6).
  - RAS Pumps (No 9).
  - SAS Pumps (No 6).
- MCC-RS2 (RAS/SAS Pumps)
  - FST Scraper (No 6).
  - RAS Pumps (No 9).
  - SAS Pumps (No 6).
- MCC-DD (Disinfection & Dechlorination)
  - NaOCl Dosing Pumps (No 6).
  - $NaHSO_3$ Dosing Pumps (No 6).

8.15.9 Phase II Electrical work description

The following electrical work is to be carried out as part of the project:

- Supply and installation of a 20kV medium-voltage panels, comprising two incoming cells and six transformer protection cells.
- Supply and installation of 20kV medium-voltage cables for the connection between the EDL room and the 20kV medium-voltage panels.
- Supply and installation of 20kV medium-voltage cables for the connection between the new 20kV medium-voltage panels and each transformer.
- Supply and installation of six ONAN-type transformers, each with a power rating of 2,500kVA.
- Supply and installation of four prime power (PRP) diesel generators with a unit power of 2,000kVA and containerized type.
- Supply and installation of Main Distribution Boards.
- Supply and installation of low-voltage cables for the connection between the transformers, the diesel generators, the gas generators, and the Main Distribution Boards, including connections.
- Supply and installation of a reactive energy compensation cabinet connected to the Main Distribution Boards.
- Supply and installation of a MCC panels at minima for RAS/SAS pumping station, Secondary sludge thickening building, etc.
- Supply and installation of low-voltage cables for the connection between the Main Distribution Boards and the MCC Panels, including connections.
- Supply and installation of a synchronization cabinet for supply of the Main Distribution Boards from the parallel diesel generators, including cables, cabling, and connections.
- Supply and installation of a synchronization cabinet for parallel supply of the Main Distribution Boards from the electrical network and the gas generators, including cables, cabling, and connections.
- Supply and installation of low-voltage cables for the connection between the MCC Panels and equipment, including connections.
- Supply and installation of a Sub-Panel Board (SPB) for each building.
- Supply and installation of low-voltage cables for the connection between the Sub-Panel Boards and equipment, including connections.
- Supply and installation of outdoor lighting, including cables, cabling, and connections.
- Supply and installation of lightning protection system.
- Supply and installation of earthing system.

8.15.10 Electrical work requirements

A private transformer substation must be built in each site.

The total capacity of the new electrical installation must be suited to the requirements defined by the Contractor plus a 20% allowance for further equipment.

This indoor-type substation shall comprise the following main items:

- Medium-Voltage Incoming cells.
- Medium-Voltage outgoing Transformer feeder cells.
- Oil power transformers.

- MV cables and MV cable terminations.
- LV cables and connections.
- Main low-voltage switchboard.
- Power Factor compensation panel.
- Motor control/command cubicles.

The Main Distribution Board (MDB) must include at minima the following main items:

- Automatic changeover switches.
- LV circuits-breakers and tripping systems.
- LV contactors and relays.
- Signaling relays and indicator lamps.
- Select mode switches (Automatic/Manual/Stop).
- Select mode switches (Run/Stop, Open/Close, etc.)
- Variable Frequency Drives.
- Motor Soft Starters.
- Lightning arrester.
- Earthing.
- Monitoring and safety accessories.

The Contractor shall also be responsible for the following:

- Costs and procedures for obtaining the agreement of the energy distributor. The services shall comply with the rules for constructing and installing subscriber substations issued by Lebanon Electricity Authority.
- The power supply network being unreliable, a generator sets will be installed in parallel with the general power supply network and can work as normal source.
- It will be able to satisfy the power supplies of the entire plant.
- The generators shall be powered by fuel tanks providing a 10 days operation's capacity.

Lightning system shall be installed.

Electrical power will be distributed via MCC panels installed in the process zone buildings described above.

Each MCC panels must include at minima the following main components:

- LV cables and connections.
- LV circuits-breakers and tripping systems.
- LV contactors and relays.
- Variable Frequency Drives.
- Motor Soft Starters.
- Signaling relays and indicator lamps.
- Select mode switches (Automatic/Manual/Stop).
- Select mode switches (Run/Stop, Open/Close, etc.)
- Lightning arrester.

- Earthing.
- Monitoring.

Each Sub-Panel Board (SPB) must include at minima the following main components:

- LV cables and connections.
- LV circuits-breakers and tripping systems.
- LV contactors and relays.
- Signaling relays and indicator lamps.
- Lightning arrester.
- Earthing.

Variable Frequency Drive (VFD) with a power rating equal to or greater than 250kW must be of the stand-alone type.

8.15.11 Generators Fuel Consumption

A preliminary estimate of the generators' fuel consumption, based on 4 hours of operation, indicates approximately 4,100 liters per day for Phase 1 and 5,200 liters per day for Phase 2 (Refer to Appendix 8.8).

8.15.12 Scada Monitoring System

The plant will incorporate a SCADA monitoring system. Details regarding this system will be included in the draft final design report.

8.16 Observations and Recommendations

In our preliminary design, we had almost implementation the recommendations of the Feasibility Study which are conservative and provide a little bit oversized plant.

The process design criteria included partial nitrification and denitrification as following:

Parameter	Influent Value	Effluent Value	Remarks
BOD_5	360 mg/L	25 mg/L	OK
TSS	450 mg/L	35 mg/L	OK
NH_4^+	Not defined	10 mg/L	Partial nitrification required
TKN	72 mg/L	30 mg/L	Partial Denitrification required
Total P	10 mg/L	10 mg/L	No removal required
Total Coliforms		2,000 MPN/mL	Disinfection Required

Moreover, it was required a SRT (Solids Retention Time) of 12 days which is a conservative value for nitrification and lead to large volume of aeration tanks.

As known, the nitrification consumes lot of oxygen and consequently power. The nitrification of each gram of NH_4^+ requires almost 3.5 times the oxygen required for the oxygen of 1 g of BOD_5. While the denitrifications will secure an oxygen credit of around 25%, it will require anoxic mixers which requires additional power.

Should the client opt to optimize on the CAPEX and OPEX via omitting the Nitrogen/Ammonia ELV criteria, the following savings can be reached:

- Aeration Tanks Volume: The SRT of 12 days requires aeration tanks volume of 230,000 m^3 (considered 336,000 m^3 in the conservative FS design). For BOD_5 oxidation only, an SRT of 5

days can be safely considered. It will require aeration tanks volume of 112,000 m^3. Moreover, anoxic tanks (total volume 57,456 m^3) will be removed.

- The oxygen requirements for different options are as following:

	With Nitrification	With Denitrification	BOD_5 removal only
Actual Oxygen Transfer Kg O_2/hr	6,684	5,339	3,228
Standard Oxygen Transfer Kg O_2/hr	14,940	11,913	9,314
Blowers capacity m^3/hr	210,895	168,446	119,721

Notes:

- *Alpha factor used in calculation of Oxygen Transfer is 0.65 for nitrification and 0.5 for BOD_5 removal only.*
- *For the option with Dentification, 12 duty blowers will be used each 330 kW i.e total 3,960 kW. Moreover, 24 mixers each 18.5 kw are used for the anoxic tanks i.e total of 444 kW. Adding up will provide 4,404 kW without considering the power requirement of Internal Recirculation pumps (which are relatively small)*
- *For the option with BOD_5 removal only, 12 blowers will be used each 250 kW i.e total 3,000 kW (46.8% less).*
- *Based on 0.27 USD/kwh, and considering 24 hours operation of the duty blowers (there 4 4 blowers standby in addition to the duty blowers), the daily saving on power is 0.27 USD/kWh x (1,404 x 0.9 kW) x 24 h/d= 8,188 USD/day ➔ 2,988,620 USD/year*
- *Cost saving on the construction of tanks.*
- *Cost saving on blowers, IR pumps and Mixers: Approx. 1.5 M USD*

Primary Sedimentation Tanks

Primary Sedimentation Tanks (PST) are designed with lamella tube in order to save on space requirements. Due to site congestions, it is recommended to build the primary sedimentation tanks for full capacity i.e. 282,000 m^3/d. If the construction of primary clarifiers is split into 2 phases, since it will be difficult to excavate and the second phase due to site congestion.

Since the expected flow to come into the coming years is within 140,000 m3/day due to incomplete network and house connections, we recommend the following:

- *Use both PSTs without lamella at the first phase. In this case, they operation an overflow flow rate of 49 m^3/m^2/hr (which is OK) and provides a theoretical removal of TSS around 48% and BOD_5 around 27%. It is to note that will lamella PST running at full capacity, the theoretical removal is 67% for TSS and 45% for BOD_5.*
- *When the plant is to be run at full capacity, lamella tubes can be installed*

TSE Effluent and Gravity Sea Outfall

The TSE from secondary treatment is brought to existing manhole MH-E2 via DN1400 pipes having total length of around 2.6 km (peak flow 5.13 m^3/s). Manhole MH-E2 discharges by gravity into the existing DN1500 sea outfall. However, after checking the design of the said outfall, it will discharge less than 3 m^3/s due to available water head. The purpose of the FS was to avoid using pumps to discharge good TSE and saving on operation cost. In this case, the existing gravity sea outfall capacity will need to be extended. We propose to consider studying the option of constructing a new sea outfall near the chlorination tank which will saving the 2.6 km of DN1400 pipes as will the capacity extension of the existing DN1500 sea outfall.

CHAPTER 9 SOIL TREATMENT

Chapter 9 - Soil Treatment

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Chapter 9 Soil Treatment

9.1 Deep Dynamic Compaction

In reference to V4-P2/Chapter 4 (Geotechnical Investigation), the new Wastewater Treatment Plant (WWTP) will be founded on loose to medium dense Manmade Fill (Silty Sandy GRAVEL of intermediate plasticity with Cobbles and Boulders, Type-S1 that is rarely used, Type-S2 that is mostly used and Type-S3 that is moderately used) and underlying medium dense to dense Seabed Sediments (Gravely very Silty fine SAND of intermediate plasticity) with water table/seawater-level at bottom level of foundation.

The above-mentioned fully saturated/submerged multilayered foundation soil having presumed bearing value of =<105kPa with modulus of subgrade reaction of =<5.5MN/m3, cannot be considered safe bearing strata for the new WWTP.

Accordingly; the multilayered foundation soil within the Areas Phase 1, Phase 2a and Phase 2b (see below Figures 9.1 to 9.3) will be densified by Deep Dynamic Compaction (DDC) from the elevation +2.00, knowing that the Manmade Fill and Underlying Seabed Sediments are favorable (Zone 1) and intermediate (Zone 2) for DDC according to FHWA-SA-95-037 (1995), and the type of foundation will be enlarged-raft, taking the heterogeneous Manmade Fill (Type-S1, Type-S2 and Type-S3, see below Figures 9.4 to 9.6) into consideration.



Figure 9.1 Area Phase 1



Figure 9.2 Area Phase 2a



Figure 9.3 Area Phase 2b



Figure 9.4 Manmade Fill Type-S1



Figure 9.5 Manmade Fill Type-S2