

Preliminary water supply system design for subdivision of Kanata Lakes, Ottawa

CVG 4113

Winter 2013

Water Distribution System Design: Progress Report I

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Table of content

Site Selection

1

Population Estimate and Projections

2

Preliminary Layout of Water Supply System

4

Estimation of Water Demand at Nodes

5

Reference

9

List of figure

Figure 1 Area selected for water supply is approximately 1/4th of the residential area. 1

Figure 2 Kanata North Area (Kanata North, 2013)

2

Figure 3 Kanata South Area (Kanata South, 2013)

3

Figure 4 Preliminary layout of the water supply system in Beaverbrook, Kanata

4

List of table

Table 1 Water demand in each junction of the network system

5

Site Selection:

Kanata is one of the largest residential areas of Ottawa. It's located at 22 km west-southwest of downtown Ottawa along Highway 417 at a latitude of 45°18' North and a longitude of 75°55' West, with an area of 139 km² . Its northern end is just to the west of the Ottawa River. Located to the west of the Greenbelt, it is one of the largest of several communities that surround central Ottawa. It is a planned community. It was designed to have a mix of densities and commercial and residential properties. It contained large amounts of open space, and was to be surrounded by a greenbelt.

Marchwood Lakeside: Known as Kanata Lakes, this area is bounded by Campeau Drive to the south, Knudson Avenue and Wedlock Way to the east, Terry Fox Drive to the west and Terry Fox Drive to the north. It has a main arterial street called Kanata Ave. which runs north-south all the way to the 417. This area has the second of two urban, 18-hole golf courses in Kanata. This area is close to the Beaverbrook: Bounded by the Kanata North Business Park to the north, March Road to the east, Campeau Drive to the south and Knudson Avenue and Weslock Way to the west. It is the first and oldest neighborhood in Kanata, Beaverbrook is known for its residential housing and community facilities.

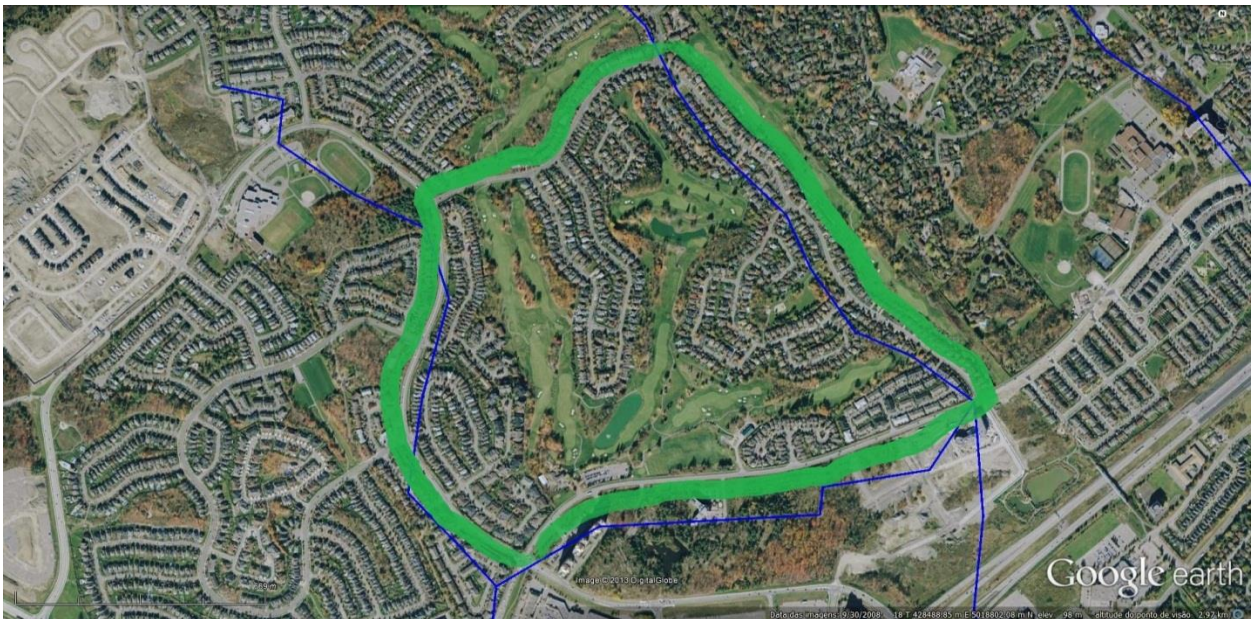


Figure 1: Area selected for water supply is approximately 1/4th of the residential area.

Population Estimates and Projections:

Population Growth:

The area of Kanata and the surrounding area were planned back in 1970.

Kanata itself can be separated into two areas, Kanata North and Kanata South.

Separating the North and South of Kanata is the Highway 417 that runs in the

middle of these areas. (Forcer City of Kanata, 5.7.1 Background, 2013) The

area of Kanata is about 230 hectares big and majority of the land developments

are currently occurring in the North area of Kanata.

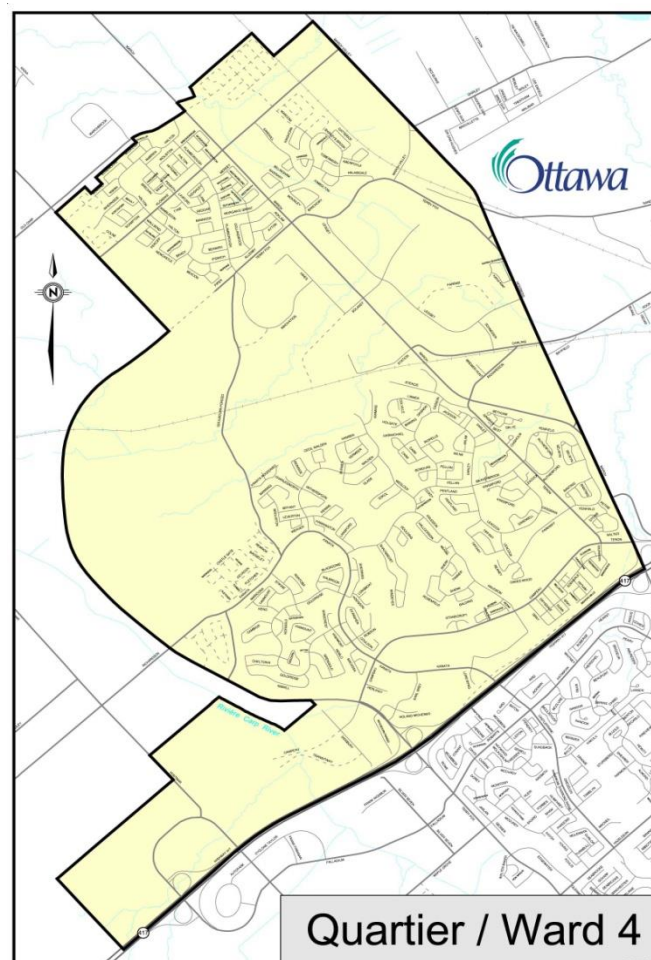


Figure 2: Kanata North Area (Kanata North, 2013)

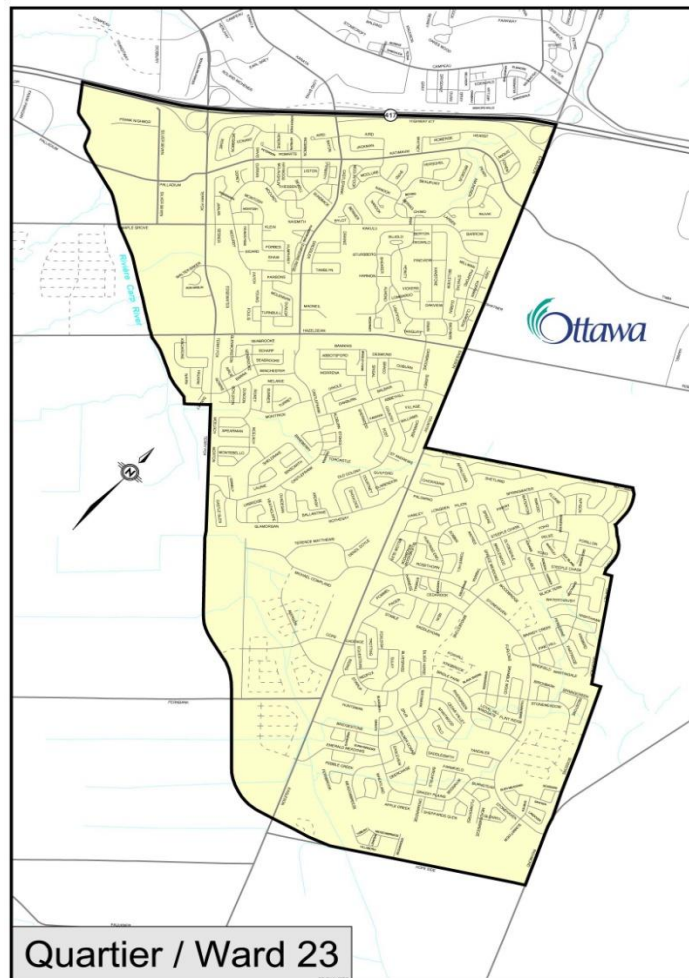


Figure 3: Kanata South Area (Kanata South, 2013)

As seen in Kanata North area figure, about 75% of the land is currently developed. Majority of the land is dedicated to residential homes. The office buildings and small warehouses are located in the middle of the map between the dense road areas. At the moment the North, East and West area are all forest. In the upcoming years the population of Kanata North will increase because of two reasons. The first reason is the construction of new single and townhouses located in the East area of Kanata North. What used to be an open field is now replaced by a long stretch of road that connects Morgan's Grant (top part of the map) to Centrum Shopping Center (bottom of the map beside the highway). With the new additional of houses comes new schools, the Kanata

North Elementary School was built a few years ago to accommodate new youths in that area. The second reason is the construction of new offices and high tech sector buildings which will lead to new jobs. A new headquarters for Research in Motion (R.I.M), a telecommunication company is currently building an office in that area. Also additional commercial buildings such as creational centers are currently being designed and built.

Preliminary Layout of the Water Distribution System:

The preliminary network layout was generated using EPANET 2.0 and consists on 117 nodes (junctions) and 128 pipes (see Figure 4).

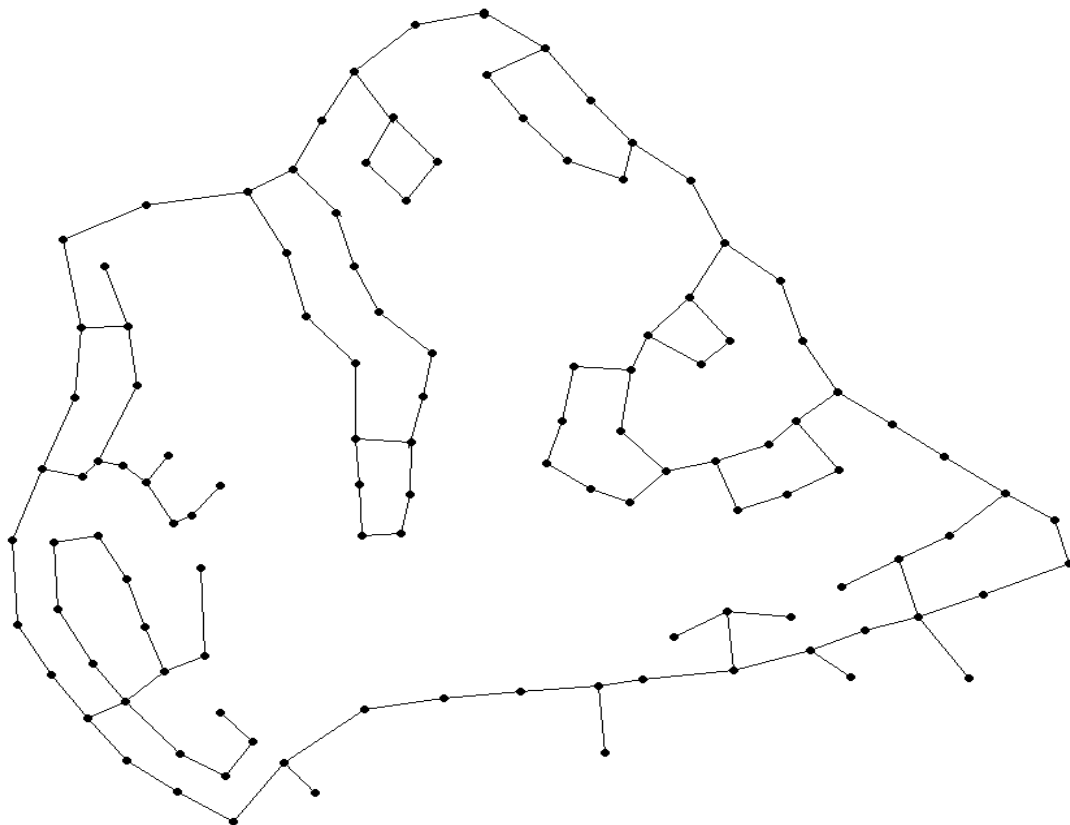


Figure 4: Preliminary layout of the water supply system in Beaverbrook, Kanata

Estimation of water demand at the junctions:

The calculation of the water demand for area in Kanata, Ottawa was done by considering the land use and population density. This particular area is part of the subdivision named Beaverbrook and is the first and oldest neighborhood in

Kanata. It consists mainly of small-lot residential single-family homes but it also comprehends the Kanata Golf & Country Club. Being the oldest neighborhood is it highly saturated with small-lots, single-family residential homes. For the calculation of the demand, the population density used was 20 persons/ha (Statistics Canada, 2001). Considering the residential usage area in the influence of the nodes and an average daily consumption per person of 329 litres/day (Statistics Canada, 2004). Additionally, a special consideration was made for the junction 20, which supplies water to the golf park. Golf Parks may utilize a lot of water for irrigation purposes during the summer. Considering the golf park green area, it was possible to calculate an average daily demand of 2 000 000 L/day during the peak weeks of the year. The following Table X shows the results of the calculations for the water demand in each junction of the network.

Table 1: Water demand in each junction of the network system

Domestic Consumption (l/per/day)	329
Daily max peak factor	2
1 hour peak factor	3
Daily min peak factor	0,5

Junctions	Area (m ²)	Single Family Small Lot (%)	Population	Demand	Total Demand (m ³ /day)
NODE 1	9274	50	23,185	7627,865	15255,73
NODE 2	8549	40	17,098	5625,242	11250,484
NODE 3	5261	40	10,522	3461,738	6923,476
NODE 4	2743	80	10,972	3609,788	7219,576
NODE 5	4686	70	16,401	5395,929	10791,858
NODE 6	6059	50	15,1475	4983,5275	9967,055
NODE 7	7074	75	26,5275	8727,5475	17455,095
NODE 8	5882	50	14,705	4837,945	9675,89
NODE 9	5243	70	18,3505	6037,3145	12074,629
NODE 10	5173	65	16,81225	5531,23025	11062,4605
NODE 11	5698	70	19,943	6561,247	13122,494
NODE 12	6862	40	13,724	4515,196	9030,392
NODE 13	6437	45	14,48325	4764,98925	9529,9785
NODE 14	4526	70	15,841	5211,689	10423,378
NODE 15	4200	85	17,85	5872,65	11745,3
NODE 16	4991	75	18,71625	6157,64625	12315,2925
NODE 17	4250	35	7,4375	2446,9375	4893,875
NODE 18	7511	85	31,92175	10502,25575	21004,5115
NODE 19	2096	15	1,572	517,188	1034,376

NODE 20	8303	10	4,1515	1365,8435	2731,687
NODE 21	8306	15	6,2295	2049,5055	4099,011
NODE 22	7291	10	3,6455	1199,3695	2398,739
NODE 23	4158	10	2,079	683,991	1367,982
NODE 24	3273	10	1,6365	538,4085	1076,817
NODE 25	6090	25	7,6125	2504,5125	5009,025
NODE 26	2090	30	3,135	1031,415	2062,83
NODE 27	1880	80	7,52	2474,08	4948,16
NODE 28	2696	45	6,066	2001995,714	4003991,428
NODE 29	4332	30	6,498	2137,842	4275,684
NODE 30	8731	70	30,5585	10053,7465	20107,493
NODE 31	6077	70	21,2695	6997,6655	13995,331
NODE 32	4818	80	19,272	6340,488	12680,976
NODE 33	5568	70	19,488	6411,552	12823,104
NODE 34	4909	55	13,49975	4441,41775	8882,8355
NODE 35	3641	90	16,3845	5390,5005	10781,001
NODE 36	4795	80	19,18	6310,22	12620,44
NODE 37	6802	80	27,208	8951,432	17902,864
NODE 38	4838	30	7,257	2387,553	4775,106
NODE 39	6876	50	17,19	5655,51	11311,02
NODE 40	3937	90	17,7165	5828,7285	11657,457
NODE 41	4843	90	21,7935	7170,0615	14340,123
NODE 42	6061	90	27,2745	8973,3105	17946,621
NODE 43	1271	65	4,13075	1359,01675	2718,0335
NODE 44	4589	40	9,178	3019,562	6039,124
NODE 45	3303	60	9,909	3260,061	6520,122
NODE 46	4816	40	9,632	3168,928	6337,856
NODE 47	1181	75	4,42875	1457,05875	2914,1175
NODE 48	1285	70	4,4975	1479,6775	2959,355
NODE 49	1036	90	4,662	1533,798	3067,596
NODE 50	1279	70	4,4765	1472,7685	2945,537
NODE 51	4911	40	9,822	3231,438	6462,876
NODE 52	1430	90	6,435	2117,115	4234,23
NODE 53	3749	40	7,498	2466,842	4933,684
NODE 54	5241	75	19,65375	6466,08375	12932,1675
NODE 55	4805	100	24,025	7904,225	15808,45
NODE 56	1687	85	7,16975	2358,84775	4717,6955
NODE 57	2295	90	10,3275	3397,7475	6795,495
NODE 58	2324	90	10,458	3440,682	6881,364
NODE 59	1989	95	9,44775	3108,30975	6216,6195
NODE 60	1880	85	7,99	2628,71	5257,42
NODE 61	2374	85	10,0895	3319,4455	6638,891
NODE 62	1789	90	8,0505	2648,6145	5297,229
NODE 63	1694	90	7,623	2507,967	5015,934
NODE 64	1706	85	7,2505	2385,4145	4770,829

NODE 65	1481	85	6,29425	2070,80825	4141,6165
NODE 66	2025	40	4,05	1332,45	2664,9
NODE 67	2712	65	8,814	2899,806	5799,612
NODE 68	4447	40	8,894	2926,126	5852,252
NODE 69	5910	45	13,2975	4374,8775	8749,755
NODE 70	3823	50	9,5575	3144,4175	6288,835
NODE 71	4769	40	9,538	3138,002	6276,004
NODE 72	3337	30	5,0055	1646,8095	3293,619
NODE 73	6543	25	8,17875	2690,80875	5381,6175
NODE 74	6191	95	29,40725	9674,98525	19349,9705
NODE 75	3119	90	14,0355	4617,6795	9235,359
NODE 76	2783	90	12,5235	4120,2315	8240,463
NODE 77	3159	100	15,795	5196,555	10393,11
NODE 78	5530	80	22,12	7277,48	14554,96
NODE 79	3417	90	15,3765	5058,8685	10117,737
NODE 80	2821	90	12,6945	4176,4905	8352,981
NODE 81	2273	85	9,66025	3178,22225	6356,4445
NODE 82	2830	90	12,735	4189,815	8379,63
NODE 83	2003	90	9,0135	2965,4415	5930,883
NODE 84	4814	50	12,035	3959,515	7919,03
NODE 85	4058	65	13,1885	4339,0165	8678,033
NODE 86	3089	90	13,9005	4573,2645	9146,529
NODE 87	2988	80	11,952	3932,208	7864,416
NODE 88	5040	20	5,04	1658,16	3316,32
NODE 89	4479	20	4,479	1473,591	2947,182
NODE 90	3799	40	7,598	2499,742	4999,484
NODE 91	2334	90	10,503	3455,487	6910,974
NODE 92	2887	95	13,71325	4511,65925	9023,3185
NODE 93	3622	30	5,433	1787,457	3574,914
NODE 94	2080	60	6,24	2052,96	4105,92
NODE 95	5931	20	5,931	1951,299	3902,598
NODE 96	3012	20	3,012	990,948	1981,896
NODE 97	4747	70	16,6145	5466,1705	10932,341
NODE 98	4036	15	3,027	995,883	1991,766
NODE 99	4629	40	9,258	3045,882	6091,764
NODE 100	2168	90	9,756	3209,724	6419,448
NODE 101	2656	80	10,624	3495,296	6990,592
NODE 102	2468	90	11,106	3653,874	7307,748
NODE 103	1996	90	8,982	2955,078	5910,156
NODE 104	2257	90	10,1565	3341,4885	6682,977

NODE 105	3936	95	18,696	6150,984	12301,968
NODE 106	2849	90	12,8205	4217,9445	8435,889
NODE 107	3486	90	15,687	5161,023	10322,046
NODE 108	3783	20	3,783	1244,607	2489,214
NODE 109	4744	10	2,372	780,388	1560,776
NODE 110	2390	15	1,7925	589,7325	1179,465
NODE 111	3156	90	14,202	4672,458	9344,916
NODE 112	3098	35	5,4215	1783,6735	3567,347
NODE 113	3143	90	14,1435	4653,2115	9306,423
NODE 114	6941	90	31,2345	10276,1505	20552,301
NODE 115	10820	85	45,985	15129,065	30258,13
NODE 116	11221	20	11,221	3691,709	7383,418
NODE 117	7933	60	23,799	7829,871	15659,742

Water supply system design for subdivision of Kanata Lakes, Ottawa

CVG 4113A

Winter 2013

Water Distribution System Design: Progress Report II

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Table of content

Primary Layout	3
Demand Estimation	4
Final Layout	4
Stimulation Results	6
Reference	10

List of figure

Figure 1 Preliminary water supply system layout	3
Figure 2 Peak factor for water-consumption flow rates	4
Figure 3 Final water supply system layout and pipe sizes	5
Figure 4: Reservoir placement	5
Figure 5: Topography (GreenKenue Ottawa Database)	6
Figure 6: Minimum hourly demand results	7
Figure 7: Maximum hourly demand results	8
Figure 8: Maximum daily demand with fire	9

Preliminary Layout

This preliminary layout is based on the nodes allocations in the first progress report. We encountered many problems with stagnation and velocity in the pipes.

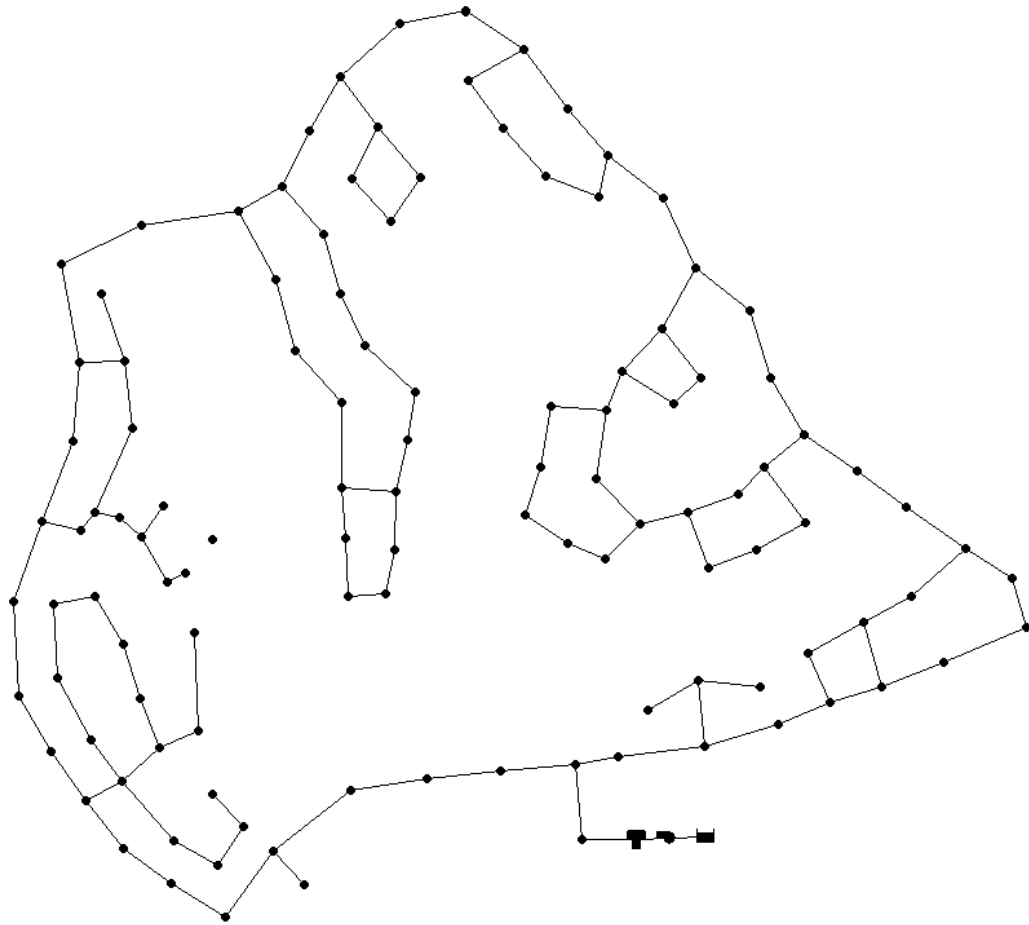


Figure 1: Preliminary water supply system layout

Demand Estimations

In the last progress report, the saturated population estimation for the selected area was made by comparing to the population in area of Beaverbrook's (similar/close area). The population estimate was around 2300 individuals.

Population	Peak factors		
	maximum daily of the year	maximum hourly of the year	minimum hourly of the year
under 500	3,00	4,50	0,40
500 – 1 000	2,75	4,13	0,40
1 001 – 2 000	2,50	3,75	0,45
2 001 – 3 000	2,25	3,38	0,45
3 001 – 10 000	2,00	3,00	0,50
10 001 – 25 000	1,90	2,85	0,60
25 001 – 50 000	1,80	2,70	0,65
50 001 – 75 000	1,75	2,62	0,65
75 001 – 150 000	1,65	2,48	0,70
over 150 000	1,50	2,25	0,70

From: Ontario Environment Ministry, *Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems and Storm Sewers* (May 1979), p. A-1.

Figure 2: Peak factor for water-consumption flow rates

Peak factors used in the simulation according to the population estimated:

- Minimum Hourly: 0.45
- Maximum Hourly: 3.38
- Daily Peak: 2.25

Final Layout

Continuing to refine the design through EPANET, changes in pipe diameters and connecting parts in the network are made to reduce stagnation problem. Reservoir's location was decided based on the water demand and the area's topography.

Most of the design is based on the area's topography. An additional reservoir was needed to supply the golf camp in the summer (high peak – the most south).

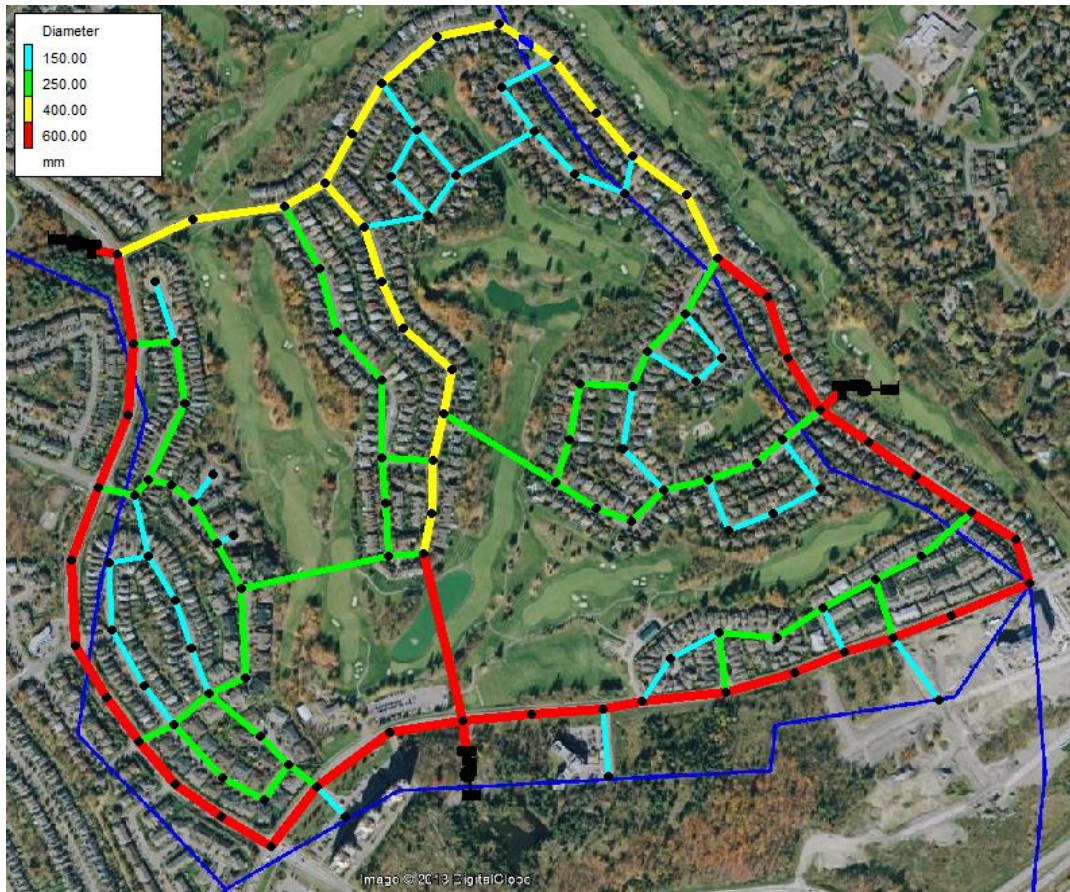


Figure 3: Final water supply system layout and pipe sizes

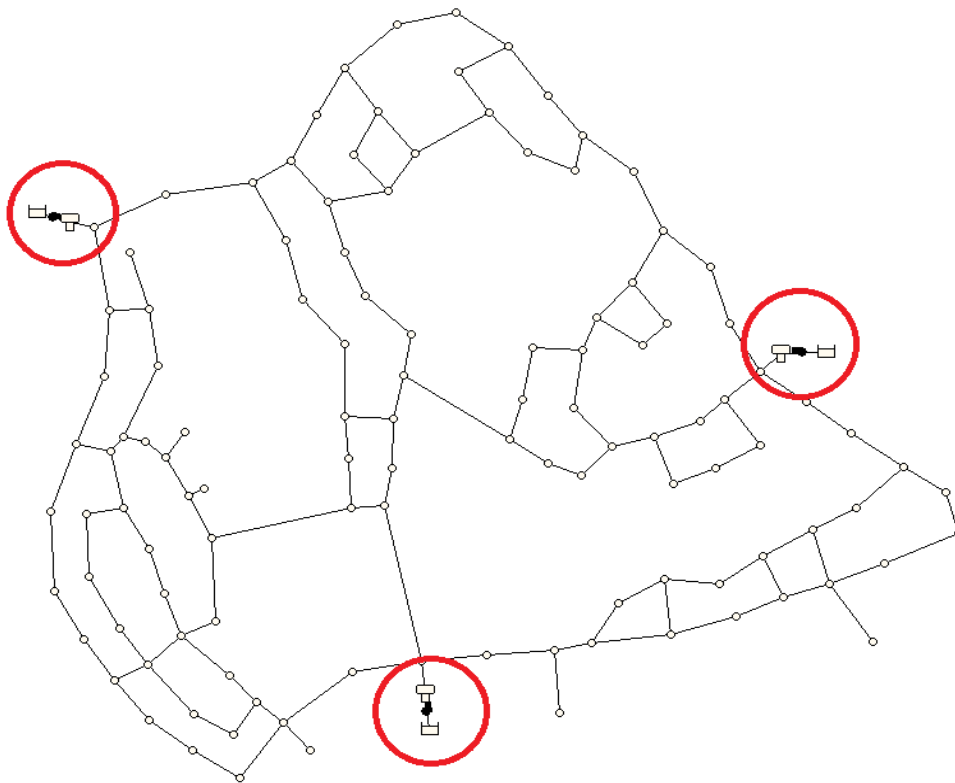


Figure 4: Reservoir placement

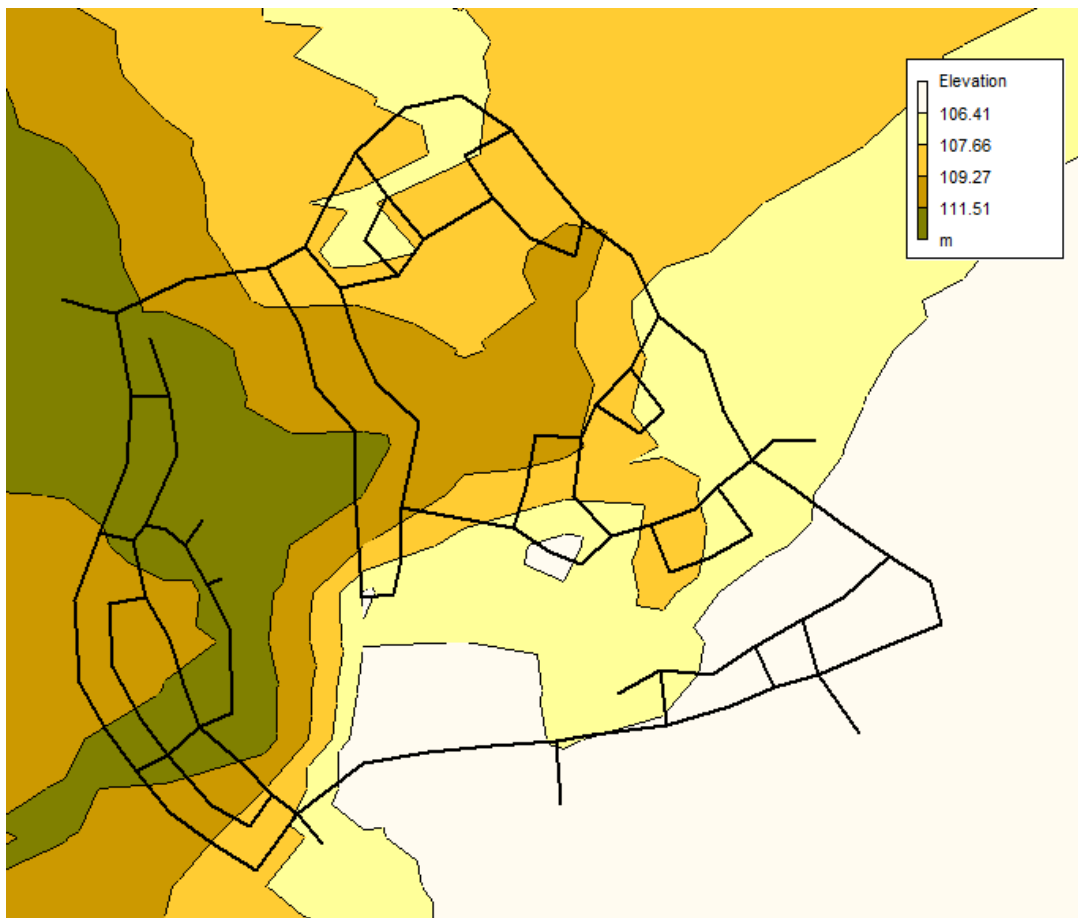


Figure 5: Topography (GreenKenue Ottawa Database)

Simulation Results

Different simulations are made to balance the network under various situations. The demand estimates for each scenario were imported into EPANET for subsequent simulations at each node. The scenarios are:

1. Minimum hourly demand
2. Maximum/Peak hourly demand
3. Maximum/Peak daily demand with fire

The population estimate of the area was approximately 2300 individuals as shown above. As the population is under 250,000, it is only necessary to stimulate one major fire along the maximum daily demand. Since it is not

conceivable to stimulate the fire event at every node, the stimulation was done to the node which proves to be the most critical for the design case. For this system, the worst scenario would be a major fire at Node 76 due to its location from the reservoirs.

1. Minimum Hourly Demand

In the following figure, dark blue shows very small velocities achieved, and light blue shows small velocities close to 0.3m/s (minimum). The entire system still works very well despite being in the minimum situation. All the pressures in pipes are fine.

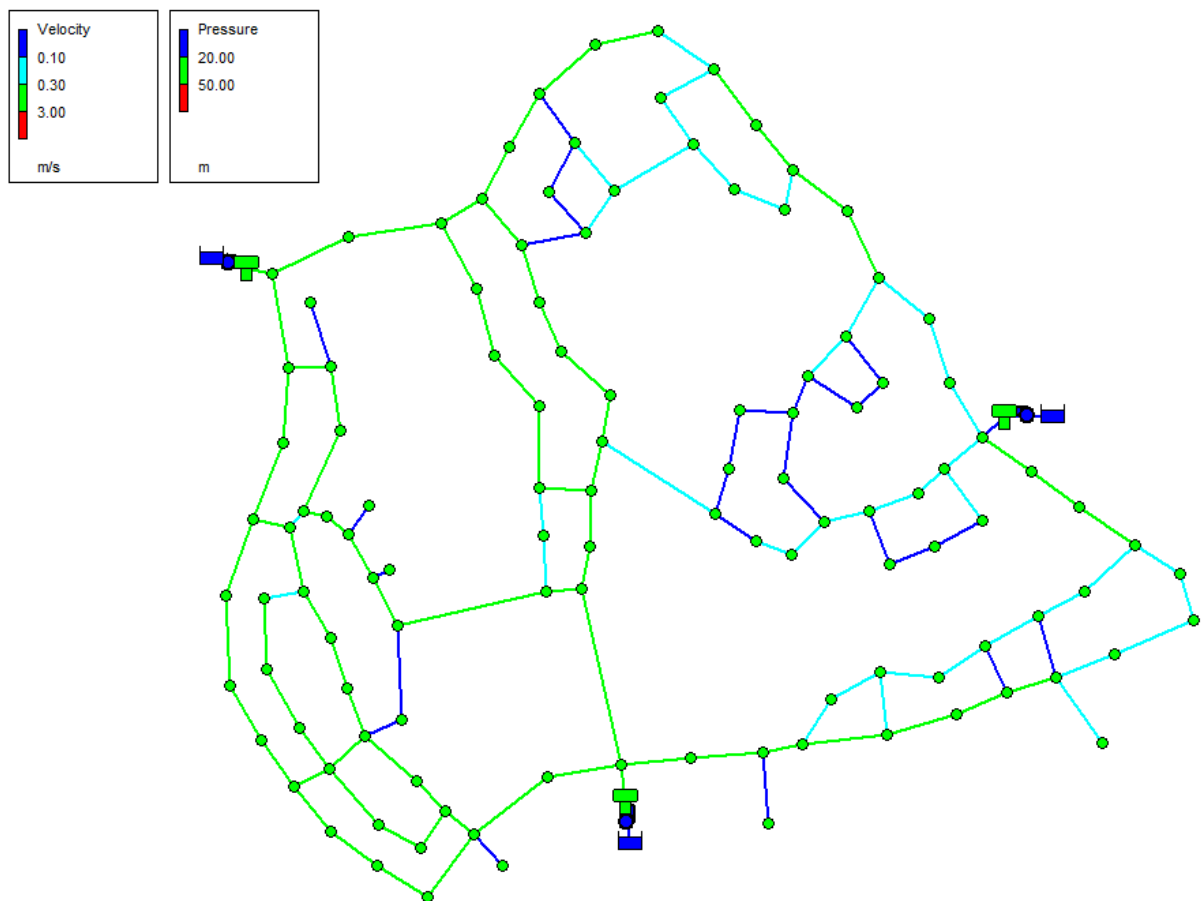


Figure 6: Minimum hourly demand results

2. Maximum Hourly Demand

Considering the peak hourly demand scenario, despite of having occasional low velocity flow in urban areas, most of the flows are close to the 0.3m/s minimum. All junctions are operating under the target pressure. The pipes close to the golf camp may need additional protection against corrosion by water pressure as they may convey water at higher speeds than 3m/s. All the pressures in the system work thoroughly.

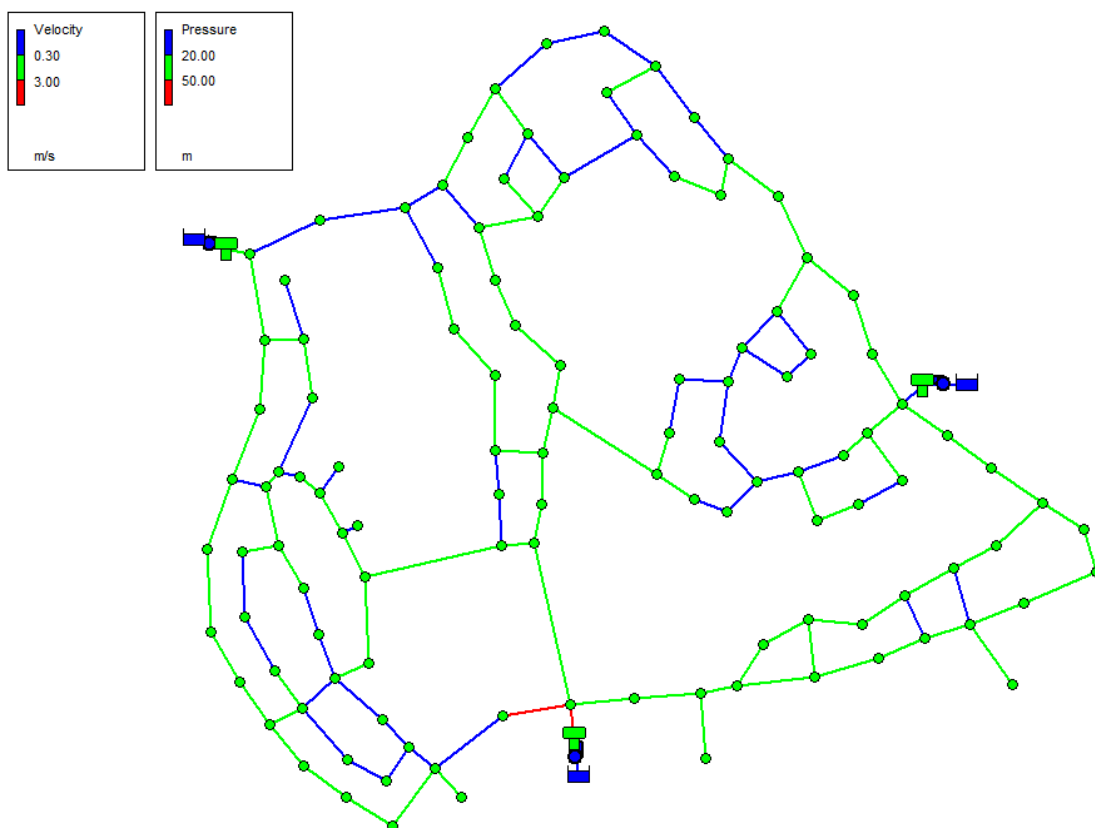


Figure 7: Maximum hourly demand results

3. Maximum Daily Demand with Fire

When considering the peak daily demand with fire scenario, the Node 76 is chosen because of its distance from the reservoirs. It is for being one of the worst locations for a fire scenario.

From the results of stimulation, the system still works well in the fire scenario. The pipes, close to the golf camp and the fire scenario, show velocities over 3m/s while the other part of the system still works fairly well. All the pipe pressures in the system remain in stable condition.

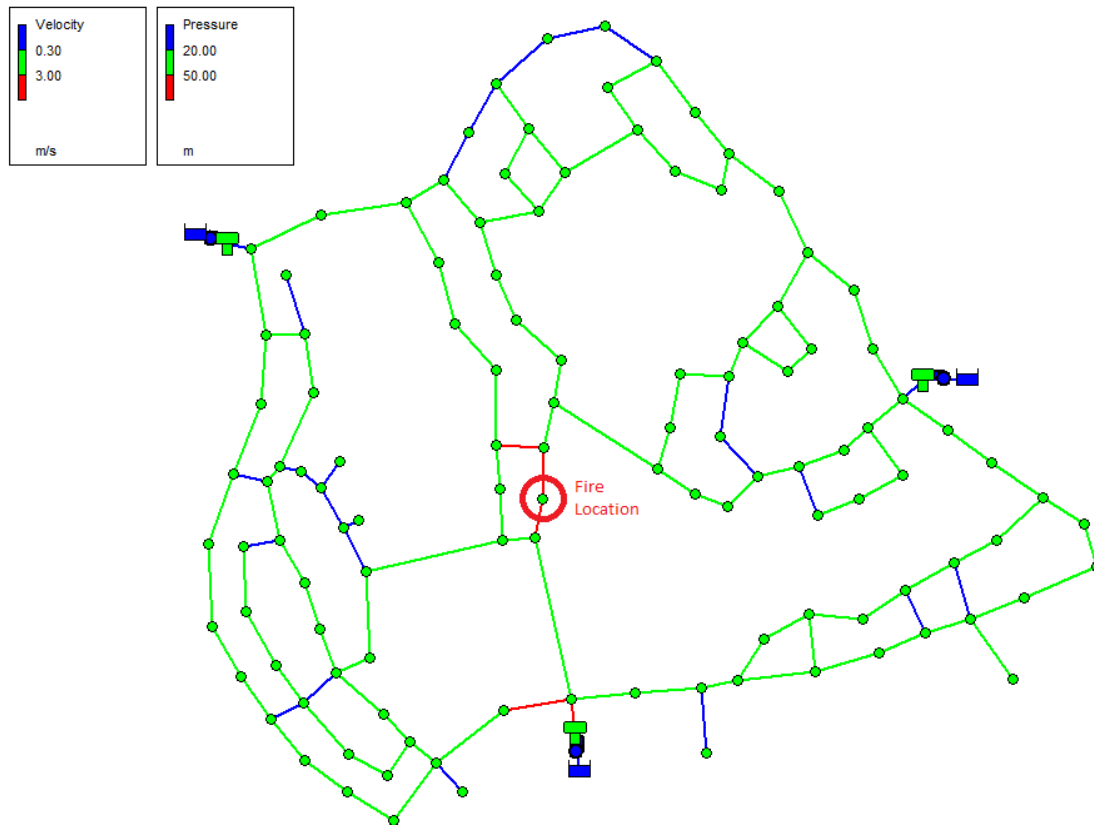


Figure 8: Maximum daily demand with fire

Preliminary water supply system design for subdivision of Kanata Lakes, Ottawa

CVG 4113

Winter 2013

Water Distribution System Design: Progress Report III

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Tuesday, March 26th, 2013

Table of content

Sanitary Sewer System.....	3
Velocity Calculation & Design Checks.....	9
Conclusion.....	10
References.....	11

List of figures

Figure 1: Sewer Collection System.....	3
Figure 2: Beaverbrook's nodes in GreenKenue's Ottawa Topography Database.....	4
Figure 3: General flow path and preliminary plan for collection in Beaverbrooks.....	5
Figure 4: Preliminary layout of the water supply system in Beaverbrook, Kanata	6
Figure 5: Pipes and Junctions Flow Collaboration in the Network.....	8
Figure 6: Special Pipe to use in the Network	10

List of tables

Table 1: Area of the Sub-Watersheds.....	7
Table 2: Elevation Data at the Nodes.....	7
Table 3: Flow Rates for each Node.....	7
Table 4: Maximum and Minimum Flow Rates that corresponds to each Pipe.....	7
Table 5: Total pipe flows.....	8
Table 6: Maximum and Minimum Velocity.....	9

Report 3 – Sanitary Sewer System

A sufficient sanitary sewer system must be designed to transport wastewater from domestic use. For the proposed design in this report, gravity is used as the main collection force because it is cheap and it avoids time lost from excavation. In order to do so, a topographical map of our selected area was used for the design in the sewer systems. An example of a branched sewer system is presented below.

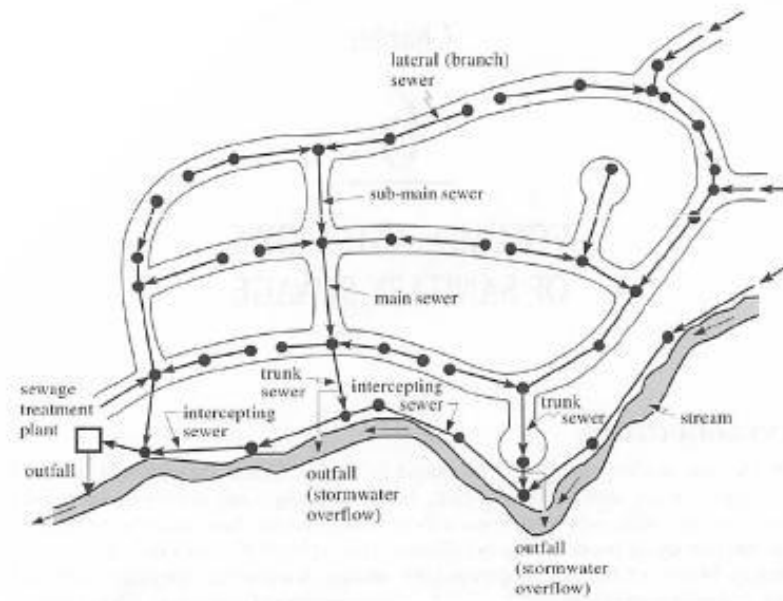


Figure 1: Sewer Collection System

To design the sewer system for our area, a program called Storm Water Management Model (SWMM) was used. SWMM is mainly used for stormwater analysis as the title suggests, but with the right information it can also generate sanitary sewer models. This program can also design any sizes of drainage systems which includes detention facilities. It can model combined sanitary sewer overflows and generate pollutant source for wasteload treatment.

For our pipes to use gravity as the main flow system, the elevation and locations of the nodes were uploaded onto another program called GreenKenue. Using GreenKenue, which is another system like SWMM, the approximate latitudes and longitudes of the nodes (junctions) and pipes (conduits) were found. A figure of the nodes with location in GreenKenue is displayed below.

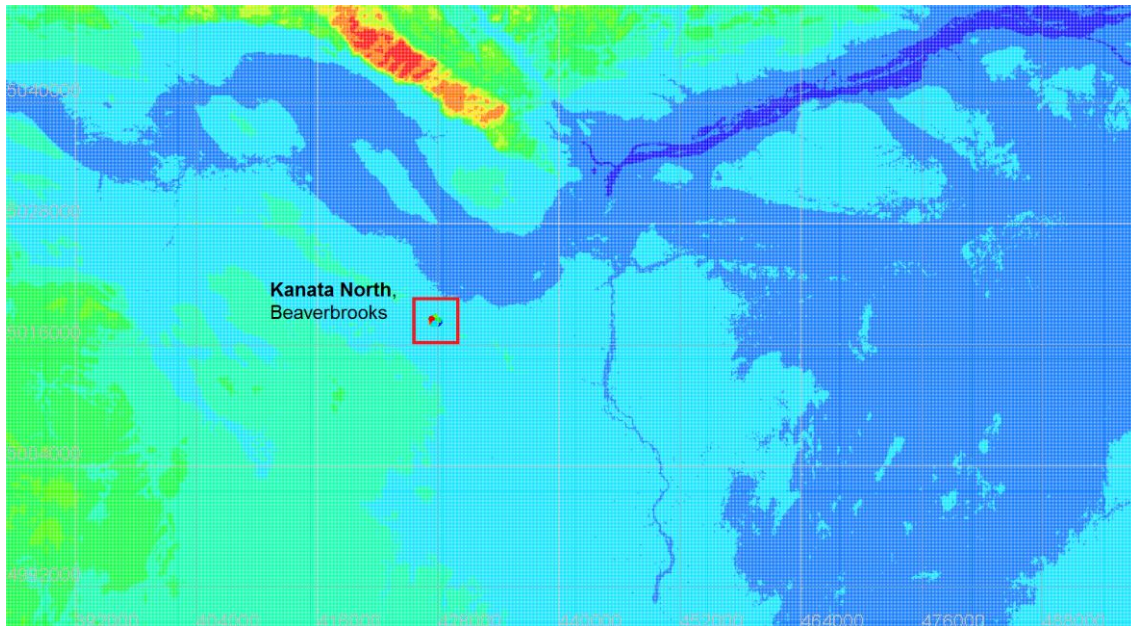


Figure 2: Beaverbrooks nodes in GreenKenue's Ottawa Topography Database

With the information obtained using GreenKenue, the flow path for Kanata North and potential sub-watersheds is presented below.

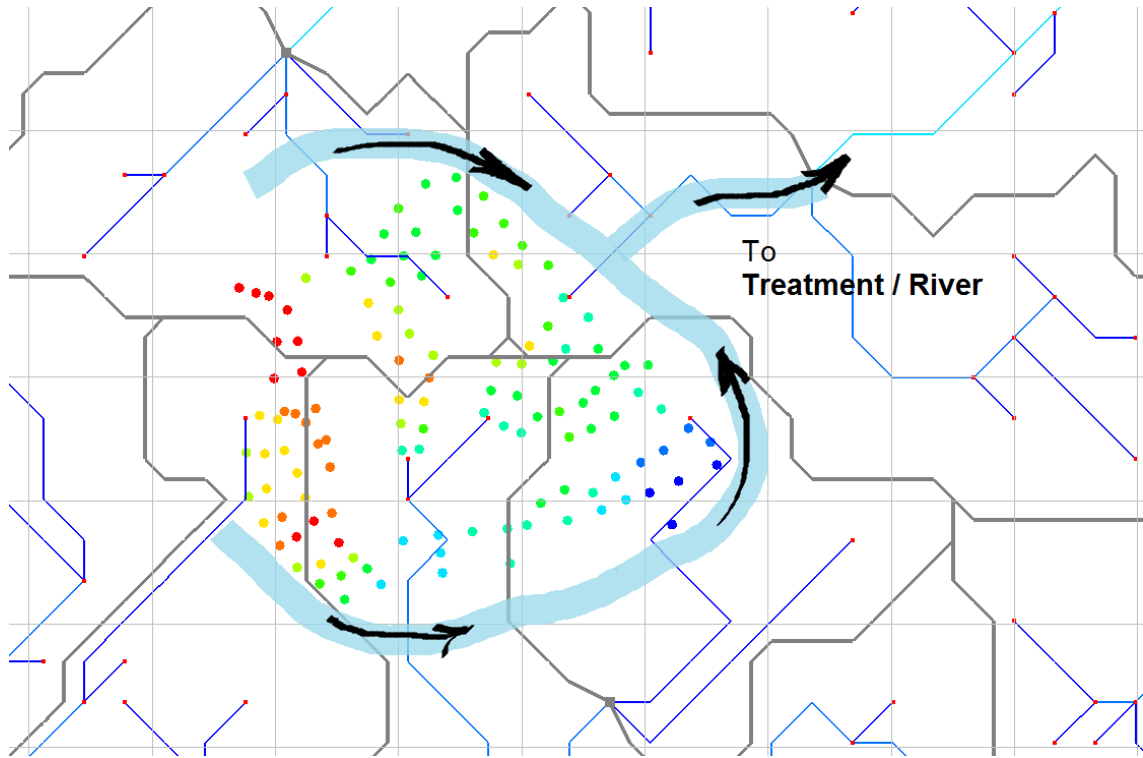


Figure 3: General flow path and preliminary plan for collection in Beaverbrooks.

For Figure 3:

- the thick light blue lines represent the proposed sewer lines
- arrows represent the direction of the flow with respect to gravity and the lay of the land
- the different color dots are the nodes represents different elevations

Uploading all the information into SWMM, a more simplified detail that shows the pipes and node with five different regions is shown below. For this design the amount of nodes is slightly reduced for easier calculations.

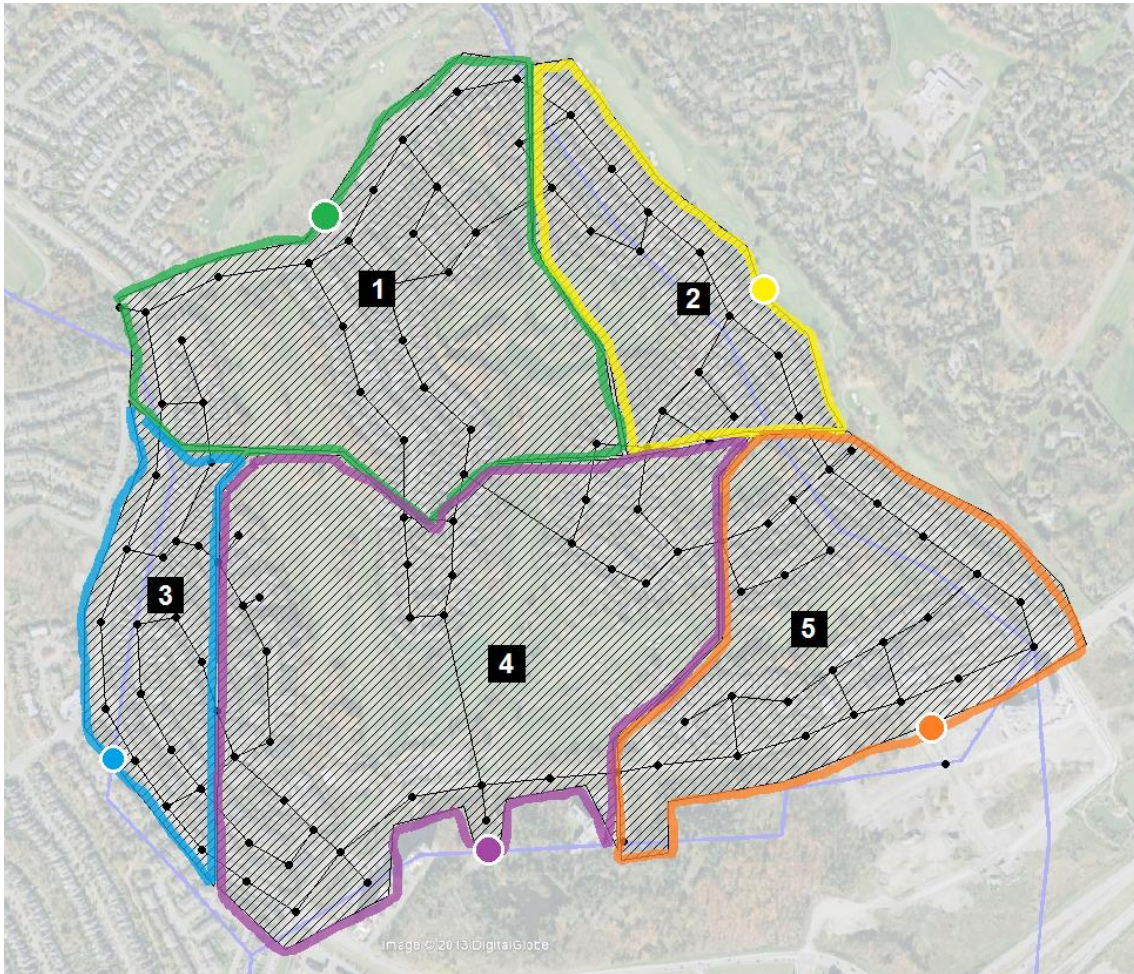


Figure 4: Sub-Watersheds and Sanitary Sewer System Project in SWMM

For Figure 4:

-different color lines are the sewer pipes and in locations where the colors meet, they are combined to be one pipe

-the large colored dots are potential outlets for each area

Once finished with the diagrams, all necessary data are calculated and recorded. To find the amount of wastewater productions at each node, the watershed areas, and elevation of the nodes are presented below. These numbers are all generated with our design in the program used as described above earlier.

Table 1: Area of the Sub-Watersheds

Sub-watershed	Area (ha)
1	20,86
2	9,93
3	7,17
4	29,54
5	16,62

Table 2: Elevation Data at the Nodes

Node	Initial Invert Elevation (2m below ground)
16	108,18
21	103,09
28	101,04
38	104,22
3	106,48

Assuming different percentages of use for each watershed (because of the Golf Park) and a density factor of 40.8 people per hectare, the estimated population is around 2,448. This number can be considered accurate as it closely matches the previous population estimation.

Table 3: Flow Rates for each Node

Node	Contributing Area (ha)	% Residential	Population	Sanitary Flow (L/s)
3	20,86	70,00	595,76	2,21
38	9,93	80,00	324,12	1,20
15	7,17	90,00	263,28	0,98
21	29,54	60,00	723,14	2,68
28	16,62	80,00	542,48	2,01

Table 4: Maximum and Minimum Flow Rates that corresponds to each Pipe

Infiltration (60L/pers.day)	Inflow (50 L/pers.day)	PF max	PF min	Q max	Q min
0,41	0,34	4,00	0,25	9,58	0,97
0,23	0,19	4,00	0,25	5,21	0,53
0,18	0,15	4,00	0,25	4,24	0,43
0,50	0,42	4,00	0,25	11,63	1,17
0,38	0,31	4,00	0,25	8,73	0,88

From Table 4, the maximum discharge is 11.63 L/s and the minimum is 1.17 L/s.

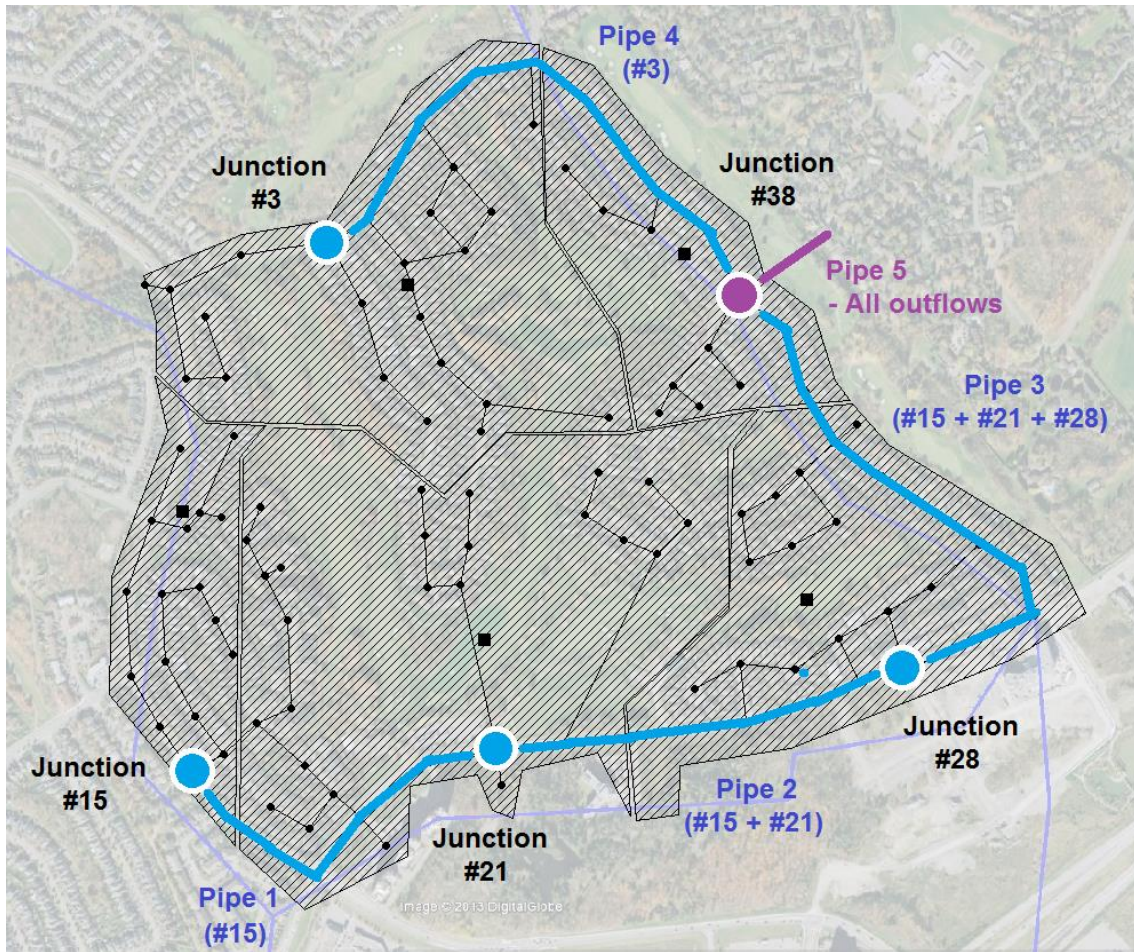


Figure 5: Pipes and Junctions Flow Collaboration in the Network

For Figure 5:

-the blue dots are the outlets that are connected to blue colored main pipe

-the purple dot and line is the outflow which directs the wastewater to the treatment plant

Table 5: Total pipe flows.

Pipe	Q max [L/s]	Q min [L/s]
#1	4,24	0,43
#2	15,87	1,60
#3	24,60	2,48
#4	9,58	0,97
#5	39,40	3,97

Table 5 represent the discharge rates of the combined pipes presented in Figure 4.

Velocity Calculation & Design Checks

For the calculation part, the first step is to get the diameter of the pipes. The theoretical diameters for the pipes were computed using the following equations:

Full pipe equation

$$V(m/s) = \frac{1}{n} \left(\frac{D}{4} \right)^{2/3} s^{0.5}$$

$$Q(m^3/s) = \frac{1}{n} \left(\frac{D}{4} \right)^{2/3} \frac{\pi D^2}{4} s^{0.5}$$

$$D(m) = \left[\frac{Q(m^3/s) * n * 4^{8/3}}{\pi S^{0.5}} \right]^{3/8}$$

V: speed (m/s)
 D: diameter (m)
 s: slope of the terrain
 n: manning's coefficient, 0.015
 Q: discharge (m³/s)
 Π: the constant pi, 3.14....

Using the equations described above, the Q_{max} and Q_{min} is calculated and presented in table 6 below.

Table 6: Maximum and Minimum Velocity

Pipe	Design Flow (max)	Design Flow (min)	Lenght (m)	Init. Elevation Begin	Init. Elevation End	Mod. Elevation end	Minimum Slope (m/m)
1	4,24	0,43	446,23	106,48	104,22	104,22	0,0043
2	15,87	1,60	326,93	108,18	103,09	103,09	0,0043
3	24,60	2,48	326,93	103,09	100,04	100,04	0,0043
4	9,58	0,97	494,59	101,04	106,22	98,22	0,0043
5	39,40	3,97	500	98,22	90,45	80	0,0043

Final Slope (m/m)	Diameter (Manning)	Selected Diameter (mm)	Full pipe flow (L/s)	Full pipe velocity (m/s)	Max V (m/s)	Min V (m/s)
0,0051	111	200	20,27	0,64	0,49	0,22
0,0156	148	200	35,53	1,13	0,70	0,55
0,0093	192	200	27,52	0,87	0,54	0,53
0,0057	148	200	21,50	0,68	0,42	0,33
0,0076	238	300	73,15	1,03	0,64	0,53

According to the design for pipes, the minimum allowed discharge cannot be less than 0.6m/s and the maximum flow cannot be greater than 3m/s. With that said, none of the pipes meet those requirements. Since the minimum velocity does not meet condition,

special designed pipes are needed to increase the velocity in the flow and prevent accumulations in the pipe. It is suggested the usage of pipes with a different cross section (Figure 5) that can be used to increase flow velocities due to smaller area in the bottom part of the pipe.



Figure 6: Special Pipe to use in the Network

The diameter of the whole network can be done by 200mm with the exception of the final outlet that leads to treatment which must be 300mm. The network works as intended considering Manning's roughness 0,015.

Conclusion

The difficulty in this report was getting the velocity close to the required range. We obtained the velocities between 0.22 and 0.70 m/s. Since the minimum velocity does not meet the requirements, special designed pipes are needed to increase the velocity in the flow and prevent accumulations in the pipe. The main problem was a higher slope than the minimum for the pipes because most joints were at the same elevation or even higher. Another problem was the pipe shape and diameter. We changed the pipes to egg shapes and the diameters to 200mm.



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Progress Report #4

Group 6

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1. Preliminary network properties

The same layout used in the sanitary system design is used in the stormwater system project, as both of them highly value the usage of gravity in order to conduct the water flow.

The preliminary setup of the network was determined to be as following:

Table 1: Preliminary stormwater system pipe specifications.

Pipe	Diameter (mm)	Slope (m/m)	Roughness
1	200	0,0051	0,015
2	200	0,0156	0,015
3	200	0,0093	0,015
4	200	0,0057	0,015

The following diagram explains the network design and the stormwater catchment watersheds. The pipes #1, #2, #3 and #4 collect the stormwater circling the region and all the flows combined are directed to the main outflow (towards the river) in pipe #5.

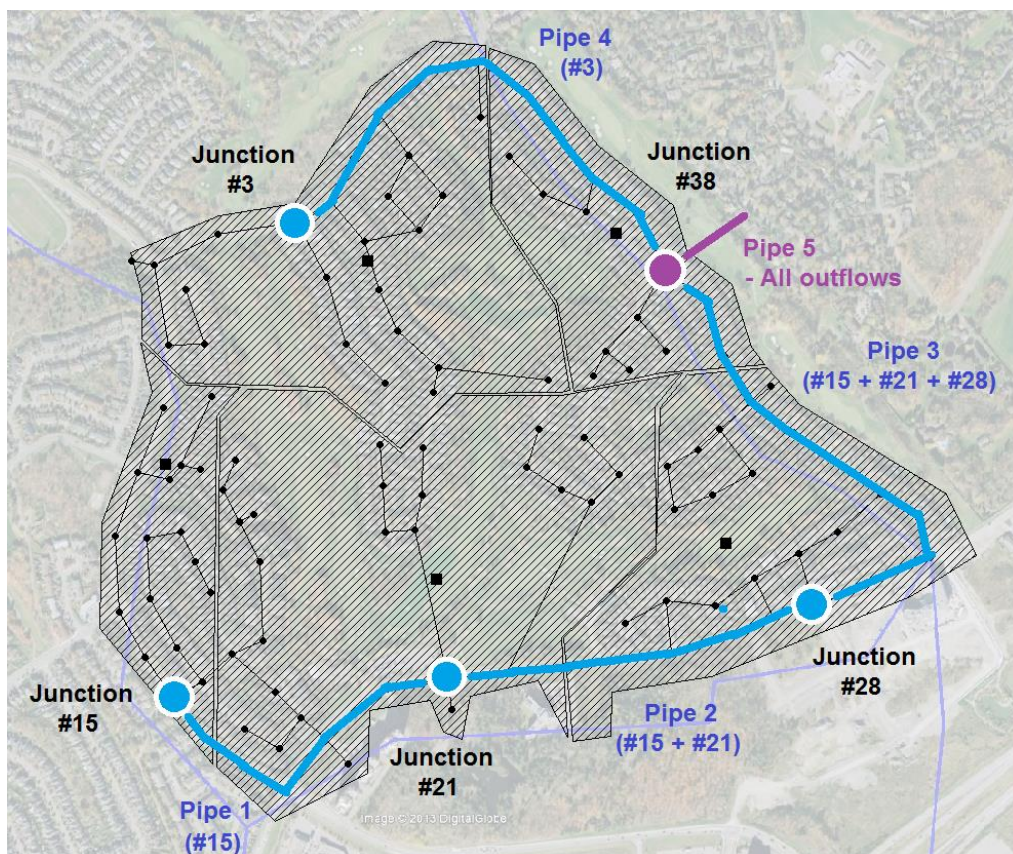


Figure 1: Preliminary stormwater system layout

2. Watershed properties

The watersheds generated based on the topography of Beaverbrooks, in Kanata, with the aid of the program GreenKenue is as follows:

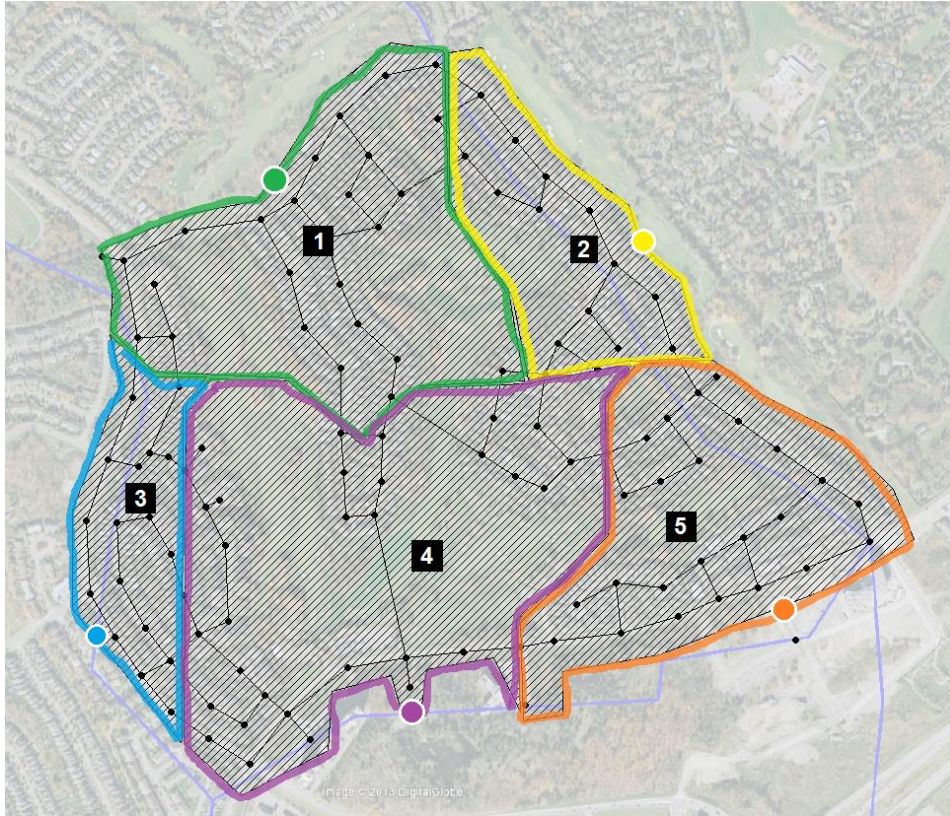


Figure 2: Sub-watersheds the Beaverbrooks, Kanata in SWMM.

After defining and numbering the watersheds in the area of interest, a few parameters are needed to be calculated in order to successfully design the synthetic storms and the performance of the stormwater network. The **slope** is an important factor as it reflects the watersheds rate of change of elevation with distance in its principal flow path. It can be calculated with the following equation:

$$S = \frac{\Delta E}{L}$$

Where ΔE is the difference in elevation in the watershed and L is the length of its principal flow path.

The area of the watershed is also necessary to calculate the volume of rainfall. In hydrologic design, the rainfall is considered to occur uniformly over the watershed. As the watersheds were exported into the EPANET SWMM program it was possible to use it to calculate the areas.

The final parameter necessary is the SCS runoff coefficient, which was developed to represent the combination of hydrologic soil group and a land use treatment class. This coefficient CN is a function of three variables – soil group, the cover complex and antecedent moisture conditions. The following table shows the complete classification.

GETTING CN FROM HSG
AND LAND USE

Table 2-2a Runoff curve numbers for urban areas ^{1/}

Cover description	Average percent impervious area ^{2/}	Curve numbers for hydrologic soil group			
		A	B	C	D
<i>Fully developed urban areas (vegetation established)</i>					
Open space (lawns, parks, golf courses, cemeteries, etc.) ^{3/} :					
Poor condition (grass cover < 50%)	68	79	86	89	
Fair condition (grass cover 50% to 75%)	49	69	79	84	
Good condition (grass cover > 75%)	39	61	74	80	
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)					
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)	98	98	98	98	
Paved; open ditches (including right-of-way)	98	98	98	98	
Gravel (including right-of-way)	83	89	92	93	
Dirt (including right-of-way)	76	85	89	91	
Dirt (including right-of-way)	72	82	87	89	
Western desert urban areas:					
Natural desert landscaping (pervious areas only) ^{4/}					
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)	69	77	85	88	
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
<i>Developing urban areas</i>					
Newly graded areas (pervious areas only, no vegetation) ^{5/}					
		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

^{1/} Average runoff condition, and $I_p = 0.25$.

^{2/} The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

Figure 2: Runoff CN for urban watersheds.

It is important to highlight that a big part of the area in study is occupied by a Golf Course, and with that in mind, properties of the sub-watersheds are not constant considering the land use. The properties are summarized in the following table:

Table 2: Watershed properties.

Watershed	Slope (m/m)	Area (ha)	CN	% Impervious
1	1,216930071	8,67	69	40
2	1,216930071	4,2	85	65
3	1,216930071	3,09	85	65
4	1,216930071	12,44	69	40
5	1,216930071	6,88	85	65

3. Simulations

This stormwater collection system is part of the minor storm system for this area, and consists of rainfall drainage sewers and sub catchments. It is meant to collect runoff that results from storm events. Using hydrologic estimation data, it is possible to calculate synthetic storms in many regions. Below follows an example of an average hour intensity for 1 hour design storm of return period 10 years.

Average 1 hour intensity: $0.35 * h$

$$i(t) = \begin{cases} h \frac{t}{a}, & t \leq a \\ h \exp\left(-\frac{t-a}{K}\right) & (mm/h) \end{cases}$$

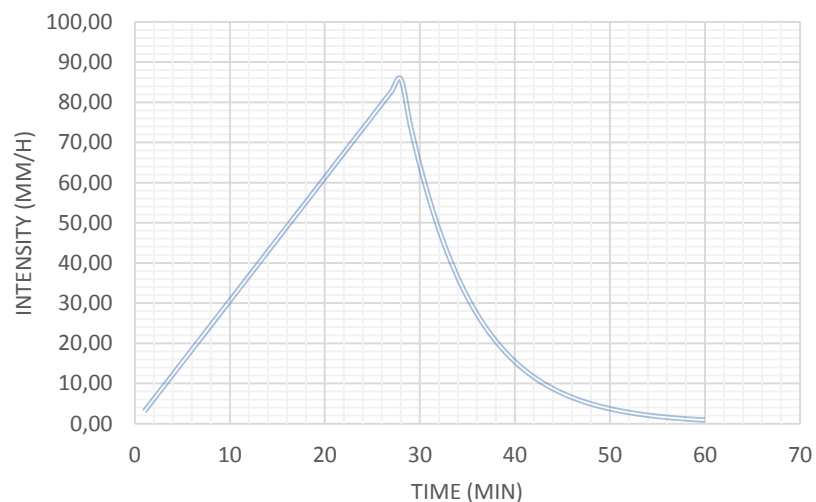
Being $K = 7$ for Ontario and h adjustable to reflect intensities from IDF curves.

The following table shows us that, for Ottawa, $a = 26$.

Table 2: Parameter “a” for Ontario.

TABLE D-7 ONTARIO VALUES FOR PARAMETER "a" (minutes)	
Ear Falls	26
Central Patricia	23
Thunder Bay	25
Rayner	27
White River	27
Timmins	24
Sudbury	21
Windsor	27
St. Thomas	24
Toronto	21
Kingston	27
Ottawa	26

Therefore, $h = \frac{30}{0,35} = 85,71$. The calculated one hour urban design storm is as follows:



Using the EPANET SWMM software, it was then possible to import this design storm and simulate the network to drain the rainfall in four different situations:

- 1) Synthetic 1 hour design storm of return period 10 years.
- 2) Synthetic 1 hour design storm of return period 100 years.
- 3) Historic storm: Hurricane Timmins
- 4) Historic storm: Hurricane Hazel

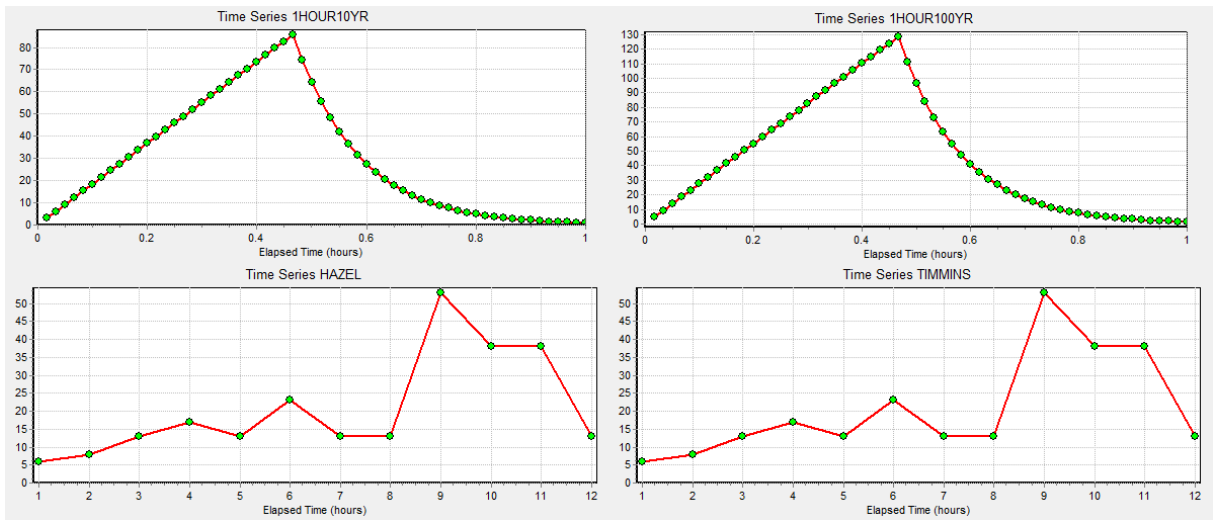


Figure 3: Storms simulated in the network.

4. Results and Conclusions

The final design of the system is displayed as follow, with the numbering of nodes and links of EPANET SWMM:

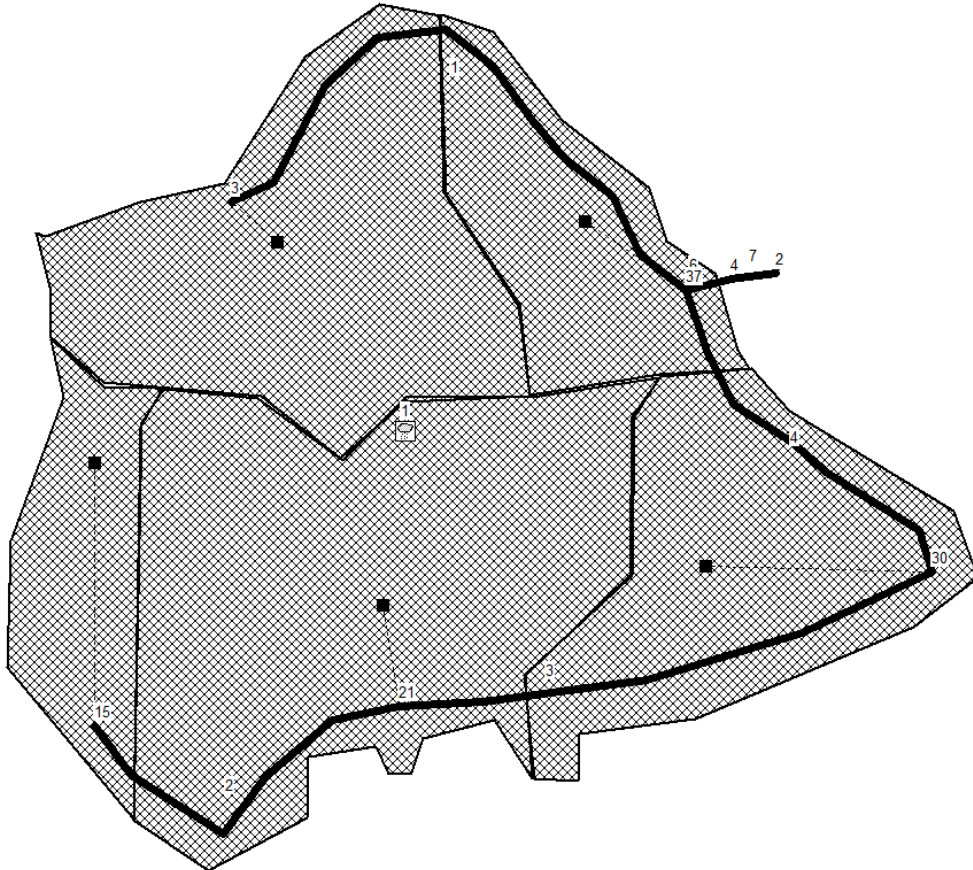


Figure 4: Final design and hydraulic components.

The suitable diameter for the stormwater network was estimated to be **1 meter**. Commercial diameters below 1 meter caused flood in the area and were not able to drain the rainfall as efficiently in the tested simulations. It is important to highlight that the diameter was estimated based on the efficiency draining the 1 hour design storm of 10 years return period.

- **Simulation:** Synthetic 1 hour design storm of return period 10 years.

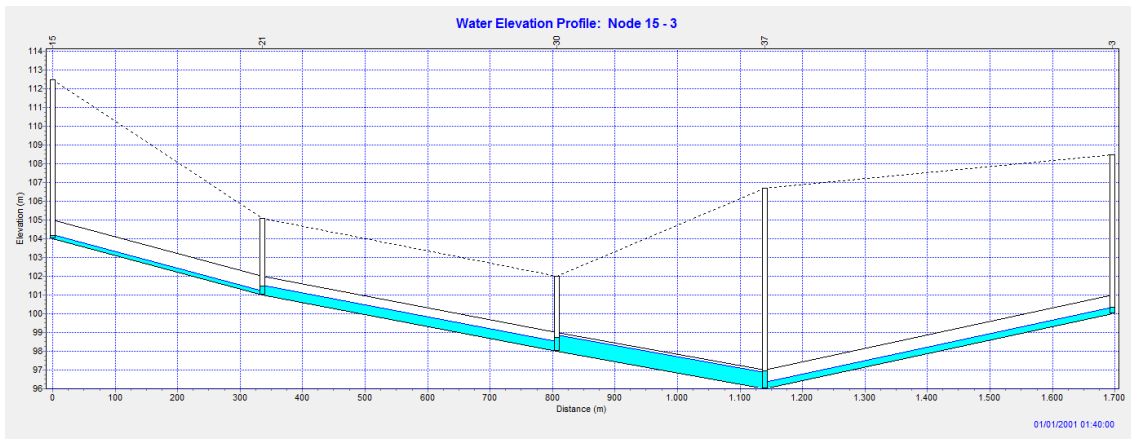


Figure 5: Network profile at the 10 years return period storm's peak.

The network successfully drains the storm without causing flooding in the area. The lowest point in the profile is the point connected to the main outflow of the network, directed to the river. The following graphic shows the flow velocities registered in the system's pipes during the storm:

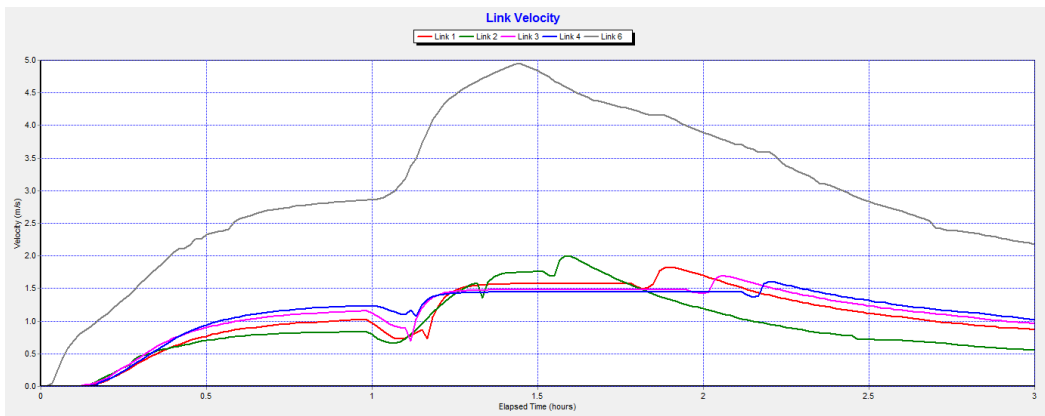


Figure 6: Velocity plot for the storm sewer pipes.

- **Simulation:** Synthetic 1 hour design storm of return period 100 years.

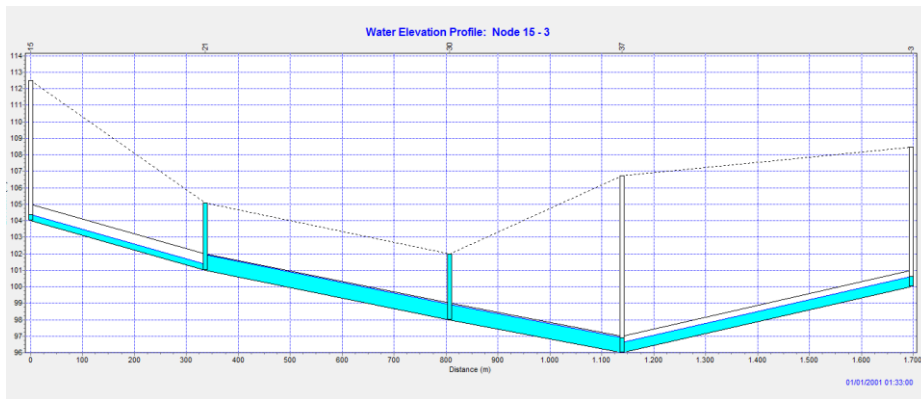


Figure 6: Network profile at the 100 years return period storm's peak.

In the peak of the 100 years return period storm, the network causes flood in two points in the south.

- **Simulation:** Hurricane Hazel and Timmins Storm.

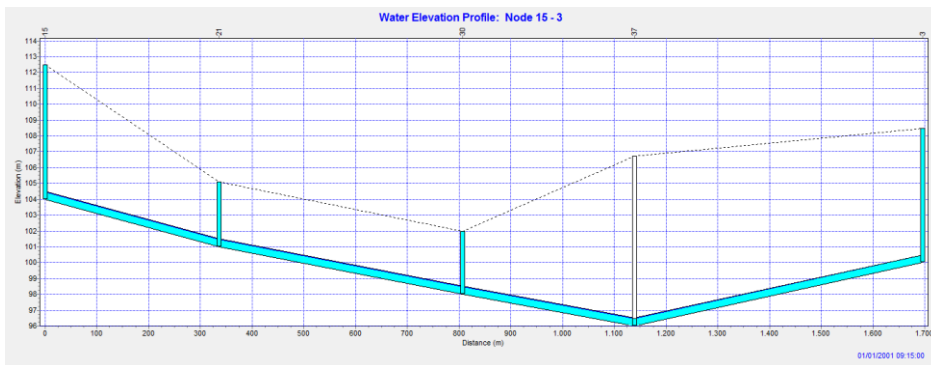


Figure 7: Network profile at the Hurricane Hazel's storm peak.

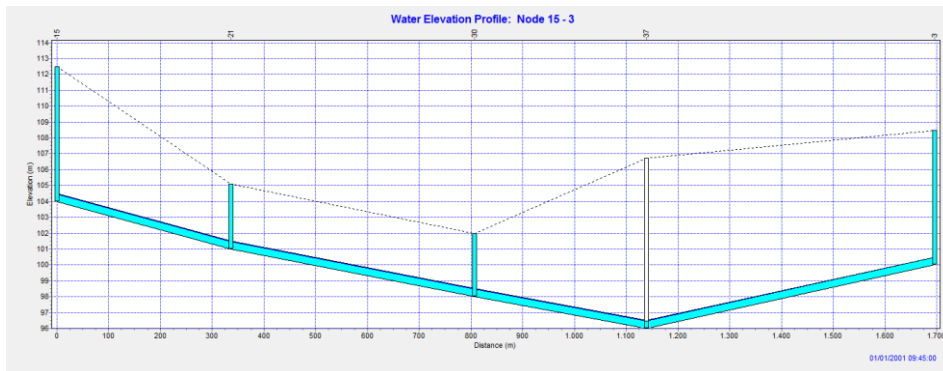


Figure 8: Network profile at the Timmins' storm peak.

In both historical storms scenarios, the network is not capable of draining the rainfall without causing major flooding in most points of the area. It is important to highlight that the network is part of the so-called “minor stormwater system”, and it must be design to alone effectively drain the 10 years return period storm. Further investments to increase efficiencie of the minor stormwater system might be too costly and not worth it considering the major stormwater system must also be designed. It is in these case of bigger storms that it is expected that the major stormwater system would be also redirecting part of the rainfall through it’s culverts and open channels to assist the drainage.

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