CE 3372 – Water Systems Design
Design Project

Contents

1 Problem Statement

A small residential subdivision is depicted in Figure 1.

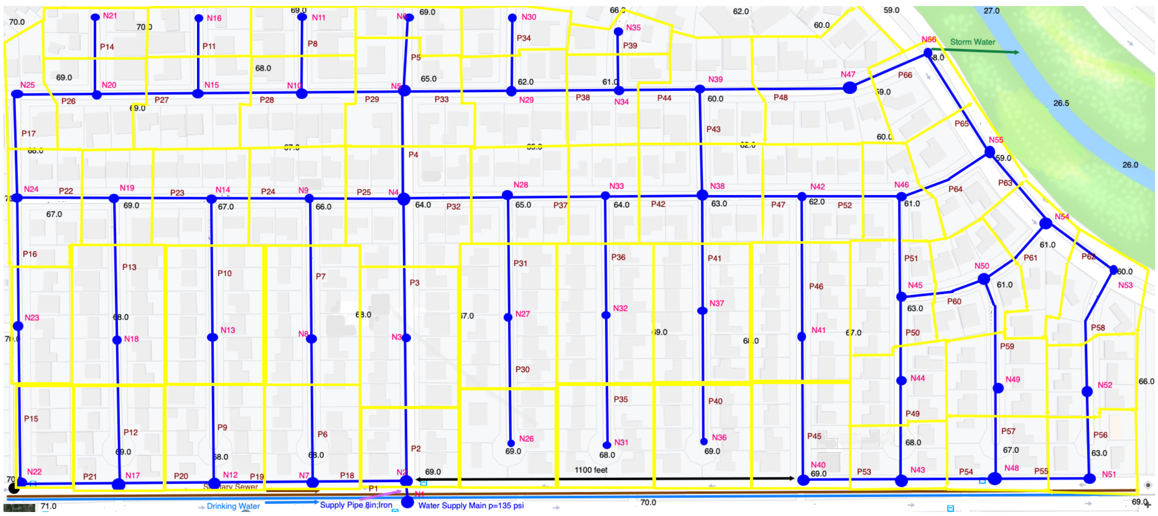

Figure 1: Somewhere USA Water Distribution (Skeleton) System

The subdivision is assumed to be within the extra-territorial jurisdiction (ETJ) of the City of Houston.

Prepare supporting hydraulic models for a water distribution system (drinking water), a sanitary sewer system, and a drainage system (stormwater). The drainage system hydraulic model should demonstrate that design criteria are satisfied at the design values (i.e. peak demand; 5-year, 3-hour precipitation event; . . .).

1.1 Drinking Water System

The drinking water is suppled from the 48-inch water main on the bottom (blue – labeled) of the figure. The water main reliably delivers water at a pressure of 135 psi. The water main delivers treated drinking water with a chloramine disinfectant residual of 8 ppm.; you do not need to consider water treatment. The water supply line (as shown) is nearly horizontal with bottom (of the pipe) elevation of 52-feet.

1.1.1 Drinking Water System Design and Report Contents

The project report component for the drinking water system should contain:

1. The required burial depths required for the water distribution pipes.
2. The minimum pipe diameters required on the water utility side of the meter.
3. A node-pipe layout map/drawing, overlain on the subdivision map. The pipe layout can be a skeleton system (you do not need to show every meter – demands should be grouped from 6-8 houses/node).
4. Produce a node elevation table (and land surface elevation table)
5. A node contour map.
6. Demand estimates (describe the values and methods).
7. Fire hydrant locations.
8. A hydraulic model that simulates the flow and pressure in the network.
9. A hydraulic simulation under normal flow (no fire) conditions. The model must be interpreted and the minimum and maximum pressures identified (along with their locations). If special items are required (pressure-reducing-valves; booster pumps; . . .) these must be described and their need explained.
10. A hydraulic simulation under (reasonable) fire-flow conditions. The model must be interpreted and the minimum and maximum pressures identified (along with their locations).
11. A cost estimate for the pipes, valves, fire hydrants, water meters, and any special components that are required based on the hydraulic simulation.
12. An estimate of the trenching requirements to install the network

1.2 Drainage System

The drainage system will discharge stormwater in the North-East corner of the study area (green arrow) into the stream indicated by the blue line segment.

1.2.1 Drainage System Design and Report Contents

The project report component for the drainage system should contain:

1. An estimate of the anticipated 5-year, 3-hour design storm for Harris County; with supporting references.
2. A node-pipe layout for the storm sewer system, overlain on the subdivision map. The

pipe nodes will be treated as inlets.

3. Inlet calculations to determine the drainage area that a single 10-foot curb-on-grade inlet would be expected to capture. Use this value to determine inlet spacing (and location) along the streets.
4. The burial depths required for the storm drain mains and laterals.
5. The drainage pipe diameters and materials.
6. Produce a node invert elevation table and land surface elevation table for the node locations.
7. A hydraulic model that simulates the flow and water surface elevation in the drainage system. The model will use sub-catchments (and CN method) for hydrology, these will attach to the nodes, which in turn connect pipes.
8. A hydraulic simulation, using "kinematic wave" option in SWMM, for the design storm.
9. A hydraulic simulation, using "dynamic wave" option, with "inertial terms == IGNORE", in SWMM for the design storm. The outfall in this simulation is "NORMAL".
10. A hydraulic simulation, using "dynamic wave" option, with "inertial terms == IGNORE", in SWMM for the design storm. The outfall in this simulation is "FIXED" with the stage (elevation) set to 50 feet.
11. A cost estimate for the inlets, pipes, junctions and any special components that are required based on the hydraulic simulation.
12. An estimate of the trenching requirements to install the storm sewer.
13. A summary, interpretation, and recommendations based on the hydraulic modeling – esp. the impact of backwater (the 3rd simulation conditions).

1.3 Sanitary Sewer System (F2020 NOT REQUIRED!)

Fall 2020 - This component is not required! The sanitary sewer system will collect wastewater from the residential homes, and discharge into the existing 96-inch sanitary sewer line on the bottom edge (orange – labeled) of the figure. The existing sanitary sewer main is sloped to the West (left) at 0.5% (0.005 dimensionless) and its invert is at elevation 42-feet at a location 3000 feet to the East of the origin (0,0).

1.3.1 Sanitary Sewer System Design and Report Contents (F2020 NOT REQUIRED!)

Fall 2020 - This component is not required! The project report component for the drainage system should contain:

1. A node-pipe layout for the sanitary sewer system, overlain on the subdivision map. The system can be skeletonized (6-8 homes can "share" a collection system node.
2. The burial depths required for the sanitary sewer mains and laterals.
3. The pipe diameters and materials.
4. Produce a node invert elevation table and land surface elevation table for the node locations.
5. A hydraulic model that simulates the flow and wastewater surface elevation in the drainage system.
6. A hydraulic simulation, using "kinematic wave" option in SWMM, for the design flow. The outfall in this simulation is "NORMAL".
7. A cost estimate for the pipes, junctions and any special components that are required based on the hydraulic simulation.
8. An estimate of the trenching requirements to install the sanitary sewer.
9. A summary, interpretation, and recommendations based on the hydraulic modeling.

2 Elevation Data

The origin is the lower left hand corner of the map, **use the elevation maps you created in earlier exercises**.

EASTING-FEET	NORTHING-FEET	ELEVATION-FEET
0.00	0.00	72.00
1089.00	0.00	71.00
1892.00	0.00	70.00
3228.50	0.00	68.00
3228.50	473.00	62.00
2942.50	594.00	61.00
2651.00	896.50	61.00
2464.00	1177.00	60.00
1892.00	1100.00	66.00
1089.00	1100.00	68.00
539.00	1100.00	70.00
0.00	1100.00	71.00
2843.50	1061.50	35.00
1089.00	594.00	69.00
1089.00	319.00	70.00
1892.00	594.00	67.30
1892.00	319.00	68.60

Figure 2: Mapping Elevation Data (Older Survey)

3 Example Reports

Example reports from prior courses are included below. In the past the project was presented in two reports, however in this case a single combined report is sufficient.

WATER DISTRIBUTION SYSTEM ANALYSIS AND DESIGN FOR CROSBY, TEXAS

TEAM [redacted]

[redacted]

Date: March 10, 2015

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EXECUTIVE SUMMARY

A water distribution network for a residential development in Crosby, Texas within Harris County has been designed. The distribution network satisfies demand across the development and adequate fire protection by the use of fire hydrants.

The analysis was completed by first determining parameters including minimum and maximum allowable flow pressures, elevations at each junction, and minimum pipe diameters depending on the number of junctions. The demand pattern used reflected average daily usage for typical Americans while the Hazen-Williams C value was assumed. Based on these assumptions and additional research, a model was generated in Environmental Protection Agency NET (EPANET), a software program that can be used to create and analyze closed conduit systems. Using EPANET, pressures, velocity, and flow rate were generated within pipes and junctions.

INTRODUCTION

Project Name and Purpose

The purpose of the "Newport Water Distribution System and Analysis for Crosby, Texas" is to create and analyze a water distribution system that can meet the demands of residents of single-family homes as well as of fire hydrants, using EPANET.

The objectives are to meet demand across the development, which means the water requirements of the development need to be met; and provide adequate fire protection to the newly annexed area, by providing enough water flow pressures in pipes.

Project Limits

The proposed development is located in Crosby, TX in Harris County within a neighborhood called Newport. The proposed development would be an addition to the Newport neighborhood bounded by N Diamondhead Blvd and Golf Club Dr, seen below in Figure 1.

Figure 1. Project limits in Crosby, Texas

Assumptions and Constraints

The limits of the project include: pressures of water flow inside the pipe must be between 35 psi and 80 psi under normal conditions with a maximum allowable velocity at peak house demand of 8 feet per second (fps) (Chanslor 2011), a minimum of 6 inch diameter pipes for less than 250 connections and 8 inches for more than 250 connections, and a minimum pressure of 20 psi for residual fire flow (Baker 2012). The minimum amount of backfill for pipes with a 12 inch diameter or smaller is 4 feet from the top of curb, with 3 feet absolute minimum (Lincoln).

To design the proposed development, some assumptions had to be made. The number of fire hydrants was assumed to be 28, due to each hydrant being spaced approximately 500 feet apart (Standard). The average demand was calculated to be 164 gcpd (gallons per capita per day) based on the average population and water use demand in Harris County (Pate 1987), so 200 gcpd was assumed to allow room for a greater capacity. Assuming Crosby, TX is generally a suburban area, the average number of residents per household is 3.10 (Houston 2014), thus 3.5 residents per household was assumed to account for possible increase in changes in population. The proposed development is assumed to be completely undeveloped, but lot lines have already been laid out, therefore lots were counted and 409 lots were assumed (Newport 2015). The lowest and highest elevations within the proposed development were assumed to be 39 and 50 feet respectively, based on previous surveys of the land.

A third party company does the pavement and construction, thus no costs are assumed for those portions of the project.

EXISTING CONDITIONS

Location and Topography

The proposed development is a neighborhood within Crosby, Texas located in southeast Texas in Harris County as seen in Figure 1. The topography is relatively level to gently undulating with elevations in the proposed development ranging between 39 ft and 50 ft. The slope from the most southwest corner to the most northeast is 0.161%. The slope from the most southwest corner to the highest elevation, which would be the northeast corner, is 0.301%. These values have been determined by the elevation of the nodes, shown in Figure 3, and the linear distance between them.

Land Use

The current land use for the proposed development is currently forestland. It is undeveloped and is covered by trees, bushes, and thick grass. The soil is mostly clay loams and clays.

HYDRAULICS

Analysis Objective

Using EPANET, a water distribution system for the proposed development was designed to meet the necessary flow demand and pressure requirements. For this subdivision of 409 residential homes, a single water line entering the subdivision from the southwest must provide adequate water pressure and flow at any given moment during the day, including fire flow during peak usage.

Hydraulic Methodology

EPANET allows the user to create a model using junctions and links that can be adjusted to fit design requirements. Data, including elevation, base demand, and pipe length and diameter, can be inputted into the model in addition to components, such as pumps, reservoirs, and valves, in order to calculate outputs such as flow and pressure.

The first step to creating the model was to set the defaults, which in this case were: the use of the Hazen-Williams loss model, units of gallons per minute, a diameter of 10 inches for the pipes, and a roughness coefficient of 100.

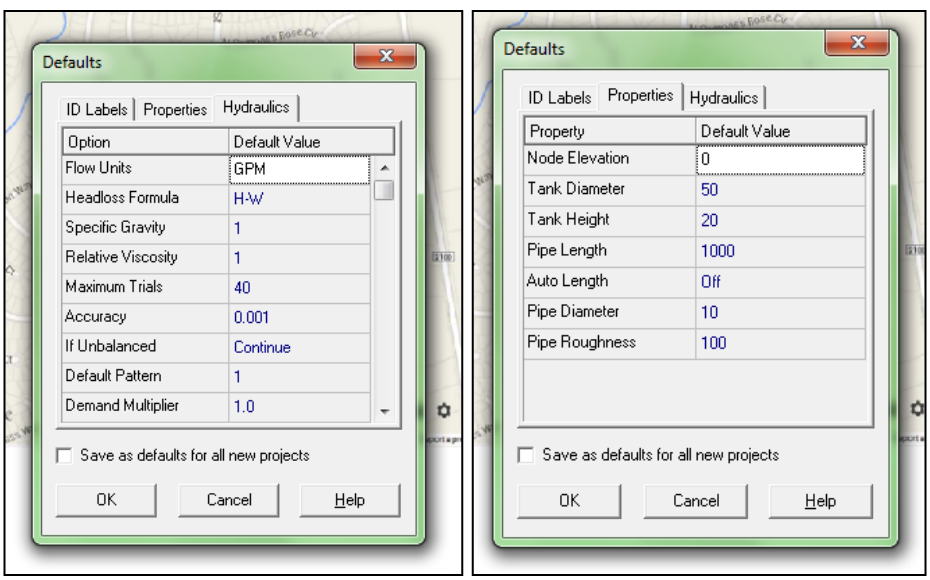

Figure 2. Defaults for EPANET Model

In order to create the model, an image of the proposed development was saved as a bmp file and then uploaded as a background. The image allowed the design engineers to draw the water distribution system using the planned roadways and house lots as guidelines. The first step in drawing the model was to input a reservoir, which acted as the source of water for the development. In this case, the source was the Luce Bayou, which entered the subdivision from the southwest and had an elevation 46 feet below the lowest elevation in the subdivision. In order to determine the lowest elevation in the subdivision, a topographical map created from prior surveying was used from which an elevation of 39 feet was found. Therefore, the elevation for the reservoir was inputted as -7 feet. The elevations for each node can be seen in Figure 3.

The next step to creating the model for the water distribution system was to determine where nodes or junctions would be located. The water necessary to meet the demand for the houses comes from these nodes, and they also serve as turning points since pipes are not curved but some streets are. Using the image of the proposed development, nodes were placed at approximately every seven houses and at road intersections, as every house does not require a node and the node should not be overloaded. The layout of pipes and nodes is shown in Figure 9. The subdivision was then divided in order to determine which nodes would serve which houses. This grouping of houses according to node entered in EPANET can be seen on Figure 4.

The locations of the fire hydrants was based on the hose length, therefore they were dispersed about 500 feet apart throughout the subdivision. A total of 28 hydrants for the neighborhood have been established. They are marked by a red circle on Figure 4. Each hydrant serves approximately six houses.

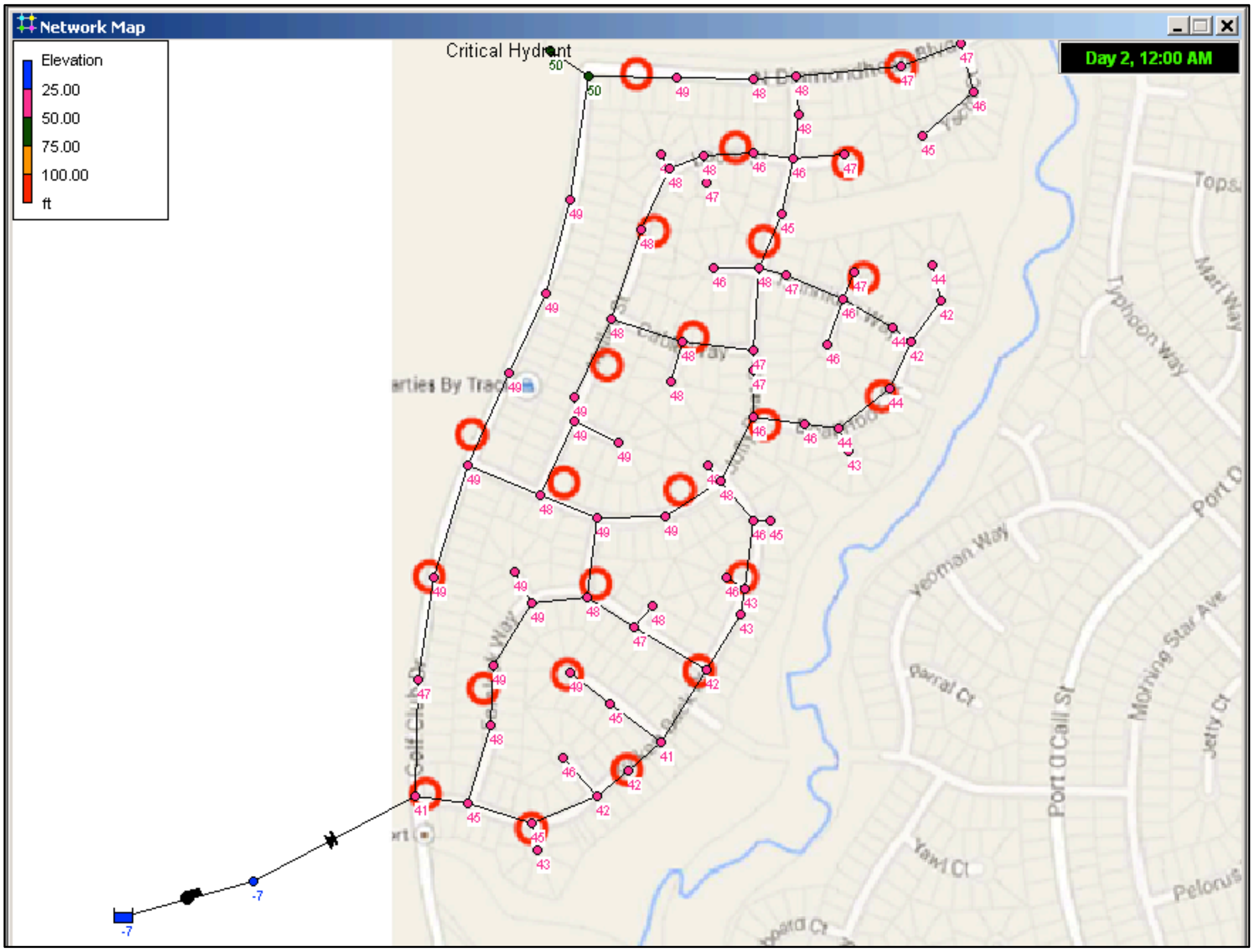

Figure 3. Node Elevations

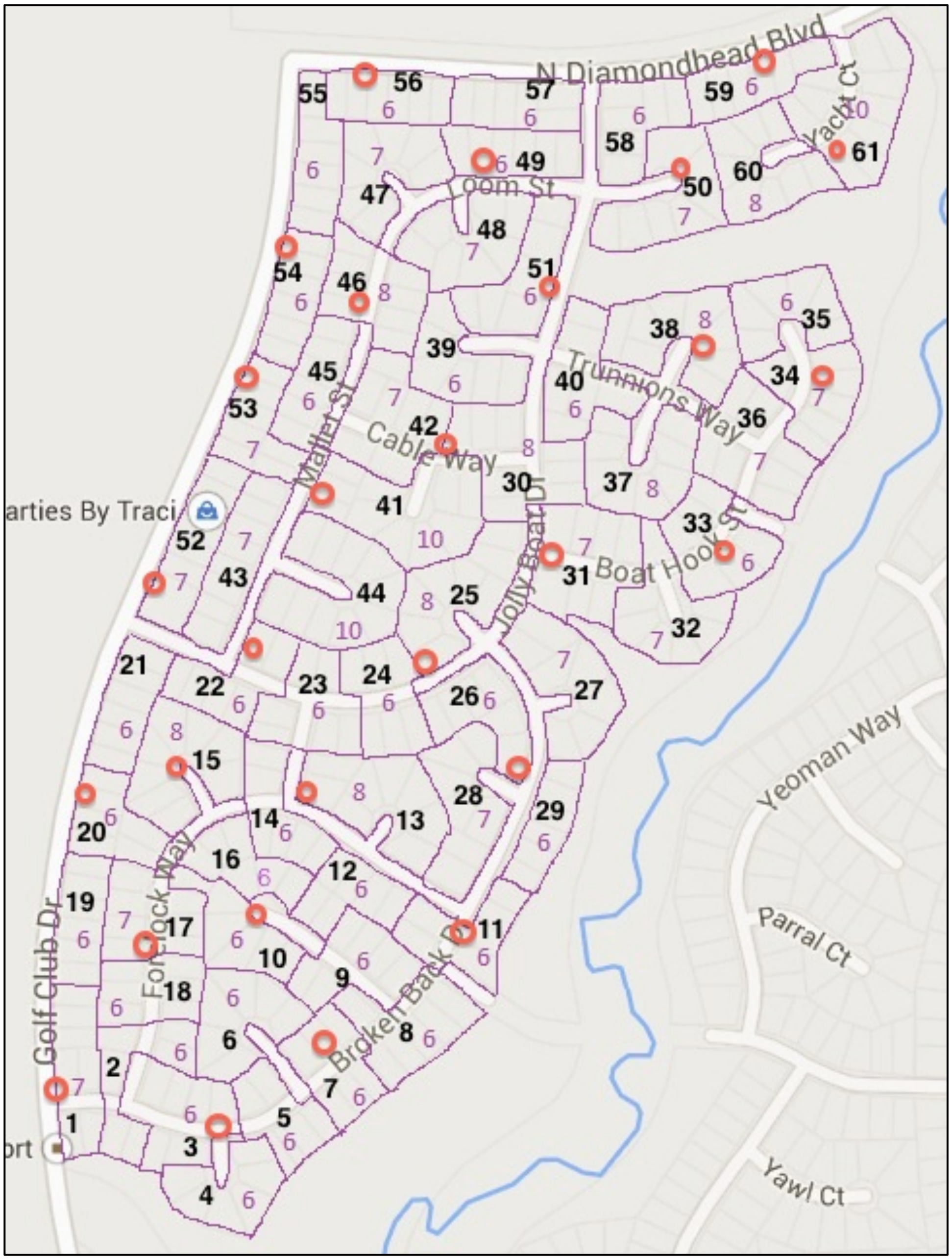

Figure 4. Houses to Corresponding Nodes with Fire Hydrants

It was estimated that each person uses approximately 200 gpd and each house has an average of 3.5 people living in it. With the number of residential homes, this means there is an estimated demand of 286,200 gpd for the entire subdivision. In order to determine the demand for each node, the number of houses it served was multiplied by the number of people per house and the amount of water each person used per day. These calculated demands were inputted as base demands for each node. If a node did not serve any houses and acted only as a junction between pipes, the base demand was entered as zero. These base demands are shown in Figure 5. Additional information needed for the nodes was the elevation of each node, which was determined using a topographical map, and inputted at each node. Values are shown in Figure 3.

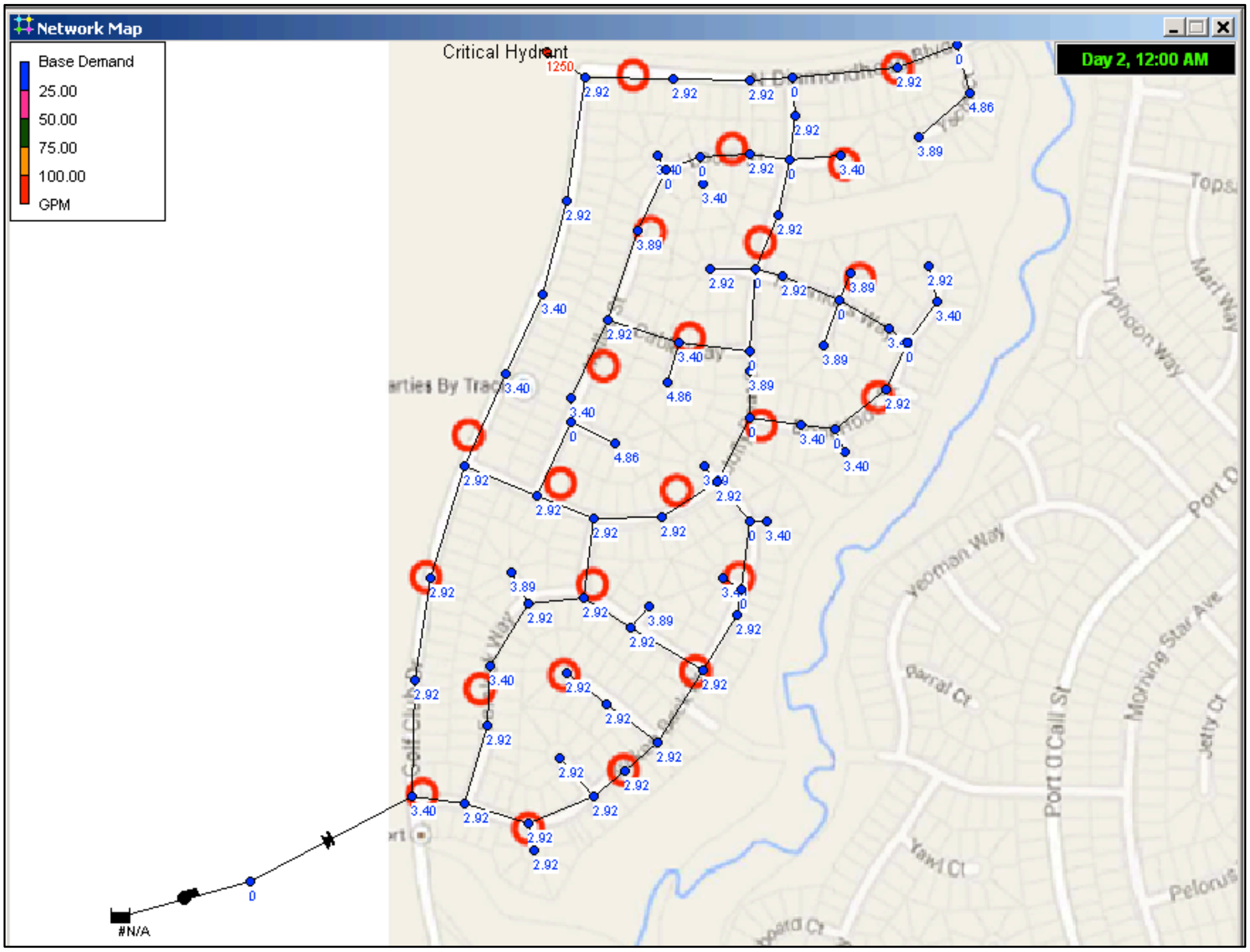

Figure 5. Base Demands as Entered in EPANET

The next step was to determine the length of the pipes connecting the nodes. These distances were estimated using the distance measurement tool on Google Maps. The distances determined by this process are shown in Figure 6.

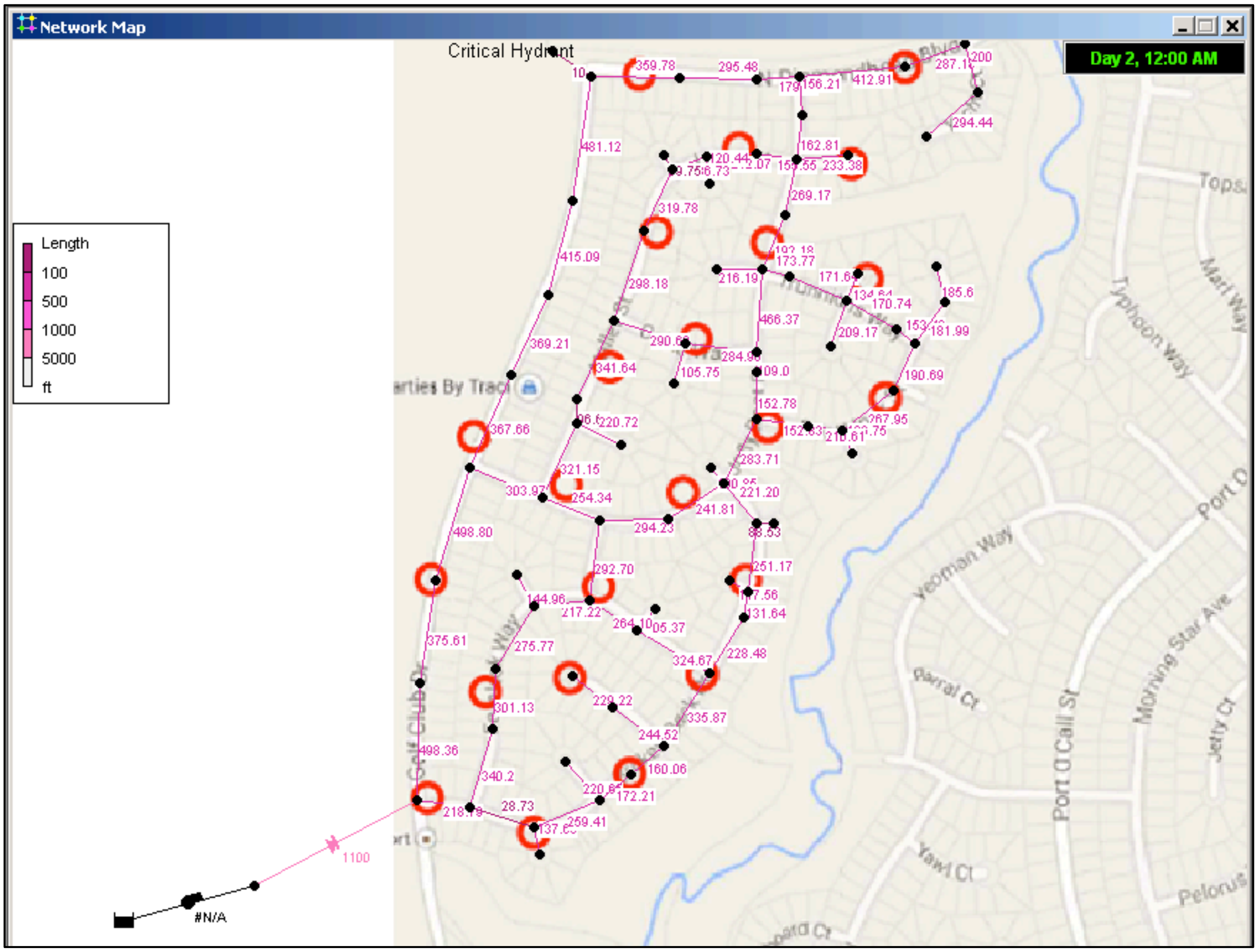

Figure 6. Pipe Lengths

Since the elevation of the reservoir was below the elevation of the development, a pump was included in the design. As pumps are links in EPANET, a node was placed outside of the development and the pump was connected between the reservoir and this node. The elevation at this node was equal to that of the reservoir, -7 ft, and the base demand was zero. In order to connect the pump to the subdivision, a pipe with a diameter of 12 inches and a length of 1100 feet was used, according to the distance from the Bayou to the subdivision. Pump curves were created using given performance curves from Cornell Manufacturing Company and were linked to the distribution model in order to run the program and observe each pump's performance.

As previously stated, the pressure in the pipes must be between 35 and 80 psi at all times of the day to maintain proper functionality and prevent contamination of water or breaches. This is assuming there are no fires. Target pressures in the distribution lines are between 50 and 65 psi. Also, velocity of water in the pipes should not exceed 8 fps to prevent pipe degradation.

In order to retain the water pressure at required levels and keep the water from flowing backwards, a check valve was added to the model. To do this, the pipe connecting the pump to the neighborhood was selected and the Initial Status was changed to CV. The setup of this valve relative to the pump and reservoir is shown in Figure 7.

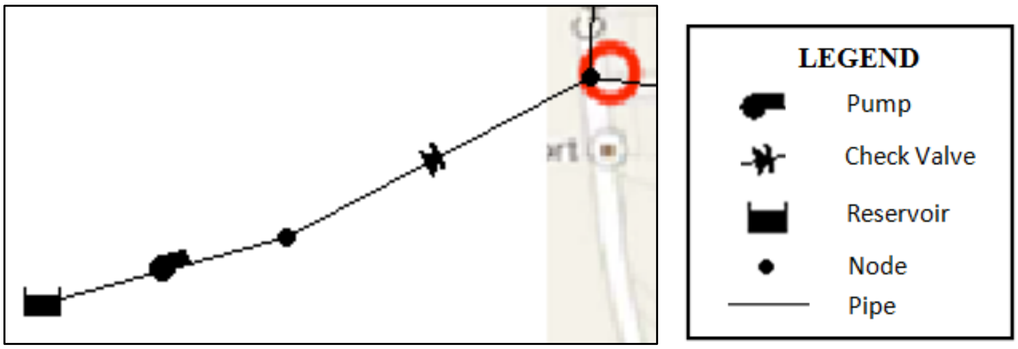

Figure 7. Reservoir, Pump, and Valve System

A pattern for water usage during different times of the day was established based on the likelihood of people using water for that set time. To illustrate, there is much less water demand at 2 AM, because most residents will be sleeping. However in the afternoon, around 7 or 8 PM, demand increases because most residents are doing things around their homes that require water. This pattern is represented in Figure 8. These peak factors are the percentage of the average demand for that hour of the day.

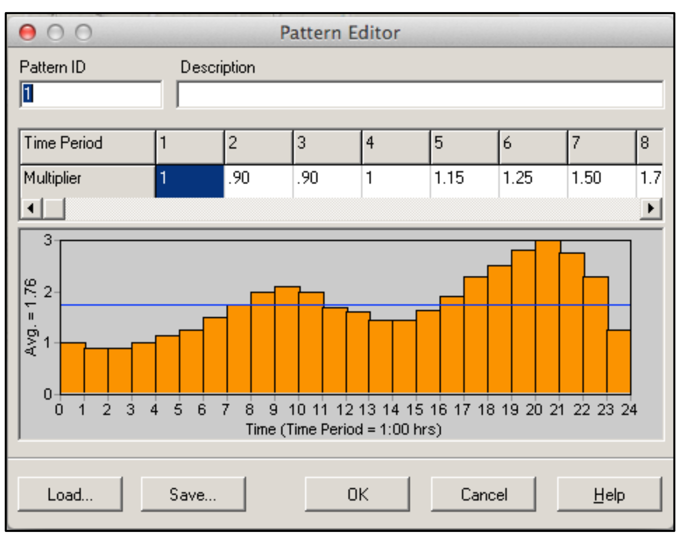

Figure 8. Demand Pattern

It was necessary to model fire flow for the neighborhood to ensure that proper pressures would be maintained. Fire flow at every node at all times of the day was not needed, since it would be highly unlikely that the whole subdivision would be on fire and that it would take a full day to put out. Fire demand was therefore modeled for the hydrant placed furthest away and at the highest elevation. The location of this “critical hydrant” can be seen in the top left corner of the system layout as it is shown in Figure 3.

The demand at this critical hydrant was modeled by standard fire flow criteria using a class factor of 1.5 for wood frame construction in the neighborhood, an estimate of the total square foot area of the largest floor in the building, and a percentage of the total area of the other floors. The average fire flow demand was found to be 1250 gpm for this subdivision. The system was tested with a fire at this location, and with the regular hourly demands at each node.

RESULTS AND RECOMMENDATIONS

Description

The basic network of proposed pipes for the area can be seen below in Figure 9.

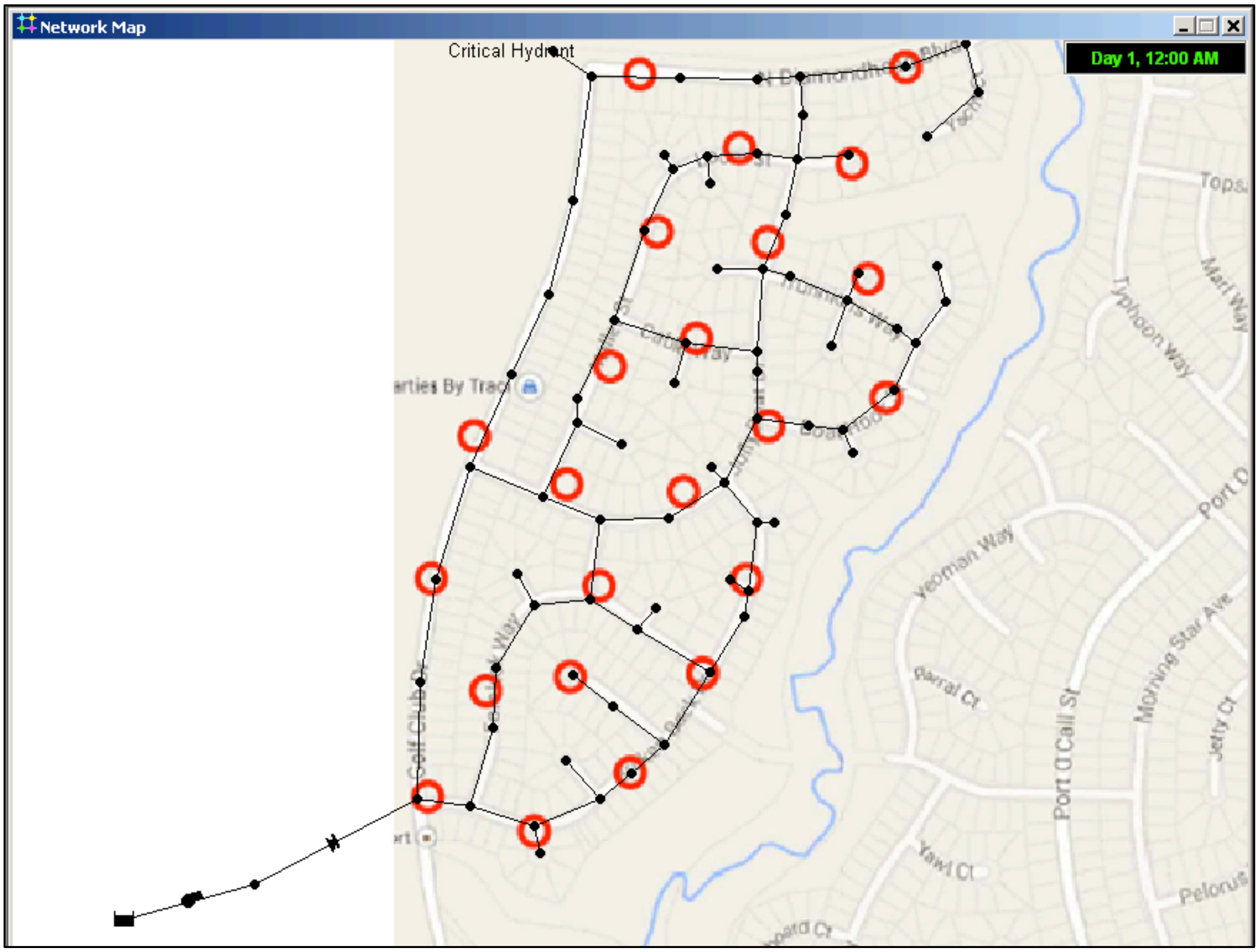

Figure 9. Proposed Pipe Setup

As stated, they are set up to follow the places were roads will be built to facilitate any necessary maintenance. A total of 79 junctions and 85 pipes will be used as described in the cost estimation portion of the report. The red circles on Figure 4 mark the location of the 28 fire hydrants, which will be spread throughout the neighborhood.

The pump chosen was a design by Cornell Manufacturing Company; the model was 5RB-D. This allowed for the smallest pressure to be 23.16 psi at the location of the critical hydrant twenty hours into the simulation if there is a fire that needs to be put out. The smallest pressure would be 60.17 psi, also at that node, without a fire in the neighborhood. As can be seen by the demand pattern shown in Figure 8, this time, around 8 PM, is when the greatest demand on the system takes place. These lowest pressure values are both above the minimum required pressure; therefore pressure will be sufficient for the daily demand as well as for possible fire requirements. The pressures at each node during this hour of highest demand are show below in Figure 10 and Figure 11.

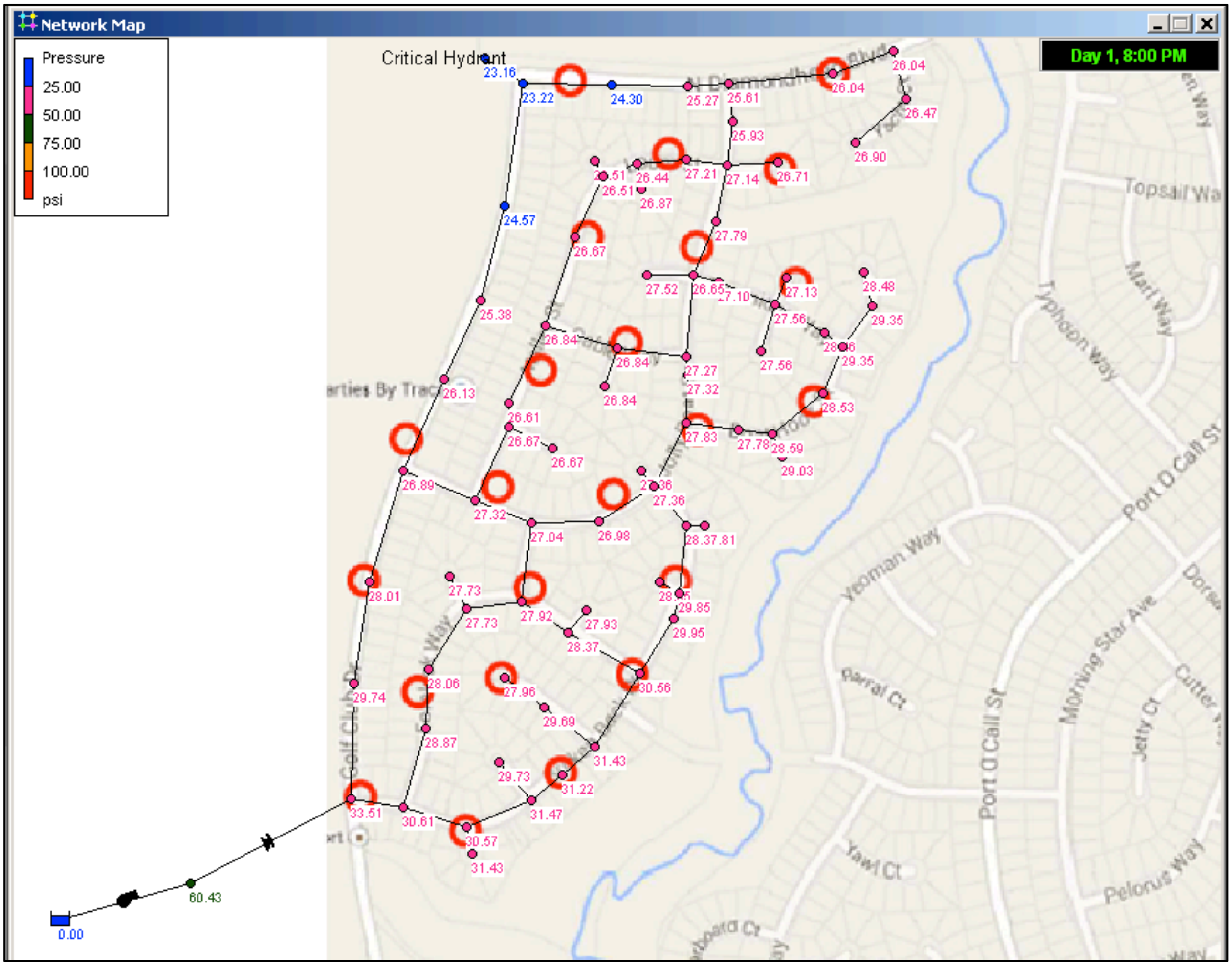

Figure 10. Smallest Pressures with Fire Flow Requirement

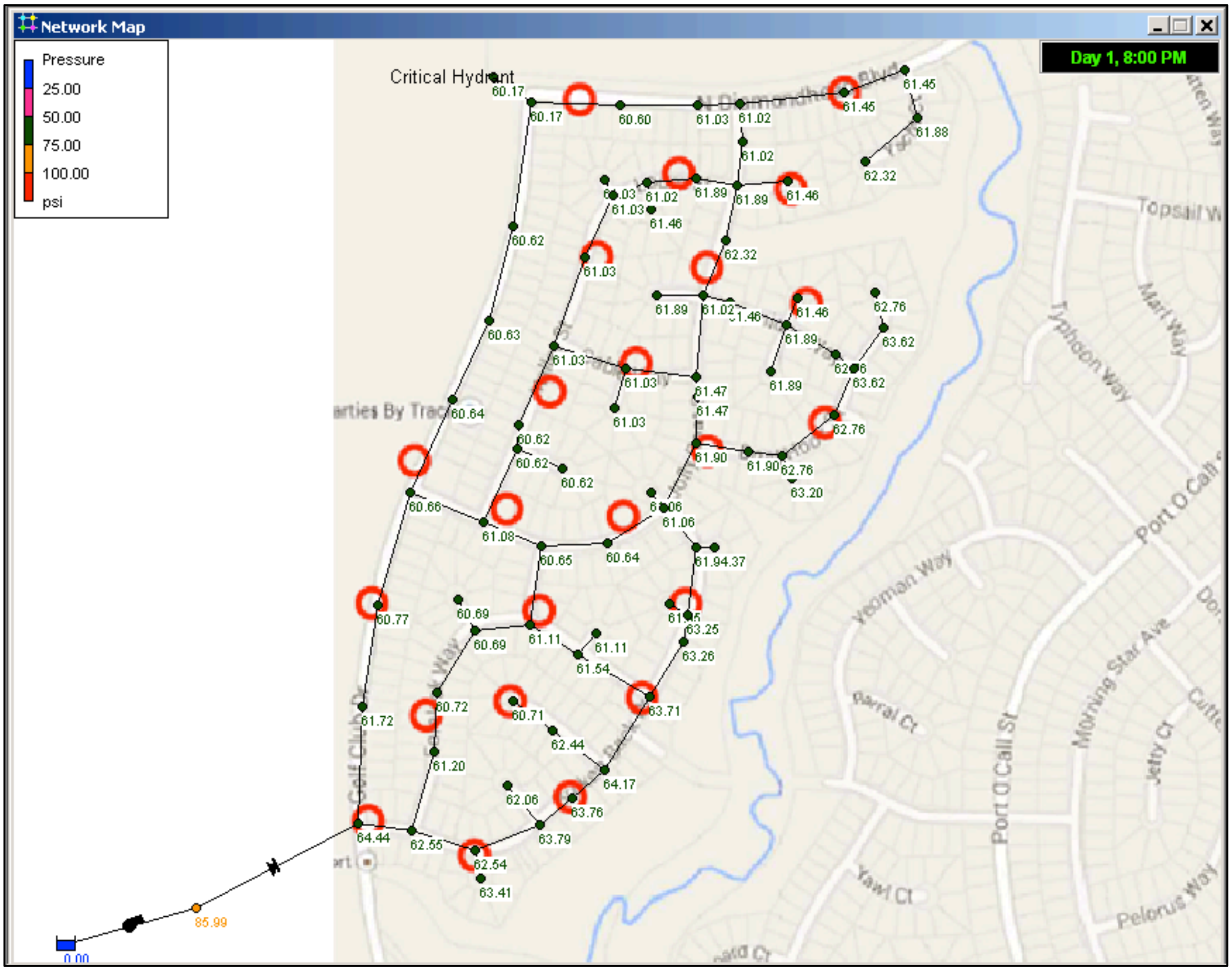

Figure 11. Smallest Pressures without Fire Flow Requirement

The simulations showed that the largest pressure in the pipe system would be 50.65 psi, at the node closest to the pump, for a situation that required fire flow. The highest without a fire requirement would be 65.77 psi at the same location. These high pressures both occur around 2 AM when, as can be seen on the demand pattern in Figure 8, the residents of the neighborhood would use very little water. These largest pressure values are both below the maximum allowable pressure for the pipe system; therefore no damage will be inflicted on the system. The pressures at each node during these hours of low demand are displayed in Figure 12 and Figure 13.

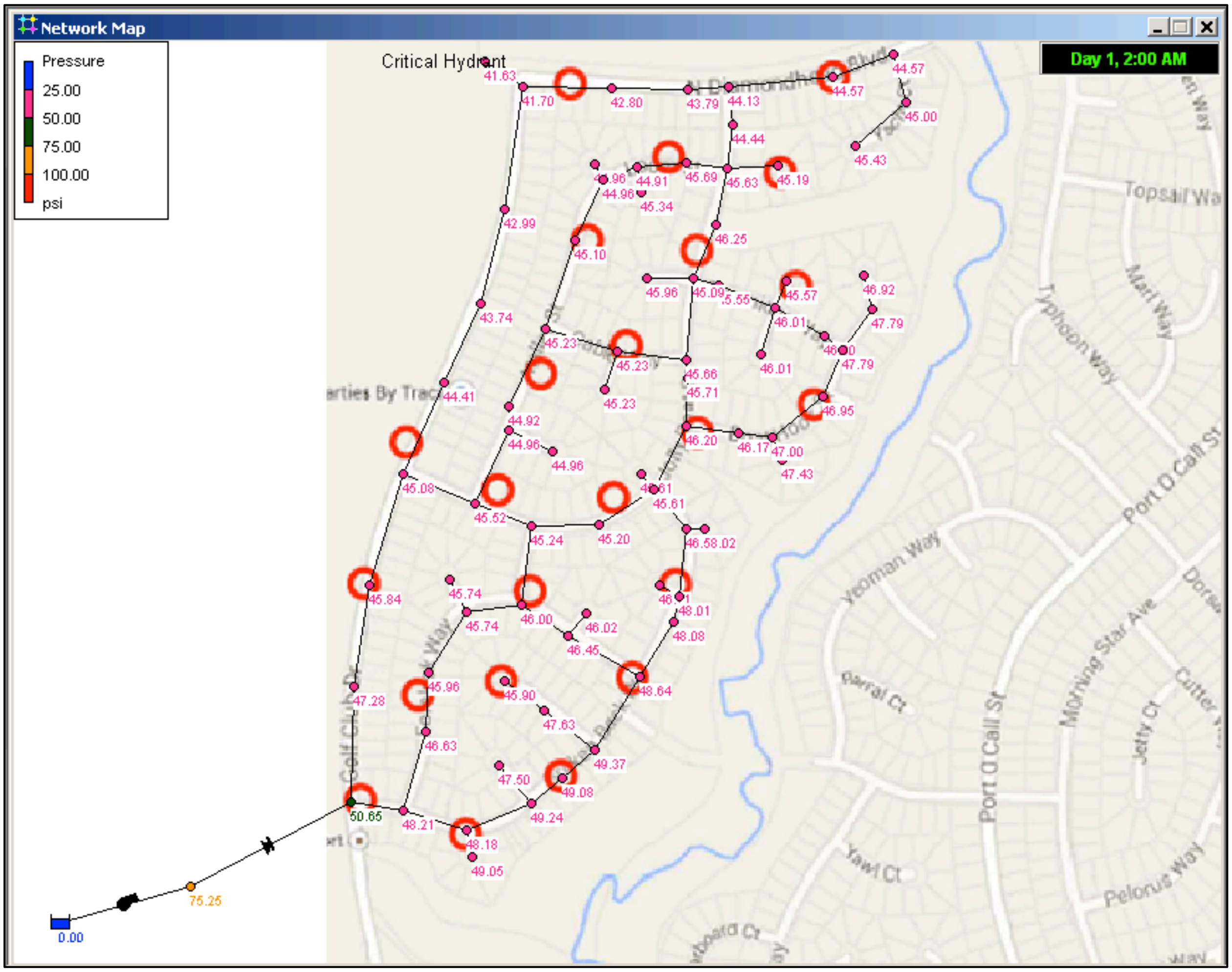

Figure 12. Largest Pressures with Fire Flow Requirement

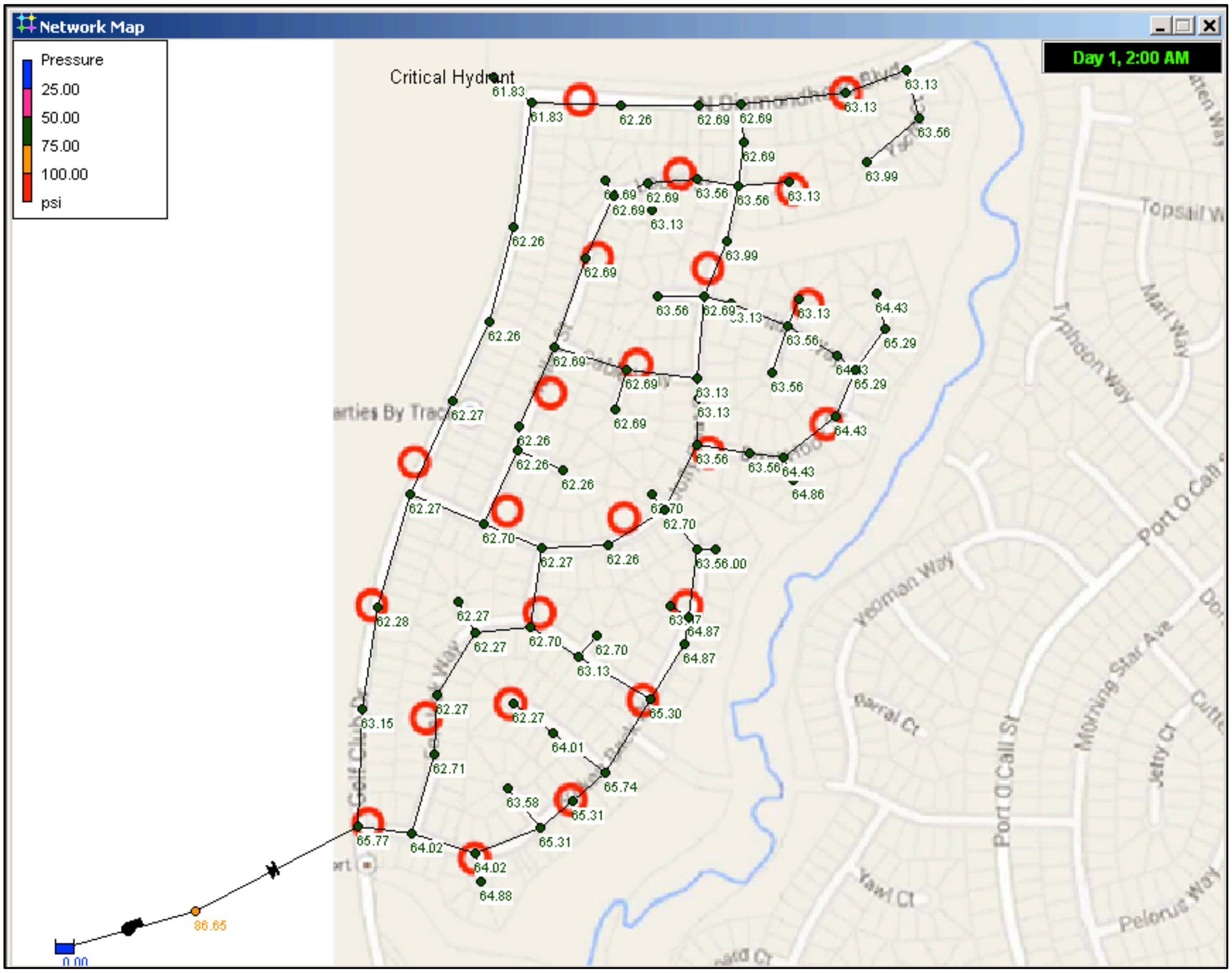

Figure 13. Largest Pressures without Fire Flow Requirement

Hydraulic Analysis

For the proposed system, the pressure at the pump and details of the pipe can be found in the table in the Appendix of this report. This represents both situations that were modeled with EPANET: with a fire flow requirement in the neighborhood and without.

The pump pressure remains steady throughout the day and sufficient discharge and flow is supplied to the system to accommodate the variant amount of water usage and maintain pressures within the range required.

It should be noted that water pressure and velocity are highest at this point of the system. This is expected due to the fact that all the water from the reservoir is being pushed through this node and pipe before it is dispersed throughout the system. The higher pressures at this location, along with the control valve in case of flow disruption, prevent backflow of the system.

Recommendations

For the proposed project, the pipes should follow the path of the roads in the neighborhood, with their respective length as seen on Figure 6. These pipes should all be of a 10-inch diameter, with the exception of the pipe with the valve, which will have a 12-inch diameter. The valve suggested is model 12-33F-CB2 by Newco. There will be 28 hydrants in the area, as marked by red circles on Figure 4. The pump recommended for this layout is by Cornell Manufacturing Company, model 5RB-D. These choices ensure proper function of the system in Harris County, since all requirements are met.

Cost Analysis

The total land area is 121 acres. Purchasing the land will cost approximately $18,500 per acre. It will have to be cleared and grubbed, which will cost approximately $4,500 per acre since it is a heavily wooded area.

Excavating will cost $42 per linear foot and backfill will cost $7 per linear foot.

The 10-inch diameter pipes will cost $88 per linear foot and the 12-inch diameter pipe will cost $128 per linear foot. Pipe lengths for the distribution system are shown in Figure 6. A Swing Check Valve made of carbon steel will be used. This valve is model 12-33F-CB2, designed by Newco, and has a 12-inch diameter. The cost for this valve is $4,327.32.

Hydrants used for the subdivision will be Clow Medallion F2545 Fire Hydrant (AWWA - ULFM). As previously stated, there will be a total of 28 each priced at $4,592. The trench depth for the hydrants should be six feet.

The pump chosen for the system is Cornell Pump Co.'s model 5RB-D. It will cost $7,500.

The cost of pavement was not included in this analysis because that is to be subcontracted out.

Overall, the project will require funds of $7,591,300.

Table 1. Preliminary Cost Estimate

LAND					
	Description	**Unit**	**Qty.**	**Unit Cost**	**Total Cost**
	DEVELOPMENT AREA	AC	121	$18,500.00	$2,238,500.00
	Subtotal Development Area				**$2,220,000.00**
	CLEARING AND GRUBBING	AC	121	$4,500.00	$544,500.00
	Subtotal Clearing and Grubbing				**$544,500.00**
	EXCAVATION	LF	20,374	$42.00	$855,708.00
	Subtotal Excavation				**$855,708.00**
	BACKFILL	LF	20,374	$7.00	$142,618.00
	Subtotal Backfill				**$142,618.00**
UTILITIES					
	Description	**Unit**	**Qty.**	**Unit Cost**	**Total Cost**
	WATER SUPPLY:				
	10-inch Waterline	LF	19274	$88.00	$1,696,112.00
	12-inch Waterline	LF	1100	$128.00	$140,800.00
	Check Valve		1	$4,327.00	$4,327.32
	Fire Hydrants		28	$4,592.00	$128,576.00
	Pump		1	$7,500.00	$7,500.00
	Subtotal Water Supply				**$1,977,315**
				SUBTOTAL	$5,740,100
				Contingencies (15%)	$861,000
				Engineering (15%)	$990,200
				TOTAL	**$7,591,300**

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APPENDIX

Appendix A. Pump and Pipe Measurements

Appendix A. Pump and Pipe Measurements

Hour	With A Fire Flow Requirement			Without A Fire Flow Requirement		
	Pump	Pipe		Pump	Pipe	
	Pressure (psi)	Flow (gpm)	Velocity (fps)	Pressure (psi)	Flow (gpm)	Velocity (fps)
0:00	74.73	1448.90	4.11	86.64	198.90	0.56
1:00	75.25	1429.01	4.05	86.65	179.01	0.51
2:00	75.25	1429.01	4.05	86.65	179.01	0.51
3:00	74.73	1448.90	4.11	86.64	198.90	0.56
4:00	73.91	1478.73	4.19	86.63	228.74	0.65
5:00	73.35	1498.63	4.25	86.62	248.63	0.71
6:00	71.86	1548.35	4.39	86.59	298.35	0.85
7:00	70.26	1598.08	4.53	86.54	348.08	0.99
8:00	68.54	1647.80	4.67	86.48	397.80	1.13
9:00	67.82	1667.69	4.73	86.45	417.69	1.18
10:00	68.54	1647.80	4.67	86.48	397.80	1.13
11:00	70.59	1588.13	4.51	86.55	338.13	0.96
12:00	71.23	1568.24	4.45	86.57	318.24	0.90
13:00	72.17	1538.40	4.36	86.60	288.41	0.82
14:00	72.17	1538.40	4.36	86.60	288.41	0.82
15:00	70.91	1578.19	4.48	86.56	328.19	0.93
16:00	69.24	1627.91	4.62	86.51	377.91	1.07
17:00	66.32	1707.47	4.84	86.38	457.47	1.30
18:00	64.74	1747.25	4.96	86.29	497.25	1.41
19:00	62.22	1806.92	5.13	86.13	556.92	1.58
20:00	60.43	1846.70	5.24	85.99	596.70	1.69
21:00	62.65	1796.97	5.10	86.16	546.98	1.55
22:00	66.32	1707.47	4.84	86.38	457.47	1.30
23:00	73.35	1498.63	4.25	86.62	248.63	0.71
24:00	74.73	1448.90	4.11	86.64	198.90	0.56

STORM WATER DRAINAGE SYSTEM ANALYSIS AND DESIGN FOR CROSBY, TEXAS

[redacted]

Date: April 21, 2015

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EXECUTIVE SUMMARY

A storm water collection system for a residential development in Crosby, Texas within Harris County has been designed. The storm water collection system successfully collects all runoff from a 2-year design storm, meaning a 50% annual exceedance probability, with no impact downstream.

The analysis was completed by first determining parameters including maximum flow capacity and velocity, elevations at each inlet, and minimum pipe diameters depending on the flow and velocity characteristics. Based on these assumptions and additional research, a model was generated in SWMM, a storm water system analyses program, and the inlets were sized. The approximate preliminary cost is $[redacted], which accounts for the cost of materials, excavation, backfill, contingencies, and engineering.

INTRODUCTION

Project Name and Purpose

The purpose of the "Newport Storm Water Collection System and Analysis for Crosby, Texas" is to create and analyze a storm water collection system that can drain runoff in a safe manner without major local flooring that does not have significantly impact downstream. This was done using the computer program SWMM from the Environmental Protection Agency.

Project Limits

The proposed development is located in Crosby, TX in Harris County within a neighborhood called Newport. The proposed development would be an addition to the Newport neighborhood bounded by Jolly Boat Dr. and Golf Club Dr, seen below in Figure 1.

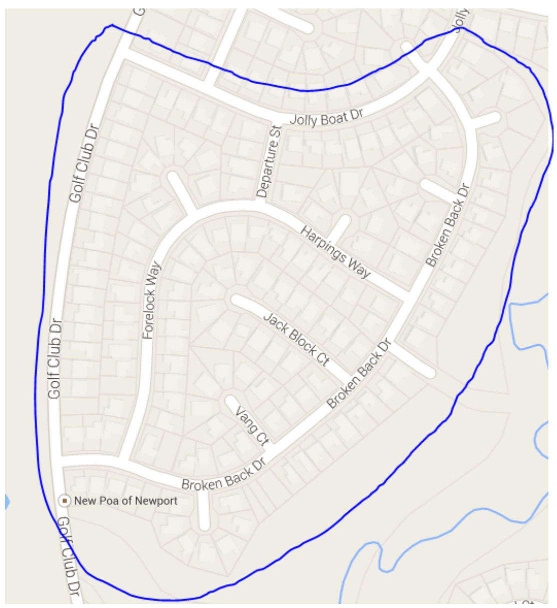

Figure 1. Project limits in Crosby, Texas

Assumptions and Constraints

The limits of the project include: the system must rely completely on gravity, the velocity in the pipes must be between 3 and 8 feet per second during the specified design storm, and the pipes must have a minimum back fill of 3 feet (Lincoln.) Additionally, no more than 700 ft of pavement can drain to an outlet from either side for a total of 1400 ft. Furthermore, minimum slopes must be met dependent on the pipe diameter used.

To design the proposed development, some assumptions had to be made. The most reasonable inlets to use were curb inlets since all the residential lots drain to the streets. Due to how close together the runoff volumes and times of concentration were between the predevelopment and post development, it was determined that there is no need for a detention structure.

A third party company does the pavement and construction, thus no costs are assumed for those portions of the project.

Location and Topography

The proposed development is a neighborhood within Crosby, Texas located in southeast Texas in Harris County as seen in Figure 1. The topography is relatively level to gently undulating with elevations in the proposed development ranging between 39 ft and 50 ft.

Land Use

The current land use for the proposed development is currently forestland. It is undeveloped and is covered by trees, bushes, and thick grass. The soil is mostly clay loams and clays.

HYDROLOGY

Analysis Objective

Using a topographical map created by Lidar, the area of the proposed development was analyzed in order to determine the demand for the storm water drainage system. For this subdivision, 13 drainage areas were delineated to drain into 13 inlet sets for the transportation of the water to an outfall located in the south east corner of the development.

Hydrologic Methodology

As per regulations set by Harris Country, the storm water drainage system was designed using a 2-year storm. In order to create the system, a hyetograph, or a graph showing rainfall for a given frequency, was created to determine the flow rate for each inlet. For this development located in Crosby, Texas, the depth for a 2-year, 24-hour storm is 4.2 inches of rainfall. Using this depth and a SCS type II curve for Texas, a hyetograph with cumulative depths was generated. Based on the cumulative depths, the amount of rainfall could be calculated generating the amount of rainfall at each time. The largest amount of rainfall for a 3-hour period begins at 10.5 hours and ended at 13.5 hours from a 24-hour rainfall event.

The next step in designing the storm water drainage system was to identify the locations of the inlets. Using a topographical map created with Lidar, general locations of inlets were chosen. These locations were then altered based on the requirement that no more than 700 ft of pavement drain into the outlet from each direction using AutoCad to measure lengths. After determining the locations of the inlets, the subcatchment areas, or the watershed areas draining to the inlet were delineated. This was done using the topographical map to determine the direction the water would flow. Next, the amount of flow the inlet would have to accommodate for was determined; this was computed using the rational method. Computations for the rational method involved determining the runoff coefficient for the site location, which was chosen as 0.4 based on the proposed development being single-family residential use. Then the area of each delineated subcatchment was measured in square feet then converted to acres. The time of concentration was calculated using the Kerby method for which the dimensionless retardance coefficient, N, was chosen to be 0.2 for poor grass or moderately packed surfaces. The based on the longest flow path for the drainage area, and then was used along with parameters for intensity for Harris County to calculate the intensity.

HYDRAULICS

Analysis Objective

Using SWMM, software used to analyze storm water collection systems, a storm water collection system was created to address flooding at peak flow. Based on a 2 year, 24 hour rainfall event, pipe slopes for a given size must have a minimum slope while the flow velocities must be kept within specified guidelines with backfill requirements met.

Hydraulic Method

Storm Water Management Model (SWMM), uses a series of subcatchment areas, junctions, and links that can be adjusted to fit design requirements. Data, including elevation, subcatchment area, flow rate, pipe length, and diameter, can be entered into the model to generate flow depths and velocities.

The first step to creating the model was to set the defaults, which in this case were: percent slope, percent impervious set at 38%, infiltration model of "CURVE_NUMBER", conduit roughness of 0.01, flow units of cubic feet per second (cfs), routing method of dynamic wave, and the force main equation of Hazen-Williams.

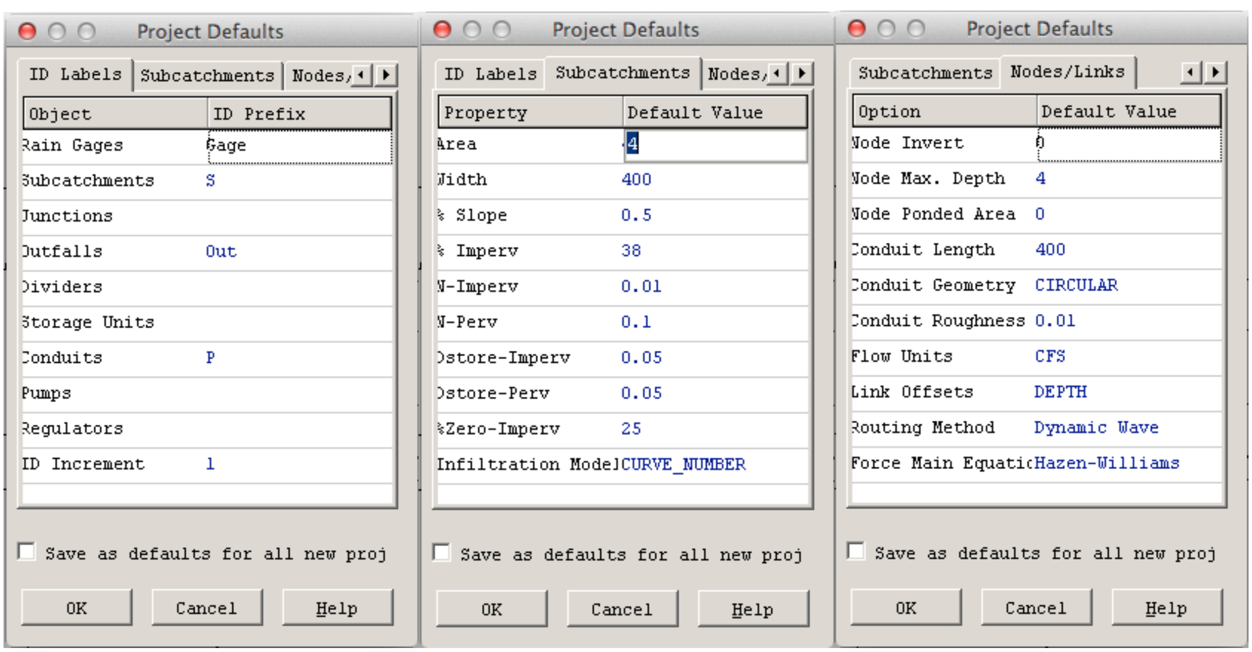

Figure 2. Project defaults

In order to create the model, an image of the proposed development was saved as a jpg file and uploaded as the background. The image allowed the design engineers to draw the storm water distribution system with subcatchment areas and the proposed roadways as guidelines. The number of pairs of inlets used for this setup is 13. Based on the location of inlets and topography, the sizes of subcatchment areas were determined. The curve number used for infiltration was determined from USDA's Web Soil Survey, which related soil properties for the subdivision. Using the class D given from this and the National Engineering Handbook that incorporated residential properties and streets, the curve number was determined to be 87.

Based on the subcatchment areas, the flow rate into each pair of inlets could be determined based on the 2-year design storm. A hyetograph representing a 2-year design storm for Harris County was created in which the peak rainfall depths were used for a 3-hour period. Creating a hyetograph allowed for values to be inputted into the rain gage, in which all values were considered to be negligible while the 3 hour rainfall depths were used.

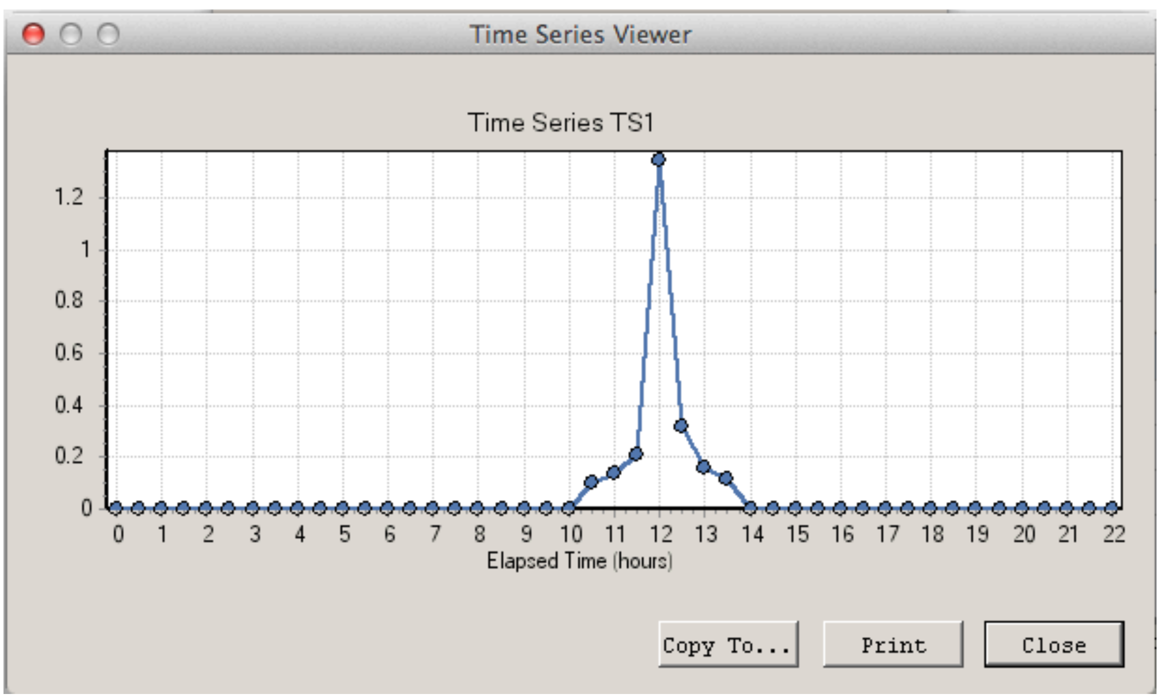

Figure 3. Rain gage time series

The elevations of each inlet were determined to find the depths of pipes based on backfill requirements of three feet minimum. The location of the outfall, at the southeast corner of the subdivision, was determined to be 25 feet. Based on minimum slopes for a given pipe size, SWMM will output velocities and overflowing pipes, also known as flooding. The flow rate velocities must be at a minimum of 3 feet per second and a maximum of 8 feet per second with no flooding. Pipe diameters and elevations could be adjusted to accommodate appropriate velocities and water surface elevations. To decrease costs, the backfill was minimized by maximizing elevations of junctions while the smallest pipe diameters were used.

In order to size the inlets, the inlet capacity was determined. The inlets used were curb-on-grade and the capacity for 10, 15, and 20 foot inlets were computed. Using the flow capacities for these inlets, the drainage areas for the different sized inlets were determined by the rational method. The inlets for the development were then chosen based on the calculated flow and area for each subcatchment.

Pipe lengths were estimated from an online map source according to the determined location of the inlets. The subcatchments and pipe layout, along with areas and lengths are shown in Figure 4.

RESULTS AND RECOMMENDATIONS

Description

The layout for the proposed storm water drainage system is shown below in Figure 4.

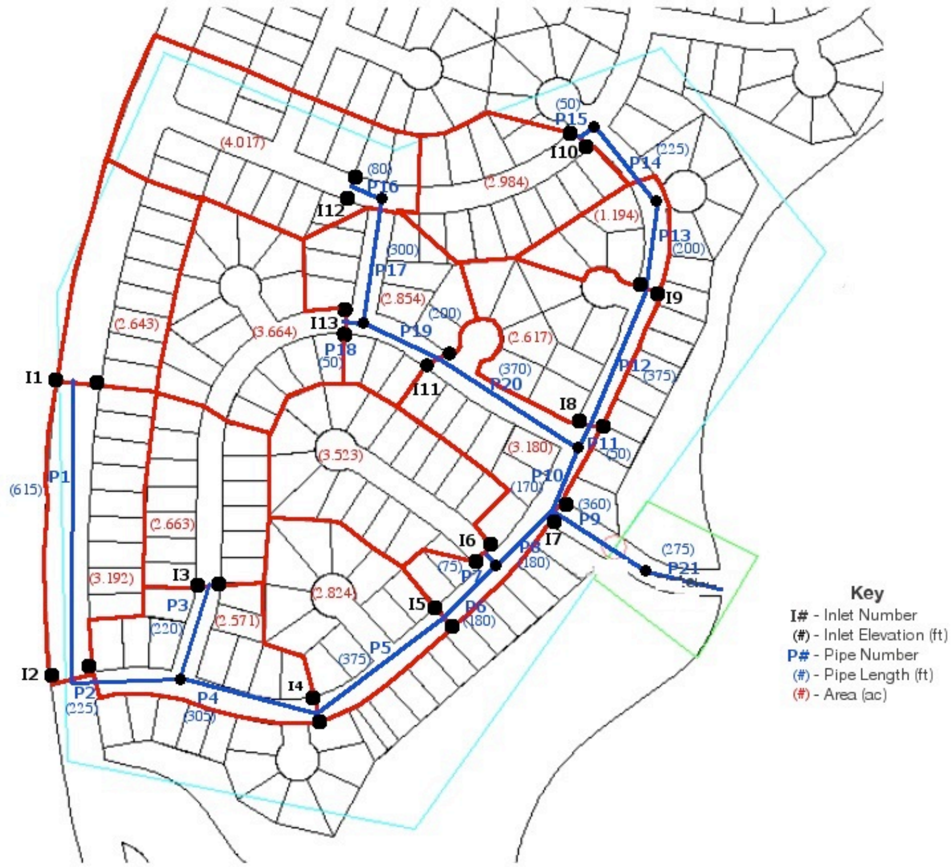

Figure 4. Proposed layout

The network consists of 13 inlets and 8 junctions, which resulted in 21 total nodes for the setup. The outfall is shown on the east most part of the subdivision. A total of 21 concrete pipes were called for. As shown, they follow the layout of the street to facilitate maintenance. Of these, three had a 10-inch, ten had a 12-inch, four had a 15-inch, two had an 18-inch, and two had a 21-inch diameter. The minimum slopes were verified according to regulation for each pipe based on these diameters. Of the 13 inlet pairs, 12 of them will be 20 ft curb-on-grade while the other one will be a 10 ft curb-on-grade.

With this system and the peak hours of the 2-year 24-hour duration storm, the network met all requirements and successfully transported the excess rainwater to the gully.

There was no flooding at any of the inlets or nodes. However there was slight surcharge at a few of them. However the height above the crown did not reach levels were there would be backflow into the streets. These values can be seen in Figure 5.

Summary Results

Node Surcharge

Click a column header to sort the

Node	Type	Hours urcharge	Max Height Above Crown	Min Depth Below Rim
I5	JUNCTIO	0.13	0.786	1.714
I6	JUNCTIO	0.25	1.190	1.977
I7	JUNCTIO	0.27	0.744	3.006
I8	JUNCTIO	0.33	1.753	1.247
I11	JUNCTIO	0.19	1.715	1.285
I13	JUNCTIO	0.07	0.623	2.377
15	JUNCTIO	0.27	1.217	1.283
16	JUNCTIO	0.30	1.464	0.536
20	JUNCTIO	0.11	1.545	1.455

Figure 5. Node surcharge

Pipe velocities also remained within the required range, as seen in Figure 6. Those with lowest velocities were the pipes leading from outlying inlets. The highest velocities were those in the pipes leading all the storm water to the outfall. This was expected since these are the ones that collect the flows. These pipes also tended to be the steepest because of the elevation of the gully compared to the rest of the subdivision.

The path that was most troublesome was the one leading from the west most inlets to the outfall on the east side of the subdivision. The node elevations and pipe diameters had to be fine tuned to achieve appropriate flow patterns. The water surface elevation profile for this path, is represented in Figure 7. Note that this is the flow pattern for the very peak of the rainfall data.

Summary Results

Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CFS	Day of Maximum Flow	Hour of Maximum Flow	Maximum Velocity	Max / Full Flow	Max / Full Depth
P3	CONDUIT	1.40	0	12:30	3.18	0.52	0.76
P1	CONDUIT	1.15	0	12:30	3.25	0.50	0.62
P18	CONDUIT	1.56	0	12:35	3.27	0.24	1.00
P17	CONDUIT	1.77	0	12:33	3.30	0.47	0.79
P2	CONDUIT	2.72	0	12:30	3.55	0.49	0.60
P7	CONDUIT	1.86	0	12:29	3.58	0.57	1.00
P13	CONDUIT	1.42	0	12:30	3.73	0.43	0.50
P14	CONDUIT	1.43	0	12:30	3.95	0.46	0.47
P4	CONDUIT	4.08	0	12:30	4.13	0.85	0.86
P19	CONDUIT	3.09	0	12:34	4.34	0.94	1.00
P5	CONDUIT	5.52	0	12:35	4.43	0.78	0.96
P11	CONDUIT	3.51	0	12:28	4.47	0.54	1.00
P6	CONDUIT	6.64	0	12:35	4.58	0.65	1.00
P12	CONDUIT	2.10	0	12:32	4.68	0.51	0.77
P15	CONDUIT	1.44	0	12:30	4.91	0.22	0.40
P16	CONDUIT	1.70	0	12:30	5.24	0.33	0.48
P20	CONDUIT	4.26	0	12:29	5.85	1.02	1.00
P10	CONDUIT	7.73	0	12:29	6.30	1.70	1.00
P8	CONDUIT	8.01	0	12:33	6.52	1.28	1.00
P21	CONDUIT	17.18	0	12:31	7.35	1.36	0.93
P9	CONDUIT	17.20	0	12:29	7.36	1.58	0.93

Figure 6. Link flow

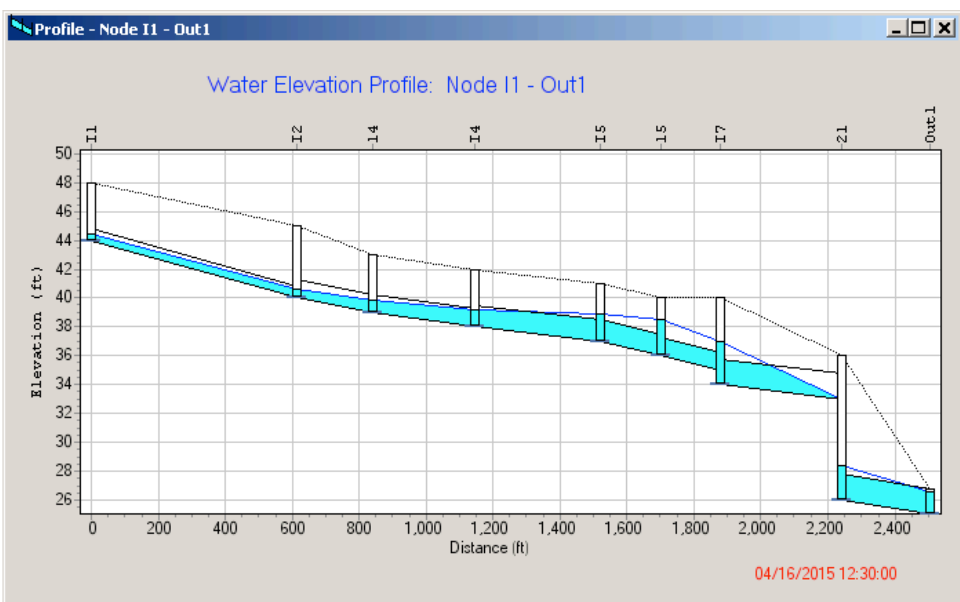

Figure 7. Profile from I1 to Outfall

Recommendations

For the proposed project, all pipes should follow the roadways in the neighborhood with the lengths shown in Figure 4. Pipe material will be reinforced concrete and should have the corresponding diameters as shown in Appendix C. There will be 13 pairs of inlets, whose locations are also shown in the figure in Appendix C. These choices ensure proper functioning of the storm water collection system for this subdivision in Harris County, since all requirements are met.

Cost Analysis

The total volume to be excavated will be approximately [redacted] cubic yards. It will cost $[redacted] per cubic yard for the first [redacted] cubic yards and $[redacted] per cubic yard for the remaining volume. Backfilling the total volume will cost $[redacted] per cubic yard. The 10 inch diameter non-reinforced concrete pipe will cost $[redacted] per linear foot; the 12 inch, 15 inch, 18 inch, and 21 inch diameter reinforced concrete piping will cost $[redacted], $[redacted], $[redacted], and $[redacted] per linear foot respectively. Type A-1 manholes with a diameter of [redacted] ft were selected. Each manhole will cost $[redacted] plus $[redacted] per vertical foot. The 10 ft curb–on-grade inlets will cost $[redacted] plus $[redacted] per vertical foot while the 20 ft curb-on-grade inlets will cost $[redacted] plus $[redacted] per vertical foot. A storm water pollution prevention plan is to be considered and will cost $[redacted]. Grubbing, clearing, pavement and the cost of the total land were not included in this analysis because it is to be subcontracted out. After contingencies and engineering, the project will require funds of approximately $[redacted]. The preliminary cost estimation is shown in Table 1.

Table 1. Preliminary Cost Estimate

LAND				
Description	**Unit**	**Qty.**	**Unit Cost**	**Total Cost**
EXCAVATION:	CY	[redacted]	[redacted]	[redacted]
	CY	[redacted]	[redacted]	[redacted]
Excavation Subtotal				[redacted]
BACKFILL:	CY	[redacted]	[redacted]	[redacted]
Backfill Subtotal				[redacted]
UTILITIES				
Description	**Unit**	[redacted]	**Unit Cost**	**Total Cost**
DRAINAGE				
10-in RCP	LF	[redacted]	[redacted]	[redacted]
12-in RCP	LF	[redacted]	[redacted]	[redacted]
15-in RCP	LF	[redacted]	[redacted]	[redacted]
18-in RCP	LF	[redacted]	[redacted]	[redacted]
21-in RCP	LF	[redacted]	[redacted]	[redacted]
Pipeline Subtotal				[redacted]
2-ft Diameter Manhole				
6-ft Depth	EA	[redacted]	[redacted]	[redacted]
6.25-ft Depth	EA	[redacted]	[redacted]	[redacted]
6.5-ft Depth	EA	[redacted]	[redacted]	[redacted]
6.75-ft Depth	EA	[redacted]	[redacted]	[redacted]
7-ft Depth	EA	[redacted]	[redacted]	[redacted]
7.5-ft Depth	EA	[redacted]	[redacted]	[redacted]
12.25-Depth	EA	[redacted]	[redacted]	[redacted]
Manhole Subtotal				[redacted]
10-ft Inlet				
6.75-ft depth	EA	[redacted]	[redacted]	[redacted]
10-ft Inlet Subtotal				[redacted]
20-ft Inlet				
6-ft Depth	EA	[redacted]	[redacted]	[redacted]
6.25-ft Depth	EA	[redacted]	[redacted]	[redacted]
6.5-ft Depth	EA	[redacted]	[redacted]	[redacted]
7-ft Depth	EA	[redacted]	[redacted]	[redacted]
7.25-ft Depth	EA	[redacted]	[redacted]	[redacted]
8-ft Depth	EA	[redacted]	[redacted]	[redacted]
20-ft Inlet Subtotal				[redacted]
ADDITIONAL ITEMS				
Pollution Prevention	EA	[redacted]	[redacted]	[redacted]
Pollution Prevention Subtotal				[redacted]
			Subtotal	[redacted]
			Contingencies (15%)	[redacted]
			Engineering (15%)	[redacted]
			TOTAL	[redacted]

4 Using SWMM for Sanitary Sewer Design

Sanitary Sewer Design Using EPA Storm Water Management Model (SWMM)

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ABSTRACT: Traditional sanitary sewer design uses detailed design tables that contain all the necessary information to complete the design. An alternative approach is to use modeling software, such as the EPA SWMM model. This paper details the application of the EPA SWMM model to design the sanitary sewer system of a proposed 62 lot development. Typically SWMM is thought of almost exclusively as a storm water system analysis tool, but this application shows how it can also be used for sanitary sewer analysis and design. The paper details how the SWMM model parameters were set to handle infiltration and inflow as well as base residential sanitary flows. Comput Appl Eng Educ 18: 203–212, 2010; Published online in Wiley InterScience (www.interscience.wiley.com); DOI 10.1002/cae.20124

Keywords: SWMM; sanitary sewer; hydraulic model

INTRODUCTION

One of the cornerstone topics in Civil Engineering hydraulic design is the design of sanitary sewer systems. Traditionally these designs are done by constructing detailed tables that contain all the necessary information to complete the design [1–4]. An example of such tables is shown in Figure 1. Note that the tables in Figure 1 contains only the information required to design a sewer for three blocks on one street, Wayne Road. While such tables can be constructed using Excel, they represent a tedious procedure that is prone to errors. These design tables are very time consuming to produce and become very unwieldy when used to design even moderate size systems.

An alternative approach is to use hydraulic modeling software. An example of such hydraulic modeling software is the EPA Storm Water Management Model (SWMM). Typically SWMM is used for designing storm water systems [5]. It can also be used for sanitary sewer modeling if the user adjusts the model parameters accordingly. This article details an application of SWMM to design a sanitary sewer system for a development that was constructed in Cass County, Missouri.

SWMM MODEL OVERVIEW

SWMM was developed by the EPA and is free, public domain software. SWMM is a dynamic rainfall-runoff

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Location					Basic Data			Domestic Flow						Infiltration/Inflow			
1	2		3		4	5	6	7	8	9	10	11	12	13	14	15	16
Street	From	MH	To	MH	L (ft)	Area (ac)	Cumm. Area (ac)	Pop. Density (pers/ac)	Cumm. Pop.	Avg Unit Flow (gal/capita /day)	Cumm Avg Flow (cfs)	Peaking Factor	Cumm. Peak Flow (cfs)	Avg Infilt. (gal/ac/day)	Cumm. Avg Infilt (cfs)	Peaking Factor	Cumm. Peak I/I (cfs)
Wayne	James	78	Prince	79	740	19.17	19.17	30	575.21	110	0.10	4.5	0.44	1400	0.04	1.8	0.07
Wayne	Prince	79	Blane	80	830	16.51	35.68	25	987.97	110	0.17	4.5	0.76	1400	0.08	1.8	0.14
Wayne	Blane	80	Linden	122	1215	12.78	48.47	30	1371.44	110	0.23	4.5	1.05	1400	0.11	1.8	0.19

Location					Design Flows		Sewer Design								
							Ground Elevation (ft)							Invert Elevation (ft)	
1	2		3		17	18	19	20	21	22	23	24	25	26	27
Street	From	MH	To	MH	Cumm. Avg Flow (cfs)	Cumm. Peak Flow (cfs)	Upper MH	Lower MH	Drop (ft)	Slope	Pipe Dia. (in)	Capacity (cfs)	Full Velocity (ft/s)	Upper MH	Lower MH
Wayne	James	78	Prince	79	0.14	0.52	115	102	13	0.0176	8	1.61	4.60	108.33	95.33
Wayne	Prince	79	Blane	80	0.25	0.90	102	99	3	0.0036	10	1.32	2.42	95.17	92.17
Wayne	Blane	80	Linden	122	0.34	1.24	99	92	7	0.0058	10	1.67	3.06	92.17	85.17

Figure 1 Example of sewer design tables.

simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas [5]. SWMM tracks the quantity and quality of runoff generated within each subcatchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps. SWMM was first developed in 1971 and has undergone several major upgrades since then. It continues to be widely used throughout the world for planning, analysis and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well. The latest version of SWMM (SWMM 5.0) features a graphical interface.

EXAMPLE APPLICATION

The example that was used in class was based on a development that was built in the Town of Raymore, in Cass County, Missouri. The development consisted of 62 lots varying in size from 1/8th of an acre to $\frac{1}{2}$ an acre, plus a swimming pool and several roads, as shown in Figure 2.

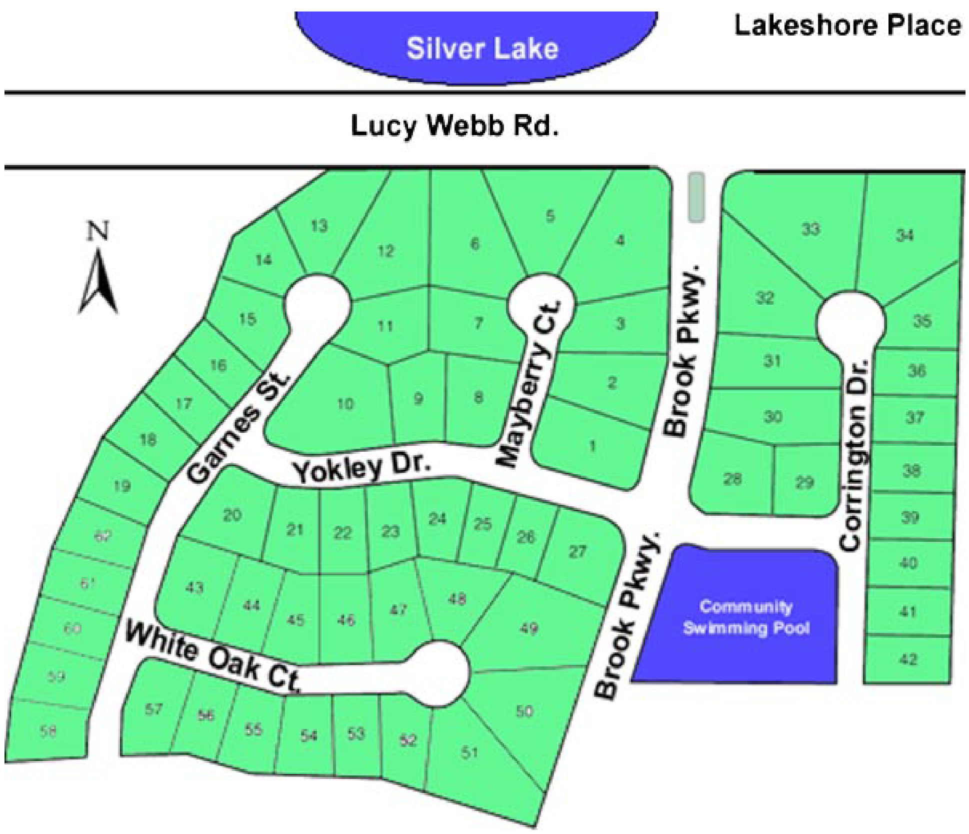


Figure 2 Site plan. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

Contour maps based on elevation data given in Appendix 1 were constructed. Design requirements were adapted from those used by the City of Ann Arbor in Michigan [6] as they included detailed specifications on storm intensity and duration, inflow, and infiltration (I/I) requirements, and typical daily sanitary flow generation rates. These requirements are given in Appendix 2 and 3 and can be summarized as:

- Design dry weather flow = 300 gpd (Single family residence)
- Design dry weather flow = 20 gpd/capita (Swimming pool)
- Peak dry weather flow = 400% design flow
- Design storm = Type II SCS, 6 hr duration, 4″ total rainfall (see Appendix 4)
- I/I = 10% of rainfall enters sewers immediately
- Flow can not exceed 90% of pipe capacity.

Additional requirements that were to be met included:

- Only 1 connection to existing trunk main on Lucy Webb Road
- Minimum depth of cover in streets to be 6 ft
- Minimum depth of cover for lateral is 3 ft
- Minimize use of easements
- Minimum pipe size in streets is 8″
- Minimum lateral size is 6″

SWMM MODEL SET UP

The site plan was imported into SWMM and used as a background map, onto which the sewer network would be superimposed. This is shown in Figure 3. The various inputs that were required are detailed below.

Rainfall

As per the City requirements, a 6 hr storm that delivers 4″ of rain was constructed based on the SCS Type II rainfall distribution. This was set up as a time series in SWMM.

Subcatchments and I/I Requirements

Each block of land, plus the pool, was defined as a subcatchment and its area calculated. The elevation

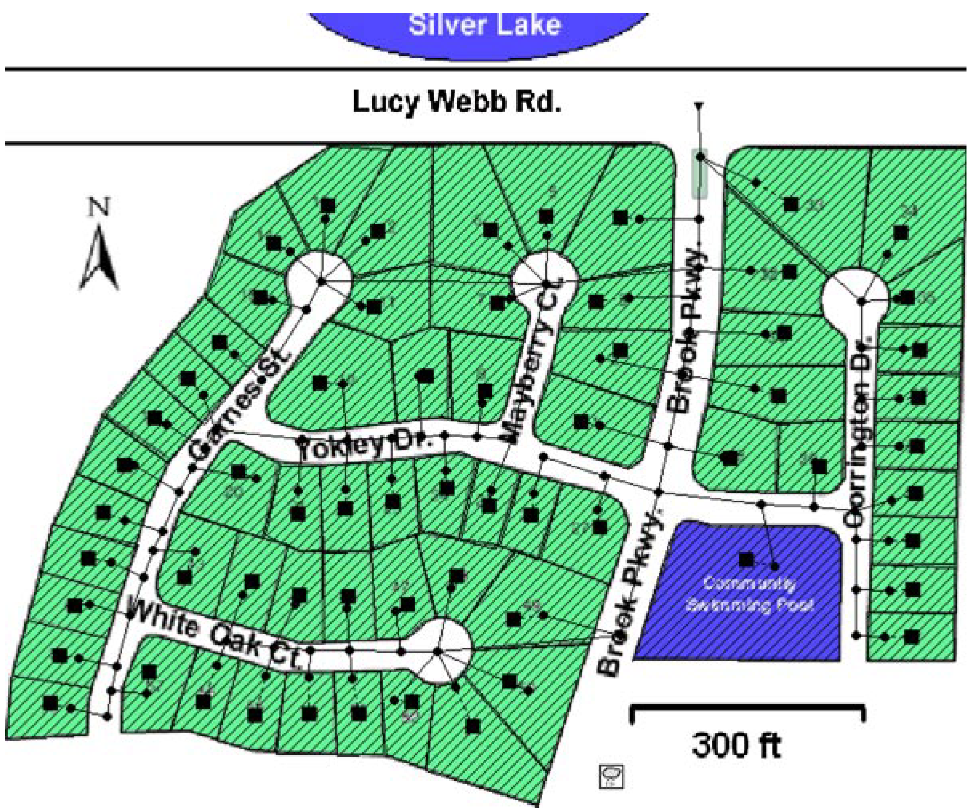


Figure 3 SWMM network. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

of the center of each block was computed and it was assumed that this elevation would represent the origin of each lateral connection, less the 3ft of cover that was required.

There were several possible ways to model the I/I requirements that 10% of the storm be allowed to enter the system immediately. The method used here was to let the 10% flow off the subcatchments and enter the junction where the laterals joined the main. This has the advantage of not forcing the I/I flow through the laterals, but instead having the flow first appear at a junction on the main. This is a realistic interpretation of what happens in reality, as manholes on the street are largely responsible for inflows in systems where roof leaders do not connect to the sanitary system.

This 10% storm runoff was achieved by setting the percent impervious of each block to 10% and forcing all of the impervious runoff to enter the system. To ensure that the runoff from the impervious area reached the system quickly, a width of 1000 ft was assigned to every subcatchment. This had the effect of reducing the effective length of the subcatchment, and therefore reduces the runoff travel time. The make sure that none of the storm was captured in depression storage, the depth of depression storage on the impervious area was set to zero.

On the other hand, none of the runoff from the pervious portion of subcatchment could be allowed to enter the system, otherwise more than the required 10% of the storm would be captured as I/I. To prevent this, the pervious area depression storage was set to a large value, 1000″. This value is large enough that any rainfall that lands on the pervious area is effectively held there in depression storage.

Junctions

The dry weather flow from the houses was input into the junction that acts as the start of the lateral connection. For each junction the invert elevation was set based on minimum cover requirements plus an additional foot to account for the pipe. The maximum depth was set as the difference between the invert and the ground elevation.

Conduits

It was assumed that PVC pipes would be used throughout the system and Manning's "n" was set to 0.01. The conduit type was circular (not "circular filled") and the depth is the pipe diameter.

Simulation Period

The model was run for 8hrs, beginning when the storm started and ending 2 hr after the storm finished. As the system is small no start time prior to the storm was necessary, especially as the crucial period occurs mid way through the design storm, 3 hr into the simulation.

MODEL RESULTS

One of the challenges of traditional design methods of sewer networks is the inherently static approach to a dynamic system. Sewer networks represent three dimensional, time variable systems. SWMM's output capabilities allow the user different options to view the system in multiple dimensions and time. This is one of the significant advantages of using SWMM, and directly impacts the users ability to see flaws in the design and make appropriate changes. An example of SWMM's output options include: color coded mapping of the system throughout a run; and the ability to plot hydraulic profiles along various sewer lines. Examples of these are detailed below.

A map of the system at peak flow is shown in Figure 4. In this figure the all three main components of the system are being tracked simultaneously: subcatchments, junctions and pipes. The user can change the legend to flag when critical design points occur. For this example the pipes would be set to turn red when their capacity reaches 90% (as per the design specifications) and the nodes would be set to turn red if they surcharge (overflow).

The route of flow that originates in Lot 58, and terminates at the connection to the existing trunk main in Lucy Webb Road (hereafter referred to as Route 1) is shown schematically in Figure 5. The corresponding SWMM generated hydraulic profile of Route 1 at peak flow is shown in Figure 6. In all of SWMM's output options the user has the ability to step through the model run in time, in effect creating an animation of the results.

An additional output feature of SWMM that is useful is the ability to look at a time series of flow at various points in the system. An example of this is shown in Figure 7. The nodes represent various manholes located along Route 1. This figure clearly demonstrates the peak flow timing within the system, as well as how the flow accumulates through the system.

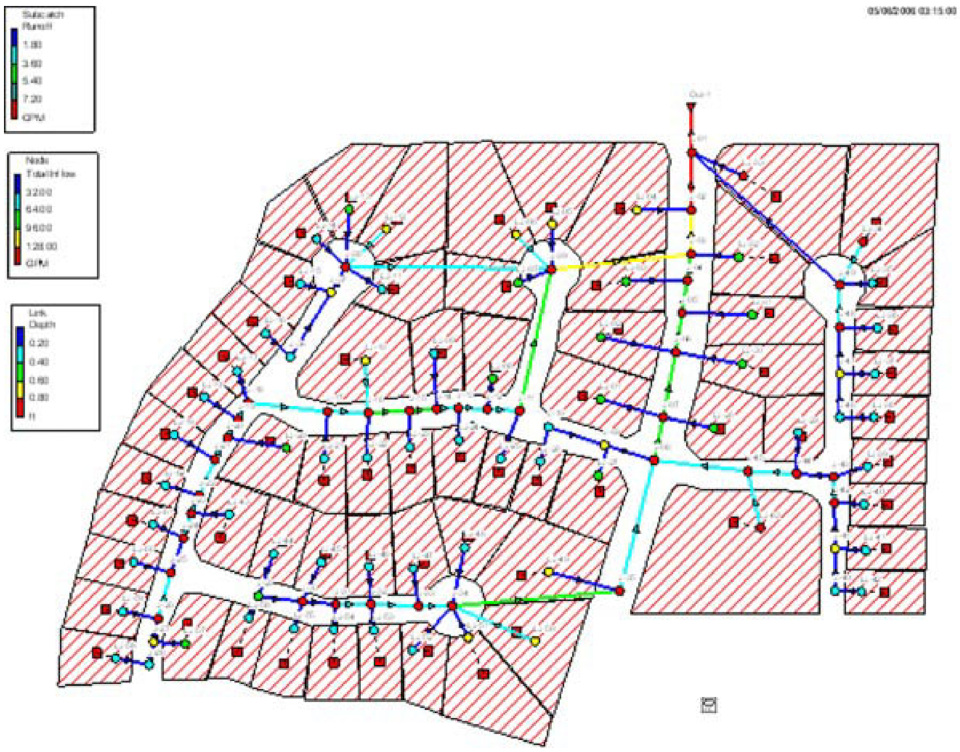


Figure 4 SWMM network at peak flow. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

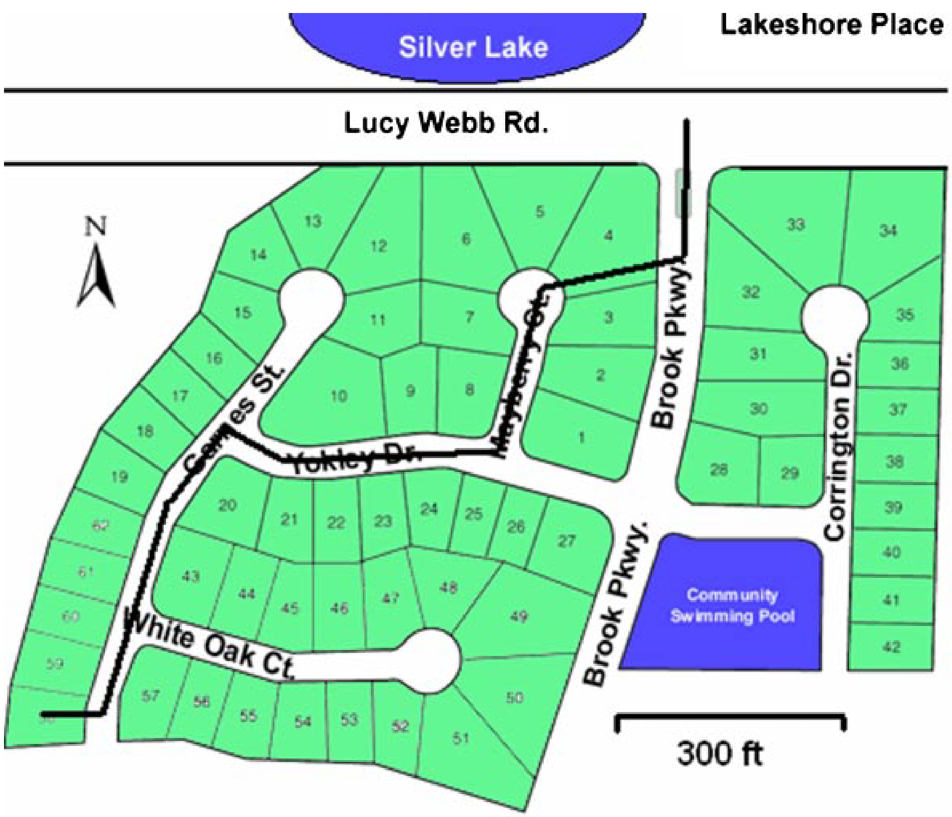


Figure 5 Flow route from Lot 58—Route 1. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

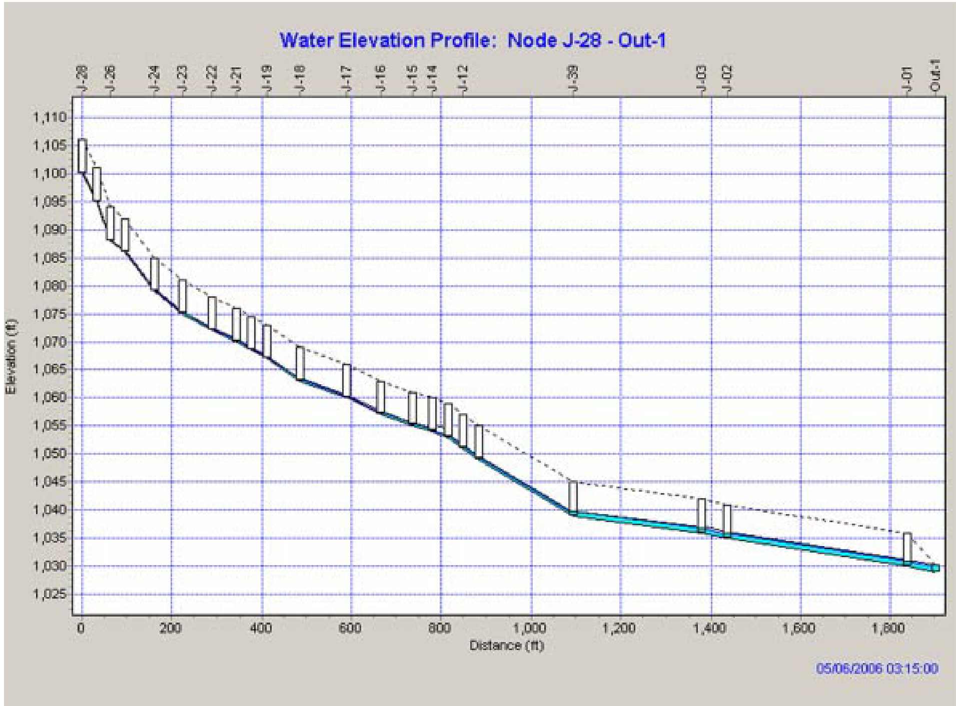


Figure 6 Hydraulic profile of Route 1. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

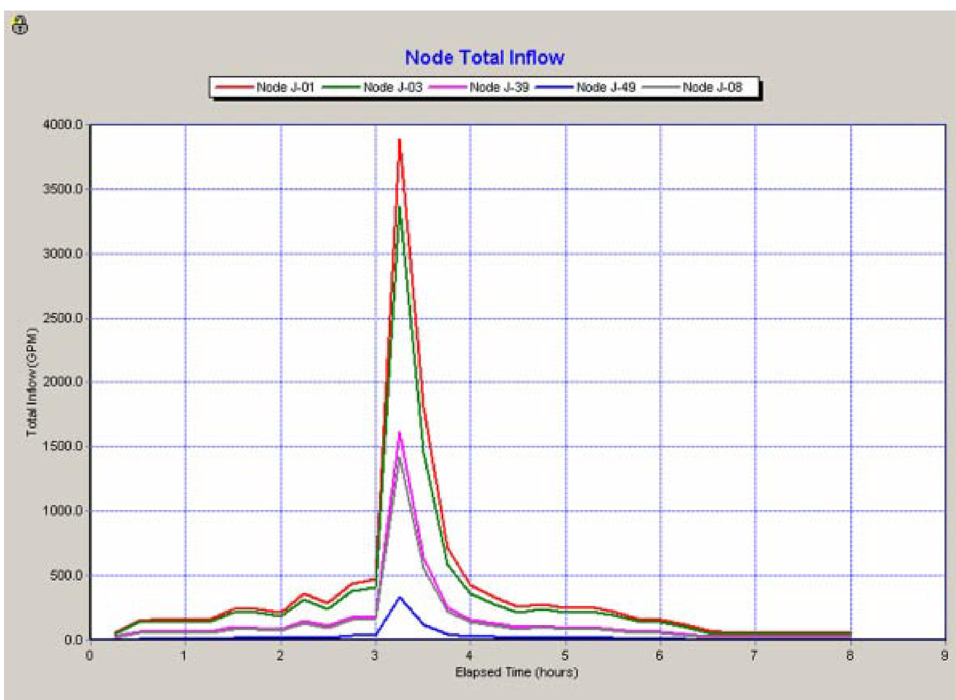


Figure 7 Time series of flow at various points along Route 1. [Color figure can be viewed in the online issue, which is available at www.interscience.wiley.com.]

CONCLUSIONS

Although primarily thought of as a storm water tool, the EPA Storm Water Management Model (SWMM) can be readily applied to the design of sanitary sewer systems if the appropriate parameters are set accordingly. In fact the use of SWMM is a much more practical way to approach a sanitary sewer design than the traditional tabular methods. As shown in the example documented in this paper, a SWMM sanitary sewer network is easy to set up and subsequently easy to modify as part of the design process. SWMM also comes with excellent post processing graphics that allows the user to view the system dynamically, both in plan and profile. Perhaps best of all is that SWMM is free, public domain software.

ACKNOWLEDGMENTS

The figures showing SWMM network and subsequent results were generated by Donald Dedrick, during the Hydraulic Design Course at Manhattan College in Spring 2006.

APPENDIX 1: SITE ELEVATION DATA

Reference Elevation Data (ft)

Silver Lake	1030.0
Lucy Webb at Brook Pkwy	1040.0
Brook Pkwy at South Pool	1060.0
SW Corner Lot 58	1120.0
SE Corner Lot 51	1070.0
SE Corner Lot 42	1080.0
NE Corner Lot 34	1050.0
NW Corner Lot 13	1050.0
NW Corner Lot 18	1080.0
SE Corner Lot 9	1060.0
SE Corner Lot 37	1065.0
NE Corner Lot 2	1050.0
NE Corner Lot 44	1085.0

APPENDIX 2: SEWER DESIGN REQUIREMENTS (BASED ON CITY OF ANN ARBOR, MI)

Table A. Design Dry Weather Flows

Type of facility or use	Design dry weather flow rate
Single family residence	300 gpd
Two family residence	600 gpd
Apartment to a single family unit (up to 400 sq. ft.)	150 gpd
Motels with kitchenettes, apartments, town houses, mobile homes, trailers, co-ops, etc. up to 600 sq. ft. of gross floor area	150 gpd/U
Motels with kitchenettes, apartments, town houses, mobile homes, trailers, co-ops, etc. with 601–1200 sq. ft. of gross floor area	225 gpd/U
Motels with kitchenettes, apartments, town houses, mobile homes, trailers, co-ops, etc with over 1200 sq. ft. of gross floor area	300 gpd/U
Motel unit less than 400 sq. ft.	100 gpd/U
Motel unit more than 400 sq. ft.	150 gpd/U
Hospital (without laundry)	150 gpd/bed
Hospital	300 gpd/bed
University housing, rooming houses, institutions	75 gpd/captia
Cafeteria (integral to an office or an industrial building)	2.5 gpd/capita
Non-medical office space	0.06 gpd/sq. ft. ground floor area
General industrial space	0.04 gpd/sq. ft. ground floor area
Medical arts (doctor, dentist, urgent care)	0.10 gpd/sq. ft. ground floor area
Auditorium/theater	5 gpd/seat
Bowling alley/tennis court	100 gpd/court-alley + food
Nursing home	150 gpd/bed
Church	1.5 gpd/captia
Restaurant (16 seat minimum or any size with dishwasher)	30 gpd/seat
Restaurant (fast food)	20 gpd/seat
Wet store—food processing	0.15 gpd/sq. ft. ground floor area
Wet store no food (barber shop, beauty salon, etc.)	0.10 gpd/sq. ft. ground floor area

(*Continued*)

Table A. (*Continued*)

Type of facility or use	Design dry weather flow rate
Dry store (no process water discharge)	0.03 gpd/sq. ft. ground floor area
Catering hall	7.5 gpd/captia
Market	0.05 gpd/sq. ft. ground floor area
Bar, tavern, disco	15 gpd/occupant + food
Bath house	5 gpd/occupant + 5 gpd/shower
Swimming pool	20 gpd/capita
Service stations	300 gpd/double hose pump
Shopping centers	0.02 gpd/sq. ft. ground floor sales area
Warehouse	0.02 gpd/sq. ft. ground floor area
Laundry	425 gpd/machine
Schools, nursery, and elementary	10 gpd/student
Schools, high, and middle	20 gpd/student
Summer camps	160 gpd/bed
Spa, country club	0.30 gpd/sq. ft. ground floor area

Standards

Sanitary sewer connection for the proposed project will be allowed if only if all of the following three requirements are met:

1. Sanitary sewer trunks and laterals do not surcharge.
2. Total of existing dry weather peaks, wet weather flows, and the project's proposed peak dry weather flow is less than 90% of sewer pipe design capacity.
3. No historical reported backups for the sanitary sewer trunk system to which the proposed project will be connected. Sanitary sewer trunk system includes trunk sewer and flow contributing laterals.

Guidelines for Calculating Dry and Wet Weather Flows

Dry Weather Flow

- Use City of Ann Arbor Table B.1 to calculate design dry weather flows (see Appendix 3)
- Project's proposed peak dry weather flow is four times design dry weather flow.
- Existing flows are available from Field Services Division, Water Utilities Department.

Wet Weather Flow

- The rain storm to be used for analysis is four inches of rain in a 6-hr period (100-year event). Use SCS Type II Distribution at 15 min intervals (see Appendix 4)
- 6–10 percent of the rainfall enters the sanitary sewer instantaneously. Model scenario run using 10% level.

APPENDIX 3: CALCULATIONS FOR DESIGN DRY WEATHER FLOWS

Table B.1 Design Dry Weather Flows

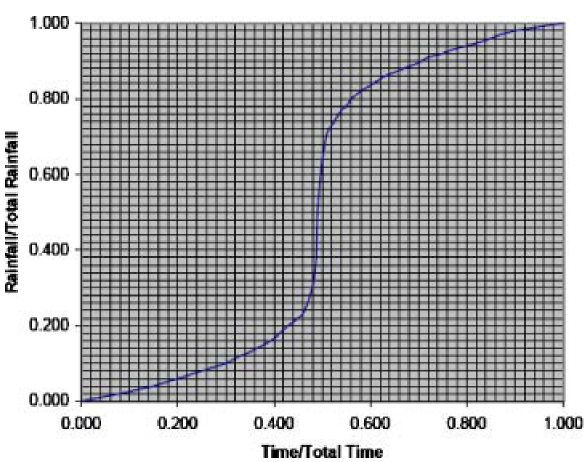
Type of facility or use	Design dry weather flow rate
Single family residence	300 gpd
Two family residence	600 gpd
Apartment to a single family unit (up to 400 sq. ft.)	150 gpd
Motels with kitchenettes, apartments, townhouses, mobile homes, trailers, co-ops, etc. up to 600 sq. ft. of gross floor area	150 gpd/U
Motels with kitchenettes, apartments, townhouses, mobile homes, trailers, co-ops, etc. up to 601–1200 sq. ft. of gross floor area	225 gpd/U
Motels with kitchenettes, apartments, townhouses, mobile homes, trailers, co-ops, etc. up to greater than 1200 sq. ft. of gross floor area	300 gpd/U
Motel unit less than 400 sq. ft.	100 gpd/U
Motel unit greater than 400 sq. ft.	150 gpd/U
Hospital (without laundry)	150 gpd/bed

(*Continued*)

Table B.1 *(Continued)*

Type of facility or use	Design dry weather flow rate
Hospital	300 gpd/bed
University housing, rooming house, institutions	75 gpd/capita
Cafeteria (integral to an office or industrial building)	2.50 gpd/capita
Non-medical office space	0.06 gpd/sf
General industrial space	0.04 gpd/sf
Medical arts (doctor, dentist, urgent care)	0.10 gpd/sf
Auditorium/Theater	5 gpd/seat
Bowling alley, tennis court	100 gpd/crt—alley
Nursing home	150 gpd/bed
Church	1.50 gpd/capita
Restaurant (16 seat minimum or any size with dishwasher)	30 gpd/seat
Restaurant (fast food)	20 gpd/seat
Wet store-food processing	0.15 gpd/sf
Wet store no food (barber shop, beauty salon, etc.)	0.10 gpd/sf
Dry store (no process water discharge)	0.03 gpd/sf
Catering hall	7.50 gpd/capita
Market	0.05 gpd/sf
Bar, tavern, disco	15 gpd/occupant
Bath house	5 gpd/occupant + 5 gpd/shower
Swimming pool	20 gpd/capita
Service stations	300 gpd/double pump
Shopping centers	0.02 gpd/sf
Warehouse	0.02 gpd/sf gr. area
Laundry	425 gpd/machine
Schools, nursery, and elementary	10 gpd/student
Schools, high, and middle	20 gpd/student
Summer camps	160 gpd/bed
Spa, country club	0.30 gpd sf gr floor area

APPENDIX 4: SCS TYPE II STORM DISTRIBUTION



Time/ total time	Rainfall/ total rainfall	Time/ total time	Rainfall/ total rainfall
0.000	0.000	0.520	0.730
0.400	0.100	0.530	0.750
0.100	0.25	0.540	0.770
0.150	0.040	0.550	0.780
0.200	0.060	0.560	0.800
0.250	0.080	0.570	0.810
0.300	0.100	0.580	0.820
0.330	0.120	0.600	0.835
0.350	0.130	0.630	0.860
0.380	0.150	0.650	0.870
0.400	0.165	0.670	0.880
0.420	0.190	0.700	0.895
0.430	0.200	0.720	0.910
0.440	0.210	0.750	0.920
0.450	0.220	0.770	0.930
0.460	0.230	0.800	0.940
0.470	0.260	0.830	0.950
0.480	0.300	0.850	0.960
0.485	0.340	0.870	0.970
0.487	0.3700	0.900	0.980
0.490	0.500	0.950	0.990
0.500	0.640	1.000	1.000

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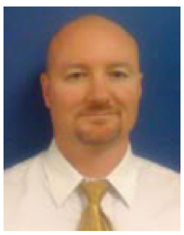
BIOGRAPHY



Scott Lowe is an Associate Professor in the Civil and Environmental Engineering Department at Manhattan College in Riverdale, New York. He has worked in the field of hydraulic engineering for many years and worked on projects throughout the US and overseas. The hydraulic design course featured in the paper also contains computer labs on water distribution system design using EPANET, river hydraulic analysis using HECRAS, and stormwater system design using HydroCAD.