Project One

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CE 441-1

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Contents

1 FACTORS

The growth of town's population is expected to increase to 11,000 over the next 10 years, which is the lifetime of this project. The population is estimated to increase by 6,000 people due to a new steel mill opening. The steel mill will not produce any excess waste that changes the estimated GPCD and BOD. Brookesville is in the outskirts of Houston, Texas. The mean temperature of the coldest month in Brookesville is 57° C.

The present sewer system consists of an 8 inch diameter trunk concrete sewer line, the friction factor n = .013 and the slope of the pipe, s = 0.004, which discharges the town's wastewater into a lake about 1 mile outside of the town. The invert elevation of this 8 inch line at the point of discharge is 90 ft. above sea level. This sewer is dilapidated, and will be replaced.

2 Introduction

There will be three new ponds, each with equal "person capacity". The ponds will be constructed adjacent to each other with piping designed so tat they may be run in series *or* in parallel. Two pumps have also been designed in order to empty one pond into the other if necessary. The lagoon will be on line with the existing sewer and the effluent weir will be 300 yards from the lake so as not to interfere with the environmental quality adjacent to the lake. Unobstructed wind sweeps will be available at this location. The slope of the overland surface at this site to the stream is negligible, and the elevation is 100 ft. The estimated sewage flow, $(Q_{flow}$, is 91 gpcd. This estimate has a 20% contingency factored in. This number was determined using the population, the history of Brookesville's GCPD, and comparison against other towns with similar characteristics.

This new sewage system is designed to achieve 85% BOD removal. The effluent BOD will be 32.25 mg/L. Each oxidation pond is designed to be 735 ft x 386 ft, the depth of water in each pond will be 4', and the total depth of each pond will be 8'. The embankments around each pond have an inner slope of 6:1 and an outer slope of 3:1. This embankment allows a 4 foot freeboard to account for any waves and has 10 feet of width between each pond to account for any maintenance or machinery (i.e. mooring machines).

PID 0 accounts for an overview of the new system, from town to lake. PID 1 accounts for the piping and schematics of the new oxidation ponds. PID 2 accounts for the excavation schematics. PID 3 accounts for the weir and manhole schematics. PID 4 accounts for the emergency pump and manifold schematics. PID 5 acounts for the head loss schematics.

3 Oxidation Ponds Dimensions and Reasoning

REFER TO P&ID 2 FOR FIGURES

$$v = 10.7 * 10^{-8} NqY(1.072^{(35-T)}$$

Where v = the total volume needed to move flow for 4-5 weeks of treatment in Acre-Ft

N = Total population

q = gpcd

Y = 5 day BOD in mg/l

T = mean air temp of coldest month (in $\circ Celsius$

85-90% BOD removal is assumed under this design.

$$v = 10.7 * 10^{-8}(11,000))(91)(179)(1.072^{(35-15.55)}$$

The total volume $v = 74.38 = 75$ acre-feet

The volume of each pond is 25 acre-feet, or 30,837 m^3.

The depth of the pond is 4 feet. This is to account for rain, and any potential overflow. This depth can easily be mixed and airated by wind action. There will be a 4 foot freeboard to account for any possible waves that could occur in the ponds. For maximum efficiency, the length of the pond is twice its width. First we divide the calculated volume of an individual pond by the depth of the pond:

$$A_p = \frac{25}{4} = 6.2Acres$$

$$A_p = 6.2acres * 43,560\frac{ft}{acre} = 27,000ft^2$$

The area of each pond in $feet^2$ is 27,000 ft^2.

Width (W) = $\sqrt{\frac{area}{2}}$ = 368 ft = 109 m

Length (L) = $2 * W$ = 735 ft = 224 m

4 Excavation Calculations

REFER TO P&ID 2 FOR FIGURES

The excavation is compacted 80%, so the expanded volume will be 120% of the compacted volume. The embankments have an inward slope of 6:1 and an outward slope of 3:1. The width of the embankment is 10 feet, which accounts for mooring machines, and other maintenance operations. The freeboard is 4 feet to account for any potential waves that could occur in the pond. This makes a total depth of 8 feet, from the bottom of excavation to the top filled point. Reference PID 2 for details. The calculations for the excavation depth and total volume that needs to be excavated are as follows:

4′ water depth + 4′ freeboard = 8′ = H + x

Where H= the height above ground and where x = the excavation depth.

B (the length between the deepest points of the water) can be represented by the following equation:

B = 10′ +9H = 10′ + (9)*(8) = 72′

Fill 1 is the dirt required for the surrounding embankments.

$$\text{Fill 1} = [\frac{B'+b'}{2} + H'] * L_{perimeter}$$

$$\text{Fill 1} = [\frac{72'+10'}{2} + 8'] * (368 * 2 + 735 * 2)$$

$$\text{Fill 1} = 1.08094x10^6 ft^3$$

b = 10 + 12H = 10 + 12(8) = 106′

Where b = the length between the top of the freeboard

Fill 2 is the dirt required to build the interior embankments.

$$\text{Fill 2} = [\frac{B'+b'}{2} + H'] * L_{perimeter}$$

$$\text{Fill 2} = [\frac{106'+10'}{2} + 8'] * (368 * 2 + 735 * 2)$$

$$\text{Fill 2} = 1.4x10^5 ft^3$$

$$\text{Tot. Fill} = \text{Fill 1} + \text{Fill 2} = 1.23 * 10^6$$

Tot.Fill * 0.8 = the total volume needed to excavate and build the three ponds

Tot.Fill = 9.84*10^5

With this information, we can calculate the depth of excavation needed, x.

$$B = (W + 12x)(L + 12x)$$

$$\frac{(368+12x)(735+12x)+x}{2} * 3 = 9.84 * 10^5$$

$$X = 2.5 \text{ ft} = 0.8 \text{ m}$$

We must excavate 2.5 feet in order to have the correct amount of dirt to fill the embankments.

5 Weirs, Manholes, Piping, Pumps, and Drainage Calculations and Reasoning

The inlet pipe will be the newly installed 18" pipe. The length piping has already been excavated to accommodate the manholes (shown in section 3), so this ultimately was the most cost-effective choice. This pipe has been chosen to be 18" since the slope is 0.004 ft/ft, and the max flow is 4.65 cfs. This is shown in the upcoming section. This implies that the velocity is 3.2 ft/sec, according to the Nomograph for Manning Formula.

The initial flow rate has been determined to be 91 gcpd. We converted the flow rate to cfs using the following method:

$$(11,000people) * (91gcpd) = 1,001,000gpd$$
$$(1,001,000) * (0.0000015) = 1.55cfs = 0.044cms$$

This number is used to calculate the time it takes for this process to complete:

$$t = \frac{V_{ponds}}{Q_{avg}}$$

Where v_{ponds} = 3,267,000 ft^3

The max average flow of this system through the 12" pipe is

$$Q_{max} = Q_{avg} * 3 = 1.55cfs * 3 = 4.65cfs = 0.132cms$$

$$t = \frac{3,267,000ft^3}{1.55cfs} = 24.5days = 3.5weeks$$

Thus the total time for the system to fill and circulate is 3.5 weeks.

5A Manhole Calculations

REFER TO P&ID 3 FOR FIGURES

Due to the size of the town being on the smaller side, the manholes were not designed with a pump. The manholes are to be places approximately 500 feet apart, with a 6' dirt cover atop of the new piping.

Manhole Height

H = Ground Cover + Water Depth Above Excavation+ Head Loss

H = 6′ +5.5′ +2′ = 13.5′ Manhole Depth = 4.11 m Manhole Depth

Length of Inlet Pipe Total Distance = $\frac{13.5'}{4'} * 1000 = 3,375ft = 1028m$

Number of Manholes Backwater Curve: $\frac{7'}{4'} * 1000 = 1750ft = 533.4m$

Number of Manholes: $\frac{1750}{500}$ = 4 Manholes

5B Manifold Calculations

REFER TO P&ID 4 FOR FIGURES

The manifold was designed to be able to withhold Q_{max}, which is 4.65 cfs. This is the most extreme scenario, as the average cfs is expected to be 1.55 cfs. This has a 200% contingency. The manifold pipe is an 18" pipe. This pipe has a total length of 50′ at the manifold, 600′ from the manhole to ponds 1 and 3, 300 feet from the manifold to pond 2, and has 2.5" diameter holes.

The number of ports on the manifold can be calculated by diving the manifold length by the distance between each port, 1.5". Thus there are 33 ports.

The design velocity of the manifold is 10 fps, as the average velocity is 3.2 ft/s, and 10 ft/s has just above a 200% contingency factored in.

Using the manning formula, we can determine that the slope (n) = 0.003. We can find the head loss between the manhole and each pond with the equation:

$$h_1 = L * n$$

$$h_1 = 600 * 0.003 = 1.8 \text{ ft}$$

The same can be done for the other ponds.

The flow for each port can be found by dividing the velocity by the number of ports: $\frac{10cfs}{33ports}$ = 0.3 cfs

The area of each port is: $3.14 * (1/12)^2 = 0.021ft^2$

The velocity of the liquid through each pipe is thus:

$$v = \frac{q}{A} = \frac{0.3}{0.021} = 13.75ft/s$$

Now we can find the head loss for the length of pipe between each port:

$$h_{Lport} = K\frac{V^2}{2g}$$

Where K = 1.0, v = the velocity, and g is gravity.

$$h_{Lport} = 1.0\frac{(13.75fps)^2}{2(32.2\frac{ft}{s^2}}$$
$$h_{Lport} = 2.93 = 3ftofheadloss$$

The head loss to the next port can be found:

$$h_{L2} = 10/2cfs = 5cfs$$

Using the Mannings Normagraph, we can determine that the slope must be 0.0025 ft/ft.

Thus, h_{L2} = 50*0.0025 = .125 per port.

The total head loss is $h_{Lport} + h_{L2} + h_1 = 3 + .125 + 1.8 = 4.925ft = 1.5m$

$\frac{0.125}{4.925} = 2.5\%$ which is well under the maximum of 20%.

5C Weir Calculations

REFER TO P&ID 3 FOR FIGURES

The length of each weird crest can be calculated with the equation,

$$Q_{cfs} = 3Lh_w^{3/2}$$

Where Q_{cfs} is the average flow in cubic feet per second, L is the length of the weir, and h is the weir head in feet for surface skimming.

$$1.55cfs = 3L * (1/12)^{3/2}$$

Cfs per pond = 1.55/3 = 0.51 cfs per pond

L = 7.16 ft = 2.2 m

The depth of the weir box is found next using the equation:

$$H_o = h_o + .25h_o = 1.25(\sqrt{h_l^2 + \frac{2Q_{cfs}^2}{gb^2 h_l}})$$

Where h_l is the diameter of the pipe plus a 2" contingency, Q_{cfs} is the flow rate in cubic feet per second per pond, g is gravity and b is the width of the weir channel. The width of the weir channel was chose to be 1.5 feet, based around the average size of the town, with a slight overestimate for contingency.

$$H_o = 1.25(\sqrt{(\frac{5}{6})^2 + \frac{2(0.51ft^3)^2}{(32.2\frac{ft}{s^2}(1.5ft)^2(\frac{5}{6}}})$$

$$H_o = 1.026 \text{ ft} = 0.31 \text{ m}$$

5D Emergency Pump Calculations

REFER TO P&ID 4 & 5 FOR FIGURES

There are two emergency pumps, one situated between ponds 1 and 2, and the other situated between pumps 2 and 3. These pumps were designed to empty one pond into another in 1-2 days. Both pumps are completely reversible, and are situated above the pond, causing the static height difference to be about 8 feet. We can calculate the flow required through the pump with the equation:

$$Q_{avg} = \frac{v_{pond}}{t}$$

$$Q_{avg} = \frac{12.5acre-ft}{2days}$$

$$Q_{avg} = 272,250\frac{ft^3}{day} \; Q_{avg} = 75.5cfs = 2.13cms$$

The pump used must have a minimum of 75 KW capacity. This power has a 20% contingency factored into it.

The total pump head forward is 18' + 8' = 26' H_{pf}

The total pump head reverse is 28' H_{pR}

FORWARD PUMP

B,D open: K = 0.2

4 Tees: K = 0.4

$$h_l f = \frac{1.55^2}{2*32.2}(0.2*2)(0.4*4) = 0.075$$

$$HP_f = \frac{Q_{pump}*(H_p f + H_p r)*\gamma\omega}{550}$$

$$HP_f = 46.5ft = 14.2m$$

REVERSE PUMP

A,C open: k = 0.2

8 bends: K = 0.25

$$h_l f = \frac{1.55^2}{2*32.2}(0.2*2)(0.25*8) = 0.090$$

$$HP_r = \frac{Q_{pump}*(H_p f + H_p r)*\gamma\omega}{550}$$

$$HP_f = 49.4ft = 15m$$

6 Cost Estimate

Everything will be multiplied by a factor of 0.85 due to Brookesville being just outside of Houston, whose cost adjustment is 0.82.

6A Cost of Raw Wastewater Pumping

C = $(4.43x10^4)Q^{0.58}$

Where Q is the design flow. Thus,

$$C = 0.09 \text{ MOD (millions of dollars)}$$

6B Sitework Including Excavation

C = $(1.96 * 10^5)Q^{0.66}$

Where Q is the design flow. Thus,

$$C = 0.48 \text{ MOD (Millions of Dollars)}$$

6C Cost of Mobilization

$$C = (6.34 * 10^4)Q^{0.69}$$

$$C = 0.13 \text{ MOD (Millions of Dollars)}$$

6D All Piping

$$C = (12.23 * 10^5)Q^{0.77}$$

$$C = 0.6 \text{ MOD (Millions of Dollars)}$$

6E Stabilization Pond Cost

$$Q = (7.08 * 10^5)Q^{0.67 * 3}$$

$$Q = 0.76 \text{ MOD (Millions of Dollars)}$$

6F Effluent Outfall - Outfall to Non-Ocean Surface Water Cost

C = $(6.10X10^4)Q^{0.66}$

C = 1.1 MOD (Millions of Dollars)

6G Intermediate Effluent, and Raw Wastewater Pumping Cost

C = $(4.43 * 10^4) * Q^{0.58}$

C = 0.9 MOD (Millions of Dollars)

6H Excavation Cost

C = $(1.33 * 10^5) * Q^{0.64}$

C = 0.34 MOD (Millions of Dollars) x 2 to account for embankments

C = 0.68 MOD (Millions of Dollars)

6I Piping Cost

21" pipe: $135.5 * (5280 + 2160) = 1,008,120 dollars = 1.01 MOD$

6J TOTAL COST

The total cost of this project is set to be:

$((0.82 + 0.09 + 0.48 + 0.13 + 0.6 + 0.9 + 1.1 + 0.76 + 0.68 + piping) * 0.82) = 4.89$ Million Dollars, plus 25% for the cost of engineering:

(4.89*1.25) = 6.109 Million Dollars (todays inflation not accounted for)