Dimensioning of a natural ponds as wastewater treatment plant for the city of Ouazzane

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Abstract—*Morocco has a mobilizable water heritage of 21 billion m^3 including 16 billion m^3 of surface water and 5 billion m^3 of groundwater. Unfortunately due to the exponential population growth and economic and social development, these water resources cannot meet the growing demand of the agriculture, industry and drinking water sectors.*

In addition, more than 546 million m^3 of raw wastewater is discharged annually into the receiving environment and only a tiny proportion is reused in the irrigation of about 7000 ha. Most of the wastewater discharged is responsible for the pollution of the environment.

To seek new water resources while curbing pollution, Morocco has undertaken a gigantic project of treatment of its wastewater. The city of Ouazzane, like other urban agglomerations, must choose an adequate system for the purification of its wastewater discharges.

The present work proposes the design and sizing of a natural lagoon WWTP based on demographic data, pollutant load, wastewater flow and availability of land for the city of Ouazzane.

The project proposes a pretreatment using a vertical manual screens of 36 bars of 10 mm diameter and spaced 12 mm and a Grit/Grease Separator of 5.6x1.8 m in size. Primary treatment with a circular primary decanter 16 m in diameter, 3 m deep and 602.88 m in volume. Four anaerobic basins 4 m deep, 8204 m^3 in volume and 58.02 x 35.35 m in size. Also four Optional ponds 2m deep, 41008m^3 deep and 107 x 192m in size. Finally, four maturation ponds of 1 m deep and 6400 m^3 of volume each.

Keywords— **Wastewaters, Epuration, Lagoons, Ouazzane, Morocco.**

I. INTRODUCTION

Our study focuses on the choice of the appropriate purification system to treat wastewaters from the city of Ouazzane. Unfortunately, several constraints will have an influence on the choice of the most suitable method for the habitat of Ouazzane. Among these, we can note: the high altitudes, a rugged topography and difficult accesses; irregularity of use; Difficult energy supply; weakness of economic and financial resources [1].

So after the examination of the studied treatment variants (Activated sludge, Lagoon, Bacterial bed, Reed plant filters ...) our choice was based on the techniques presenting a good robustness of treatment, a level of technicality easy to implement and reasonable investment and operating costs. As a result, the process that will be used for the purification of the effluent volumes to be treated is a natural lagoon.

In agreement with ONEP, we included a pretreatment, primary treatment (anaerobic lagoons) and secondary treatment (optional lagoons) and we added a tertiary treatment (ripening lagoons). In what follows in the present study, the objective was to calculate or deduce the volume load, the pollutant load, the surface load, the flow rates of the raw sewage and the dimensions of the different lagoons, the time of sewage stays, the total surface area of the WWTP.

II.   STUDY AREA

2.1   Geographical Position

The city of Ouazzane is located in the northern limit of the Gharb plain. The topography of the city is contrasted: Upstream, on the hill, the slopes are of the order of a few percent. Further downstream, at the level of the urban site, the general slope is weaker until the outlet in the low-slope agricultural plain [2].

The city of Ouazzane is 171 km from Rabat and 134 km from Fez. It is crossed by the RN13 national road connecting Meknes - Tetouan and RR408 regional road. The center is part of the geographical area of the Pre-rif and from an administrative point of view, the center of Ouazzane comes from the region of Tangier-Tetouan. His average Lambert coordinates are: X = 484.00 and Y = 466.40 (**Figure 1**).
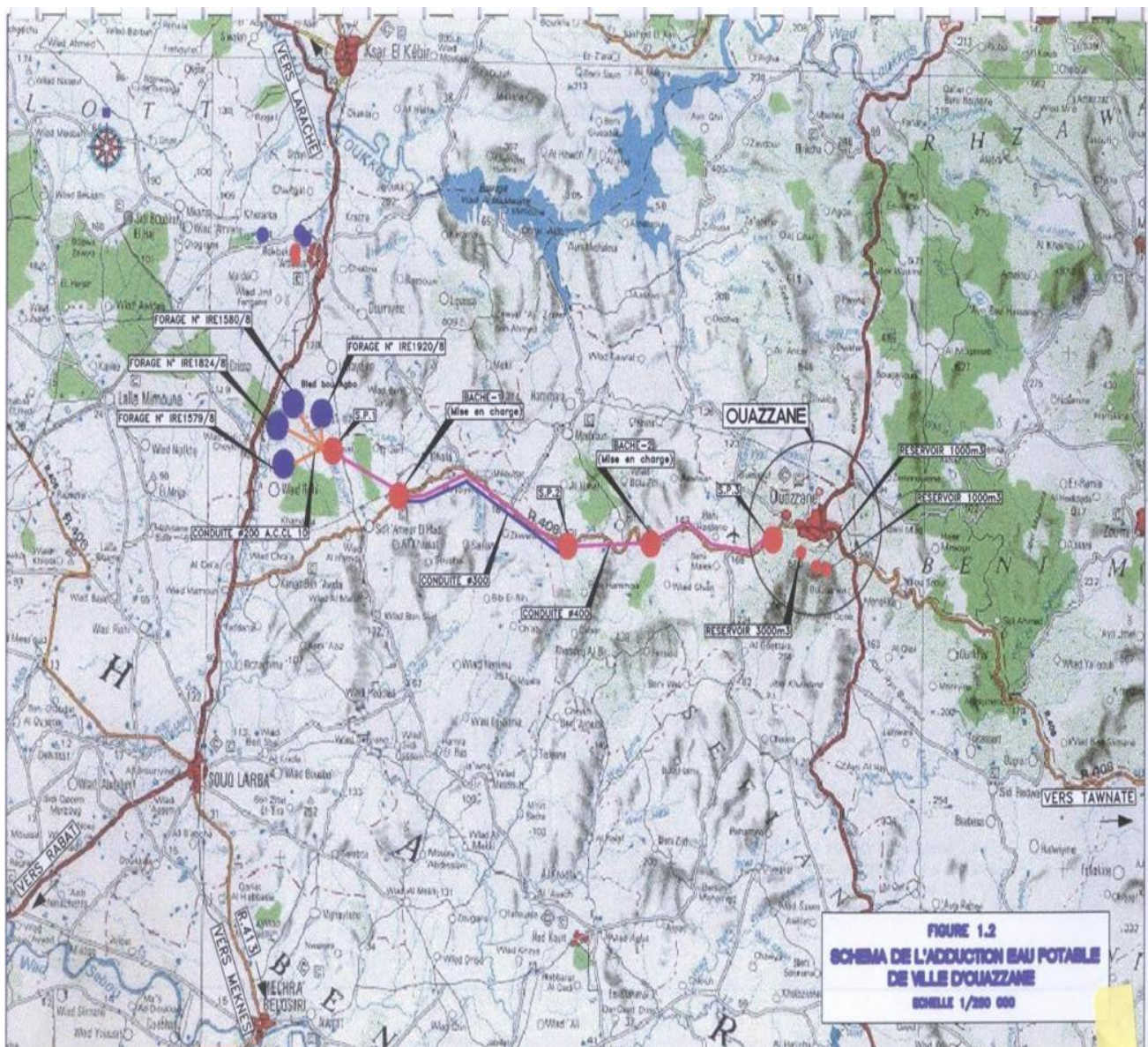

FIGURE 1: GEOGRAPHICAL LOCALIZATION OF OUAZZANE CITY

2.2   Meteorological Datas

The city of Ouazzane is located in the western part of the pre-rif mountain. It is characterized by a subhumid climate with moderate winter: the average temperature is around 18 °C, the evapotranspiration is 700 to 900 mm. Maximum, minimum

and average monthly temperatures are presented in **Table 1**. The average monthly precipitation recorded is given in Table 2 and the average annual rainfall is about 885 mm [3].

TABLE 1
MONTHLY VARIATION OF TEMPERATURE IN OUAZZANE (°C).

Months	Jan	Feb	Mar	Abr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
Max	16,2	18,1	21	23,2	26,6	30,7	35 ,6	36	32,6	28,5	22	17,6	25 ,7
Min	5,3	5,9	7,9	9,1	11	13,9	15,9	16,2	14,8	11,9	8,7	6,6	10,6
Mean	10,8	12,0	14,5	16,2	18,8	22,3	25,8	25,8	23,7	20,2	15,4	12,1	18,1

TABLE 2
AVERAGE ANNUAL RAINFALL IN OUAZZANE

Months	Jan	Feb	Mar	Abr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
P (mm)	129	118	119	74	45	13	1	1	14	70	127	174	73,75

2.3 Hydrology

The Ouazzane city is built on the northern flank of the syncline Bouhlal mountain, which is formed at the base by a practically impermeable schisto-sandstone series on which rests a more permeable sandstone series as one rises. Towards the top, the contact of the two series is marked by the resurgence of many sources.

The basin of Bou Agba is lined at the bottom by marls of the Miocene then come above the conglomeratic formations more or less clayey or sandy of Villafranchian age and old quaternary which outcrop largely on the edges of the basin. The River M'Da upstream of Souk El Larbaa, enters the basin of Bou-Agba from the East and comes south at Sidi Bou-Knadel to continue thereafter on the plain of Gharb. The River M'Da enters this basin from the east to Grouna and comes south to the threshold of Sidi Bou Knadel to then enter the plain of Gharb [4].

The tablecloth of Bou-Agba stands in the pebbles of Villafranchien, which are outcropping at the edge of the bowl and fleeing in the center under the sandy clay quaternary cover. The thickness of the pebbles is 50 m at the edges and 100 to 200 m at the center. The sheet is free on the borders (50 km^2) and in charge in the center (25 km^2). The waters of the deep aquifer of Bou-Agba are sweet of good quality (less than 0.5 g / L of total salts) and calcium carbonate type.

In addition, the geotechnical surveys undertaken in Ouazzane recently showed at the level of the city and its sewerage network the ground is rather clayey, without presence of tablecloth at the level of the projected WWTP. The terrain is also clayey, impervious since the permeability coefficient revealed on the WWTP site, by the measurements is of the order of 10-9 cm/s (10-11 m/s).

The drinking water supply of the city of Ouazzane is provided by 4 boreholes located about 40 km from the city so far from the planned site of the WWTP in the basin of Bou-Agba which houses a large underground water reserve [5].

2.4 Economic Activity

The agricultural vocation of the Ouazzane city contributes to the relative dynamism of the local economy. Ten rural communes belong to the very wide circle of Ouazzane. The Ouazzane region is especially famous for its olive trees. In fact, among the 17,000 hectares cultivated by arboriculture, there are 15,000 hectares of olives and around 1,500,000 trees producing up to 25,000 tons of olives per year. There is also an area of 600 hectares exploited in apricots and with an annual production of 1500 tons / year, and 1000 hectares of fig trees with 1500 tons /year. The animal production of red meat reaches 1243 tons/year. The herd consists of 48,000 head of cattle, 181,000 head of sheep and 14500 head of goats.

The city of Ouazzane does not have any industrial zone structured, however it is necessary to note the existence of six oil mills of which five oil mills are of minor importance and a Moroccan Industrial Oil Company of Ouazzane (SIMOD) which is the most important oil mill of the city. It was founded in 1968 and covers an area of 3 hectares. The purpose of the said company is the crushing of olives and the conditioning of the oil. The company exports extra virgin oil to the United States and Europe. Finally, a carob crushing plant which is the company Medi of Caroube (SMEDCA) created in 1989, its activities include the crushing of carob, the production of pipe blanks and the production of medicinal plants [6].

Craft has long been an important sector of economic activity in the city of Ouazzane since it occupies more than a quarter of the workforce. Of all the activities, local commerce occupies a dominant place. This imbalance is accentuated by the predominance of the grocery store. The tourist potential of Ouazzane is quite important. It revolves around at least six axes: Zaouia, Lake Bou Déroua, the tomb of Rabbi Amran located 10 km away in the village of Asjen, hunting, Tangier - Fès road and handicrafts. Despite this, Ouazzane lives on the sidelines of the national tourist activity. The city has only six unclassified hotels and only a few dozen beds.

III.   STUDY METHODS

3.1   Basic Data for the Design of the Treatment Plant

Before considering a project to install a wastewater treatment plant (WWTP), it is necessary to collect the most general information concerning the estimation of drinking water consumption, the flow of wastewater, the load in BOD_5, the content of suspended matter, and even the temperature and solar radiation according to the seasons [7,8].

The city of Ouazzane is fed from four boreholes made at the level of the water table of the plain of Bou Agba 40 km west of the city. Capture, adduction and distribution are provided by ONEE. Currently a project to reinforce the Ouazzane city water supply from the LouKous is being implemented by ONEP. It has a distribution network powered from:

* Very high floor network: tank with a capacity of 200 m^3;

* High-level network: two tanks with a capacity of 1000 m^3;

* Network beats floor: two tanks with a capacity of 1000 and 3000 m^3;

* Network very beating floor: a tank with a capacity of 750 m^3.

We need to know the current population of the Ouazzane region, and its growth rate and projection into the future. Also we need the rate of connection to drinking water and the connection rate to the sewerage network.

3.1.1   Consumption of drinking water:

Pcd = Pt x Tbp

Pcd: Population connected to drinking water expressed in inhabitant (hab).

Pt: Total population (forecast) expressed in inhabitant (hab).

Tcd: Connection rate to drinking water expressed in%.

3.1.2   wastewater discharges:

Pcs = Pt x Tcs

Pcs: Population connected to the sewerage network expressed in inhabitant (hab).

Pt: Total population (forecast) expressed in inhabitant (hab).

Tcs: Connection rate to the sewerage network expressed in%.

The daily consumption of the population connected to drinking water must be estimated in terms of (m^3 /day). The examination of the needs sheets established by ONEE made it possible to determine the calculation assumptions to evaluate the evolution of drinking water consumption. For the determination of future consumption, the following types of users were distinguished: connected domestic users, administrative users and industrial users. The Ouazzane AEP network connection rate increased from 96% in 2006 to 98% in 2020.

The average household consumption in 2004 is 60 L / person / day. For our forecasts, we take into account the average specific consumption of 65 L / person / day in 2006.

According to ONEP data on the overall consumption of Ouazzane administrations, an average consumption of 6L/person/day is deduced. For the forecasts of the consumption of the administrative users, we start from the forecasts of ONEP of 6L/hab/d. The definition of the industrial user of ONEP is very vast and goes from trade (restaurants, cafes ... etc) to the small industry, the craft industry and tourism. We consider a consumption, at present, of 6 L / hab / d for the manufacturers of Ouazzane.

Concp = Dpcd x Pcd / 1000

Conscp: Daily consumption of the population connected to drinking water expressed in (m^3 / d).

Dpcd: Daily allocation of the population connected to drinking water expressed in (L / hab / d).

Pcd: Population connected to drinking water expressed in inhabitant (hab).

The sewage flow rates are calculated from the daily consumption of drinking water affected by a sewage discharge rate. The average daily flow is that emanating from the drinking water consumption of the connected population multiplied by the rejection rate or refund rate. The rejection rate is a function of the type of habitat, in the villa zone, it can go down to 0.5 but it can exceed 100% of the consumption if the water used does not have for only origin the water distributed by ONEP, we can adopt an average rejection rate equal to 0.8.

3.1.3   Average flow of wastewater (Qww)

Qww = Conscp x Trejc = qkP. 10^{-3}

Qww: Wastewater flows expressed in (m^3 /day):

Trejc (k) : The rejection rate expressed in% (80 to 90%).

 Conscp: Daily consumption of the population connected to drinking water expressed in (m^3 / d).

q: Consumption of water Liter/person   P: Population connected

3.1.4   Peak hourly rate of wastewaters Qph: The hourly peak coefficient is given by the formula:

Qph = 1.5 + (2.5 / $\sqrt{}$Qww)

With Qww = average flow (in L / s)

3.1.5   Daily point flow of wastewater Qp

The daily peak flow Qp is expressed in cubic meter / day (m3 / d).

Qp = Cp x Qww

Qp: Wastewater point flow rate expressed in cubic meter / day (m3 / d)

 Cp: The point coefficient, is taken equal to 1.3. It is given by the following formula:

Cp = 1.5 + 2.5 / $\sqrt{}$Qww

3.1.6   Pollutant load Cp:

Cp = Pcs x Ratio / 1000 (Kg/day)

Ratio (gDBO5 / hab / d): Pollution ratio

3.1.7   Concentration of BOD$_5$

BOD$_5$ is expressed in mg of O$_2$ / L.

Concentration in BOD$_5$ = pollutant load x 1000 / Qww

3.1.8   Equivalent inhabitant (EH)

EH = Polluting load X 1000 / Ratio

3.1.9   Calculating stormwater flows

Various formulas for evaluating stormwater flow exist. We will use, in the present study, the superficial formula of CAQUOT [9].

3.2   Dimensioning of Pretreatment

The purpose of the pretreatment is to remove the coarsest solids or particles, which may interfere with subsequent treatments or damage the equipment. Pre-treatment includes Screenings, Grit and Grease separator.

3.2.1 Dimensioning of the "Screening".

For the choice of the manual bar screens it is necessary to calculate its total area **St** and the area between the bars **Sb** (expressed in m²), and the number of bars **n**, and their width **l** (expressed in m) by the following formula.

$$S_t = \frac{S_v}{\left(l/_{i+e}\right)}$$

$$n = \frac{S_b}{(e \times h)}$$

$$l = \frac{S_t}{h}$$

$$S_b = \frac{S_v \times e}{i}$$

With:

St: total area St (m²): **Sv**: Vacuum section

e: Thickness of the bar expressed in cm.   **i**: Spacing between the bars expressed in cm.

n: The number of rungs   **h**: The Height of the Degreaser expressed in cm.

l: the width of the bar (m)   **Sb**: the area between the bars (m²)

3.2.2 Design of the Grit/Grease

For the choice of the Grit/Grease separator one has to calculate its horizontal surface **Sh** (expressed in m²), its vertical surface **Svt** (expressed in m²), the width **l** (expressed in m), the length **L** (expressed in m), the Volume **V** (expressed in m³), and the residence time **Ts** (expressed in s).

$$S_h = \frac{Q_p}{C_s}$$

$$S_{vt} = \frac{Q_p}{V_h}$$

$$l = \frac{S_{vt}}{H}$$

$$L = \frac{S_h}{l}$$

$$V = L \times l \times H$$

$$T_s = \frac{V}{Q_p}$$

With:

Sh : The horizontal surface (m²)   **Cs**: Surface load expressed in m / h

Qp: Wastewater point flow rate expressed in cubic meter / day (m³ / d)

Svt : The vertical Surface (m²)   **Vh**: Horizontal flow velocity expressed in m / s.

l : The width (m):   **H** : Height expressed in m.   **L**: The length (m)

V: The Volume (m³)   **Ts**: The residence time (s):

3.2.3 The dimensioning of the Oil, Grease and Fat Traps

For the choice of the Grease separator one must calculate its horizontal surface **Sh** (expressed in m²), its vertical surface **Svt** (expressed in m²), the width **l** (expressed in m), the length **L** (expressed in m), the Volume **V** (expressed in m³), and the residence time **Ts** (expressed in s).

$$S_h = \frac{Q_p}{C_s}$$

$$S_{vt} = \frac{Q_p}{V_h}$$

$$l = \frac{S_{vt}}{H}$$

$$L = \frac{S_h}{l}$$

$$V = L \times l \times H$$

$$T_s = \frac{V}{Q_p}$$

With:

Sh: The horizontal surface (m²)   **Cs**: Surface load expressed in m / h

Svt: The vertical Surface (m²)   **Vh**: Horizontal flow velocity expressed in m / s.

l: The width (m)   **H**: Height expressed in m   **L**: The length (m)

V: The Volume (m³)   **Ts**: The residence time (s)

3.2.4 Dimensioning of the Primary Decanter

The primary treatment with a circular decanter is intended to complete the removal of non-decanted fines by grit removal. The decanter is sized on the basis of the dry point flow rate according to the following equations:

Sdec = Qp x Va

Vdec = Sdec x Hdec

Sdec: surface of the decanter in m^2

Qp: peak flow in dry weather m^3 / h

Va: rate of climb limit set at 1.2 m / h

Hdec: the cylindrical height is fixed at 3 m.

R dec: decanter radius in m calculated from the Sdec surface

3.3   Dimensioning of Ponds

3.3.1 Dimensioning of the Anaerobic ponds

A.   The volume load (Cv):

The primary treatment chosen is a natural lagoon. To size anaerobic lagoons, criteria based on the volume load, the residence time, the depth of the basins and the treatment efficiency are used. Gloyna has shown that the decrease in BOD_5 is a function of the retention time which is itself influenced by the temperature [10].

The volume load is expressed in g of BOD5 / m3 / day and must be between 50 <300 g / m3 / day [11]. According to Mara and Pearson [12] the volume load varies mainly with temperature (**Table 3**). It is recommended in general, volume loads of 100 g / m3 / day in winter to 300 g / m3 / day in summer.

TABLE 3
VARIATION OF VOLUMETRIC CHARGE Cv AS A FUNCTION OF TEMPERATURE FOR ANAEROBIC BASINS

Temperature (°C)	Volumic load (g /m^3/jour)
T ≤ 10	100
10 < T ≤ 20	20*T -100
T > 20	300

B.   Volume of the water slice

The volume of the water slice is calculated as follows:

$$Vt = \frac{Cp}{Cv}$$

$$V_A = \frac{C_{OX}\, Q_{mean}}{C_{V_A}}$$

With:

Ve: volume of the water slice   **Cv**: Volume load (g / m3 / d)   **Cp**: pollutant load g / d

C.   Volume per basin

$$V_a = \frac{V_t}{n}$$

With:

Vt: total volume of anaerobic basins in m^3.   **n**: number of basins chosen.

D.   Total surface needed

Area **S**, anaerobic basins is calculated by the following formula:

$$S = \frac{C_o \times Q}{P \times C_v}$$

With:

S: basin area expressed in m^2.

Co: Incoming concentration of BOD5 (mg / L).

Cv: Volume load (g DBO5 / m3 / day).

Q: Wastewater flow (m^3 / day)

P: Depth of basin.

$$S_t = \frac{V_t}{P_a}$$

Vt: total volume of anaerobic basins   **Pa**: basin depth of 3 to 4 m

E.   Surface per anaerobic basin:

$$Sa = Va/Pa$$

$$S_a = \frac{S_t}{n}$$

With

n: Number of basins in parallel   **Va**: volume of the anaerobic basin   **Pa**: depth of the basin

F.   Length halfway up the water:

$$L1 = (Sa)^{0,5}$$

G.   Length of water:

$$L2 = L1 + (2 \times P/2 \times Slope)$$

L1: length at mid-depth.

P: Depth adopted.

Slope: internal slope of bank.

H.   Bottom length:

$$L3 = L1 - (2 \times P/2 \times Slope)$$

L1: Length at mid-depth.

P: Depth adopted.

Slope: internal slope of the bank.

I.   Length at the peak:

$$L4 = L2 + (2 \times Cr \times Slope)$$

L2: Length wire water.

Cr: adopted ridge.

Slope: internal slope of the bank.

J.   Residence time:

$$T_s = \frac{V_a}{Q}$$

According to Ceremher [11] **Ts** must be between 3 to 5 days.

Va: Volume of anaerobic basin m^3.

Q: discharge of wastewater at the entrance of anaerobes (in m^3 / d)

K.   Effluent charge:

$$C_e = \frac{C_a}{K_t \times T_s + 1}$$

Ce: Effluent charge

Ca: Tributary charge.

Kt: Elimination coefficient of BOD$_5$.

Ts: Residence time.

3.3.2   Dimensioning of Facultative Ponds

In order to size the optional basins, criteria based on the surface charge **Cs**, the residence time **Ts**, the depth of the basins **Pf** and the purification efficiency **Rf** are used. Two dimensioning criteria are used for the optional basins namely:

   ❖   The surface load in kg / ha / day: minimum 1000 kg / ha / d   beyond which: anaerobiosis.

   ❖   Residence or retention time.

A.   Surface charge Cs (in kg / ha / d).

According to Mc Garry & Pescod [13] the surface charge **Cs** is given by the following formula:

$$C_{s(F)} = \frac{(60.3 \times (1.0993)^T)}{1.3}$$

According to Mara and Pearson, [12] the **SF** surface is given by the following equation:

$$S_{(F)} = 10000 \times \frac{\text{Charge polluante}_{\left(Kg/j\right)}}{C_{S(F)}}$$

Cs: Surface load (in kg / ha / d).   **T**: Residence time

SF : The required area (in m^2).

$$S_F = \frac{C_F}{C_s}$$

With :

SF: Total area needed (in ha).

CF: Load at the entrance of the optional pools.

Cs: Superficial charge.

B.   Volume of basins:

$$VF = SF \times PF$$

C.   Residence time (days).

According to Mara and Pearson, [12] the residence time is given by the following formula:

$$T_{S(F)} = \frac{S_{(F)} \times P_F}{Q}$$

If **Ts** <15 days: Take **Ts**(f) = 15 days and recalculate the depth **P** by the relation:

$$S_F = 15 \times \frac{Q}{P_F}$$

D.   Depth of basins (m)

According to Mara and Pearson [12], the Depth of basins (in m) should be 1.2 to 2 m. P_F= 1.5 m.

E.   Purification performance

The BOD_5 of the effluent at the outlet of the optional pond is the same as that of the entrance of the ripening basin:

$$C_2 = \frac{C_1}{1 + T_{a(F)} \times T_{S(F)}}$$

The rate of abatement of BOD5 at the level of the optional pond depends solely on the temperature, it is determined by:

$$T_a = (1 - \frac{1}{1 + K \times T_{S(F)}}) \times 100$$

K according to Marais [14] and Demillac [15].

$$T_{a(F)} = 0.3 \times (1.05)^{(T-20)}$$

For wastewater that will be discharged into the natural environment, the BOD_5 at the outlet of the optional pond **C2** must be less than 120 mg / L, if this is not the case it will be necessary to increase the residence time T_{SF} to have **C2** = 120 mg / L.

3.3.3   Dimensioning of Maturation Ponds

The maturation pond is a shallow, aerobic basin whose role is the elimination of pathogens. The sizing of this basin is done by the following relations:

Volume of the maturation basin Vm in m^3

Vm = Q$_{ww}$ x Tsm

Surface of the maturation basin Sm in m^2

Sm = Vm / Pm

Residence time in the maturation basin Tsm in days

Tsm = (1 / β x K) x log (Lsf / Lsm)

With:

Pm: depth of the maturation basin

Lsf: DBO_5 at the optional basin exit

Lsm: BOD_5 at the outlet of the maturation pond.

IV.   RESULTS AND DISCUSSION

4.1   Evaluation of the Flow of Parasitic Water

After analysis of the results of the various characterization and consultation campaigns with ONEP, we selected 35% of the average discharge for the parasitic waters in 2006. This rate decreases to 20% in 2020 and 20% in 2040. This regression is explained by the improvement of the networks after completion of the project and intervention of ONEP for the management of the sanitation service.

4.2   Volume of Ouazzane Sewage

In Morocco, the discharge rates usually used for domestic wastewater are between 70% and 85% of the volume consumed (**Table 4**) ONEP / GTZ, [16]. According to the reconnaissance on the spot and in other cities of the country, we adopted for the city of Ouazzane a rate of rejection of 80% for the domestic, administrative and industrial uses.

The average connection rate is 91% for the year 2004. As hypothesis for the evolution of the connection rate, we consider an increase from 96% in 2008 to 98% in 2030. The total discharges of the city of Ouazzane are presented in **Table 5**. The calculation of wastewater discharges has been carried out for the River Rha and Rabat River basins, since the treatment plant will be sized only for the treatment of discharges of these two basins (**Table 6**).

4.3   Estimation of the Pollutant Load of Ouazzane

For the pollutant load calculation we adopted the national plan of liquid sanitation ratios for cities of similar size (27 to 30 g BOD_5 / hab / d). For the calculation of the polluting load coming from collectives and administrative users, high schools with boarding schools, hospitals, dispensaries and slaughterhouses were taken into account. The contribution of different users in the collective sector is as follows:

High school with internship: 25 g / pupil / day

Military (barracks): 40 g / effective / day

Hospital: 60 g / bed / day

Abattoirs: Sheep and goats: 500 g BOD5 / head / day; Cattle: 3000 g BOD5 / head / day; Moorish baths: 20 g / visitor / day.

The lagoon purification process recreates the conditions of self-purification of the natural environment. The proper functioning of a lagoon depends on the balance between different groups of animal and plant species (bacteria, zooplankton, algae and aquatic plants). Photosynthesis plays a driving role.

TABLE 4
OVERALL CONSUMPTION OF DRINKING WATER

	2006	2007	2008	2010	2015	2020	2025	2030	2035	2040	2048
Consumption											
population (hab)	59255	59907	60566	61905	65386	69386	72945	77046	81378	85953	93815
Connection Rate %	96	96	96	98	98	98	98	98	98	98	98
Population Connected (hab)	56885	57510	58143	60667	64078	67681	71486	75505	79750	84234	91939
Population Not Connected (hab)	2370	2396	2423	1238	1308	1381	1459	1541	1628	1719	1876
Endowments L/hab/d											
Population Connected (hab)	65	65	65	65	65	65	65	65	65	65	65
Population Not Connected (hab)	15	15	15	15	15	15	15	15	15	15	15
Administrative	6	6	6	6	6	6	6	6	6	6	6
Industrials	6	6	6	6	6	6	6	6	6	6	6
Consumptions (m³/d)											
Population Connected	3697	2738	3779	3943	4165	4399	4647	4908	5184	5475	5976
Population Not Connected	36	36	36	19	20	21	22	23	24	26	28
Endowments L/hab/j	63	63	63	64	64	64	64	64	64	64	64
Consumption domestic	3733	3774	3816	3962	4185	4420	4668	4931	5208	5501	6004
Consumption administrative	356	359	363	371	392	414	438	462	488	516	563
Consumption industrial	356	359	363	371	392	414	438	462	488	516	563
Total Consumption m³/d	4444	4493	4542	4705	4969	5249	5544	5856	6185	6532	7130
Net Endowments L/hab/d	75	75	75	76	76	76	76	76	76	76	76

Indeed, algae produce oxygen by photosynthesis. This oxygen is used by bacteria to mineralize and assimilate organic matter, hence the production of carbon dioxide, nitrates, and phosphates. Decantable materials are deposited at the bottom of the lagoon. They are regularly extracted from the system, in order to keep the installations running smoothly. To prevent coarse materials from accumulating in the basins, a screening facility is recommended upstream.

We have also taken into account the polluting flows of the businesses in the domestic expenses. For industrial companies, we consider only water consuming establishments and tourist establishments. In these cases, the costs are estimated by considering the specific conditions of the establishments, the industry or the industrial zone in question.

The specific pollutant fluxes in g / hab / day equivalent for the different parameters are presented in Table 6 below:

➢ **BOD$_5$: a** pollutant flux of 27 g BOD$_5$ / person / day in 2006 and 30 g / person / day for 2020 is adopted. This increase results from an improvement in the standard of living of the population of Ouazzane;

➢ **COD:** for the COD, it is adopted a value of 80 g / hab / d for the year 2006. We adopted the value of the National Scheme which increases proportionally with the value of BOD$_5$ as follows:

COD = 2.9 x BOD$_5$ to 89 g COD / inhab / day in 2030;

➢ **MES:** for MES, we adopted the values recommended in the National Scheme. The value of 27 g /hab /d is adopted for the year 2006, and 30 g / hab / d in 2020. This value corresponds to MES = 1x BOD$_5$.

TABLE 5
RESULTS OF THE GLOBAL WASTEWATER DISCHARGE OF OUAZZANE

	2006	2007	2008	2010	2015	2020	2025	2030	2035	2040	2048
WASTEWATER DISCHARGE											
DOMESTIC REJECTIONS											
POPULATIONS (inhabitant)	59255	59907	60566	61905	65386	69062	72945	77046	81378	85953	93815
Connection rate to the sewer %	91	91	91	92	93	95	96	97	97	98	98
Sewer return rate %	80	80	80	80	80	80	80	80	80	80	80
Domestic Rejection (m^3/d)	2718	2748	2777	2916	3113	3359	3585	3826	4042	4313	4707
ADMINISTRATIVE Rejection											
Sewer return rate %	80	80	80	80	80	80	80	80	80	80	80
Connection rate to the sewer %	100	100	100	100	100	100	100	100	100	100	100
Administrative Rejection (m^3/d)	284	288	291	297	314	331	350	370	391	413	450
Sewer return rate %	80	80	80	80	80	80	80	80	80	80	80
Connection rate to the sewer %	100	100	100	100	100	100	100	100	100	100	100
Industrial Rejection (m^3/d)	284	288	291	297	314	331	350	370	391	413	450
TOTAL REJECT (m^3/d)	2873	3324	3359	3510	3741	4021	4285	4566	4824	5139	5607

TABLE 6
RESULTS OF THE GLOBAL CONSUMPTION AND WASTEWATER DISCHARGE OF OUAZZANE

	2006	2007	2008	2010	2011	2012	2015	2017	2020	2025	2030	2035	2040
OUED REHA BASIN **RABAT BASIN**													
Totale Population EH	59152	59803	60461	61798	62478	63165	65273	66717	68943	72819	76913	81237	85805
Totale Consumption m^3/d	4436	4485	4535	4697	4748	4801	4961	5070	5340	5534	5845	6174	6521
Connection rate to the sewer %	91	91	91	92	92	92	93	94	95	96	97	97	98
Total population connected to the sewer EH	49994	54421	55020	56854	57479	58112	61093	63310	65906	70340	74835	79043	84089
Global specific flow DBO5 g / Eq.hab / d	27	27	27	28	28	28	29	29	30	30	30	30	30
Total Equivalent Inhabitants EH	59085	59715	60412	62316	63002	63695	67558	70387	72437	77238	81430	86008	91353
Overall average rate m^3/d	3281	3317	3353	3504	3543	3582	3735	3851	4015	4278	4558	4814	5129
Parasitic water level %	35	35	35	30	28	26	24	22	20	20	20	20	20
Average flow rate including parasitic water m^3/d	4429	4478	4527	4555	4535	4513	4631	4699	4818	5134	5470	5777	6155
// // // // L/s	51	52	52	53	52	52	54	54	56	59	63	67	71
Effluent charge of DBO5 kg/d	1440	1472	1489	1576	1593	1611	1739	1794	1962	2092	2234	2358	2531
// // // mg/L	325	329	329	346	351	357	375	382	407	407	408	408	411
EffluentCharge of DCO kg/d	4274	4368	4419	4679	4730	4782	5162	5327	5822	6210	6630	7000	7512
// // // // mg/L	965	975	976	1027	1043	1060	1115	1134	1208	1210	1212	1212	1221
Effluent Charge of MES kg/d	1422	1453	1470	1557	1574	1591	1717	1772	1937	2066	2206	2329	2500
// // // // // mg/L	321	325	325	342	345	353	371	377	402	402	403	403	406

4.4   Dimensioning of the "Screening And Grit"

The sizing of the screening disposal is the same for most WWTPs. A coarse grid is provided for the discharge of sewage from the pipeline so that coarse material contained in the wastewater can be retained. It is recommended to use for this what is commonly called a cage grid.

In order to ensure operation that has the least possible problems, it is planned to distribute the effluent to be treated under normal operating conditions on two different paths. This will allow, in case of revision work on one of the grids, a certain screening of wastewater arriving at the treatment plant through the presence of a second grid. The two grilles will each be sized to support 50% of the maximum hydraulic load.

To avoid the occurrence of hydraulic problems in the event of a prolonged failure of a grid, it will be planned to set up parallel to the two grids a bypass. The spacing between the bars of the fine grids is generally between 5 mm and 20 mm. The smaller these distances, the greater the volume of materials retained.

Thus, the establishment of grids too fine makes the system more sensitive to mechanical failures caused by the jamming of solids. These considerations lead us to propose a screening chamber which will be equipped with a grid consisting of circular bars with a diameter of 10 mm, which will have an inclination of 1: 3 and a height of 0.8 m (**Table 7**).

Grid cleaning will be manual to avoid high operating and maintenance costs. To facilitate the continuous cleaning of the grids the screening chamber will have an access bridge equipped with a perforated channel. This channel will remove residues with a rake and evacuate the water in the channel screening. The collected residues are evacuated in a container ($1m^3$), which will be placed next to the screening chamber. The collected residues will be transported to the dump. It is proposed to size a manual screening unit, with manual cleaning (**Table 7**).

TABLE 7
CRITERIA OF DIMENSIONING OF SCREENING

Maximum speed of passage in rainy weather	<	2,0	m/s
Minimum speed of passage in dry weather	>	0,5	m/s
Guaranteed passing speeds required for a degree of clogging of the grid	=	25	%
Flow velocity in the approach and exit channels of the grid for Qph	>	0,5	m/s
Screening residues	=	env. 5	L/hab/a
Number of channels	1		
Type	Manual Cleaning		
Width	0,50 m		
Height	0,80 m		
Max hydraulic load	71 L/s		
Tilt of the bars	1 : 3		
Section of the bars	Circular		
Thickness of the bars	10 mm		
Spacing of the bars	12 mm		

The main function of the sand trap is to protect sand treatment facilities, so that sand deposit does not reduce the volume needed for effective treatment. In addition to this function, the grit eliminates the need to protect the pumps and other equipment from abrasion, thus significantly reducing maintenance costs.

The last important function is to allow a good separation of the sand and the organic matter in order to maintain the quantity of sand thus produced at a low level. This separation will avoid additional costs for the evacuation of large volumes of sand and will reduce the release of unpleasant odors associated with the evacuation of this material. However, a good separation of the organic and mineral fractions can be ensured in a grit chamber only by air insufflation. That's why we recommend the installation of a ventilated sand trap (**Table 8**).

The sand that settles in the grit chamber is driven by a scraper into the collecting hopper. The scraper moves with a scraper bridge mounted on rails. A mammoth pump sucks the sludge that settles into the hopper and drives it to a classifier. It is in the latter that the sand is separated from the water. This water is then reintroduced into the clarification process while the sand is led to a reservoir that must be emptied regularly.

<div align="center">

TABLE 8
CRITERIA FOR SIZING THE GRIT/GREASE

</div>

Parameters		Value	Unit
Residence time for Q_{max}	>	5	min
Residence time for Q_{ph}	>	10	min
Horizontal speed in the grit chamber with Q_{max}	<	0,1	m/s
Specific energy for air insufflation	~	5	W/m³
Sand retained by the sand trap	~	0,06	L/m³
Dimensioning flow	=	71	L/s
Area	=	11	m²
Length	=	5,64	m
Width	=	1,88	m

4.5   Dimensioing of Primary Decanter

The primary treatment with a circular decanter is intended to complete the removal of unwanted course materials by grit removal. The primary clarifier is sized on the basis of the dry point flow (Table 9). The settling lagoon has a circular geometry, the diameter is 16.00 m; the radius of 8.00 m and the depth of 3.00 m. The total volume is 602.88 m.

<div align="center">

TABLE 9
CRITERIA OF DIMENSIONING OF PRIMARY DECANTER

</div>

	Unit	Q (m³/h) Calculated Value	Value After correction
Débits	Q (m3/d) 7267	302,78	302,78
Taux de débordement	τ (m³/ h / m²)	1,5	1,33
Temps de rétention de l'eau dans le décanteur	tr (hour)	2	2
Depth	H (m)	3	3
Total Surface	S (m²)	201,86	201
Diameter	D (m)	16,04	16
Ray	R (m)	8,02	8

4.6   Dimensioning of Lagoon Basins

For the "natural lagooning" process, there are numerous references and models for sizing basins without there finally being any that is universal. The most common sizing variant of the 3 types of ponds used in the natural lagoon system: anaerobic, optional, maturation.

4.6.1   Anaerobic Basins

These are settling basins with anaerobic conditions. Treatment consists mainly of sedimentation of suspended solids and partial digestion of readily biodegradable organic material. The volume of the pond is chosen large enough to ensure a storage of sludge of at least one year (**Table 10**). The residence time of the water must at least exceed the day, and the volume required for the deposit of sludge must be included. In addition, according to MARA [17] a volume load of 350 kg/m³ /d should not be exceeded to limit odor nuisance.

The pre-treated wastewater passes through the Venturi canal and will be distributed in the distribution zone on the two pipes that will bring the wastewater to the four anaerobic basins. The distribution system will be equipped with a cofferdam so that, in case of cleaning of an anaerobic basin, the effluent can be diverted to the other basins.

The basins will be rectangular in shape with a bottom length of 44.02 m with a slope of 1: 2 (V / H). The maximum length at the crest of the dike will be 60.02 m. In order to avoid overflows due to the wind, we reserve a freeboard of 0.5 m minimum. Feeding each basin will be provided by a DN250 pipe entering the basin at the first third of the depth, 2 meters below the water level. Communication from one basin to another will work through a look with a spillway (cofferdam) towards the connecting pipe to the proposed optional pond.

It is recommended to clean the ponds once a year that are to say to evacuate the sludge that has been decanted. Otherwise the additional volume of sludge will reduce the useful volume of the pond and the necessary retention time will no longer be respected. Then the sludge will be pumped off with a sludge pump and transported to the drying beds.

TABLE 10
SIZING CHARACTERISTICS OF ANAEROBIC BASINS

Parameters		Value	Unit
Volumic Load	=	30 - 300	kg/m^3/d
Depth	=	03-04	m
Residence time	=	03-05	day
Areal Load	=	1000	kg/ha/d
Number of basins		4	parallel basins
Dimensioning of basins at mirror		58,02 × 35,35	m
Depth of water		3,50	m
Minimum freeboard		0,50	m
Depth of basin		4,00	m
Basin Volume		8204,02	m^3
Interior slope		1 : 2	

4.6.2 Optional Basins

Wastewater treated in anaerobic basins is routed through DN 250 mm PVC pipes to two series structures that distribute the water to the entrances to optional lagoons. Like anaerobic basins, optional basins operate in parallel. The geometry of the lagoon aims to have a regular shape, the length is approximately 1.5 to 2 times the width (**Table 11**). The total depth of the basins is 2.00 m with a freeboard of 0.5 m. The dykes will have an inclination of 1: 2. The feeding of each basin will be done by a DN 250 pipe which returns to a height of 0.75 m compared to the coast of the bottom of the basin. The exit of the basin will be made through a dumping threshold formed of cofferdams. Thus, we will be able to adjust the water level in the basins. The cofferdams will be easily manipulated aluminum bars of height 5 cm.

TABLE 11
DIMENSIONING OF OPTIONAL BASINS

Parameters	Value	Unit
Number of basins	4	parallel basins
Dimensioning of each basin to mirror	106,84×192,31	m
Depth of Water	1,50	m
Basin Depth	2,00	m
Basin Volume	41092,8	m^3
Interior Slope	1:2	

4.6.3 Maturation/ Storage Basins

The maturation basins are shallow and aerobic basins whose role is the elimination of pathogens and will be used for the storage of treated wastewater for agricultural valorization. The sizing of these basins is summarized in **Table 11**. The wastewater treated in the optional basins passes through DN 250 mm pipes to four series of works which distribute the water to the inputs of the maturation lagoons. Like anaerobic and facultative basins, maturation basins operate in parallel.

The maturation lagoon has a rectangular geometry, the length is approximately 1.5 to 2 times the width. The total depth of the basins is 1.50 m with a freeboard of 0.50 m. The dikes will have a slope of 1: 2

TABLE 12
DIMENSIONING OF MATURATION LAGOON

Number of basins	4 parallel basins
Dimensioning of each basin to mirror	160.00 ×40.00 m
Depth of Water	1,00 m
Depth of Basin	1,50 m
Basin Volume	6400
Interior Slope	1:2

4.7 Drying Beds

The sludge will be dehydrated on drying beds corresponding to the usual methods in Morocco. Drying beds made of concrete with layers of gravel and sand are recommended. This configuration allows sufficient dewatering of the sludge which can then be used in agriculture. The beds will be provided with perforated PVC drainage pipes (DN 150) with 3/4 openings.

The pipes are laid in drain trenches and surrounded by gravel. The drainage pipes lead the dirty infiltrated water towards a DN 250 pipe, which brings them back gravity to the distributor upstream of the anaerobic basins. The number of drying beds was determined according to the interval and frequency of cleaning.

TABLE 10
DIMENSIONING OF DRYING BEDS

Parameters		Value	Unit
Quantity of sludge per drain		2993	m^3
Depth of beds		0,6	m
Length of beds		25	m
Width of the beds		10	m
Total number of beds		30	

V. CONCLUSION

When using an extensive process such as natural lagooning, the quantities of sludge produced are less than in the case of the use of an intensive process. However, sludge is not removed continuously but periodically. This is why we need to build important infrastructures for the evacuation of sludge. These infrastructures are not used regularly but must be maintained, they occupy space and require investments.

Natural lagooning requires the installation of very important infrastructures as well as the reasonable investment cost. Nevertheless it requires little maintenance and few qualified personnel compared to the bacterial bed and activated sludge. This process gives a very good robustness and reliability of treatment in addition it presents olfactory nuisance (smell), considerably in summer because of the presence of the wind Chergui having the direction of the EST towards the West).

According to the multi-criteria comparison of the available variants, it seems that lagooning is the most appropriate method for the treatment of wastewater from the city of Ouazzane.

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