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The Hydraulic Design of the Grit Chamber of the Urmia Wastewater Treatment Plant with Hydraulic Approach

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ABSTRACT: The main objectives of the treatment plant are reducing pollutant substances and reusing them for various applications. Due to amount of inlet sewage, one can say that whatever volume of inlet sewage increases, the accuracy of calculations and designing rises and operation of it is done in best possible way. Hydraulic designing of each system has many advantages with minimum need to pumping systems and based on using natural slope of land and gravity motion of flow. The aim of providing hydraulic flow in wastewater treatment plants is evaluating speed, capacity and eventually height drops per unit and determining hydraulic profile of the sewage surface. The wastewater treatment plant of Urmia city is designed with Biolac system. This process is under monopoly of German companies. The initial and secondary sedimentation systems are designed seamlessly and rectangular which is separated by a polyethylene separator baffles each other. In this study we have attempt to reform the system of initial sedimentation and compare results with Biolac system.

Key words: Initial Sedimentation, Biolac System, Hydraulic Designing, Urmia

INTRODUCTION

All the communities in daily activities produce waste substances that may be solid, liquid or gas form. Liquid waste materials are so-called sewers or sewage. Sewer is the water that is contaminated by various applications. About 99.9 percentage of sewers is water and only about 0.1 percentage of it is consist of impurities such as suspended solids substance and colloidal and solution (Nadafi 2011).

Wastewater is not just sewage. All the water used in the home that goes down the drains or into the sewage collection system is wastewater. This includes water from baths, showers, sinks, dishwashers, washing machines and toilets. Small businesses and industries often contribute large amounts of wastewater to sewage collection systems; others operate their own wastewater treatment systems. In combined municipal sewage systems, water from storm drains is also added to the municipal wastewater stream. Wastewater is about 99 percent water by weight and is generally referred to as influent as it enters the wastewater treatment facility. “Domestic wastewater” is wastewater that comes primarily from individuals, and does not generally include industrial or agricultural wastewater (Sarafraz *et al.,* 2007).

Understanding specifications of wastewater is of the basic processes in designing sewage purification systems. Without reliable data in this case, designs will be met with many problems such as increasing cost, and operation and maintenance problems and loss in reaching purification goals (Afuoni and Erfanmanesh, 2012).

According to Miller's comments, any change in characteristics of air, soil, water and food that have adverse effects on health, environment, human activities and other organisms is called pollution and any water that loses its quality for particular use, is turned into sewers. So the sewer is water which consumed and during this process, some suspended and dissolved substances entered in it (Tovrovski and Maathai, 2010).

Sludge can be defined as mud or soft sediment resulting from settling of solid substances exists in sewage. Also, the sludge can be assumed as mixtures of massed granular particles, which hydrodynamically exhibit such as a single particle, and observed that these masses can be present, have no contact with other masses (Metcalf and Eddy, 2011).

Growing urbanization, population growth and urban and industrial development, problem as water shortage (water scarcity) occurs. To solve this problem, strategies have been contemplated for purifying existing water in environment and transform it into safe and delicious water before delivering industrial or urban drinking water networks and purifying domestic wastewater and industrial effluents before draining them into the nature. The major results of raw water treatment and waste water treatment are including producing water and treatment wastewater, sludge and byproducts (Monzavi, 2011).

The purified water for urban or industrial and wastewater of treatment plant is transferred to the acceptor water or lands and sludge prior to final disposal is refined for re-use purposes. In conventional wastewater treatment methods such as active sludge processes and stalactite filter, a high volume of initial sludge is added into secondary sediment sludge (active sludge). In active sludge process, secondary sludge is a microbial biomass which is produced by the metabolism of organic matter, and microbial product in sediment wastewater is about 50% and about 20% of this mass returned to the system and with initial sludge guided to disposal system. In Stalactite filters, the hydraulic load is less and less sludge is produced and there is no ancillary returning recycle system. Generally, high volumes of sludge with a concentration of 1 to 4 percent of solid substance are produced in treatment processes which are created one of the main problems of excretion. The reason is that the excess waste sludge is a combination of organic matter and microbial cells that may be degradable with other microorganisms (Gourjar (2012),

Planning and designing of wastewater treatment facilities is first and effective step in the implementation of efficient Waste water treatment plants technically and economically. Obviously using systematic and efficient methods have a great impact on success of the project. Waste water treatment plants, consisting of a chain of physical, chemical and biological processes which The main purpose of guiding them is the removal of contaminants present from one side and remove the remaining organic material in the sludge treatment processes on the other hand, before unloading them into the environment. On physical filtration (mechanical) stage of wastewater, the very coarse aggregate solid particles floating or suspended existing in wastewater are removed firstly. Also isolation of sand particles, are often mineral and contaminated with fats and oils, is done. The final stage of physical purifying is related to initial settling (sedimentation) pool. In some cases, to improve efficiency of the solid-liquid phase separation, a small portion of the surplus returned sludge is returned to initial sedimentation pool or Chemical coagulant substances are injected to wastewater at this point. The ability of loading organic pollutant substances depend heavily on the amount of sludge in the aeration pool, because the amount of sludge available at aeration pool depends heavily on the performance of secondary sedimentation basin (pool) under varying hydraulic loading conditions and returned sludge (Bedelians Golikandi, 2012).

MATERIAL AND METHODS

The purification works at Manger provide both primary and secondary treatment processes. Primary treatment removes most of the solids from the effluent, but doesn't remove or degrade the dissolved organic matter. Secondary treatment uses microorganisms to convert these organics to simple compounds, and uses the energy of the sun to destroy pathogens2. The effluent is then safe to be discharged into the Manukau Harbour. The entire process is shown diagrammatically in (Figure 1). The works have been designed to take advantage of the natural features of the site. Oxidation ponds provide very economical secondary treatment and these were chosen because a suitable area of harbor mudflats could be formed into ponds and because Auckland has the sunny climate necessary for the efficient working of the ponds. Conditions in the ponds promote the growth of unicellular algae: minute plants which, like any other plants, absorb carbon dioxide in daylight and give off oxygen by photosynthesis. This oxygen oxidase's the organics, thus purifying the sewage by reducing its oxygen demand.

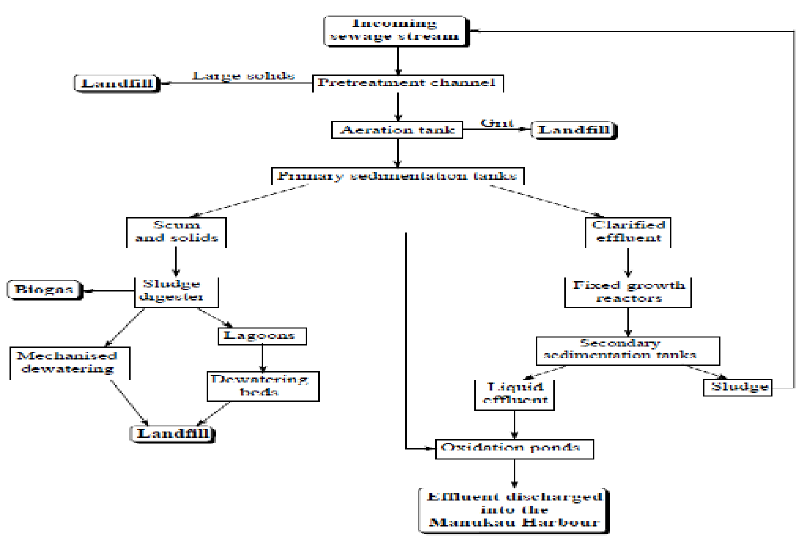

Fig. 1. Sewage treatment flow diagram.

Table 1. Typical wastewater treatment processes and removal method.

Treatment methods	**Removal mechanisms**	**Typical processes**
Physical Treatment	Screening **,** Filtering, Difference of gravity, Thermo-energy, Electric energy, Reverse osmosis	Screen**,** Filtration, Settling , flotation, Evaporation, drying, Electrolysis, Reverse osmosis membrane
Chemical treatment	Oxidation reaction**,** Reduction reaction, Double decomposition	Oxidation**,** Reduction, Neutralization, coagulation
Physical chemical treatment	Phase boundary potential**,** Adsorption, Ion exchange, Electrochemical reaction, Super critical phase	Coagulation-settling**,** Coagulation-flotation, Activated carbon adsorption, Ion exchange resin and membrane, Electric Dialysis, Electrolysis, Super critical water oxidation
Biological treatment	Aerobic decomposition**,** Anaerobic decomposition, Anaerobic-aerobic reaction	Activated sludge process, Denitrification, Phosphorous removal**,** Anaerobic digestion process**,** Denitrification, Biological phosphorous removal

The typical processes and removal methods are shown in Table 1 while the screened residues, Separated oil, sludge, etc. generated during wastewater treatment are partly used for livestock Feed, fertilizer, and other purposes, and they are primarily reduced in volume by dewatering, drying, or incineration for disposal as industrial waste (Eckkenfelder 1965). The conceptual relation between the treatment technologies and the treatment requirements in food processing factories are schematically shown in Fig. 1. As shown clearly in the Figure, the major process used for treating wastewater is biological. In the pre-treatment stage, a screen is often used to remove floating materials such as labels and plastic sheets. A gravity oil separator is provided for oil containing wastewater generated by edible oil production. After the pre-treatment stage, normal level BOD is decomposed by an aerobic biological treatment, while high level BOD of several thousands to tens of thousands is diluted prior to treatment. In recent years this high level BOD wastewater tends to be treated, without dilution, by an anaerobic biological process in the pre-treatment stage, and then re-treated by an aerobic biological process. Introducing an anaerobic biological process benefits by reducing the load for the later stage aerobic biological process, converting organic materials in wastewater into fuel gas, downsizing the settling tank because of not using diluting water, and preventing sludge bulking. The BOD removal rate in the anaerobic biological process is normally between 80 and 90%. Then, the remaining BOD is removed by the aerobic biological process, which has a removal rate in the 95 to 99% range. When a factory is located in a sewer-serviced area, an aerobically pre-treated wastewater can be discharged directly to the sewer. When the factory's location is in a non-serviced area and the effluent quality is regulated strictly, then a tertiary treatment is required to reduce BOD, COD, and SS. In such cases, sand filtration, and coagulation-flocculation- sedimentation and activated carbon absorption are, singly or in combination, added for the tertiary treatment (Bryant and Wiseman 2003). Urmia Wastewater treatment plant is constructed vicinity to Reyhan Abad village in the approximate altitude (1330 m) from sea level.

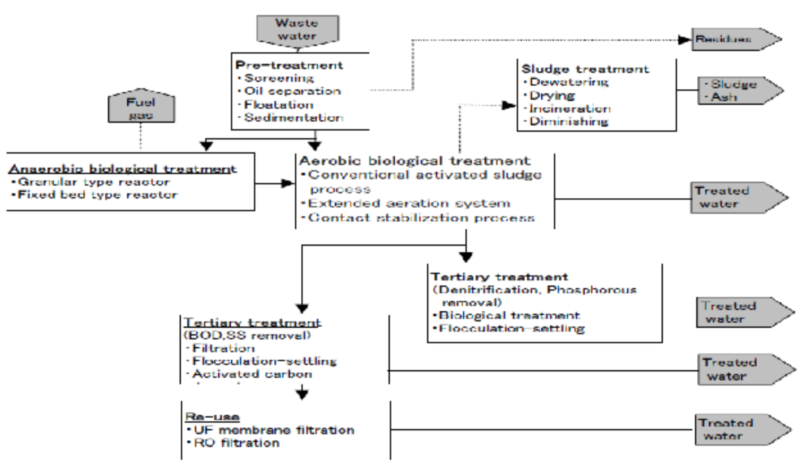

Fig. 2. Typical wastewater treatment system.

The average annual rainfall height in the city of Urmia is (370 mm), minimum temperature (-22°C) and the absolute maximum (38.4°C), minimum monthly average (-3.6°C), the maximum monthly average (31.31°C) and the average daily minimum (-2°C) and maximum (24°C). Its Area is (135 ha). Wastewater treatment plant is in five equal modules, each with a capacity of two hundred thousand people (200,000 people) and the population covered by the plant for the horizon year (1400) has predicted one million people. The amount of daily waste with acceleration is (0.575 square feet safely), the average daily wastewater without acceleration is equal to (4000 m) and a maximum of moment wastewater is 83,160 cubic meters and about 0.962 meters per second. BOD the effluent slop of this system is below 10 mg per liter, while in the other processes, BOD output is between 50 to 30 milligrams per liter. The length of sewage transmission lines is 51.5 km. Inlet pipe diameter is 1200 mm, length of input channels sewer is 5 m, sewer inlet channel width 2.5 m, there are two integrated Biolac poolin two modules with the same conditions, the length of the Bio Locke pool for each module is 120 mm, the width of Bio Locke pool for each module is 50 meters, the height (depth) of Bio Locke pool for each module is 4 m, the number of Fine screens is 6 with similar performance, the maximum capacity of fine screen is 170 liters per second, the number of pond sludge drying is 14, length of sludge drying pond is 140 meters, sludge drying ponds width is 22 m, height (depth) of sludge drying pond is 4 meters.

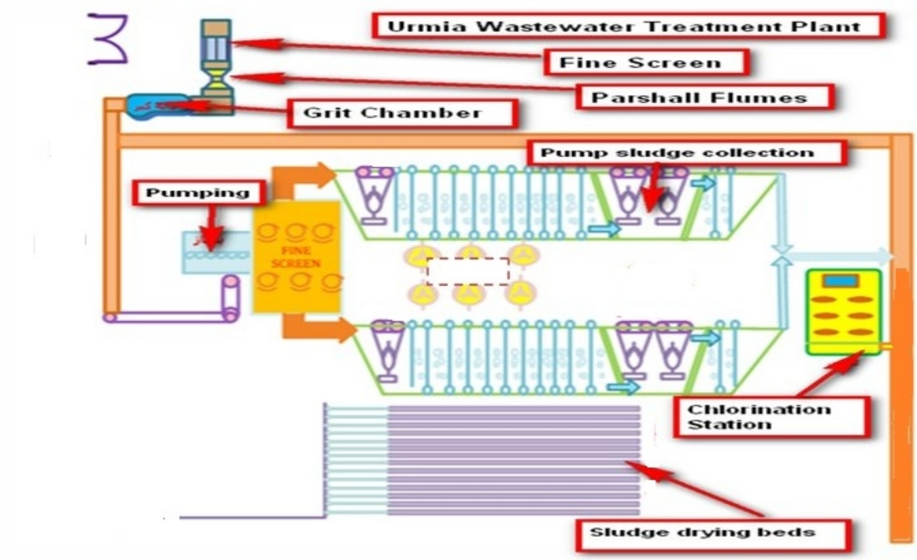

Fig. 3. Schematic picture of Urmia wastewater treatment plant.

The statistics of estimating the prospective population of Urmia in 2031 is equal to 1000.000 and about %5 of this population is considered for non-covered conditions of sewage refineries. For every person, the amount of output sewage is equal to 198 Lit per a second in 2006 according to the PBO (parliamentary Budget Officer) standards. Since of consuming increase, from 1 percent upon the output sewage amount is added for coming years , but with regard to the designation condition of the 2031 about %1.05 extra amount (for non-water leakage condition) is considered. The amount of leakage per a cubic meter of the length of the network is equal to 23 cubic meters per a day for every cubic meter. By summing up of these values, the total average value of the generated sewage in 2016 will be equal to 0.883 cubic meters per a second and in 2031 it is estimated to be equal to 2.529 cubic meters per a second. Table 2 illustrates the amount of current population and perspective population. To predict the future of city's population comprehensive and detailed plans are used. Numerous items such as population growth trend in previous years, civil and industrial programs considered for future, immigration possibility (receive or repel) and current population and its change, has effects.

Also, Table 2 shows the amount of leakage, average sewage and total average sewage. The average daily total usage during a year for a person of city's population is called average usage per capita. Screening equipment is located in the start of physical treatment (mechanical). Type, combination and the amount of rubbish is related to incoming sewage's combination, network pipes' steep, the vastness of covered zone and special conditions of the area. Based on internal standard of Iran, the speed between rods in mechanical cleaners is 0.6 to 1 m/s, rod's width is 8-10 mm, depth of rods is 50-75 mm, the open space between rods 10-50 mm, steep of rods to horizon 75-85 degree, and allowed height drop is 150 mm.

Table 2. Technical specifications of a wastewater refinery of Urmia city (Mohammadi and Rahimzade 2013).

Year	2006	2016	2021	2031
Population of city (people)	580000	707,000	780,000	1,000,000
Population covered (people)	350,000	650,000	650,000	950,000
Sewage with no leakage water (person liter per days)	198	203	203	210
Leakage water (cubic meters per day)	23	23	23	23
Average wastewater (Lit/day)	218	223	228	256
The average of total produced wastewater (Lit/Sec)	883	1678	1955	2529

RESULTS AND DISCUSSIONS

A. The hydraulic design of the grit chamber

Grit removal is the elimination of sands, solid and non-organic materials in wastewater. The lack of elimination of such materials could lead to clogged pipes and channels, the mechanical equipment erosion, the reduced productive volume of other treatment units and the increased sludge discharge in sludge digesters. In order to remove the grits three types of removers with horizontal flow grit chamber, the aeration grit chamber and the vortex flow grit chamber are used. The hydraulics of the grit chamber does not differ in the systems with fat removers or without removing the fat. The facilities used to separate sand and granular materials from waste water containing mineral grains include deep grit removers, the grit remover channels (the longitudinal grit removal), the circular grit removers and the grit removers equipped with aeration.

B. The basics of design and selecting the grit removers

According to Stein (1983) in eleven municipal wastewater treatment plants in Germany, in dry weather, there are 10-60 g/m3 granular mineral contents in the wastewater inlet and the diameter of the minerals is 0.09-3 mm. Therefore, the facilities and equipment must be designed in such a manner that the bulk of fine sand grains are separated from the wastewater. The amount of deposited materials is considered as 0.02-0.2 liters per cubic meter of wastewater. This value is four liters per person per year in average the highest amount of which id due to rainfall, especially after a period in the summer. In such cases, the material greater than 0.3 mm deposited in the pipes of network can be washed and discharged in the water treatment plant.

According to Londog theory, the maximum monthly amount equals 0.2-4.9 liters of grain per person. Eymhaf suggests two liters per person per year, for areas with high population density, and for other areas five liters per person per year. Choosing the appropriate grit remover depends on various factors, the main ones of which include operating and executable index. The operating index includes the amount and volatility of waste entering the plant, the separation of different granular deposits, the sand grains and other minerals in the wastewater, the type of equipment related to the inflow of the grit remover, The method of the transference of grain deposited in the plant and the location required for the various facility units and other processes required in grit removal stage, the activities required for the maintenance of facilities and using the such facilities as a biological pre-stage treatment .

C. Hydraulic flow within the grit chamber with horizontal flow

This grit removal system is used in small plants. In this case, the wastewater flow moves slowly and the grits are deposited with a free fall to the bottom of the channel. The grit removal channel is built with a mild, no or negative slope which is designed for the washing or discharge. Other related features such as the surface load and the depth and length of the grit removal channel are obtained based on the velocity of falling particles. The channel length is a function of depth necessary for sedimentation and usually considering an additional length equal to the least theoretical length percentage is optimal for compensating the turbulence in the inlet and outlet wastewater.

D. Designing the grit chamber

In order to design the grit chamber of the wastewater treatment in Urmia city we consider the following steps:

Two grit removal units with spiral flow that each remover would be designed for half the maximum instantaneous flood. The minimum aeration capacity is considered 7.8 lit/sec within the channel length and the width of each channel is 3.5 m based on Table 3(2.5-7m). Velocity at the inlet and outlet structures should be in the range of 0.3 meters per second and retention time (2 to 5 minutes) is 4 minutes in this case. Since the very low velocity within the grit removal channel, the height loss within the length is insignificant and the height loss is determined based on the inlet and outlet discharges. Two grit removal channels work independently. So the air injection diffusers are placed within the channel and 0.6m above the bottom. The channel bottom is considered with a mild slope which is built toward the grit collecting channel on one side of it. The velocity of flow rise when the two channels are working equals:

The surface area of each channel: $= 3.5\times 13 = 45.5m^2$

The surface rise $\frac{m^3}{m^2.day} = \frac{0.481(\frac{m^3}{sec})\times 86400(\frac{sec}{day})}{45.5(m^2)} = 913.372$

This value is considerably lower than the flow rise velocity limit to remove particles larger than 0.21mm diameter (1730 cubic meters per square meter per a day or 1.2m per a minute). The surface rise velocity in the case that one of the grit removal channels is being serviced and the Flows pass a channel equal:

The surface rise velocity $\frac{m^3}{m^2.day} = 2\times 913.372 = 1826.75$

Table 3.The results of hydraulic design of the grit removal channel in Urmia the wastewater treatment.

The maximum instantaneous flood for each grit removal channel	0.481 m^2/sec
The depth of the grit removal channel	4.45m
The ratio of length to width of the channel	1:4
The length of the grit removal channel	13m
The width of the grit removal channel	3.5 m
The surface area of each grit removal channel	45.5 m^2
The exact retention time in maximum instantaneous flood when both units are functioning	5.46 min
The exact retention time in maximum instantaneous flood when one of the units is functioning	2.88 min

Table 4. Common specifications for the design of the various parts of the grit removal channel with aeration.

Design factor	**The allowed range**	**descriptions**
Depth	2-5m	The width of the grit removal channel is limited to provide the possibility of spiral movement in the result of air injection
Length	7.5-20m	
Width	2.5-7m	
W/D	1:5-1:1	
L/W	1:5-1:2.5	
Surface load	0.6-0.8m/sec	-
The retention time in maximum instantaneous flood	2-5 min	If the grit removal channel is used for the initial aeration and the elimination of particles with diameter above 0.21mm is considered the retention time is greater
The inlet and outlet structure	-	The inlet and outlet structures must be designed in a way that prevents the formation of short connected flows or turbulent flow. The inlet must enter the flow into a spiral pattern into the channel. Also the outlet has a 90 degree angle with the inlet flow (fig 7-8). The inlet and outlet structure are in a way that they keep the velocity at 0.3 m/sec.
The barrier pages	-	The longitudinal and transverse pages help the spiral movement of the flow and increase efficiency of grit removal. If the length is higher than the width using the barrier page is necessary.
Channel geometry	-	The air diffusers, channel bottom slope, the well and the facilities of collecting the grits affect the geometry of the channel.

E. The geometries of the input structure

For the input structure the submerged inlet channel with 1m width is used so that the flow is divided into two grit removal channels. Each channel has an aperture with 1*1 dimensions that lead the flow submerged into the grit removal channel close to air injection diffusers. A barrier page is considered within each channels inlet to divert the flow into the spiral pattern. The energy relation is as follows:

$$\Delta H = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} + h_l$$

Where:

V_1: The average velocity in the inlet channel (meters per second)

V_2: The average velocity in the grit removal channel (meters per second)

ΔH : The height difference between the open water surface in the input channel and the grit removal channel (meter)

h_l: Height loss in the channel and the inlet channel.

Since the height loss in the inlet channel and velocity height have insignificant values, h_l is approximately equal to the piezometer difference in cross-section. Therefore it is possible to replace the h_l and ΔZ (The difference in the discharge of the water level in both sides of the inlet valve into the grit removal channel). So according to the dimensions of the aperture, the required height which is ΔZ is gained for the Q discharge.

$$Q - C_d \times A \times \sqrt{2g\Delta Z} \quad \ldots(2)$$

Where:

A is the sectional area of the aperture (square meters)

C_d indicates the discharge factor which is 0.61 for the angular square.

Generally V_1 and V_2 are small values and the value of $[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}]$ is ignorable. In other words for the maximum instantaneous flood we have:

$Q = C.L.H^{\frac{3}{2}} \Rightarrow 0.481 = 0.61 \times 3.5 \times H^{\frac{3}{2}} \Rightarrow H = 0.38m$

So the hypothetical water depth equals:

Hypothetical water depth: 3.65+0.38+0.1= 4.13m

The hypothetical water depth is 4.13. the width of the channel is 1m. So we have:

$$V_1 = \frac{0.962}{1(\text{Channel width}) \times 4.13(\text{The hypothetical water depth})} = 0.23\,\frac{\text{m}}{\text{sec}}$$

$$V_2 = \frac{0.962}{3.5(\text{The width of the grit removal channel}) \times 3.84(\text{The hypothetical water depth in the grit removal channel})} = 0.071\,\frac{\text{m}}{\text{sec}}$$

This is insignificant and ignorable. In any case by mixing the discharge and water level loss the Equation (3) is obtained:

$$\Delta H = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \left(\frac{Q}{C_d A\sqrt{2g}}\right)^2 \quad \ldots(3)$$

In the designed maximum instantaneous flood of 0.926 m^2/sec, when one of the channels is working we have:

$$\Delta H = \left[\left(\frac{0.962}{0.61 \times 1m \times 1m \times \sqrt{2 \times 10}}\right)^2\right] = 0.121m$$

By choosing one input with greater dimensions or factor the value of ΔH is reduced.

The outlet structure consists of a rectangular weir structure, drainage, an outlet box and an outlet pipe. The outlet weir is 2.5 long and the dimensions of the drainage equal 1.5 m (width) *2.5 m (length). The outlet weir is shared between two grit removal channels and considered as 2.3*1.5 m. The weir length must be less than the width of the grit removal box, so the weir length is 2.5. The adjustable valves are installed on the outlet box so that they prevent the flow from entering them when each one of the channels is not working. The outlet pipe leads transfers the flow into a dividing box that divides the wastewater into the initial sediment tanks. The height above the outlet weir is calculated with the maximum instantaneous flood when both channels are working. The height above the outlet weir is calculated using the following relation:

$$Q = \frac{2}{3} C_d L' \sqrt{2gH^3} \quad \ldots(4)$$

Where:

Q: The flow discharge on the weir (cubic meters per second)

H: The height of the weir (m)

C_d: The discharge coefficient which is equal 0.6

L' is the effective length the value of which is calculated by $L' = L - 0.1nH$. In this calculation L is the weir length and n is the number of contractions (shrinkage or inlet) which equals 1. In the maximum instantaneous flood when both channel are working (the discharge equals 0.481 cubic meters per a second) based on the trial and error method we assume $L' = 2.44$ in the first trial:

$H = 0.23m \Rightarrow L' = 2.5 - 0.1 \times 0.23m = 2.477m$

The second trial is done by the value obtained for L':

$H = 0.227m \Rightarrow L' = 2.5 - 0.1 \times 0.227 = 2.477m$

The value of $L'$is obtained by the second trial. Therefore the depth of the water above the weir equals 0.23m. Thus the height of the weir crest above the grid removal channel bottom equals:

The elevation of the crest above the bottom= 3.65-0.23 = 3.42m.

In the case that the whole flow is passed through one of the channel we have:

$$L' = 2.46m \Rightarrow \left[\frac{0.481\frac{m^3}{sec} \times \frac{3}{2}}{0.6 \times 2.46 \times \sqrt{2 \times 10\frac{m}{s^2}}}\right]^{\frac{2}{3}} = 0.228m$$

$L' = 2.5 - 0.1 \times 0.228 = 2.476m$

The water depth in the channel in the maximum instantaneous flood when one of the channels is out of service equals:

Water depth = The elevation of the crest above the bottom of the grid removal channel+ the height above the weir = 3.37+0.45=3.82

F. Calculating the output drainage

In order to obtain the depth of maximum water in the collecting channel we consider that one of the grid removal channels is out of service and all the flows pass through one of the channels. In addition, given the fact that the water level in the outlet box is determined by the downstream, in this case the water depth in the point of flow discharge into the outlet box (the downstream end of the collecting channel) is constant and equals 1.5m. Thus, the water depth in the upstream end of the channel y_1 is obtained by the relation (5):

$$y_1 = \sqrt{y_2^2 + \frac{2Q^2}{g.b^2.y_2}}$$

(5)

If the value of y_2 equals 1.5m and the acceleration of gravity is equal to 10 square meters per second, we have:

$$y_1 = \sqrt{1.5^2 + \frac{2\times0.962^2}{10\times1.5^2\times1.5}} = 1.518m$$

To consider the effect of friction we usually increase the value of y_1 12%. In addition 0.15m is added to the channel height in order to consider the free height. So the height of the whole collecting channel equals:

The height of the whole collecting channel=$1.12\times 1.518 + 0.15 = 1.85m$

G. Total losses of hydraulic load in the grid removal chamber

The loss of hydraulic load in the grid removal chamber is obtained by the Loss of hydraulic load in the inlet structure, the Loss of hydraulic load in the outlet structure, Loss of hydraulic load in the grid removal channel and the Loss of hydraulic load in the barrier pages. The loss of hydraulic load in the grid removal channel is due to low velocity of the flow and the short length of the channel is insignificant. For the Loss of hydraulic load in the inlet and outlet barrier pages that block the flow the following relation is considered:

$$h_L = C_D \times \frac{V_2^2}{2g} \times \frac{A_b}{A} \qquad \ldots(6)$$

Where:

h_l: The height loss created by the barrier page (baffled) - (m)

V_2: Horizontal velocity component in the non-inhibited grid removal channel (m/sec)

A_b: The vertical image of the barrier surface (square meters)

A: The surface area of the grid removal cross section (square meters)

C_D: The drag coefficient that equals 1.9 for the pages.

If the barrier page covers 50% of the area of the grit removal channel, the hydraulic load loss caused by the barrier page in maximum instantaneous flood when one of the channels is out of service is calculated as follows:

The velocity within the grid removal channel: $=\frac{Q}{A} = \frac{0.962\frac{m^3}{sec}}{3.5m(\textbf{Width})\times3.84\text{m}(\textbf{Depth Flow})} = 0.074\frac{m}{sec}$

Thus the loss of hydraulic load is calculated through the relation (5-12) as follows:

$$h_{L=}\frac{1.9\times0.074^2}{2\times10} \times \frac{0.5}{1} = 0.0005m$$

The hydraulic load loss is ignored due to the insignificance of the energy waste.

SUGGESTIONS

(i) All the design elements is use of empirical approaches to other countries and no design value is reassessed in the area. It is proposed to be designed to determine the coefficients in this country compared to other countries, practical design values can be obtained.

(ii) Using precision measuring input and output water quality in different types of wastewater treatment systems.

(iii) Evaluation of climate and environment, the efficiency of wastewater treatment.

(iv) The effect of each of the sewage treatment plants reduces pollutants in wastewater.

(v) Different regionalization of water and wastewater based on quality and offer early treatment (treatment operations) before entering the wastewater treatment plant.

(vi) Optimization methods for wastewater treatment in communities large and small, based on the economic value of the Value Engineering Approach

(vii) Information and Culture of wastewater treatment techniques in the consumer society.

REFERENCES

Afuoni. M, Erfanmanesh. M, (2012). “Environmental Pollution”Arkaneh Danesh Publications, Isfahan, Iran.

Bedelians Golikandi.G. (2012). “Wastewater Treatment Plant Designing”, Aeezh Publication, Tehran,Iran.

Bryant C and Wiseman C (2003). Efforts to predict BOD for process control. *In Proceedings, Tappi International Environment Conference.*

Eckenfelder, (1965). Biological Wast Treatment, translated by Iwai, p 68 Korona.

Gourjar, B.R. (2012). “Sludge Treatment and Disposal”, Mokhtarzadeh Azar. R (Translator), Hafeez Publishing, Rural Water and Wastewater Company of West Azerbaijan Province, Iran.

Metcalf and Eddy, (2011). “Wastewater Engineering”, A. Abrisham, and et al, Markazeh Nashre Daneshgahi Publishing, Design and Research Consulting Engineers Company of Isfahan Water and Wastewater, Iran.

Mohammadi R. Rahimzade MH. (2013). Environmental Report for Wastewater Treatment Plant. Shafab Consulting Engineers. Urmia. Iran.

Monzavi.MT, (2011). “Municipal Wastewater”, Tehran University Publications, Iran.

Nadafi. K, (2011). “Wastewater Treatment”, construction and training organization, the Ministry of Energy, Iran.

Sarafraz, S., Khani M.M. and Yaghmaeian K. (2007). Quality and quantity survey of hospital wastewaters in Hormozgan province. *Iran. J. Environ. Health. Sci. Eng.,* **4**(1): 43-50.

Tovrovski, E.S. and Maathai, P.K. (2010). “Treatment of Sewage Sludge", Mousavi. GR (Translator), Hafeez Publications, Iran.