Project Three

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CE 441-1

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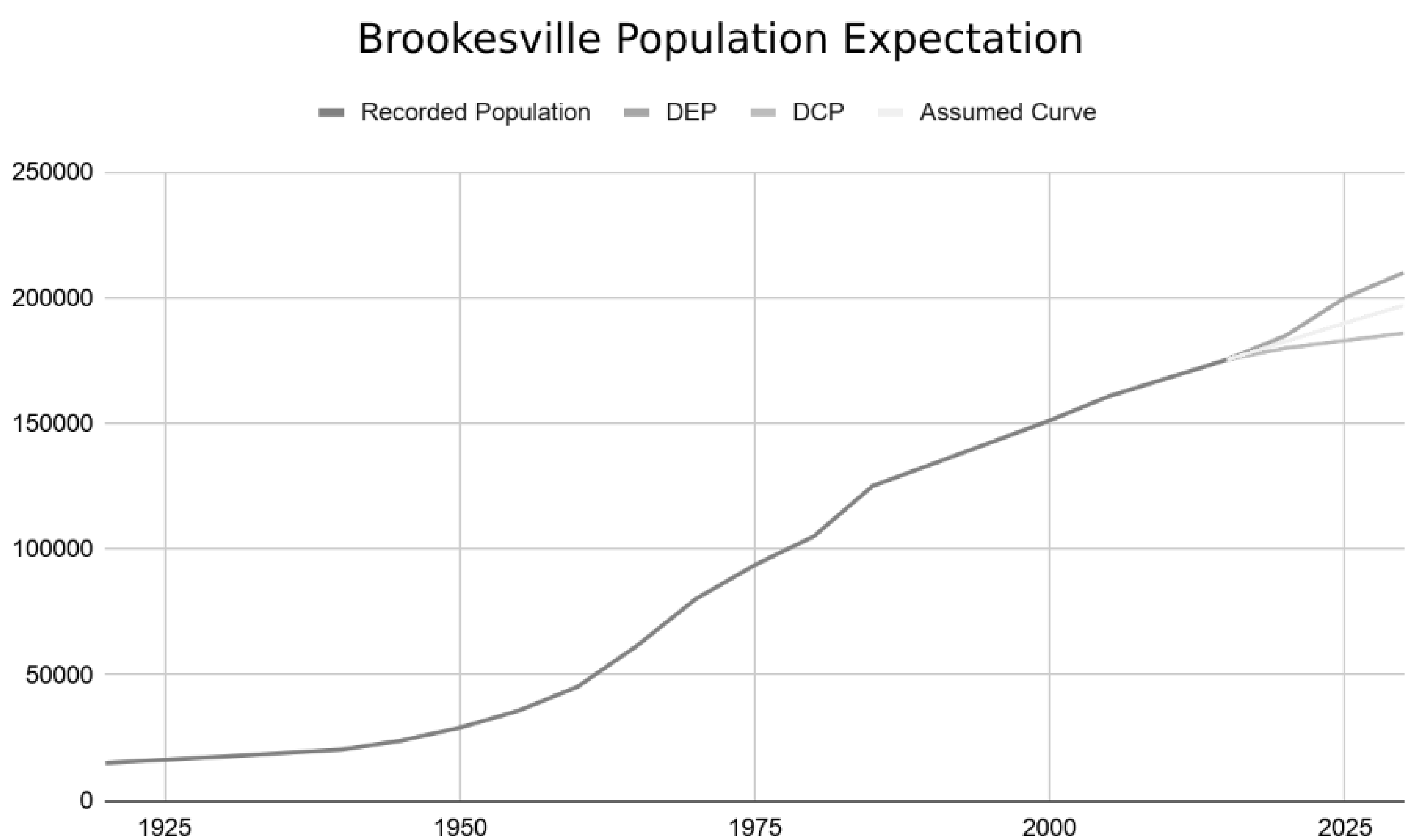
Wastewater Treatment Project 3

1 Introduction

Cooper Engineering Corporation has been contracted to design a new primary treatment plant for the city of Brookestown outside of Orlando, Florida. This treatment plant will consist of a partial flume, bar screen, grit chamber, pumping well and primary sedimentation tank. It was designed with a minimum lifetime of 12 years. Brookestown's population is currently 125,000 residents. These are full-time, year-round residents. Brookestown is expanding rapidly as a new nuclear plant has brought many jobs, businesses, and families to the area. Brookestown is expected to have a population of 174,000 by the year 2034. This is a slight overestimate according to the the growth rates of similar cities with new industries, meaning it *should* have a lifetime longer than 12 years.



2 Wastewater Flow Rate

The following table breaks down the wastewater flow rate of Brookestown:

Type of Establishment	Quantity	Unit Flow	Total Flow
Airports	1	398000	398000
Assembly Halls	5	75000	375000
Bars and Taverns	11	1130	12430
Bars and Taverns	7	27000	189000
Church small	6	80000	480000
Dance Halls	3	24500	73500
Factories (dry)	6	116000	696000
Factories cafeterias	5	130000	650000
Hotel,luxury	4	11200	44800
Hotel, avrage	5	43000	215000
Hospitals	4	24000	96000
Laundry	21	9300	195300
Nuclear Power Plant	1	1000000	1000000
Offices	9	240000	2160000
Restaurant 24h	23	3750	86250
Restaurant, average	28	13750	385000
School, day	3	205000	615000
School, elementary	6	162250	973500
School, high	8	184500	1476000
Service Stations	105	1300	136500
Shopping Centers	1	653400	653400
Theaters	3	**25000**	75000
Residential	1	**23100000**	23100000
		Total	34.09

The residential flow rate is expected to be 34.1 MGD.

3 Interceptor Design

Before any wastewater enters the treatment plant, the towns piping intercepts at one large interceptor. There is a manhole that is close to the interceptor with a large pump that lifts the wastewater a δH of 35 ft. After the first manhole, manholes are built every 300 feet. The interceptor has the ability to handle the town's maximum and minimum flows. The manning equation was used to calculate the minimum size diameter of the pump so that it can perform its duties.The detailed diagram of the interceptor can be found in PID 1.

$$Q_{maximum} = \frac{1.49}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}} \text{ where}$$

Where $Q_{maximum}$ = Maximum flow to the Treatment Plan

n = mannings friction coefficient

R = the hydraulic radius of the pipe = $\frac{D}{2}$

S = longitudinal slope of the pipe

Thus, the required diameter of the pipe is determined to be 44”, and the velocity of water at maximum flow is determined to be 7.36 ft/s.

The minimum velocity of wastewater through a 44" pipe was calculated to be 6.84 ft/s, and the average flow is determined to be 7.1 ft/s.

4 Parshall Flume Design

A Parshall flume controls and regulates the outlet flow of the interceptor. The type of partial flume used depends on the ratio of Q_{max} to Q_{min}, which was found to be approximately 2.5. This ratio is less than three, meaning the design of the channel is narrow and deep, and there will only be one channel. The velocity range of such a channel is 1.25 fps to 2.5 fps. The design of the partial flume was determined by the manufacturers data, which can be found in Appendix 1. The minimum and maximum height of the partial flume was determined by the following equation:

$$Q = 4 * W * h^{1.522 * W^{0.026}}$$

Where Q = the flow of wastewater (cfs)

W = width of the partial flume

h = the height of water flowing over the crest of the flume

The width of the Parshall Flume was decided to be 6.5 ft. Thus, according to the maximum and minimum flow, h was determined to be 2ft. 11in, which was rounded to 3 ft.

5 Bar Screen Design

The bar screen will be mechanically cleaned. The following criteria were used to design the bar screen:

-The maximum head loss of the screen is 0.5 ft

-The minimum head loss at the screen is 0.1 ft

-The circular spectral face bar is coefficient $\beta = 1.83$

-The angle of the screen to the vertical is 45 °

-The maximum cross sectional value of the bars varies between 1.4" and 5/8"

-The minimal clear spacing between the bars varies between ¾" and 3".

To find the minimum required spacing between the bars and maximum crosssectional width of the bars facing the flow the following equation was utilized:

$$h = \beta * (\frac{w}{b})^{\frac{a}{b}}$$

H_L = head loss on the screen

β = the bar coefficient

h_v = velocity of water approaching the bar rack

w = minimum clear spacing of the bars

b = maximum cross sectional width of individual bars

W and b parameters of the bar rack were fixed to be 2.5" and 1" respectively. Engineering drawing of the mechanically cleaned bar rack can be viewed in Appendix II.

6 Grit Chamber

The grit chamber was designed to remove 65-mesh material (diameter = 0.21 mm) with settling velocity of 3.7 fpm. The decision was made to place the grit chamber *before* the pumping well in order to utilized gravity flow and to protect the pumps from pumping material into primary sedimentation tank. The cross sectional cut of the designed grit chamber can be found in appendix II.

The following assumptions were made in the design of the grit chamber:

- Width of the chamber T = 9ft
- Horizontal velocity of the waterwater in the chamber is 1 ft/sec
- Settling velocity of the suspended particles is 3.7 ft/min
- Water exits the grit chamber through the weir of width a at critical velocity and depth. The following equation was used to find the height of flow, **H**:

$$H = \frac{3}{2} * \frac{Q}{TV} \text{ where}$$

H = height of water inside grit chamber

Q = flow of the water in the grit chamber

$\frac{3}{2TV}$ = cross sectional area of the grit chamber

H was found to be **H = x ft**

The ratio of the length of the grit chambers length to height should be the same as the settling ration of the particles (3.7 ft/min) to the horizontal velocity of water in the chamber (1 ft/sec). Thus;

$$\mathrm{L} = \frac{V_{settling}}{V_{horizontal}} * \mathbf{H} \text{ where}$$

L = length of grit chamber

$V_{settling}$ = settling velocity of particles

$V_{horizontal}$ = horizontal velocity of water

H = height of the grit chamber

Solving the equation yields the result: **L** = 170 ft. Appendix II contains detailed drawings of the grit chamber.

Critical velocity of the outlet of the grit chamber was found by utilizing Bernoulli's equation:

$$\frac{V_c^2}{2g} = \frac{dc}{2} \text{ where}$$

H = height of the water inside of the flume

V= the horizontal velocity of water in the flume

g = gravity

V_c = critical velocity

Critical Velocity V_c = 21 ft/sec.

7 Weir Well Design

Once wastewater exits the grit chamber, it flows it the pumping (wet) well. The well is designed for a 15 minute detention time. The following assumptions are factored into the design:

- Maximum detention time of well: 15 min
- Width of well: 15 ft
- Rate of pumping: $Q_{max} = 56.2$ cfs
- Minimum rate of inflow: $Q_{min} = 23.8$ cfs

The volume of the well is thus the difference between the maximum rate of pumping and minimum rate of inflow multiplied by the detention time:

$$V_{well} = T_{detention} * (Q_{max} - Q_{min})$$

Thus the volume of the well is 29,000 cubic feet. Since its width is assumed to be 15 ft, and the ratio between the length and depth is 1:1, the length and depth of the well are 44 ft each. Detailed drawings can be found in Appendix II.

8 Primary Sedimentation Tank

The following assumptions were made in the design of the primary sedimentation tank:

-Required BOD5 removal percentage is 30%

-The width of the single primary sedimentation tank is 50 feet

- Length of the primary sedimentation tank is 200 ft

The surface area of the tank is 12000 sq ft. In order to remove 30% of the BOD5, maximum surface loading of 1400 gpd/sqft is required. With the maximum flow of 921.1E5 gpd, the minimum surface area required is:

$$A = \frac{921 * 10^5}{1400 \frac{gpd}{ft^2}} = 65790 ft^2$$

With the given square footage, 3 sedimentation tanks are required to remove the 30% BOD removal in 1.5 hrs. The volume of the tanks must be:

$$V = Q * T \text{ where}$$

V = the volume of water in the primary sedimentation tank

T = the required detention time

Q = the maximum flow

The depth of water was calculated by dividing the volume by the surface area, D = 10ft. The adjusted retention time was computed by dividing the volume of the single tank by the maximum flow, coming out to t = 1 hr and 29 min. This was rounded to 1 hr and 30 minutes.

9 Effluent Weir Design

The following assumptions were made during the effluent weir design:

- The required height of water above the weir crest is 2"

-The preferred maximum weir loading rate is 200,000 gpd/ft

- A rectangular weir is implemented

- The required length of the weir was determined using the following equation:

$$Q = C * L * h^{1.5} \text{ where}$$

Q = the outlet flow of the primary sedimentation tank

L = the combined length of the weir

h = the height of water above the weir crest

C = the weir coefficient

The required length of effluent weir per tank is found to be 110 ft based off of maximum flow. The design utilizes two 55 ft weirs per tank.

10 Pump Design

After water enters the pumping well it is pumped into the primary sedimentation tank. The following assumptions were made in the design of the pumping system:

- The head loss due to friction is 10 ft
- The worst case scenario is used in the calculations
- The efficiency of the pump is 85% of the minimum water level in the wet well

The pumping well and primary sedimentation design can be found in Appendix II

The minimum total head the pump is capable of generating is the minimum water level + the freeboard + the maximum water level in the primary sedimentation tank + the head loss due to friction. This number comes out to be 60 ft.

The required horsepower is calculated with the following equation:

$$H_p = \frac{Q*h*\psi}{550*\mu} \text{ where}$$

H_p is the horsepower of the pump

Q is the maximum flow through the pump

h = the required head to be delivered by the pump

ψ is the unit weight of water ($\frac{lb}{ft^3}$ = 62.4

μ is the efficiency of the pump

The required horsepower produced by the pump is 450 Hp. Two pumps will be installed, a primary pump and a backup pump. Both pumps will be 450 HP, 85% efficient, 200 PM, and be of the Radial kind. The second pump is only to be utilized if the first pump fails or requires maintenance.

11 Cost Estimate

Everything will be multiplied by a factor of 1.34 due to Brookesville being just outside of San Diego, whose cost adjustment is 1.34.

11A Cost of Effluent Outfall Control

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

Where Q is the design flow. Thus,

C = 8.9 MOD (millions of dollars)

11B Sitework Including Excavation

C = $(1.96 * 10^5)Q^{1.09}$

Where Q is the design flow. Thus,

C = 18 MOD (Millions of Dollars)

11C Cost of Mobilization

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

C = 7.18 MOD (Millions of Dollars)

11D All Piping

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

C = 24.3 MOD (Millions of Dollars)

11E Primary Settlement Cost

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

Q = 1.96 MOD (Millions of Dollars)

11F Preliminary Treatment

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

C = 5.79 MOD (Millions of Dollars)

11G TOTAL COST

The total cost of this project is set to be:$ 66.13

Million Dollars, plus 25% for the cost of engineering: = 82.6 Million Dollars