Adamsville Village

Storm and Combined Services

CIV340 Group 14

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Executive Summary

Adamsville Village is a proposed development that will house up to 185 residential units. This report summarizes the design procedure, layout, and analysis of municipal services for this development, all of which are in accordance with the Town of Adamsville Design Criteria and Standards Model.

The analysis for the water distribution system includes simulations on EPANET for Minimum Hour, Maximum Hour, Maximum Day, and Maximum Day plus Fire Flows. The results determined that the proposed system services all the parks and homes in the village with water at the minimum pressures and velocity requirements for the region. Pipes servicing locations of higher elevations and those connected to the boundary hydrants were designed to be 400mm in diameter. Other pipes in the system are 250mm and 300mm in diameter to minimize costs. The total municipal system will costs about $922,900 of capital investment to implement.

Furthermore, the calculated analysis for the sanitary system was conducted to determine the sewage demand flow through each pipe and corresponding maximum velocity. The chosen PVC pipe diameters were kept to a minimum of 250mm throughout the entire system, and through analysis it was confirmed that each pipe either met the minimum velocity and/or the minimum slope. The pipe diameters and slopes aim to minimize the excavation and material costs of the system.

The analysis for the storm drainage system was done to ensure it can carry runoff from all roadways, residential, commercial, and industrial properties with a dual drainage system consisting of a major and minor system. To minimize costs, when possible, pipes serving residential areas were designed with a minimum value of 300mm, but larger sizes were used when demand required. All maintenance holes followed the minimum invert drops for inlet and outlet pipes, to minimize the excavation and construction costs.

The design of both the storm and sanitary systems were developed as to not interfere with the proposed water distribution system, maintaining at least the vertical minimum clearance between the water distribution pipes and the sewers.

Based on the results on the analysis of the water distribution, sanitary sewer and storm drainage systems, the proposed developments will be able to meet and/or exceed all requirements set by the Town of Adamsville.

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1.0 Introduction

Adamsville Village is a proposed development of about 14.4 hectares, that will eventually house up to 185 residential units. The purpose of this report is to evaluate the feasibility of the proposed watermain distribution system, sanitary sewer system, and storm sewer system. The development will be serviced by two existing watermains, one located 30m west of the intersection of Adams Avenue and Karney Road, and the other is located at the intersection of Fabian Papa Way and Meyer Road. The storm and sanitary sewer systems will each be serviced by a single existing manhole located on the intersection of Adams Avenue and Karney Street. Figure 1.1 shows the plan view of the proposed development of Adamsville Village.

Figure 1.1: Plan view of the Adamsville Village

2.0 Water Distribution System

2.1 Design Objectives

The objective is to ensure that all parks and residential homes in the village are provided with water at the desired pressures, while attempting to minimize costs associated with hydrants and watermain pipes. Watermains are designed to ensure that this can be done. The total water demand of the village was determined to obtain the total flow required at specific points in the system. The parameters and assumptions made to develop the design of the system are explained in the following sections and refer to the Adamsville Design Criteria and Standards Model [1].

2.1.1 Water Demand Parameters

All values used for calculated water demand parameters were obtained from the Town of Adamsville Design Criteria and Standards Model. The following assumptions were made:

- Total of 185 residential units counted based on the plan view given.
- All houses were classified as detached with a density of 4 people per unit (ppu).
- Single-family dwellings minimum fire flow demands were used for the residential units.
- Daily demands for parks were calculated by approximating the water usage of a park to that of a school, which has a daily demand of 0.5 L/s/ha.
- Fire demand for the parks was taken to be 100 L/s - the same fire flow given to residential units. This decision was made considering the low density of structures in parks and their low flammability.
- Fire flows were calculated assuming only one residential unit or park per zone were on fire at a designated moment in time.

Table 2.1.1 and 2.1.2 summarize the key flows and factors used to determine demand of the subdivision.

Table 2.1.1: Flows and Average Daily Demands

Population Density (ppu)	4
Residential Daily Demand (L/c/d)	450
Residential Fire Flow (L/s)	100
Park Daily Demand (L/s/ha)	0.5
Park Fire Flow (L/s)	100

Table 2.1.2: Peak Factors

Minimum Hourly Demand	0.4
Maximum Daily Demand	2.75
Maximum Hourly Demand	4.13

Table 2.1.3 lists the minimum and maximum pressure requirements for the water in the Town of Adamsville Design Criteria and Standards Model.

Table 2.1.3: Permissible Pressures

Demand Type	Minimum Pressure (kPa)	Maximum Pressure (kPa)
Maximum Day	350	-
Minimum Hour	-	700
Maximum Hour	275	-
Maximum Day + Fire	140	-

2.1.2 Pipe Diameter Roughness and Velocities

When considering pipe diameters, the corresponding Hazen-William Roughness Coefficients found above were taken into consideration for each different size of pipe as shown in Table 2.4. All velocities are kept below 3 m/s, as recommended in the Adamsville Design Criteria and Standards Model.

Table 2.1.4: Hazen-Williams Roughness Coefficients

Pipe Diameter	150	200 - 250	300 - 600	> 600
Hazen - Williams Roughness Coefficient	100	110	120	130

2.2 Water Demand

The town was divided into zones to calculate the demand flow at specific points in the system. The red lines in Figure 2.2. demonstrate the zones Adamsville was divided into, with each zone labelled with a letter from A to R. The demand for all the houses in each zone was found and can be seen in Table 2.2.

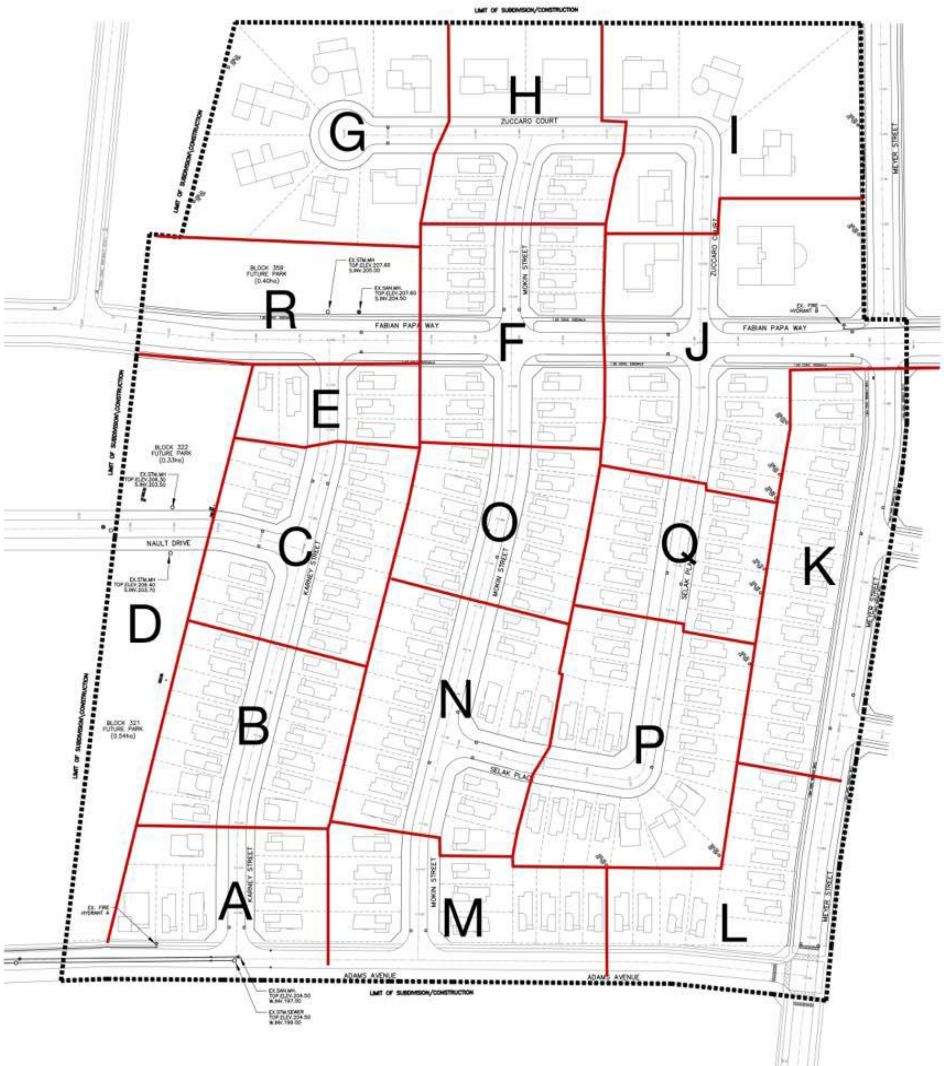


Figure 2.2: Subdivision of Adamsville into Zones

Table 2.2: Total demand flow of nodes in each zone

Zone	# Houses	Park Area (ha)	Population	Total Q_{avg} (L/s)	Fire Flow (L/s)	$Q_{max,\ hour}$ (L/s)	$Q_{max,\ day} + Q_{fire}$ (L/s)
A	8	0	32	40	100	165.2	210
B	15	0	60	75	100	309.75	306.25
C	15	0	60	75	100	309.75	306.25
D	0	0.87	0	0	100	0	100
E	3	0	12	15	100	61.95	141.25
F	14	0	56	70	100	289.1	292.5
G	6	0	24	30	100	123.9	182.5
H	8	0	32	40	100	165.2	210
I	4	0	16	20	100	82.6	155
J	11	0	44	55	100	227.15	251.25
K	16	0	64	80	100	330.4	320
L	10	0	40	50	100	206.5	237.5
M	11	0	44	55	100	227.15	251.25
N	18	0	72	90	100	371.7	347.5
O	13	0	52	65	100	268.45	278.75
P	19	0	76	95	100	392.35	361.25
Q	12	0	48	60	100	247.8	265
R	2	0.4	8	10	100	41.3	127.5

The design demand governing each zone is the max day flow and fire flow. Detailed calculations for how these flows were determined can be found in Appendix B.3.

2.3 Hydrant Flow Tests and Boundary Conditions

The two hydrant flow results were found based on measurements taken 30 m west of Adams Avenue and Karney Road and Fabian Papa Way and Meyer Road. Table 2.3 summarizes the results and unit conversions required to plot the hydrant flow tests used to interpolate the minimum flow required. Figures 2.3.1 and 2.3.2 summarize the plots.

Table 2.3: Hydrant Flow Test Results

Adams Avenue and Karney Road				Fabian Papa Way and Meyer Road			
Flows (usgpm)	Flows (L/s)	Pressure (kPa)	Pressure (m)	Flows (usgpm)	Flows (L/s)	Pressure (kPa)	Pressure (m)
0	0.00	57	40.06	0	0.00	65	45.69
630	39.74	43	30.22	635	40.06	43	30.22
1030	64.98	16	11.25	950	59.93	16	11.25

Sample Flow Conversion: $630usgpm \times 3.785 \frac{L}{gallon} \times \frac{1minute}{60s} = 39.75\ L/s$

Sample Pressure Conversion: $43\ psi\ \times 6.895 \frac{kPa}{psi} \times \frac{1}{9.81} = 30.22m$

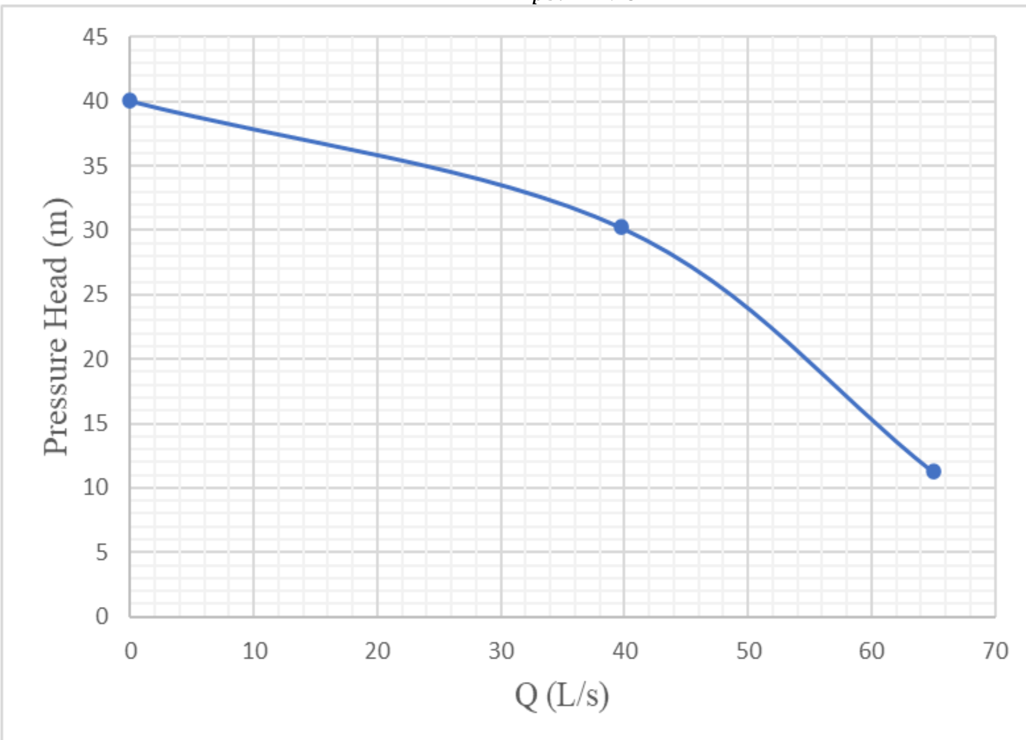


Figure 2.3.1: Hydrant Flow Test at West of Adams Avenue and Karney Road

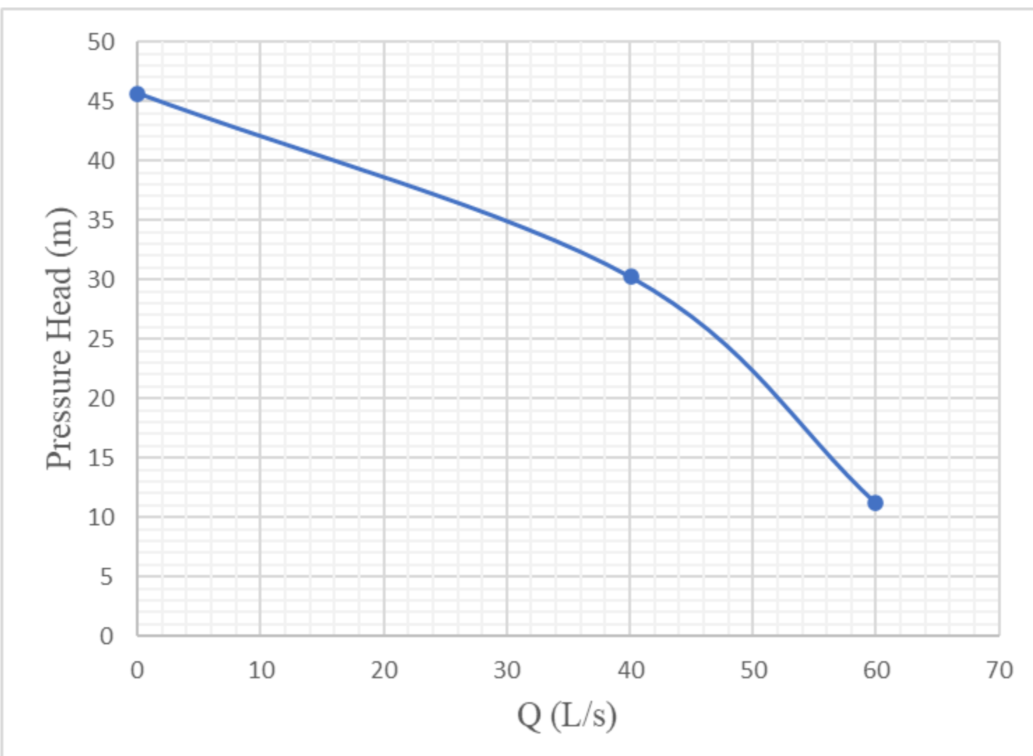


Figure 2.3.2: Hydrant Flow Test at Fabian Papa Way and Meyer Road

2.4 System Layout and Design

Figure 2.4.1 displays an approximate system layout using EPANET, where all nodes and links are labelled. There is one node defined for every zone in Figure 2.2 with the corresponding letter. These nodes in the system also represent the hydrant locations and will therefore be referred to as such moving forward. Each pipe has also been labelled with a number ranging from 1 to 26, which can be seen in Figure 2.4.1. An exact depiction of the system layout following all regulations can be seen in Appendix A.1.

Figure 2.4.1: System layout in EPANET

2.4.1 Hydrant Locations

As shown in Figure 2.4.1, at least one hydrant exists in every zone, corresponding to the locations on EPANET where base demands for the zones were inputted. To comply with the maximum spacing requirements set by the Town of Adamsville, three extra hydrants were placed in zones K, L and M - labeled EX1, EX2, and EX3, respectively. The addition of these ensures that all existing hydrants are located no more than 120 meters from each other. Furthermore, all hydrants have been placed at least one meter away from the closest lot line. Ground elevation data for each of these hydrants can be seen in Appendix B.2.

2.4.2 Watermain Location and Selection

Watermains were placed on the East and/or North side of all the roads, under sidewalks when present, or road verges, within the 3 meters easement of the property. Watermains were positioned such that they ran through all streets and delivered flow to all houses in the subdivision, making use of a looped network to maximize flow from both entrance hydrants. Pipes between the entrance hydrants meet at multiple points in the network. All mains are designed to be placed at a minimum clear cover of 1.7 meters below ground surface. The length, nominal, and actual diameter for each of the pipes can be seen in Appendix B.2.

2.4.3 Pipe Diameters

Pipes were chosen based on maintaining the minimum pressure head at delivery points as well as remaining below the maximum threshold of water velocity of 3 m/s in the pipes. The diameters for each pipe are shown in Appendix B.2. Most main pipes follow the recommended pipe diameter for watermains of 300mm and 400mm. Due to economic conservatism, some pipes were chosen to be 250mm where the delivery pressure head during fire flow simulations allowed, thus, decreasing the cost of the overall system while still ensuring guidelines, minimum pressure and maximum velocity requirements were met.

Initially a base simulation was run with only 400mm pipes. When running fire flow tests, all hydrants were tested with their respective maximum day demand, except for one with a combination of maximum day demand and fire flow. It was noticed that when the fire demand was placed at a few critical junctions (H and G), minimum pressure was not being achieved. These hydrants are located at higher elevations compared to the others, as seen in Table 7, which appears to be the cause of the lower pressure heads. Due to this, pipes around these nodes were upgraded to the next recommended size of 400mm. This ensures that sufficient flow and pressure is supplied to the surrounding homes at higher elevations in case of a fire.

2.5 Hydraulic Model Results

To ensure maximum and minimum permissible pressure requirements were met, several different demand conditions were tested on the EPANET model. A summary of these tests can be seen in Appendix B.4, displaying the range of pressures experienced by all hydrants in the system under the specified demand conditions. All hydrants meet the minimum and maximum permissible pressures outlined in Table 2.3.1. The maximum velocity experienced in any pipe under any of the three demand conditions was 0.12 m/s, complying with our maximum velocity requirement of 3m/s.

Table 2.5.1: Pressures for Minimum and Maximum Hourly Flows, and Maximum Daily Flow

Demand Condition	Pressure Range (kPa)
Minimum Hourly Flow	429.19 – 467.25
Maximum Hourly Flow	396.62 – 424.58
Maximum Daily Flow	414.86 – 452.93

Detailed results can be seen in Appendix B.4. These include pressures at each hydrant, as well as the velocities, head losses and flows along each of the pipes. Local losses were neglected due to all velocities being maintained below 3 m/s, complying with the Town of Adamsville requirements.

Furthermore, critical locations were tested for fire flow and maximum day demand. Hydrants H and G were first considered critical locations due to their high elevations, as well as the low pressures they experienced when testing maximum daily and hourly flow. When running fire flow test, both hydrants experienced a pressure of 140.18kPa, meeting the minimum requirement of 140kPa.

In addition to this, Hydrants D and R were also classified as critical due to their location on the dead end of pipes. When fire flow tests were conducted at these two locations, each hydrant experienced a respective pressure of 149.99kPa and 144.99kPa. The lowest pressures in the system in both conditions were experienced by Hydrant H – at 141.46kPa and 141.07kPa. As such, all hydrants achieved the minimum permissible fire flow pressure. At neither of these locations does the velocity in the pipes exceed 0.78m/s. The responses of all hydrants and pipes due to fire flow at these hydrants are presented in Appendix B.5.

2.6 Response Curves at Critical Hydrant Locations

As discussed previously, the critical hydrant locations are ones at the highest elevation, H and G, and the dead ends, D and R. To calculate response curves, it was assumed that residual pressure referred to the pressure head at each node. Fire flows ranging from 0 to 100 L/s were defined at each hydrant and the respective pressure heads were measured. When running the simulations, the demand at all other hydrants was set to their maximum daily flow.

Table 10: Hydrant H

Flow (L/s)	Residual Head (m)
0	42.29
25	35.84
50	31.84
75	24.54
100	14.29

Table 11: Hydrant G

Flow (L/s)	Residual Head (m)
100	14.34
75	24.65
50	31.98
25	36.01
0	42.47

Table 12: Hydrant D

Flow (L/s)	Residual Head (m)
0	44.04
25	37.53
50	33.38
75	25.85
100	15.29

Table 13: Hydrant R

Flow (L/s)	Residual Head (m)
0	43.08
25	36.6
50	32.55
75	25.16
100	14.78

2.7 Cost Estimation of System

The estimated total capital cost to implement this system to service Adamsville is $1,001,127. The unit prices per meter of each pipe size was used to obtain the total prices based on the lengths of the pipes. Detailed calculations can be found in Appendix B.6. The initial design of the system included only 300mm and 400mm nominal diameter pipes. To make the system more economical, some pipes were reduced to a diameter of 250 mm. After several pipes were downsized, it was determined that the system was functional and meeting all requirements with the updated pipe diameters. Further downsizing would have resulted in pressures lower than the permissible.

3.0 Sanitary System Design

3.1 Design Objectives

The primary objective of the proposed sanitary sewer system is to carry all the domestic, commercial, and industrial sewage produced in Adamsville Village. All flows in the sewer system will be directed by gravity towards the maintenance hole at the intersection of Adams Avenue and Karney Street.

Sections 2.2 and 2.3 outline the constraints and requirements defined by the Town of Adamsville Design Criteria and Standards Model. All these requirements were followed in the design of the proposed sewer system.

3.1.1 Pipe Design Parameters

When determining the diameter and slope of each pipe in the system, the following design parameters were used [1]:

- A minimum nominal pipe diameter of 250 mm.
- The minimum velocity under design flow conditions shall be 0.75%. If this requirement cannot be met, a minimum slope of 1% shall be used.
- The maximum permissible velocity both design flow conditions and full flow conditions shall be 3.0 m/s.
- Downstream pipe diameters shall always be greater or equal to the largest upstream pipe diameter.

3.1.2 System Layout Design Parameters

Sanitary sewers laid out in the system should generally be placed 3.0 m below ground elevation to service basements by gravity. The following depth of cover shall be used for frost protection; however, they can be set at shallower depths if insulated according to the Ontario Building Code (OBC A-7.3.5.4):

- For rigid pipes, a minimum cover of 1.2m to the spring line.
- For flexible pipes, a minimum cover of 1.5m to the crown.

Minimum clearances to watermains shall be in accordance with the following Ontario Ministry of Environment (MOE) guidelines [2]:

- Watermains and sewers shall have a minimum horizontal separation of 2.5m and shall not be laid in the same trench.
- Watermains crossing above a sewer need to have a 0.5m clearance between the invert of the watermain and the obvert of the sewer.
- If the watermain must cross below the sewer, there must be a 0.5m spacing from the invert of the sewer to the crown of the watermain. In addition, there must be a support in place so that the sewer doesn't load the water pipe. The watermain pipe shall be centered at the crossing to maximize distance to the first joint.

3.1.3 Maintenance Hole Design Parameters

Maintenance holes, also referred to as M.H in this report, should be located at the end of each line, at changes in pipe size, or material and abrupt changes in alignment. Regular spaced maintenance holes are still required regardless of these changes. These spacing requirements are summarized in Table 3.1.1.

Table 3.1.1: Maintenance Hole Spacing Requirements

Pipe Diameter (mm)	Maximum Spacing (m)
250 – 450	110
450 – 750	150
>750	Subject to town approval

Furthermore, all maintenance holes shall be placed a minimum of 1.5 m from the curb to prevent issues with road maintenance and/or reconstruction. They are also located in local low elevation areas.

At each maintenance hole, the obvert of the outlet pipe shall be lower than the obverts of any inlet pipes. Additionally, minimum drops are required to offset hydraulic losses. These requirements can be found in Table 3.1.2.

Table 3.1.2 Minimum Inlet to Outlet Drop Requirements

Type of Bend	Minimum Drop (m)
Straight – run	0.03
90^{o} bend	0.06
45^{o} bend	0.09

Drop structures must be added to maintenance holes where the inlet drop exceeds 0.6m. An external 'tee' drop structure (OSPD 1003.010) will be used in any case where this occurs.

3.2 System Layout

A layout of the sanitary sewer system can be seen in Appendix A.1 General Plan of Services. Pipes were laid following street elevations, where possible, and directed towards the external maintenance hole at the intersection of Adams Avenue and Karney Street. In addition, pipes were placed on primarily on the north and east side of the road while keeping a minimum horizontal distance of 2.5 m from the watermain. As outlined in Table 3.2.1, maintenance holes were placed at the end of each line and bend while maintaining a maximum spacing of 110 m.

3.2.1 Tributary Areas

Tributary areas were found for each pipe spanning adjacent maintenance holes, which can be seen in Appendix A.4 Sanitary Drainage Plan. These areas were used to determine the total demand of each pipe.

3.3 Design Flows

Design flows were calculated for each pipe based on the demand derived from its tributary area, as well as all the cumulative demand from the pipes upstream of the pipe of interest. Detailed calculations for Sections 3.3.1, 3.3.2 and 3.3.3 can be found in Appendix B.C.

3.3.1 Residential Demand

All values used for residential sanitary flow demand were obtained from the Town of Adamsville Design Criteria and Standards Model. The following assumptions were made:

- A total of 185 residential units counted based on the plan view given.
- All houses were classified as detached with a density of 4 people per unit (ppu).

Table 3.3.1. Flows and Average Daily Demands

Population Density (ppu)	4
Residential Daily Demand (L/c/d)	450

Residential flow was approximated for each area using the cumulative population that generates sewage passing through the maintenance hole. The cumulative populations of upstream areas were included in the calculation of total residential flow along pipes.

Peak factors for each pipe flow were determined using the Harmon equation. The equation, seen below, uses cumulative population expressed in thousands.

$$K = 1 + \frac{14}{4 + p^{0.5}}$$

As outlined in the Adamsville Design criteria, the minimum and maximum peak factors were taken to be 2 and 4, respectively.

3.3.2 Industrial / Commercial / Institutional (ICI)

Daily demands for the three parks were calculated by approximating the sewage generation of a park to that of a commercial establishment. This approximation was made due to the similarity in use between the two, relating inflows and outflows of visitors and hours of operation. This accounts for the possibility of high flow during holidays and events.

Table 3.3.2 Flows and Average Daily Demands

Population Density (persons/hectare)	75
Sewage Generation Rate (L/s/ha)	0.29

3.3.3 Infiltration Allowance

Infiltration allowance of 0.23 L/s/ha was applied to the total area of all properties that were connected upstream of the given pipe section.

3.3.4 Final Design Flows

A summary of the total demand for each pipe segment can be found in Table 3.5. This includes the cumulative maximum residential flow and maximum commercial flow with peak factors applied, as well as the cumulative upstream infiltration. The equation for maximum flow can be seen below, where K is the peaking factor, I is the infiltration allowance and A is cumulative area.

$$Q = K \times (Q_{avg,residential} + Q_{avg,commercial}) + IA$$

Detailed calculations for this segment can be found in Appendix C.1.1 The maximum flows were used as design flows for design of the sewer pipes.

As outlined in the Town of Adamsville Engineering Design Criteria and Standards Manual and as described by the client Angela Mokin, the minimum and maximum design flows are taken to be the same.

The maximum sewage production flows per tributary area from one maintenance hole to another can be found in Appendix C.1.1.

3.4 Pipe Design

The pipes in the system were designed based on the sewage flow demands and in accordance with the requirements set by the Town of Adamsville detailed in the design objectives section. Appendix C.3 outlines the diameters, slopes, and velocities of all pipes in the system, all made from PVC.

The system was designed by setting all the pipe diameters to 250mm and slopes to 1%. Where minimum velocities were met, such as pipes spanning M.H 6A to 7A and 7A to 8A, slopes were reduced to reduce excavation and try to achieve subcritical flow. In cases where a pipe's velocity did not meet the minimum requirement of 0.75m/s, the pipes remained at a minimum slope of 1%. These sizes and slopes allowed for economic conservatism across the system while still abiding by requirements and maintaining functionality.

Detailed calculations for velocity can be found in Appendix C.1.2: Sample Minimum Velocity Calculations for Maintenance Hole 1 to 2. As seen in Table 3.6, most pipes did not meet the minimum velocity requirements. This requirement was only met by the following pipes:

- M.H.6A to M.H.7A
- M.H.7A to M.H.8A
- M.H.8A to M.H.24A
- M.H.24A to Outflow

All other pipes must undergo more frequent maintenance and cleaning to ensure there is no sewage settlement due to the velocity being below the minimum requirement.

It is important to note that since minimum and maximum design flows are equal, so will be the minimum and maximum velocities.

3.4.1 Supercritical and Subcritical Flows

For each pipe, critical flow depths were calculated and the compared against the actual flow depth. This was achieved by comparing the ratios of critical flow depth over diameter, $\frac{y_{critical}}{D}$, and actual flow depth over diameter, $\frac{d}{D}$. Flow was denoted as supercritical when $\frac{d}{D}$ was lower than $\frac{y_{critical}}{D}$. This can be seen in Appendix C.3.

In summary, five pipes were found to have supercritical flow. Although this is not desirable, it is not possible to achieve subcritical flow given the minimum slope and diameter requirements.

3.5 Maintenance Hole Elevations

The following sections outline the characteristics of each maintenance hole in the system, including invert elevations, depth of cover, drop structures, and inlet drops**.**

3.5.1 Clear Cover

Table C.3.4 summarizes the elevation of the ground at each maintenance hole, the depth of each pipe invert and the cover to the crown. Detailed calculations on how these values were obtained can be found in Appendix C.1.4: Sample Invert Calculations for Maintenance Hole 1 to 2. In Appendix C.3.4: Maintenance Hole Elevations, Invert Elevations and Cover to Crown, drop structures have already been applied.

All pipes were placed to obtain a clear cover of at least 3 m to ensure that basements are adequately serviced. Pipes only servicing parks, such as the between M.H.20A and M.H.21A, were placed 1.5 m below the ground elevation to remain below frost depth.

3.5.2 Drop Structures

A preliminary design of the system found that maintenance holes 6, 8, 21 and 24 had inlets with drops larger than 0.6m. This occurred due the maintenance holes having multiple inlet sewers. As required by the Adamsville Village, drop structures were added to these maintenance holes. Table 3.5 summarizes the details of the drop structures added.

<u>Table 3.5 Drop Structure Details</u>

M.H. From	M.H. To	M.H. To Invert Elevation without Drop Structure (m)	Drop Structure (m)	M.H. To Invert Elevation with Drop Structure (m)
5A	6A	201.08	1	200.08
7A	8A	199.13	1	198.13
20A	21A	203.22	0.7	202.52
23A	24A	200.36	3.2	197.16

An additional maintenance hole, M.H.17A, also had multiple inlets. However, a drop structure was not added outside of the maintenance hole due to interference with a watermain. A drop shaft was added along the pipe instead, which is detailed in Section 7.

3.5.3 Maintenance Hole Drops

After applying drop structures, all the invert drops for sewers entering maintenance holes meet the requirements outlined in the objects. Appendix C.3.5: Inlet Drops shows the required minimum drop for each of these, as well as the actual inlet drop. As shown, all inlet drops meet the minimum drops and do not exceed 0.6 m**.**

3.6 Clearance to Watermains

Table 3.6 states the criterion followed to ensure that the sanitary systems are a safe distance away from the watermains.

Table 3.6. Criteria for watermain and sanitary sewer clearance distance

Criteria	Clearance Distance (m)	Notes
Minimum Horizontal Separation	2.5	Distance between the closest two edges
Minimum Vertical Clearance	0.5	Distance between invert of water pipes and obvert of sewers

All sewers were placed at a minimum horizontal distance of 2.5 m from the watermains. Most of the sewers were placed maintaining a minimum cover of 3 m to the ground. Since watermains were all placed 1.7 m below the ground elevation, the vertical clearance is automatically met for most of the system.

Based on a preliminary design, it was determined that the minimum required watermain clearance was not met by the sanitary sewer spanning from M.H.15A to M.H.17A, shown in Figure 2. To comply with this requirement, an external tee drop shaft of 0.5m was placed one meter away from M.H.15A's outlet. By doing so, the minimum clearance was achieved.

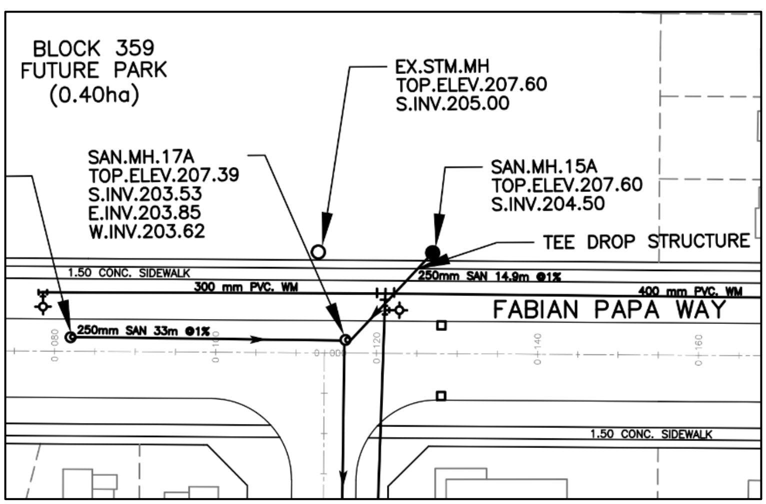


Figure 3.6. Sewer Pipe MH.15A to MH.17A Crossing Watermain

3.7 Material Specifications

All Canadian Standards Association (CAN/CSA) specifications, American Society for Testing and Materials (ASTM) specifications and Ontario Provincial Standard Specifications (OPSS) referenced hereto are to be the latest revision. [1]

3.7.1 Sewer Pipe Material

Polyvinyl chloride (PVC) pipes manufactured to CSA Standard B-182.2 (ASTM Specification D-3034) are used to service the entire system. The pipes and fittings must have a Standard Dimension Ratio of 35 and minimum stiffness of 320 kPa. Every pipe in the system has a diameter of 250 mm.

PVC pipes are acceptable for pipe diameters of 250 mm and for use in areas including residential, commercial, and industrial, which make it ideal for this design.

3.7.2 Bedding and Backfill

Bedding materials shall be in accordance with OPSD 802.010 with Granular ‘A’ bedding and OPSD 802.030 Class ‘B’ with Granular ‘A’ bedding for concrete pipes unless otherwise recommended by a qualified geotechnical engineer.

3.8 Cost Evaluation

The proposed sanitary system design adequately services the development of Adamsville while staying within the design constraints. All pipe diameters servicing the town were kept to the minimum diameter of 250 mm – which prevents unnecessary additional costs associated with larger diameter pipes. Furthermore, pipes are only placed where needed to service housing units or parks, reducing the overall length of the sewer system.

Furthermore, an attempt was made at reducing excavation costs. A clear cover depth of around 3 m was maintained throughout the majority residential system to ensure that houses are serviced by gravity flow alone. Additionally, a clear cover depth of 1.5 m was used for pipes that only serviced the parks in the system, such as, the pipe spanning from M.H.20A – M.H.21A, which reduces excavation costs.

Six pipes had to be placed with a clear cover exceeding 5 m, resulting in increased excavation cost. However, these were necessary to maintain a 1% minimum slope where velocities were not met. The existing maintenance hole along Adams Avenue had an invert of 7m below ground level, proving that this excavation is feasible.

Smaller slopes than 1% were used in pipes where the minimum velocity criteria were met, such as, pipes M.H.8A – M.H.24A, M.H.6A – M.H.7A, and M.H.7A – M.H.8A. This also helped reduce excavation costs.

4.0 Storm Drainage System Design

4.1 Design Objectives

The primary objective of the storm drainage system is to remove surface runoff from all roadways, residential, commercial, and industrial properties. The surface runoff is to be conveyed by a dual drainage system consisting of a major and minor system. The minor system is to be comprised of an underground network of pipes designed to ensure the safety of traffic and pedestrians by removing surface runoff from storms with a 5-year return period. The major system will consist of a surface network of roadways and overland flow channels to prevent flooding during 100-year storms. The system is designed to direct the discharge to a natural receiving system approved by the Town and the Conservation Authority.

The following sections outline the constraints and requirements defined by the Town of Adamsville Design Criteria and Standards Model [1]. All requirements were followed in the design of the proposed storm drainage system.

4.1.1 Meteorology

The rational method is used to determine the rainfall intensities experienced in the Town of Adamsville using the following equation:

$$l = \frac{A}{(T_c + B)^C}$$

The parameters for the equation were obtained using an intensity-duration-frequency (IDF) curve and are as shown in Table 4.1.1. The minor system was designed for a storm with a return period of 5 years, and the major system is designed for a storm with a return period of 100 years.

Table 4.1.1. IDF Curve Constants for Major and Minor System Design

Return Period	**IDF Equation Constants**		
	A	B	C
5-yr	1105.505	8.570	0.829
100-yr	2022.828	10.034	0.837

4.1.2 Peak Flow Equation

Minor storm sewer systems flows were determined using the following equation:

$$Q_{design} = 2.778 * C * l * A$$

The rainfall runoff coefficients (C) corresponding to the type of land use are summarized in Table 4.1.2. Area weighted averages of the C values were used when more than one type of land was present in a given watershed.

Table 4.1.2. Runoff Coefficients Used for Designing the Minor System.

Land Use	Runoff Coefficient, C
Parks, cemeteries, or Open Space (> 4 hectares)	0.2
Parks, cemeteries, or Open Space (< 4 hectares)	0.25
Single Family Residential (> 18 m frontage)	0.4
Single Family Residential (< 18 m frontage)	0.45
Arterial Roadways	0.65

4.1.3 Minimum Slope, Velocity and Diameter Requirements

The minimum slope of the pipes for the minor system were determined based on the pipe diameter. Details are shown in Table 4.1.3. The velocity in all pipes should not exceed 4.5 m/s.

Table 4.1.3. Minimum Slope Requirements Based on Pipe Diameter.

Storm sewer Size	Minimum Slope
Up to 375 mm	0.40%
450 mm to 525 mm	0.30%
600 mm to 1200 mm	0.20%
1350 mm and greater	0.15%

The minimum pipe diameters for storm sewers are shown in Table 4.1.4. Additionally, the diameter of any upstream pipe shall not exceed the diameter of any pipe downstream.

Table 4.1.4. Minimum Pipe Diameters.

Land Use Type	Minimum Diameter (mm)
Residential	300
Commercial and Industrial	375

The following depth of cover in table 4.1.5 shall be used for frost protection; however, they can be set at shallower depths if insulated according to the Ontario Building Code (OBC A-7.3.5.4).

Table 4.1.5. Cover Requirements

Rigid Pipe	1.2 m to the spring line
Flexible Pipe	1.5 m to the crown

4.1.4 Maintenance Hole Requirements

Maintenance holes shall be placed at each change in alignment, slope, or pipe material, as well as all pipe junctions and at minimum spacing intervals along the pipe to permit entry for maintenance. The spacing requirements for maintenance holes are summarized in Table 4.1.6.

Table 4.1.6. Maximum Maintenance Hole Spacing based on Diameter.

Storm Sewer Size	Maximum Maintenance Hole Spacing
300 mm to 750 mm	110 m
825 mm to 1050 mm	125 m
1200 mm and greater	150 m

Minimum drops between the inlet and outlet pipes are required to offset hydraulic losses. These requirements can be found in Table 4.1.7.

Table 4.1.7. Minimum Inlet to Outlet Drop Requirements.

Alignment	Minimum Drop (m)
Straight Run	0.03
Up to 45º Deflection	0.05
45º to 90º Deflection	0.075

The following criteria was also maintained to ensure safe use and operation of maintenance holes:

- At each maintenance hole, the obvert of the outlet pipe shall be in line or lower than the obverts of any inlet pipes.
- The maximum change in direction of flow in maintenance holes is 90° for sewers 900 mm and smaller and 45° for pipes over 900 mm.
- Drop structures must be added to maintenance holes where the inlet drop exceeds 1.0 m. An external ‘tee’ drop structure (OSPD 1003.010) will be used in any case where this occurs.
- All maintenance hole covers shall be located on the side of the maintenance hole parallel to the flow for straight runs, or on the upstream side of the maintenance hole at all junctions.
- When the depth of a maintenance hole exceeds 5.0 m, safety gratings shall be provided.
- All maintenance holes shall be placed a minimum of 1.5 m from the curb to prevent issues with road maintenance and/or reconstruction.

4.1.5 Catchbasin Requirements

The following criteria shall be met when locating catchbasins:

- Ponding depth at a catchbasin shall not exceed 0.3 m.

- Double catchbasins shall be used at sag points.
- Rear lot catchbasins connect to maintenance holes via catchbasin leads.

Table 4.8 summarizes the maximum spacing requirements for catchbasins.

Table 4.1.8. Roadway Catchbasin Location and Spacing.

Pavement Width	Maximum Spacing	
	Less than 4% grade	Equal to or greater than 4% grade
Less than or equal to 10 m	90 m	75 m
More than 10 m	75 m	60 m

Catchbasins located close to a maintenance hole shall have leads that connected directly to the maintenance hole. Catchbasin leads longer than 20 m must be connected to a maintenance hole. The maximum length of a catchbasin lead is 30 m. Table 4.1.9 shows the slope and diameter requirements for catchbasin leads.

Table 4.1.9. Minimum Diameter and Slope Requirements for Catchbasin Leads.

Catchbasin Type	Diameter and Slope
Single catchbasins	250 mm @ 1.0%
Double catchbasins	300 mm @ 1.0%

4.2 Minor Storm System Layout

A layout of the minor storm sewer system can be seen in Appendix A.1. Pipes were laid following street elevations, where possible, and directed towards the external maintenance hole at the intersection of Adams Avenue and Karney Street. In addition, pipes were placed on the South and West sides of the road while keeping a minimum horizontal distance of 2.5m between watermains and sewers.

4.2.1 Catchbasin Locations and Leads

Catchbasins were placed to service all roads. Double catchbasins were placed upstream of crosswalks at intersections to ensure no ponding occurred in the intersection and where pedestrians would be walking. Catchbasins were also placed in locations to satisfy the maximum spacing requirement in section 4.1.5 (one additional catchbasin per street) and double catch basins were placed in all sag locations.

4.2.2 Watershed Areas

Two different types of watershed areas were determined in the development. The first areas determined were those that contributed to Rear Lot Catchbasins (RLCB). These were found by looking at where the proposed swales started and by analyzing the grading plan to find high elevation points in each residential lot. The boundaries of these watersheds spanned from high point to high point.

Watersheds were also determined for each sewer pipe. These watersheds include all areas where water would drain onto the road between two maintenance holes. Maintenance holes were placed in accordance with Section 4.1.4 and span watersheds.

4.3 Design Flows

Design flows were calculated for each pipe by taking the sum of the flows from all upstream watersheds. Peak runoff flows were calculated using the formula and coefficients defined in Section 4.1.2. Peak flow calculations can be found in Appendix D.1.3. The maximum design flow in the system was found to be 1305.3 L/s in the pipe leaving the residential development (leaving M.H.36). The following equation was used:

$$Q_{design} = 2.778 * C * l * A$$

Sections 4.3.1 and 4.3.2 describe the processes used to determine C, A, and I for this system.

4.3.1 Runoff Coefficients

The runoff coefficients (C) were approximated for each watershed by taking an area weighted average of the land use type in the respective watershed. The area multiplied by the runoff coefficient represents the effective area of runoff. For the calculation of Q_{design}, a sum of all upstream effective runoff coefficients was used. Sample area and C calculations can be found in Appendix D.1.1.

Most houses on Zuccaro Court are single homes with a frontage of over 18 m, therefore a C of 0.4 was used. The remaining houses south of Zuccaro Court have a frontage of less than 18m, and therefore use a C of 0.45. All parks in the development have areas smaller than 4 ha, and thus use a C of 0.25.

4.3.2 Intensity and Time of Concentration

A minimum time of concentration (T_c) of 10 minutes was used when calculating the rainfall intensity for all watersheds with no upstream flow. For downstream watersheds with one direct upstream flow inlet, the T_c was taken to be the sum of the T_c of the immediate upstream watershed and the time of travel through the preceding pipe. Sample calculation for travel time in pipe can be found in Appendix D.1.4.

For watersheds in which there were more than one incoming upstream pipe, the T_c plus the travel time in pipe for each incoming upstream pipe was calculated. The larger of these values was then used as T_c. A table summarizing the T_c values can be found in Appendix E.2: Storm Design Sheet.

After determining the Tc for each watershed, the formula defined in Section 4.1.2 was used to obtain the rainfall intensity, which was used in calculating the design flows. The peak runoff design flow for each pipe in the system can also be found in the Storm Design Sheet. A sample calculation for Tc can be found in Appendix D.1.2.

4.4 Pipe and Catchbasin Lead Design

The pipes in the system were designed based on the storm peak flows determined and in accordance with the requirements set by the Town of Adamsville detailed in Section 4.1.3 and 4.1.5. Appendix E.2 outlines the diameters, slopes, and velocities of all pipes and catchbasin leads in the system. Pipes that are 450 mm or smaller in diameter are PVC pipes while the remainder are concrete pipes.

4.4.1 Pipe Diameters and Slopes

The system was initially designed by setting all the pipe diameters servicing residential areas to the minimum value of 300 mm and slopes to the respective minimum of 4%. Watersheds containing park areas were set to a diameter of 375 mm at a 4% slope according to the minimum requirements in Table 4.1.4. The capacity of the pipes defined were then calculated using the following formula:

$$Q_{capacity} = \frac{1}{n} * A * (D/4)^{\frac{2}{3}} * S^{\frac{1}{2}}$$

All pipes were taken to be PVC or concrete which corresponds to a Mannings 'n' of 0.013 according to the city of Adamsville Design Standards.

Pipes in which the design demand exceeded capacity; diameter was increased to the next available gravity sewer pipe size until flow capacity exceeded design demand. All downstream pipes were adjusted to ensure that no upstream pipe diameters exceeded downstream diameters. Slopes were adjusted to maintain minimum cover of 1.5 m to the crown to meet frost depth requirements and ensure that all inlet obverts are above or aligned with the outlet obverts, as specified in section 4.1.4. The depth of cover above crown for each manhole can be seen in Appendix D.2.1. All pipe diameters and velocities can be found in Appendix E.2.

Flow velocities were calculated using the method in Appendix D.1.3. It was determined that all pipe velocities were below the maximum of 4.5 m/s, meeting the requirements set forth by Town of Adamsville.

4.4.2 Catchbasin Leads

Catchbasin leads were connected directly to the minor storm sewer pipes when there was no closer maintenance hole and when the leads were shorter than 20 m, as specified in section 4.1.5. When the leads were more than 20 m long, the given lead was connected to the minor system via a maintenance hole.

Flow was directed from rear lot catch basins (RLCB) to maintenance holes through leads which were designed in the same manner as the storm sewer pipes outlined in the subsequent sections. Some RLCB leads exceed the maximum length of 30 m, however, these were connected to the nearest road maintenance hole that minimized the number of properties crossed. Minimum diameter and slope in table 4.1.9 were maintained for the RLCB leads.

The RLCB leads that exceed 30 m are the following (by originating RLCB): A, B, M, E, F, G, H, I, K, C, D, J, and L. Their lengths are noted in the storm design sheet, found in Appendix E.2

4.5 Maintenance Hole Design

All maintenance holes were designed to be 1200 mm in diameter, excluding MH 6, MH 8, and MH 36 which had larger diameters designed in accordance with OPSD 701.021 based on the inlet and outlet pipe diameters. All three of these manholes consist of a three-way junction with a maximum pipe hole exceeding 700 mm, which requires the maintenance hole to be greater than 1200 mm. Table 4.5.1 outlines their correct diameters.

Table 4.5.1: Maintenance Holes Exceeding 1200 mm

M.H	Diameter (mm)
6	1500
8	1800
36	1800

4.5.1 Spacing and Locations

Maintenance holes were placed at each change in alignment, slope, or pipe material. Additionally, maintenance holes were added in locations to meet the maximum spacing requirements as specified in section 4.1.4, and where RLCB leads were connected to storm sewer pipes. The distances between all maintenance holes do not exceed 110 m. The largest spacing between maintenance holes is 102 m.

4.6 Maintenance Hole Elevations

The elevation of the ground at each maintenance hole, the depth of each pipe invert and the cover to the crown can be found in the Appendix D.2.1. The following sections outline the characteristics of each maintenance hole in the system, including invert and obvert drops, depth of cover, and drop structures.

4.6.1 Invert and Obvert Drops

To reduce unnecessary costs associated with excavation and maintenance hole construction, all maintenance holes follow the minimum invert drops for inlet and outlet pipes. The minimum drops were calculated in two ways, where the largest of the two was chosen:

1. Minimum drops associated with an angle change between inlet and outlet pipes. In this case, the minimum drops range from 0.03 to 0.075m.
2. Minimum drops resulting from matching the obvert of inlet and outlet pipes of different diameters. For example, if an incoming pipe has a diameter of 250mm but the outlet pipe has a diameter of 300mm, the resulting invert drop from matching crowns is 50mm.
    a. If the inlet and outlet pipe had the same diameter, this was not considered, since the obvert elevation for the outlet would automatically be below the obvert of the inlet.

For maintenance holes with multiple inlet pipes, this procedure was performed for each of them. Once the minimum drop for each inlet pipe was found, the required outlet invert elevation for each case was calculated. Then, the lowest of these invert elevations was used as the invert elevation for the actual outlet pipe.

Invert elevations can be seen in Appendix D.2.1. A sample calculation for the maintenance hole with the largest outlet diameter can be seen in Appendix D.3.

4.6.2 Clear Cover

All pipes were placed to obtain a clear cover of about 3 m with an absolute minimum of 1.5 metres to remain below frost depth. To meet the required vertical watermain clearance of 0.5 m, clear cover of the RLCB leads were set to depths of around 2.5m to 3m below ground. This also ensured that the final pipe in the system reached the existing maintenance hole (MH.36) abiding by the minimum drop and obvert requirements.

4.6.3 Drop Structures

Tee drop structures were added to the storm sewer system when obvert drops between inlet and outlet pipes exceeded 1 m, and where required to meet minimum clearance between watermains and sewers. Table 4.6.2 describes the drop structures in the system and their location.

Table 4.6.2. Drop Structure Details

M.H. From	M.H. To	Drop Structure (m)	Location	Reason
11	12	0.5	Upstream of Crossing #10	Watermain Clearance
32	33	0.3	Upstream of Crossing #5	Watermain Clearance
39	33	0.7	Entering M.H.33	Watermain Clearance
Rear L	27	0.8	Entering M.H.8	Maximum Obvert Exceeded by 1.11m
35	36	1.5	Entering M.H.36	Maximum Obvert Exceeded by 2.17m

4.6 Pipe Crossings and Clearance to Watermains

All crossings between watermain distribution pipes and sewers (both sanitary and storm) are noted in Appendix A.1 and named, as seen in Table 4.6.1. All watermain elevation were taken to be 1.7m below ground elevation and are sloped according to the development street gradings. To find minimum clearance, two distances were considered; 1) the distance between the bottom of the watermain and the sanitary sewer obvert and 2) the distance between the bottom of the watermain and the obvert of the storm sewers. The smaller of these two values was considered the minimum clearance.

Table 4.6.1: All Crossings Between Watermain and Sewers

Crossing #	WAT.TOP ELEV	WAT.BOT ELEV	SAN.INV ELEV	SANI.OBV ELEV	STM.INV ELEV	STM.OBV ELEV	Minimum Clearance
1	206.00	205.60	202.97	201.94	201.69	202.22	3.38
2	203.48	203.18	200.11	200.36	200.73	201.33	1.85
3	203.20	202.80	198.13	198.38	200.07	200.90	1.91
4	202.785	202.385	197.16	197.41	201.63	202.23	**0.16**
5	203.88	203.58	202.29	202.54	202.6	203.05	0.53
6	204.67	204.37	-	-	203.484	203.859	0.51
7	205.7	206	-	-	204.05	204.425	1.58
8	205.9	205.6	-	-	204.58	204.955	0.65
9	205.69	205.39	203.85	204.1	-	-	1.29
10	205.35	204.95	203.13	203.38	203.79	204.24	0.71
11	205.17	204.92	-	-	202.83	203.08	1.84
12	205.04	204.79	-	-	202.37	202.62	2.17
13	204.97	204.72	-	-	202.14	202.39	2.33
14	204.75	204.5	-	-	202.04	202.29	2.21
15	204.68	204.43	-	-	201.82	202.07	2.36
16	204.47	204.22	-	-	201.69	201.94	2.28
17	203.9	203.65	201.09	201.34	201.16	201.76	1.89
18	202.67	202.42	-	-	201.24	201.49	0.93
19	202.9	202.65	-	-	201.66	201.91	0.74

As specified by the Town of Adamsville, the minimum vertical clearance between water distribution and sewers is 0.5m. All crossings, except for Crossing 4, meet this requirement. To enable adequate clearance, the watermain in Crossing 4 will have a watermain bend of 3m, as seen in Table 4.6.2.

Table 4.6.2: Effect of Watermain Bend on Crossing 4

Crossing #	WAT.TOP ELEV	WAT.BOT ELEV	SAN.INV ELEV	SANI.OBV ELEV	SAN Clearance (m)	STM.INV ELEV	STM.OBV ELEV	STM Clearance (m)
4	199.79	199.385	197.16	197.41	1.975	201.63	202.23	2.245

This places the watermain below the storm sewer and above the sanitary sewer, meeting clearance requirement in both cases.

4.7 Material Specifications

The following sections specify the material used for the pipes in the minor system. All Canadian Standards Association (CAN/CSA) specifications, American Society for Testing and Materials (ASTM) specifications and Ontario Provincial Standard Specifications (OPSS) referenced here to are to be the latest revision.

4.7.1 Minor System Pipe Material

All pipes of diameter 450 mm or less are designed to be Polyvinyl Chloride (PVC) pipes. All other pipes are concrete pipes.

The following standards must be followed for the PVC pipes:

- For 250 mm to 375 mm (inclusive), pipe to be manufactured to the latest edition of CSA Standard B-182.2 (ASTM Specification D 3034) with rubber gasketed bell and spigot joints. Pipe and fittings shall have a maximum Standard Dimension Ratio of 35 (SDR-35) and a minimum pipe stiffness of 320 kPa, or higher strength as may be required by the design.
- For 450 mm and larger, pipe to be manufactured to the latest edition of CSA Standard B-182.2 (ASTM Specification F 679 (T-1)) or CSA Standard B-182.4 (ASTM Specification F 794) with rubber gasketed bell and spigot joints. Pipe and fittings shall have a maximum Standard Dimension Ratio of 35 (SDR-35) and a minimum pipe stiffness of 320 kPa, or higher strength as may be required by the design.

All PVC pipes and rubber gasketed joints shall conform to the requirements of OPSS 1841 and OPSD 806.040 & 806.060 (regarding maximum fill / cover).

All pipes with diameters exceeding 450 mm will be concrete pipes that follow the following standards:

- For up to 900 mm (inclusive), pipe to be manufactured to the latest editions of CSA Standards A-257.1 or A-257.2 (whichever applies) and A-257.3, including corresponding appendices.
- For greater than 900 mm, pipe to be manufactured to the latest edition of CSA Standard A-257.2. Joints shall conform to the latest edition of CSA Standard A-257.3.
- All standard strength and extra strength non-reinforced concrete pipes shall conform to CSA A-257 Series and ASTM C-14, C-76, and C655.
- All pipe fittings and joints shall conform to the requirements of CSA A-257 Series and OPSS 1820.
- Class of pipe to be used shall be in accordance with the design requirements.
- All concrete pipes shall be supplied from a pre-qualified plant registered with the Ontario Concrete Pipe Association (OCPA).

4.7.2 Bedding and Backfill Bedding

Materials shall be in accordance with OPSD 802.010 with Granular ‘A’ bedding and OPSD 802.030 Class ‘B’ with Granular ‘A’ bedding for concrete pipes unless otherwise recommended by a qualified geotechnical engineer.
4.8 Cost Evaluation

4.8 Major Storm System Design and Layout

The major system was designed for a storm with a 100-year return period. These are the main design objectives:

- The roadway network shall be designed to convey overland flow to stormwater abilities or natural stream corridors.
- A maximum flow depth of 0.3m, measured from the gutter.
- Any sag points in the system must limit maximum ponding depth of 0.3m.
- All overland flow channels must convey the storm peak flow without flooding adjacent properties.
    - The maximum velocity in the channel during the 100-year storm must be 2.5m/s.

The proposed major storm system in Adamsville Village follows the proposed grading provided. The flow is conveyed along the roadway following the proposed grading. Sag points are limited to a depth of 0.30 m throughout the system. See Appendix A.4 for the overland flow conveyance.

The following locations contain local sag points, where water will pool during a 100-year storm:

- South of Lot 163 on Adams Avenue.
- East of Lot 164 on Meyer Street,
- South of Lot 299 on Adams Avenue.
- East of Lot 299 on Karney Street.
- South of Lot 315 on Nault Drive.
- East of Lot 134 on Mokin Street.
- South of Lot 154 on Zuccaro Court.
- East of Lot 245 on Mokin Street.

Based on the development grading, these are the locations where water will exit the development:

- Eastwards of Adams Avenue
- Westwards on Meyer Street Intersections (West of Lot 179 and Lot 171)
- Eastwards of Fabian Papa Way

5.0 Conclusion

Based on the results from the detailed calculated analysis, it has been determined that the water distribution, sanitary sewer and storm drainage systems proposed in this report meet the design criteria for the Adamsville Village development and will adequately service the town.

The water distribution system makes use of the two existing watermains, the one located near the intersection of Adams Avenue and Karney Road, and the other located at the intersection of Fabian Papa Way and Meyer Road. The estimated total capital cost to install the water distribution system has been calculated to be around $922,900.00 has been optimized to include all necessary pipes and fire hydrants that are required to service the village.

The placement of all storm and sanitary maintenance holes and pipes were designed around the constraints of the system's exiting maintenance holes at the intersection of Adams Avenue and Karney Street. All sanitary and storm flows from the entire town will pass through these maintenance holes to exit Adamsville Village.

6.0 References

[1] Town of Adamsville: Engineering Design Criteria and Standards Manual. (2022).

[2] MOE Guideline F-6, "Sewer and Watermain Installation: Separation Distance Requirements", and MOE Procedure F-6-1, "Procedures to Govern Separation of Sewers and Watermains", April 1994.

Appendix A: AutoCAD Drawings

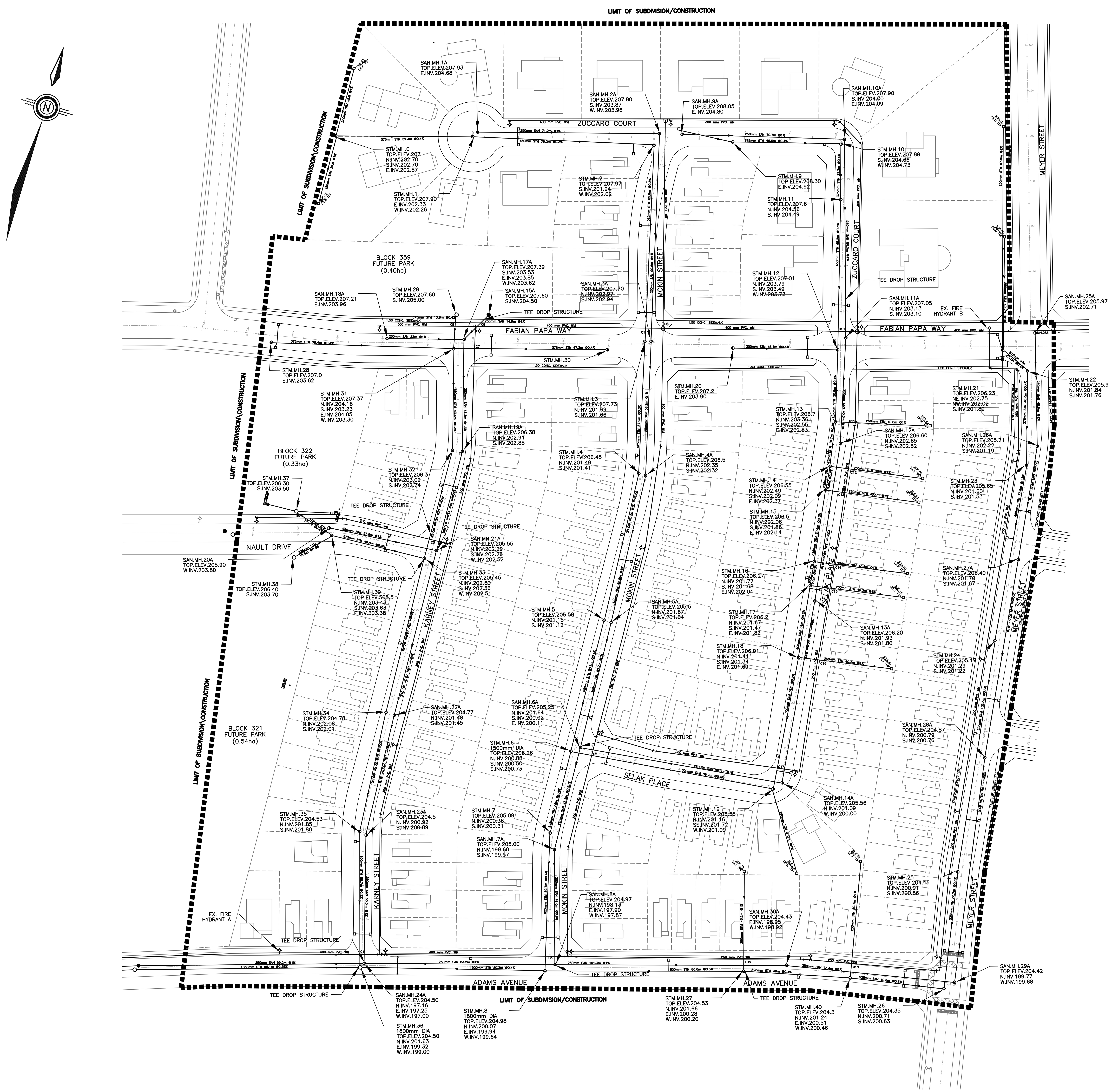
Appendix A.1: **General Plan of Services**

Appendix A.2: Plan and Profile View 1

Appendix A.3: Plan and Profile View 2

Appendix A.4: Sanitary Drainage Area

Appendix A.4: Storm Drainage Plans



METRIC
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE SHOWN

KEY PLAN N.T.S.

BENCH MARK:
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE DERIVED FROM THE CITY OF TORONTO BENCHMARK No.123456789 HAVING AN ELEVATION OF 100.000m. PLEASE REFER TO ORIGINAL TOPOGRAPHICAL SURVEY BY XXXXXXX SURVEYORS INC. (PROJECT NO. XXXXXXXXXX) FOR FURTHER DETAILS.

LEGEND

123.45	EXISTING ELEVATION
123.45	PROPOSED ELEVATION
CB. 123.45	PROP TOP OF CATCHBASIN ELEVATION
CB.INV. 123.45	PROP ELEVATION OF CATCHBASIN INVERT
2.0%	DRAINAGE FLOW DIRECTION AND SLOPE
	PROP OVERLAND FLOW DIRECTION
CB.1	CATCHBASIN
MH.1	STORM MANHOLE
MH.1A	SANITARY MANHOLE
EX.FH	EX. VALVED HYDRANT
	WATER SERVICE CONNECTION
	PROPOSED SANITARY SEWER
	PROPOSED STORM SEWER
0.20	DRAINAGE AREA (ha)
0.55	RUNOFF COEFFICIENT
0.20	DRAINAGE AREA (ha)
20	POPULATION

NO.	REVISION	BY	DATE
0	ISSUED FOR TEAM COORDINATION	A.M.	01 Jan 2022

REVISIONS

CLIENT:
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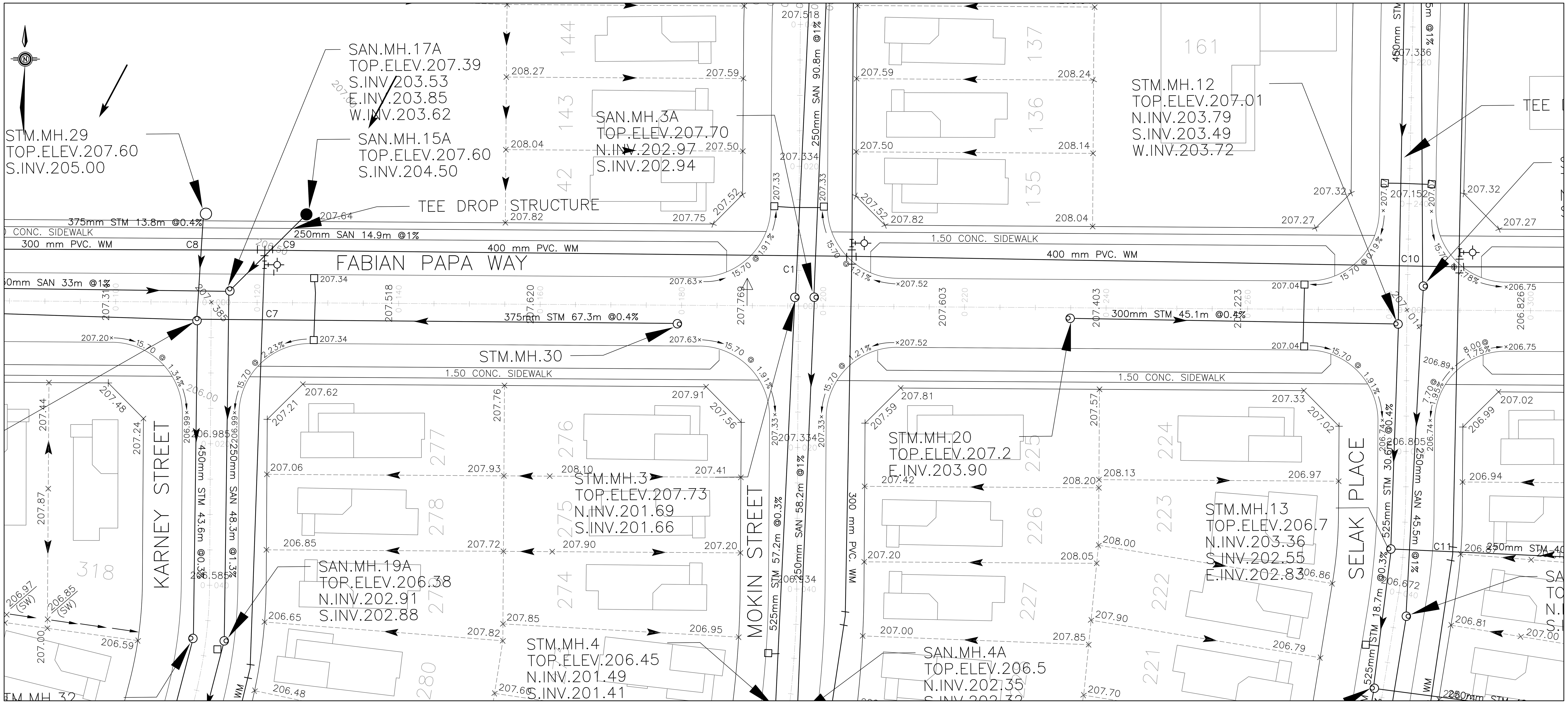
MUNICIPALITY:
ADAMSVILLE
ENGINEERING AND CONSTRUCTION SERVICES

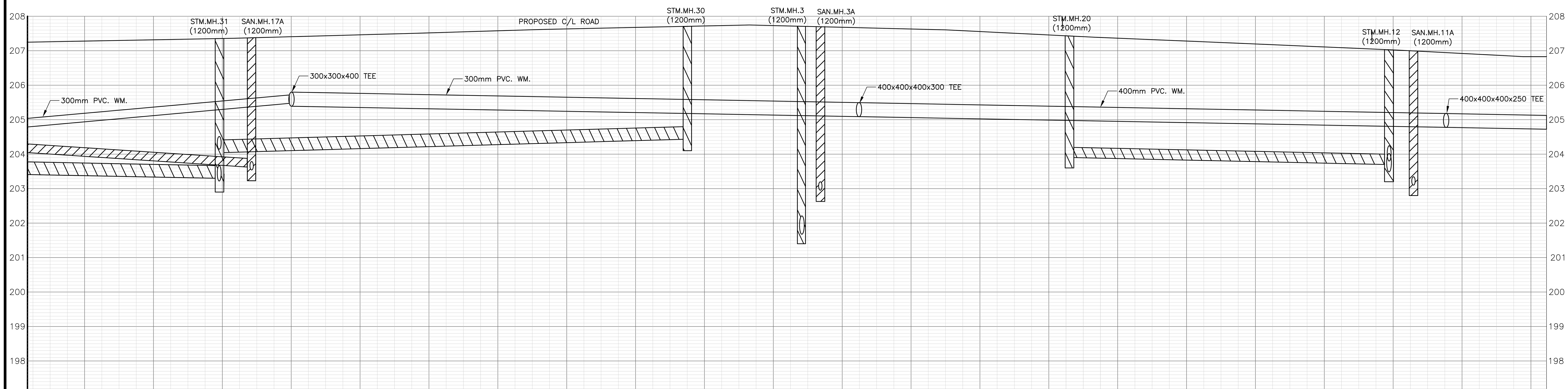
PROJECT NAME:
ADAMSVILLE VILLAGE
DRAFT PLAN SUBDIVISION

DRAWING TITLE:
GENERAL PLAN

DWN. BY:	X.X.
DESIGNED BY:	X.X.
CHECKED BY:	X.X.
SCALE:	1:750
DATE:	XXX 2022
SHEET NO:	1 OF 6

PROJECT NO.	DWG NO.	REV. NO.
22000	GP-1	0





STORM INVERT	375mm STM 79.4m 0.4%	W.203.30 S.203.23 E.204.05 N.204.16	375mm STM 67.3m 0.4%	W.204.32	S.201.66 N.201.69		E.203.90	300mm STM 45.1m 0.4%	W.203.71 S.203.49 N.204.29		STORM INVERT
SANITARY INVERT	250mm SAN 33m 1%	W.203.62 S.203.53 E.203.85			S.202.94 N.202.97				S.203.10 N.203.13		SANITARY INVERT
PROPOSED GRADES	207.314	207.385	207.518	207.620	207.769	207.603	207.403	207.223	207.014	206.826	PROPOSED GRADES
C/L CHAINAGE	0+100	0+120	0+140	0+160	0+180	0+200	0+220	0+240	0+260	0+280 / 0+300	C/L CHAINAGE

METRIC
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE SHOWN

KEY PLAN N.T.S.

BENCH MARK:
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LEGEND
- 123.45 EXISTING ELEVATION
- 123.45 PROPOSED ELEVATION
- CB. 123.45 PROP TOP OF CATCHBASIN ELEVATION
- CB.INV. 123.45 PROP ELEVATION OF CATCHBASIN INVERT
- 2.0% DRAINAGE FLOW DIRECTION AND SLOPE
- PROP OVERLAND FLOW DIRECTION
- CB.1 CATCHBASIN
- MH.1 STORM MANHOLE
- MH.1A SANITARY MANHOLE
- EX.FH EX. VALVED HYDRANT
- WATER SERVICE CONNECTION
- PROPOSED SANITARY SEWER
- PROPOSED STORM SEWER
- 0.20 / 0.55 DRAINAGE AREA (ha) / RUNOFF COEFFICIENT
- 0.20 / 20 DRAINAGE AREA (ha) / POPULATION

NO.	REVISION	BY	DATE
0	ISSUED FOR TEAM COORDINATION	A.M.	01 JAN 2022

REVISIONS

CLIENT:
fabian papa & partners
A Division of FP&P HydroTek Inc.
216 Chrislea Road, Suite ... Vaughan, Ontario

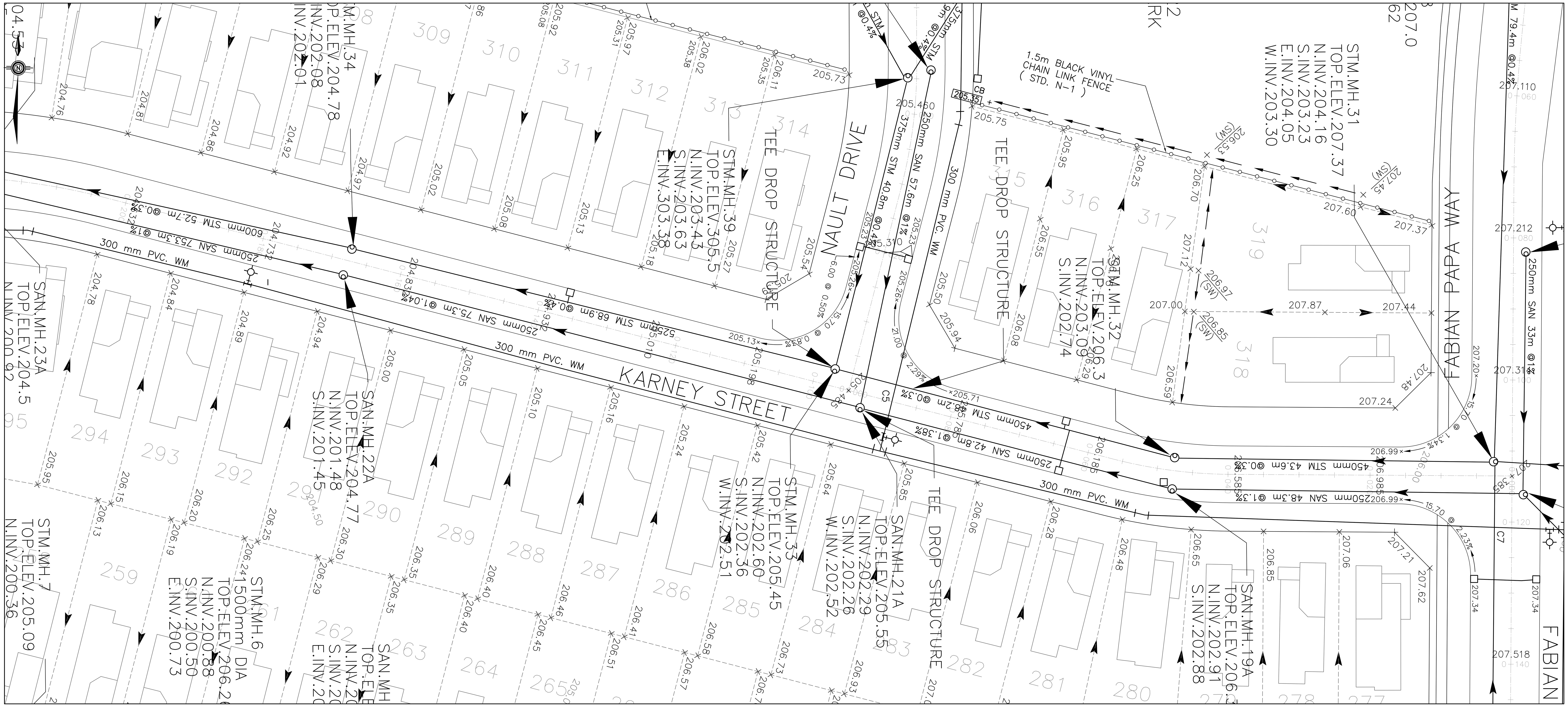
MUNICIPALITY:
ADAMSVILLE ENGINEERING AND CONSTRUCTION SERVICES

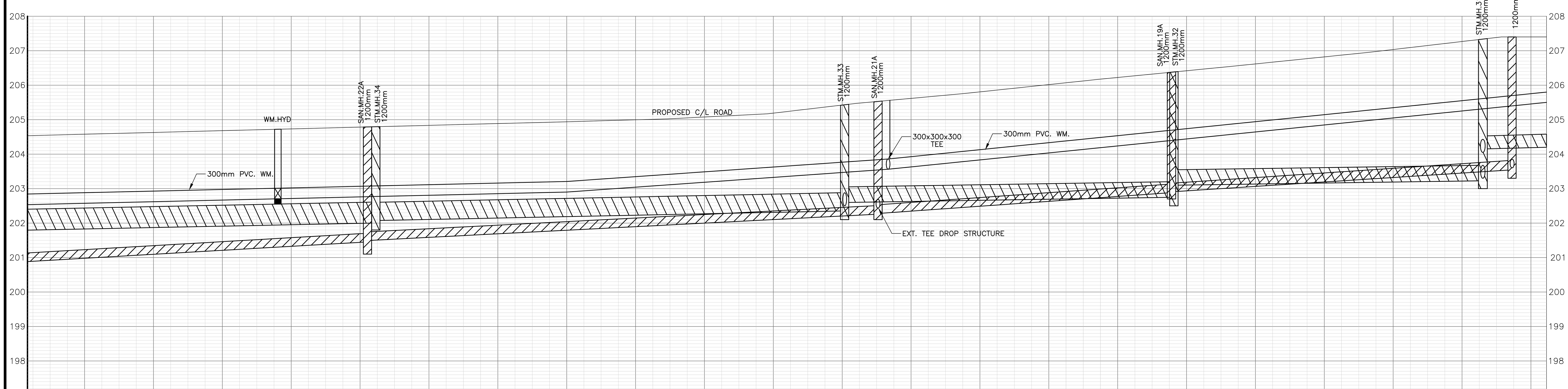
PROJECT NAME:
ADAMSVILLE VILLAGE
DRAFT PLAN SUBDIVISION

DRAWING TITLE:
FABIAN PAPA WAY PROFILE
CHAINAGE 0+085 TO 0+300

DWN. BY:	X.X.
DESIGNED BY:	X.X.
CHECKED BY:	X.X.
SCALE:	1:250
DATE:	XXX 2022
SHEET NO:	5 OF 6

PROJECT NO.	DWG NO.	REV. NO.
22000	PP-1	0





STORM INVERT	600mm STM. 52.7m @0.4% (S.201.85, N.202.09)	525mm STM. 68.9m @0.4% (W.202.51, S.202.36, N.202.60)	450mm STM. 48.2m @0.3% (S.202.74, N.203.09)	450mm STM. 43.6m @0.3% (W.203.30, S.203.23, N.204.16, E.204.05)
SANITARY INVERT	250mm SAN. 75.3m @1% (S.202.01, N.202.09)	250mm SAN. 100.5m @1.04% (W.203.29, S.202.26, N.202.29)	250mm SAN. 57m @1.38% (S.202.88, N.202.91)	250mm SAN. 42.8m @1.30% (W.203.62, S.203.53, E.203.85)

PROPOSED GRADES: 204.632, 204.732, 204.832, 204.932, 205.010, 205.199, 205.485, 205.785, 206.185, 206.585, 206.985, 207.385

C/L CHAINAGE: 0+200, 0+180, 0+160, 0+140, 0+120, 0+100, 0+080, 0+060, 0+040, 0+020, 0+000

METRIC
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE SHOWN

KEY PLAN N.T.S.

BENCH MARK:
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE DERIVED FROM THE CITY OF TORONTO BENCHMARK No.123456789 HAVING AN ELEVATION OF 100.000m. PLEASE REFER TO ORIGINAL TOPOGRAPHICAL SURVEY BY XXXXXXX SURVEYORS INC. (PROJECT NO. XXXXXXXXXX) FOR FURTHER DETAILS.

LEGEND
- 123.45 EXISTING ELEVATION
- 123.45 PROPOSED ELEVATION
- 123.45 PROP TOP OF CATCHBASIN ELEVATION
- 123.45 PROP ELEVATION OF CATCHBASIN INVERT
- DRAINAGE FLOW DIRECTION AND SLOPE
- PROP OVERLAND FLOW DIRECTION
- CB.1 CATCHBASIN
- MH.1 STORM MANHOLE
- MH.1A SANITARY MANHOLE
- EX.FH EX. VALVED HYDRANT
- WATER SERVICE CONNECTION
- PROPOSED SANITARY SEWER
- PROPOSED STORM SEWER
- 0.20 DRAINAGE AREA (ha) / 0.55 RUNOFF COEFFICIENT
- 0.20 DRAINAGE AREA (ha) / 20 POPULATION

NO.	REVISION	BY	DATE
0	ISSUED FOR TEAM COORDINATION	A.M.	01 JAN 2022

REVISIONS

CLIENT: fabian papa & partners, A Division of FP&P HydroTek Inc., 216 Chrislea Road, Suite ..., Vaughan, Ontario, L4L ...

MUNICIPALITY: ADAMSVILLE ENGINEERING AND CONSTRUCTION SERVICES

PROJECT NAME: ADAMSVILLE VILLAGE
DRAFT PLAN SUBDIVISION

DRAWING TITLE: KARNEY LANE PROFILE
CHAINAGE 0+000 TO 0+210

DWN. BY:	X.X.
DESIGNED BY:	X.X.
CHECKED BY:	X.X.
SCALE:	1:250
DATE:	XXX 2022
SHEET NO:	6 OF 6

PROJECT NO.	DWG NO.	REV. NO.
22000	PP-2	0

METRIC
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE SHOWN

KEY PLAN N.T.S.

BENCH MARK:
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE DERIVED FROM THE CITY OF TORONTO BENCHMARK No.123456789 HAVING AN ELEVATION OF 100.000m. PLEASE REFER TO ORIGINAL TOPOGRAPHICAL SURVEY BY XXXXXXX SURVEYORS INC. (PROJECT NO. XXXXXXXXXX) FOR FURTHER DETAILS.

LEGEND

123.45	EXISTING ELEVATION
123.45	PROPOSED ELEVATION
CB. 123.45	PROP TOP OF CATCHBASIN ELEVATION
CB.INV. 123.45	PROP ELEVATION OF CATCHBASIN INVERT
2.0%	DRAINAGE FLOW DIRECTION AND SLOPE
	PROP OVERLAND FLOW DIRECTION
CB.1	CATCHBASIN
MH.1	STORM MANHOLE
MH.1A	SANITARY MANHOLE
EX.FH	EX. VALVED HYDRANT
	WATER SERVICE CONNECTION
	PROPOSED SANITARY SEWER
	PROPOSED STORM SEWER
0.20 / 0.55	DRAINAGE AREA (ha) / RUNOFF COEFFICIENT
0.20 / 20	DRAINAGE AREA (ha) / POPULATION

NO.	REVISION	BY	DATE
0	ISSUED FOR TEAM COORDINATION	A.M.	01 Jan 2022

REVISIONS

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A Division of FP&P HydraTek Inc.
216 Chrislea Road, Suite 204
Vaughan, Ontario, L4L 8S5
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www.fabianpapa.com

MUNICIPALITY:
ADAMSVILLE
ENGINEERING AND CONSTRUCTION SERVICES

PROJECT NAME:
ADAMSVILLE VILLAGE
DRAFT PLAN SUBDIVISION

DRAWING TITLE:
SANITARY DRAINAGE AREA PLAN

DWN. BY:	X.X.
DESIGNED BY:	X.X.
CHECKED BY:	X.X.
SCALE:	1:750
DATE:	XXX 2022
SHEET NO:	4 OF 6

PROJECT NO.	DWG NO.	REV. NO.
22000	SSA-1	0

METRIC
ALL DIMENSIONS AND ELEVATIONS ARE IN METRES UNLESS OTHERWISE SHOWN

KEY PLAN — N.T.S.

BENCH MARK:
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE DERIVED FROM THE CITY OF TORONTO BENCHMARK No.123456789 HAVING AN ELEVATION OF 100.000m. PLEASE REFER TO ORIGINAL TOPOGRAPHICAL SURVEY BY XXXXXXX SURVEYORS INC. (PROJECT NO. XXXXXXXXXX) FOR FURTHER DETAILS.

LEGEND

- 123.45 EXISTING ELEVATION
- 123.45 PROPOSED ELEVATION
- CB. 123.45 PROP TOP OF CATCHBASIN ELEVATION
- CB.INV. 123.45 PROP ELEVATION OF CATCHBASIN INVERT
- 2.0% DRAINAGE FLOW DIRECTION AND SLOPE
- PROP OVERLAND FLOW DIRECTION
- CB.1 CATCHBASIN
- MH.1 STORM MANHOLE
- MH.1A SANITARY MANHOLE
- EX.FH EX. VALVED HYDRANT
- WATER SERVICE CONNECTION
- PROPOSED SANITARY SEWER
- PROPOSED STORM SEWER
- 0.20 / 0.55 DRAINAGE AREA (ha) / RUNOFF COEFFICIENT
- 0.20 / 20 DRAINAGE AREA (ha) / POPULATION

NO.	REVISION	BY	DATE
0	ISSUED FOR TEAM COORDINATION	A.M.	01 Jan 2022

REVISIONS

CLIENT:
fp&p fabian papa & partners
A Division of FP&P HydroTek Inc.
216 Chrislea Road, Suite ... Vaughan, Ontario, L4L ...

MUNICIPALITY:
ADAMSVILLE
ENGINEERING AND CONSTRUCTION SERVICES

PROJECT NAME:
ADAMSVILLE VILLAGE
DRAFT PLAN SUBDIVISION

DRAWING TITLE:
STORM DRAINAGE AREA PLAN

DWN. BY:	X.X.
DESIGNED BY:	X.X.
CHECKED BY:	X.X.
SCALE:	1:750
DATE:	XXX 2022
SHEET NO:	3 OF 6

PROJECT NO.	DWG NO.	REV. NO.
22000	SDA-1	0

Appendix B - Water Distribution

Appendix B.2 – Data Tables

Appendix B.2.1: Water Flow Demand Calculations

Zone	Elevation	Number of Houses	Number of Parks	Population	Residential Daily Demand (L/s)	Park Area (ha)	Institutional Daily Demand (L/s)	Total Q_{avg} (L/s)	Fire Flow (L/s)	$Q_{max, hour}$ (L/s)	$Q_{max, day} + Q_{fire}$ (L/s)	Design Demand (L/s)
A	204.5	8	0	32	0.167	0	0	0.167	100	0.688	100.458	100.458
B	204.89	15	0	60	0.313	0	0	0.313	100	1.291	100.859	100.859
C	205.8	15	0	60	0.313	0	0	0.313	100	1.291	100.859	100.859
D	206.38	0	2	0	0.000	0.87	0.435	0.435	100	1.797	101.196	101.196
E	207.6	3	0	12	0.063	0	0	0.063	100	0.258	100.172	100.172
F	207.5	14	0	56	0.292	0	0	0.292	100	1.205	100.802	100.802
G	207.96	6	0	24	0.125	0	0	0.125	100	0.516	100.344	100.344
H	208.14	8	0	32	0.167	0	0	0.167	100	0.688	100.458	100.458
I	208	4	0	16	0.083	0	0	0.083	100	0.344	100.229	100.229
J	207.2	11	0	44	0.229	0	0	0.229	100	0.946	100.630	100.630
K	205.25	16	0	64	0.333	0	0	0.333	100	1.377	100.917	100.917
L	204.3	10	0	40	0.208	0	0	0.208	100	0.860	100.573	100.573
M	205.06	11	0	44	0.229	0	0	0.229	100	0.946	100.630	100.630
N	205.4	18	0	72	0.375	0	0	0.375	100	1.549	101.031	101.031
O	206.15	13	0	52	0.271	0	0	0.271	100	1.119	100.745	100.745
P	205.8	19	0	76	0.396	0	0	0.396	100	1.635	101.089	101.089
Q	206.49	12	0	48	0.250	0	0	0.250	100	1.033	100.688	100.688
R	207.35	2	1	8	0.042	0.4	0.2	0.242	100	0.998	100.665	100.665

Appendix B.2.2: Ground Elevations of Hydrants

Hydrant	Elevation
A	204.5
B	204.89
C	205.8
D	206.38
E	207.6
F	207.5
G	207.96
H	208.14
I	208
J	207.2
K	205.25
L	204.3
M	205.06
N	205.4
O	206.15
P	205.8
Q	206.49
R	207.35
EX1	204.59
EX2	204.9
EX3	205.9
FHA	204.26
FHB	206.39

Appendix.B.2.3: Diameter and lengths of watermain pipes

Pipe	Length	Nominal Diameter (mm)	Actual Diameter (mm)
1	43.6	400	406
2	90.5	300	305
3	92.7	300	305
4	87	300	305
5	98.8	300	305
6	42.9	300	305
7	82.4	400	406
8	89.5	300	406
9	74.1	400	406
10	76.4	300	305
11	92.1	400	406
12	58.3	400	406
13	76.7	250	254
14	88.8	250	254
15	67.7	250	254
16	70.2	250	254
17	73.9	250	254
18	102.7	250	254
19	98.4	300	305
20	85	300	305
21	100	300	305
22	90.5	250	254
23	97.1	250	254
24	101.4	250	254
25	85.9	400	406
26	84.5	400	406

Appendix B.3 – Sample Calculations for Zone A

$$Elevation = 204.5$$

The elevation was determined through reading the contour lines that overlap the area.

$$\#\,Houses = 8, \#\,of\ Parks = 0$$

The number of houses and parks were accounted for in each zone by counting.

$$Population = (4\,ppu) \times (8\,houses) = 32\,people$$

To determine the population of each zone, the population density of 4 ppu was multiplied by the number of houses found in that zone. 4 ppu was used due to the assumption all units were single family dwellings.

$$Residential\ Daily\ Demand = \frac{(population) * (residential\ daily\ demand)}{24 \times 3600}$$

$$Residential\ Daily\ Demand = (32\,c) \times (450\,L/c/d)\ /\ (24\,hr * 3600\,sec) = 0.167\,L/s$$

To calculate the residential daily demand, the population of the zone multiplied by the residential daily demand.

$$Park\ Area = 0\,, Institutional\ Daily\ Demand = 0$$

$$Institutional\ Daily\ Demand = (Park\ Area) \times (0.5\,L/s/ha)$$

Zone A had no parks so both values were zero. In the case the zone had a park, it was treated as a school and calculated the institutional demand with a park daily demand of 0.5 L/s/ha.

$$Total\ Q\ average = (Residential\ Daily\ Demand) + (Institutional\ Daily\ Demand)$$

$$Total\ Q\ average = 0.167\,L/s + 0\,L/s = 0.167\,L/s$$

The total average flow was determined by adding the residential daily demand of the zone and its institutional daily demand.

$$Q\ max, hour = (pfp, max\ hour) \times (Total\ Q\ average) = (4.13) \times (0.167\,L/s) = 0.688\,L/s$$

To determine the max hour flow, the total average flow was multiplied by the peak factor given for maximum hour (4.13).

$$Q\ max, day + Q\ fire = (Fire\ Flow) + (Total\ Q\ average * pf, max\ day)$$

$$Q\ max, day + Q\ fire = (100\,L/s) + (0.167L/s \times 2.75) = 100.458\,L/s$$

To determine the max day flow, the total average flow multiplied by the peak factor for maximum day (2.75) was added to the fire flow. For all the zones the fire flow was 100 L/s, including parks.

$$Design\ Demand = MAX(Q\ max, hour : Q\ max, day + Q\ fire)$$

The design demand for each zone was taken as the max day flow and fire flow as it is the highest.

Appendix B.4– Hydraulic Model Results

Appendix B.4.1: Link results at maximum hour, minimum hour, and maximum day

Maximum Hour Demand Results

Link	Flow (L/s)	Velocity (m/s)	Headloss (m/km)
1	0	0	0
2	0.36	0	0
3	-0.93	0.01	0
4	1.8	0.02	0
5	-4.02	0.06	0.02
6	1	0.01	0
7	-5.28	0.04	0.01
8	-2.11	0.02	0
9	-0.52	0	0
10	-3.31	0.05	0.01
11	-3.66	0.03	0
12	-15.49	0.12	0.05
13	3.06	0.06	0.03
14	3.06	0.06	0.03
15	1.68	0.03	0.01
16	1.68	0.03	0.01
17	-0.82	0.02	0
18	-0.82	0.02	0
19	1.17	0.02	0
20	2.62	0.04	0.01
21	3.74	0.05	0.02
22	0.1	0	0
23	1.73	0.03	0.01
24	2.77	0.05	0.03
25	-8.12	0.06	0.02
26	-1.05	0.01	0

Minimum Hour Demand Results

Link	Flow (L/s)	Velocity (m/s)	Headloss (m/km)
1	0.00	0.00	0.00
2	0.03	0.00	0.00
3	-0.09	0.00	0.00
4	0.17	0.00	0.00
5	-0.39	0.01	0.00
6	0.10	0.00	0.00
7	-0.51	0.00	0.00
8	-0.20	0.00	0.00
9	-0.05	0.00	0.00
10	-0.32	0.00	0.00
11	-0.35	0.00	0.00
12	-1.50	0.00	0.00
13	0.30	0.01	0.00
14	0.30	0.01	0.00
15	0.16	0.00	0.00
16	0.16	0.00	0.00
17	-0.08	0.00	0.00
18	-0.08	0.00	0.00
19	0.11	0.00	0.00
20	0.25	0.01	0.00
21	0.36	0.00	0.00
22	0.01	0.00	0.00
23	0.18	0.00	0.00
24	0.27	0.01	0.00
25	-0.79	0.01	0.00
26	-0.10	0.00	0.00

Maximum Day Demand Results

Links	Flow (L/s)	Velocity (m/s)	Head Loss (m/km)
1	0.00	0.00	0.00
2	0.24	0.00	0.00
3	-0.62	0.01	0.00
4	1.20	0.02	0.00
5	-2.68	0.04	0.01
6	0.67	0.01	0.00
7	-3.52	0.03	0.00
8	-1.40	0.01	0.00
9	-0.34	0.00	0.00
10	-2.21	0.03	0.01
11	-2.43	0.02	0.00
12	-10.31	0.08	0.03
13	2.04	0.04	0.01
14	2.04	0.04	0.01
15	1.12	0.02	0.00
16	1.12	0.02	0.00
17	-0.55	0.01	0.00
18	-0.55	0.01	0.00
19	0.78	0.01	0.00
20	1.75	0.02	0.00
21	2.49	0.03	0.01
22	0.07	0.00	0.00
23	1.15	0.02	0.01
24	1.84	0.04	0.01
25	-5.41	0.04	0.01
26	-0.70	0.01	0.00

Appendix B.4.2: Node results at maximum hour, minimum hour and maximum day

Maximum Hour Demand Results

Node	Pressure (m)	Pressure (kPa)
A	44.06	432.23
B	43.67	428.40
C	42.76	419.48
D	42.18	413.79
E	40.97	401.92
F	41.07	402.90
G	40.61	398.38
H	40.43	396.62
I	40.57	397.99
J	41.37	405.84
K	43.32	424.97
L	44.27	434.29
M	43.5	426.74
N	43.16	423.40
O	42.42	416.14
P	42.77	419.57
Q	42.27	414.67
R	41.22	404.37
FHB	42.18	413.79
FHA	44.3	434.58
EX1	43.98	431.44
EX2	43.67	428.40
EX3	42.67	418.59

Minimum Hour Demand Results

Node	Pressure (m)	Pressure (kPa)
A	47.39	464.90
B	47.00	461.07
C	46.09	452.14
D	45.51	446.45
E	44.39	435.47
F	44.39	435.47
G	43.93	430.95
H	43.75	429.19
I	43.89	430.56
J	44.69	438.41
K	46.64	457.54
L	47.59	466.86
M	46.83	459.40
N	46.49	456.07
O	45.74	448.71
P	46.09	452.14
Q	45.59	447.24
R	44.54	436.94
FHB	45.50	446.36
FHA	47.63	467.25
EX1	47.30	464.01
EX2	46.99	460.97
EX3	45.99	451.16

Maximum Day Demand Results

Node	Pressure (m)	Pressure (kPa)
A	45.93	450.57
B	45.54	446.75
C	44.63	437.82
D	44.04	432.03
E	42.83	420.16
F	42.93	421.14
G	42.47	416.63
H	42.29	414.86
I	42.43	416.24
J	43.23	424.09
K	45.18	443.22
L	46.13	452.54
M	45.37	445.08
N	45.03	441.74
O	44.28	434.39
P	44.63	437.82
Q	44.13	432.92
R	43.08	422.61
FHB	44.04	432.03
FHA	46.17	452.93
EX1	45.84	449.69
EX2	45.53	446.65
EX3	44.53	436.84

Appendix B.5 – Fire Flow Test Results

Appendix B.5.1: Link results for fire flow at critical points, while maintain all other nodes at maximum daily flow

Hydrant G Results

Links	Flow (L/s)	Velocity (m/s)	Head Loss (m/km)
1	57.05	0.44	0.6
2	23.35	0.32	0.46
3	22.49	0.31	0.43
4	1.2	0.02	0
5	20.43	0.28	0.36
6	0.67	0.01	0
7	19.59	0.15	0.08
8	67.23	0.52	0.82
9	-100.34	0.78	1.72
10	-33.58	0.46	0.91
11	-33.8	0.26	0.23
12	-60.38	0.47	0.67
13	-5.08	0.1	0.08
14	-5.08	0.1	0.08
15	-6	0.12	0.11
16	-6	0.12	0.11
17	6.57	0.13	0.13
18	6.57	0.13	0.13
19	-26.04	0.36	0.57
20	-17.89	0.24	0.28
21	-17.15	0.23	0.26
22	-7.11	0.14	0.15
23	-6.03	0.12	0.11
24	-5.34	0.11	0.09
25	-31.29	0.24	0.2
26	33.24	0.26	0.22

Hydrant D Results

Links	Flow (L/s)	Velocity (m/s)	Head Loss (m/km)
1	57.09	0.44	0.6
2	47.11	0.64	1.71
3	46.25	0.63	1.65
4	101.2	1.39	7.03
5	-55.81	0.76	2.33
6	0.67	0.01	0
7	-56.65	0.44	0.6
8	-12.89	0.1	0.04
9	-0.34	0	0
10	-13.69	0.19	0.17
11	-13.92	0.11	0.04
12	-51.09	0.39	0.49
13	4.16	0.08	0.05
14	4.16	0.08	0.05
15	3.25	0.06	0.03
16	3.25	0.06	0.03
17	-2.67	0.05	0.02
18	-2.67	0.05	0.02
19	-11.57	0.16	0.13
20	-10.7	0.15	0.11
21	-9.96	0.14	0.1
22	0.16	0	0
23	1.25	0.02	0.01
24	1.94	0.04	0.01
25	-34.6	0.27	0.24
26	9.53	0.27	0.02

Hydrant H Results

Link	Flow (L/s)	Velocity (m/s)	Headloss (m/km)
1	57.05	0.44	0.6
2	23.35	0.32	0.46
3	22.49	0.31	0.43
4	1.2	0.02	0
5	20.43	0.28	0.36
6	0.67	0.01	0
7	19.59	0.15	0.08
8	67.23	0.52	0.82
9	-0.34	0	0
10	-33.57	0.46	0.91
11	-33.8	0.26	0.23
12	-60.38	0.47	0.67
13	-5.08	0.1	0.08
14	-5.08	0.1	0.08
15	-6	0.12	0.11
16	-6	0.12	0.11
17	6.57	0.13	0.13
18	6.57	0.13	0.13
19	-26.04	0.36	0.57
20	-17.89	0.24	0.28
21	-17.15	0.23	0.26
22	-7.11	0.14	0.15
23	-6.03	0.12	0.11
24	-5.34	0.11	0.09
25	-31.29	0.24	0.2
26	33.24	0.26	0.22

Hydrant R Results

Link	Flow (L/s)	Velocity (m/s)	Headloss (m/km)
1	57.07	0.44	0.6
2	29.14	0.4	0.7
3	28.27	0.39	0.66
4	1.2	0.02	0
5	26.22	0.36	0.58
6	100.66	1.38	6.96
7	-74.62	0.58	0.99
8	-16.24	0.13	0.06
9	-0.34	0	0
10	-17.04	0.23	0.26
11	-17.27	0.13	0.07
12	-58.06	0.45	0.62
13	-2.78	0.05	0.03
14	-2.78	0.05	0.03
15	-3.7	0.07	0.04
16	-3.7	0.07	0.04
17	4.27	0.08	0.06
18	4.27	0.08	0.06
19	-22.57	0.31	0.44
20	-16.82	0.23	0.25
21	-16.08	0.22	0.23
22	-4.27	0.09	0.07
23	-3.63	0.07	0.04
24	-2.94	0.06	0.03
25	-43.11	0.33	0.36
26	27.47	0.21	0.16

Appendix B.5.2: Node results for fire flow at critical points, while maintain all other nodes at maximum daily flow

Hydrant G Results

Node	Pressure (m)	Pressure (kPa)
A	18.13	177.86
B	17.69	173.54
C	16.74	164.22
D	16.16	158.53
E	14.92	146.37
F	15	147.15
G	14.34	140.68
H	14.29	140.18
I	14.5	142.25
J	15.32	150.29
K	17.32	169.91
L	18.29	179.42
M	17.55	172.17
N	17.15	168.24
O	16.38	160.69
P	16.74	164.22
Q	16.23	159.22
R	15.16	148.72
FHB	16.17	158.63
FHA	18.39	180.41
EX1	18.01	176.68
EX2	17.68	173.44
EX3	16.66	163.43

Hydrant D Results

Node	Pressure (m)	Pressure (kPa)
A	18.09	177.46
B	17.54	172.07
C	16.48	161.67
D	15.29	149.99
E	14.91	146.27
F	15.06	147.74
G	14.6	143.23
H	14.42	141.46
I	14.57	142.93
J	15.38	150.88
K	17.35	170.20
L	18.29	179.42
M	17.53	171.97
N	17.18	168.54
O	16.42	161.08
P	16.78	164.61
Q	16.28	159.71
R	15.16	148.72
FHB	16.22	159.12
FHA	18.36	180.11
EX1	18	176.58
EX2	17.69	173.54
EX3	16.7	163.83

Hydrant H Results

Node	Pressure (m)	Pressure (kPa)
A	18.13	177.86
B	17.7	173.64
C	16.75	164.32
D	16.17	158.63
E	14.91	146.27
F	15	147.15
G	14.47	141.95
H	14.29	140.18
I	14.5	142.25
J	15.32	150.29
K	17.32	169.91
L	18.29	179.42
M	17.55	172.17
N	17.15	168.24
O	16.38	160.69
P	16.74	164.22
Q	16.23	159.22
R	15.16	148.72
FHA	16.17	158.63
FHB	18.39	180.41
EX1	18.01	176.68
EX2	17.68	173.44
Ex3	16.67	163.53

Hydrant R Results

Node	Pressure (m)	Pressure (kPa)
A	18.11	177.66
B	17.66	173.24
C	16.69	163.73
D	16.11	158.04
E	14.83	145.48
F	15.01	147.25
G	14.56	142.83
H	14.38	141.07
I	14.54	142.64
J	15.34	150.49
K	17.33	170.01
L	18.29	179.42
M	17.54	172.07
N	17.16	168.34
O	16.39	160.79
P	16.75	164.32
Q	16.25	159.41
R	14.78	144.99
FHA	16.19	158.82
FHB	18.38	180.31
EX1	18	176.58
EX2	17.69	173.54
EX3	16.68	163.63

Appendix B.6– Cost Estimation

The lengths of all the links that correspond to each pipe size were added up and multiplied by the unit price. The costs of each pipe size were then added to get the final cost of $922,877.00.

Item	Quantity	Unit	Unit Price	Amount
150mm	0	m	260	0
200mm	0	m	320	0
250mm	769	m	370	284530
300mm	771.7	m	410	316397
400mm	453.9	m	500	226950
Fire hydrants	20	-	4750	95000
			Total Cost	**$922,877.00**

Appendix C - Sanitary Sewers

Appendix C.1 – Sample Calculations

Appendix C..1.1: Sample Demand Calculations for Maintenance Hole 1.A to 2.A

The area tributary to the pipe connecting M.H.1A and M.H.2A was determined by creating a polygon for each zone in the AutoCAD file, which was then converted to units of hectares.

$$Area \ = \ 12019 \, m^2$$

$$Area \ = \ 12019 \, m^2 \, x \, \frac{ha}{10000 \, m^2} = \ 1.2019 \, ha$$

The number of houses and parks in each zone were visually counted.

$$\# \, Residential \ Units \ = \ 7$$

$$\# \, of \ Parks \ = \ 0$$

To determine the population of each zone, the population density of 4 ppu was multiplied by the number of houses found in that zone. This density was used due to the assumption that all units were single family dwellings.

$$Population \ = \ (4 \, ppu) \times (7 \, houses) \ = \ 28 \, people$$

To determine the total population tributary to each pipe, the populations of every upstream section serviced were added to the population in the drainage area of the pipe being analysed. For M.H.1A to M.H.2A, there are no upstream sections. Since the population in the drainage area of this pipe itself is 28, the total population can be calculated.

$$Total \ Population = \ 0 + 28 = 28 \, people$$

A similar approach was taken to find the total area[1].

$$Total \ Area \ = \ 1.201 \, ha$$

To calculate the residential daily demand, the total population of the zone was multiplied by the residential daily demand.

$$Residential \ Daily \ Demand \ = \frac{population \times residential \ daily \ demand}{24 \times 3600}$$

[1] Alternatively, to find the total population serviced by the pipe spanning M.H.2A to M.H.3A, upstream sections need to be considered. In this case, the only upstream section is M.H.1A to M.H.2A. Thus, the total population is 28 (from the upstream section) in addition to the 56 people in the drainage area of M.H.2A to M.H.3A.

$$Residential\ Daily\ Demand\ = \frac{28\ c\ \times\ 450\ L/c/d}{24\ hr\ \times 3600\ s} = 0.15\ L/s$$

The next step is to calculate the commercial daily demand, which can be found by multiplying the park area by the daily sewage generation rate. The average daily sewage generation rate of commercial areas was determined to be 0.29 $L/s/ha$. The drainage area of M.H.1A to M.H.2A contained no parks.

$$Park\ Area\ =\ 0\ ha$$

$$Commercial\ Daily\ Demand\ =\ (Park\ Area)\ \times\ \left(\frac{0.29\frac{L}{s}}{ha}\right)$$

$$Commercial\ Daily\ Demand\ = 0 \times 0.29 = 0\ L/s$$

To determine the infiltration in the drainage area, the total area was multiplied by the infiltration allowance of 20,000 L/d/ha (0.23 L/s/ha).

$$Infiltration =\ (Total\ Area)\ \times\ (0.23\ L/s/ha)$$

$$Infiltration =\ (1.202\ ha) \times\ (0.23\ L/s/ha) =\ 0.276\frac{L}{s}$$

The next step is to calculate the peaking factor using Harmon formula. Population, p, is expressed in thousands.

$$K, Peaking\ Factor =\ 1 + \frac{14}{4 + \sqrt{p}}$$

$$K, Peaking\ Factor =\ 1 + \frac{14}{4 + \sqrt{\frac{28}{1000}}} =\ 4.35$$

Given that the maximum peaking factor is 4, this is the value taken.

$$Peaking\ Factor =\ \ 4.0$$

Finally, the design flow can be calculating using the following formula:

$$Q = K \times \left(Q_{avg.residential} + Q_{avg.commercial}\right) + IA$$

$$Q_{design} =\ (Peak\ factor) \times\ (Residential\ Daily\ Demand + Park\ Demand) + Infiltration$$

$$Q_{design} =\ (4) \times (0.15\ L/s + 0) + 0.276\frac{L}{s} = 0.86\frac{L}{s}$$

As outlined in the Town of Adamsville Engineering Design Criteria and Standards Manual and as described by the client Angela Mokin, the minimum and maximum design flows are taken to be the same.

Appendix C.1.2: Sample Minimum Velocity Calculations for Maintenance Hole 1 to 2

The first step in calculating the minimum velocity was to find the hydraulic capacity of our sewers based on Manning's formula for full flowing pipes. The length of the pipe was determined via AutoCAD and was found to be 71.2m. Manning's roughness coefficient was taken to be 0.013 and the slope taken to be 1% - which translated to a fraction 0.01.

$$Pipe\ Length = 71.2\ m$$

$$Pipe\ Dimater = 250\ mm$$

$$Actual\ Diameter = 254\ mm$$

$$Q_{full} = \frac{\pi}{n} \times \frac{D^{\frac{8}{3}}}{4^{\frac{5}{3}}} \times S^{\frac{1}{2}}$$

$$Q_{full} = \frac{\pi}{0.013} \times \frac{0.254^{\frac{8}{3}}}{4^{\frac{5}{3}}} \times 0.01^{\frac{1}{2}} = 0.062038 \frac{m^3}{s}$$

$$Q_{full} = 62.038 \frac{L}{s}$$

To determine the velocity at full capacity, the equation below was followed. Q

$$V_{full} = \frac{Q_{full}}{\frac{\pi\ x\ D^2}{4}}$$

$$V_{full} = \frac{0.0623038}{\frac{\pi\ x\ 0.254^2}{4}} = 1.2244 \frac{m}{s}$$

From M.H.1A to M.H.2A, the velocity of a full flowing pipe was found to be 1.22m/s

To get the actual velocity flowing in the pipe, the three ratios of $\frac{q}{Q}, \frac{d}{D}, \frac{v}{V}$ needed to be found. In this case, the ratio of $\frac{q}{Q}$ was 0.3469 m/s. This ratio was then used to find the remaining three ratios using the hydraulic elements chart which was digitalized into a polynomial function.

$$\frac{q}{Q} = \frac{0.86 \frac{L}{s}}{62.0388 \frac{L}{s}} = 0.0139$$

$$\frac{d}{D} = 0.0854$$

$$\frac{v}{V} = 0.2833$$

The actual velocity was calculated by multiplying the $\frac{v}{V}$ ratio by the full velocity in the pipe

$$v_{min} = \frac{v}{V}\ x\ V_{full} = 0.2833\ x\ 1.244\frac{m}{s} = 0.3469\frac{m}{s}$$

Appendix C.1.3: Sample Calculations to Determine Flow Condition in Maintenance Hole 1 to 2

To determine the flow conditions in the sewer between M.H. 1A and 2A, we must compare the $\frac{\boldsymbol{y_{critical}}}{\boldsymbol{D}}$ and $\frac{d}{D}$ ratios. The $\frac{d}{D}$ ratio was calculated previously and was found to be 0.0854. To calculate $\frac{y_{critical}}{D}$, we can use the formula for critical depth, seen below. In this equation, D represents the actual diameter of the sewer (mm) and Q_{full} (m/s).

$$y_{critical} = 0.483\ \times \left(\frac{Q_{design}}{D}\right)^{\frac{2}{3}} + 0.083D$$

$$\frac{y_{critical}}{D} = \frac{0.483\ \times \left(\frac{Q_{design}}{D}\right)^{\frac{2}{3}} + 0.083D}{D}$$

$$\frac{y_{critical}}{D} = \frac{(0.483\ \times \left(\left(\frac{0.0008598}{0.254}\right)^{\frac{2}{3}}\right) + (0.0008599\ x\ 0.254\ ))}{0.254} = 0.0830$$

Since $\frac{y_{critical}}{D}$ is smaller than $\frac{d}{D}$, the flow is subcritical.

Appendix C.1.4: Sample Invert Calculations for Maintenance Hole 1 to 2

To calculate the invert elevation of M.H.1A, we first need to find the ground elevation, which was 207.93m. Assuming a minimum clear cover of 3m and an actual diameter of 0.254m:

$$From\ M.H\ Invert\ elevation = Ground\ elevation - 3m - diameter$$

$$M.H1A\ Invert\ Elevation = 207.93\ m - 3\ m - 0.254\ m = 204.68\ m$$

To find the invert elevation of M.H.2A, we first need to multiply the slope fraction by the pipe length to find the change in vertical elevation. Then, then we can subtract this change in vertical elevation by the invert elevation of M.H.1A (the upstream pipe).

$$To\ M.H\ \ Invert\ Elevation = From\ M.H\ Invert\ Elevation - (Slope\ fraction)\ \times (Length\ of\ pipe)$$

$$M.H.2A\ Invert\ elevation = 204.68\ m - 0.01\ \times\ 71.2\ m = 203.96\ m$$

To find the depth of cover of M.H.1A, we can use the ground elevation at the point and then subtract from that both the invert election and the pipe diameter.

$$Cover\ above\ Crown = Ground\ Elevation - (Invert\ Elevation + Diameter)$$

$$M.H.1A\ Cover\ above\ Crown = 207.93\ m - (204.67\ m + 0.254\ m) = 3\ m$$

The next step in the process is to calculate the Invert Elevation of the outlet sewer of M.H.2A. In Table 8, this can be seen in the second row (under 'From M.H').

First, we must determine the angle change, which was found to be a 90° bend. According to the design criteria, a 90° bend carries a minimum drop across maintenance hole of 0.09 m.

$$Angle\ change\ in\ MH = 90\ degrees$$

$$Minimum\ downstream\ drop = 0.09\ m$$

Thus, the invert elevation of the sewer leaving M.H.2A is found by subtracting the minimum drop from the invert elevation of the sewer entering M.H.2A (found in the previous row).

$$M.H.2A\ Invert\ elevation = 203.96\ m - 0.09m = 203.87\ m$$

A different scenario occurs when there are multiple inlets – which is described in Appendix B.5.

Appendix C.1.4 Sample Invert Calculations for Multiple Inlets

Maintenance hole 6A has two inlet pipes spanning from M.H 5A and 14A respectively. Table C.1.4 shows the calculations used to determine what elevation the outlet at M.H 6 should be at to maintain minimum drop requirements.

Table C.1.4. Calculations for Outlet Elevation at M.H.6A

Pipe Segment (M.H. to M.H)	Length of Pipe Segment (m)	Upstream Invert Elevation (m)	Slope	Downstream Invert Elevation (without any drops)	Bend	Drop required (m)	Elevation of Outlet Invert Required
5 – 6	55.7	201.64	0.01	$201.64 - 0.01 \times 55.7 = 201.081$	0	0.03	$201.081 - 0.03$ = 201.051
14 – 6	88.3	200.996	0.01	$200.996 - 0.01 \times 88.3 = 200.113$	90	0.09	$200.113 - 0.09$ = 200.023

From Table 12, the 200.023 m elevation was picked as the outlet invert elevation to ensure that it is at a sufficient depth below the inlet pipe from M.H14A to 6A.

The incoming downstream invert elevation of pipe from M.H.5A – 6 is 201.081 m as seen in Table 12. This leads to the following maximum inlet and outlet elevation difference at M.H.6A:

$$Maximum\ Invert\ Elevation\ Difference\ (m) = 201.081 - 200.023 = 1.058$$

This inlet and outlet drop difference exceeds the maximum of 0.6m. A drop structure of 1 m was added to reduce this difference from 1.058 to 0.058m. This ensures a minimum drop of 0.03 as required for straight – run pipes while staying below the maximum drop of 0.6 m.

Appendix C.2 – Tables of Data

Appendix C.2.1 Maximum sewage production flows per tributary area

From MH	To MH	Design Flow (L/s)
1A	2A	0.86
2A	3A	2.19
3A	4A	2.97
4A	5A	4.19
5A	6A	4.76
6A	7A	10.44
7A	8A	11.11
9A	10A	0.40
10A	11A	1.00
11A	12A	1.50
12A	13A	2.81
13A	14A	4.05
14A	6A	5.10
15A	17A	0.57
18A	17A	0.24
17A	19A	1.20
19A	21A	1.86
20A	21A	0.49
21A	22A	3.57
22A	23A	4.42
23A	24A	4.91
25A	26A	0.31
26A	27A	0.70
27A	28A	1.48
28A	29A	2.17
29A	30A	2.39
30A	8A	2.98
8A	24A	13.96
24A	Outflow	19.62

Appendix C.2.2 Length, diameter, slope, and velocity of each pipe

From MH	To MH	Length (m)	Diameter (mm)	Slope (%)	Velocity (m/s)
1A	2A	71.2	250	1.00	0.3469
2A	3A	90.8	250	1.00	0.4795
3A	4A	58.2	250	1.00	0.5340
4A	5A	65.6	250	1.00	0.5990
5A	6A	55.7	250	1.00	0.6238
6A	7A	45.5	250	0.92	0.7531
7A	8A	49.6	250	0.90	0.7584
9A	10A	70.7	250	1.00	0.2815
10A	11A	86.5	250	1.00	0.3641
11A	12A	45.5	250	1.00	0.4193
12A	13A	69.4	250	1.00	0.5240
13A	14A	80.9	250	1.00	0.5927
14A	6A	88.3	250	1.00	0.6374
15A	17A	14.9	250	1.00	0.3067
18A	17A	33.0	250	1.00	0.2559
17A	19A	48.2	250	1.30	0.4223
19A	21A	42.8	250	1.38	0.5017
20A	21A	57.6	250	1.00	0.2956
21A	22A	75.3	250	1.00	0.5686
22A	23A	53.3	250	1.00	0.6097
23A	24A	55.5	250	1.00	0.6301
25A	26A	48.9	250	1.00	0.2680
26A	27A	49.6	250	1.00	0.3260
27A	28A	87.5	250	1.00	0.4165
28A	29A	99.1	250	1.00	0.4785

Appendix C.2.3 Flow Conditions

From MH	To MH	dDdD	ycriticalDycriticalD	Super or Sub Critical
1A	2A	0.0854	0.083	Sub
2A	3A	0.1389	0.120	Sub
3A	4A	0.1661	0.138	Sub
4A	5A	0.2030	0.163	Sub
5A	6A	0.2183	0.174	Sub
6A	7A	0.3319	0.267	Sub
7A	8A	0.3429	0.276	Sub
9A	10A	0.0643	0.066	Super
10A	11A	0.0915	0.087	Sub
11A	12A	0.1126	0.102	Sub
12A	13A	0.1609	0.135	Sub
13A	14A	0.1992	0.161	Sub
14A	6A	0.2270	0.181	Sub
15A	17A	0.0721	0.072	Super
18A	17A	0.0568	0.058	Super
17A	19A	0.0938	0.094	Sub
19A	21A	0.1158	0.112	Sub
20A	21A	0.0686	0.070	Super
21A	22A	0.1851	0.151	Sub
22A	23A	0.2095	0.168	Sub
23A	24A	0.2223	0.177	Sub
25A	26A	0.0603	0.062	Super
26A	27A	0.0784	0.078	Sub
27A	28A	0.1115	0.102	Sub
28A	29A	0.1384	0.120	Sub
29A	30A	0.1462	0.125	Sub
30A	8A	0.1664	0.138	Sub
8A	24A	0.3960	0.315	Sub
24A	Outflow	0.4360	0.385	Sub

<u>Appendix C.2.4 Maintenance Hole Elevations, invert Elevations and Cover to Crown</u>

From Maintenance Hole	To Maintenance Hole	Maintenance Hole Elevation (m)	Invert Elevation (m)		Depth of Cover above Crown of From MH (m)
			From MH	To MH	
1A	2A	207.93	204.68	203.96	3.00
2A	3A	207.80	203.87	202.97	3.67
3A	4A	207.70	202.94	202.35	4.51
4A	5A	206.50	202.32	201.67	3.92
5A	6A	205.50	201.64	200.08	3.61
6A	7A	205.25	200.02	199.60	4.97
7A	8A	205.00	199.57	198.13	5.17
9A	10A	208.05	204.80	204.09	3.00
10A	11A	207.90	204.00	203.13	3.65
11A	12A	207.05	203.10	202.65	3.69
12A	13A	206.60	202.62	201.93	3.73
13A	14A	206.20	201.90	201.09	4.05
14A	6A	205.56	201.00	200.11	4.31
15A	17A	207.60	204.50	203.85	2.85
18A	17A	207.21	203.96	203.63	3.00
17A	19A	207.39	203.54	202.91	3.59
19A	21A	206.38	202.88	202.29	3.24
20A	21A	205.90	204.15	202.87	1.50
21A	22A	205.55	202.26	201.51	3.04
22A	23A	204.77	201.48	200.94	3.04
23A	24A	204.50	200.91	197.16	3.33
25A	26A	205.97	202.71	202.22	3.00
26A	27A	205.71	202.19	201.70	3.26
27A	28A	205.40	201.67	200.79	3.48
28A	29A	204.87	200.76	199.77	3.85
29A	30A	204.42	199.68	198.95	4.49
30A	8A	204.43	198.92	197.90	5.26
8A	24A	204.97	197.87	197.25	6.84
24A	Outflow	204.50	197.00	196.01	7.25

Appendix C.2.5 Inlet Drops

From MH	To MH	To MH Bend	Minimum Drop (m)	Downstream Inlet Drop (m)
1A	2A	90º	0.09	0.09
2A	3A	Straight-Run	0.03	0.03
3A	4A	Straight-Run	0.03	0.03
4A	5A	Straight-Run	0.03	0.03
5A	6A	Straight-Run	0.03	0.06
6A	7A	Straight-Run	0.03	0.03
7A	8A	90º	0.09	0.26
9A	10A	90º	0.09	0.09
10A	11A	Straight-Run	0.03	0.03
11A	12A	Straight-Run	0.03	0.03
12A	13A	Straight-Run	0.03	0.03
13A	14A	90º	0.09	0.09
14A	6A	90º	0.09	0.09
15A	17A	45º	0.06	0.31
18A	17A	90º	0.09	0.09
17A	19A	Straight-Run	0.03	0.03
19A	21A	Straight-Run	0.03	0.03
20A	21A	90º	0.09	0.03
21A	22A	Straight-Run	0.03	0.03
22A	23A	Straight-Run	0.03	0.03
23A	24A	90º	0.09	0.16
25A	26A	Straight-Run	0.03	0.03
26A	27A	Straight-Run	0.03	0.03
27A	28A	Straight-Run	0.03	0.03
28A	29A	90º	0.09	0.09
29A	30A	Straight-Run	0.03	0.03
30A	8A	Straight-Run	0.03	0.03
8A	24A	Straight-Run	0.03	0.03
24A	Outflow	Straight-Run	0.03	0.03

Appendix D - Storm Sewers

Appendix D.1– Sample Calculations

Appendix D.1.1: Sample Area Calculations for Maintenance Hole 1 to 2

The area tributary to the pipe connecting M.H.1 and M.H.2 was determined by creating a polygon for each zone in the AutoCAD file, which was then converted to units of hectares.

$$Area\ =\ 5504\,m^2$$

$$Area\ =\ 5504\,m^2\ x\ \frac{ha}{10000\,m^2} =\ 0.5504\,ha$$

The $C_{effective}$ value was determined by examining the tributary area on AutoCAD and determining how much percentage is taken by roads and with that the coverage of single homes was known. In this case 80% of the area was dedicated to the homes and the rest of 20% was for roads. With these values they were multiplied by their respective runoff coefficients and added together.

$$C_{eff} = 0.8\ \times\ (C\ of\ single\ home > 18\,m\,frontage) + 0.2\ \times (C\ of\ arterial\ roadways)$$

$$C_{eff} = (0.8\ \times\ 0.4) + (0.2\ \times\ 0.65) = 0.45$$

$$Area\ \times\ C_{eff} = 0.5504\ \times\ 0.45 = 2476.8\,ha$$

The cumulative AC_{eff} of Maintenance Hole 1 to 2 was found by adding the cumulative AC_{eff} of the upstream sections (in this case M.H.0 to M.H.1) and the AC_{avg} of the area itself (M.H.1 to M.H.2)

$$Cumm.\ AC_{eff}\ \ of\ M.H\ 1\ to\ M.H\ 2 =\ \left(AC_{eff}\ \ of\ M.H.1\ to\ M.H.2\right) + (Upstream\ \ AC_{eff}\ )$$

$$Cumm.\ AC_{eff}\ \ of\ M.H\ 1\ to\ M.H\ 2 = 2476.8\ ha + 5340.05\ ha$$

$$= 7983.33\ ha$$

Appendix D.1.2: Intensity and Time of Concentration Calculations for Maintenance Hole 1 to 2

To find the time of concentration of M.H.1 and 2, we must consider two scenarios. The first is the T_c of the upstream watersheds. In this case, the T_c of M.H.0 to M.H.1 (the section directly upstream of M.H.1 to M.H.2) is 10.66 minutes. We must also consider the time it takes for water to travel through the pipe connecting these two upstream M.H, in this case 1.0516 minutes (length of pipe divided by velocity of flow in pipe)

The second scenario we must consider is the T_c of the watershed itself. In this case, all beginning areas have a T_c of 10 minutes. Now that we have considered both scenarios, we must choose the largest of the two.

$$T_c = \max\{10, T_c \; of \; MH\,0 \; to \; M.H\,1 + \; Time \; in \; Pipe, MH.0 \; to \; MH.1\}$$

$$T_c = \max\{10, 10.66 + 1.0516\} = 11.71 \, min$$

The rainfall intensity was determined through the equation found below where the parameters were obtained from the IDF curve. The values of A, B, and C were obtained from the IDF equation constants provide for a return period of 5-yr since the minor systems was designed for a storm with a return period of 5.

$$Intensity = \frac{A}{(T_c + B)^C}$$

$$Intensity = \frac{1105.505}{(11.71 \, min + 8.57)^{0.829}} = 91.21 \frac{mm}{hr}$$

Appendix D.1.3: Sample Flow and Velocity Calculations for Maintenance Hole 1 to 2

The first step in calculating the minimum velocity was to find the hydraulic capacity of our sewers based on Manning's formula for full flowing pipes. Manning's roughness coefficient was taken to be 0.013 and the slope taken to be 0.3% - which translated to a fraction 0.003. The diameter of the pipe was taken to be 450 mm.

$$Slope = 0.003$$

$$Nominal \; Pipe \; Dimater = 450 \, mm$$

$$Actual \; Pipe \; Dimater = 454 \, mm$$

$$Manning \; Number, n = 0.013$$

$$Nominal \; Capacity, Q_{full} = \frac{\pi}{n} \times \frac{D^{\frac{8}{3}}}{4^{\frac{5}{3}}} \times S^{\frac{1}{2}}$$

$$Nominal \; Capacity, Q_{full} = \frac{\pi}{0.013} \times \frac{(0.457)^{\frac{8}{3}}}{4^{\frac{5}{3}}} \times (0.003)^{\frac{1}{2}} = 162.72 \frac{L}{s}$$

Once the nominal capacity is found, the velocity in a full flowing pipe can be calculated by dividing it by the area.

$$V_{full} = \frac{Q_{full}}{\frac{\pi \, x \, D^2}{4}}$$

$$V_{full} = \frac{\left(\frac{162.72\frac{L}{s}}{1000}\right)}{\frac{\pi \ x \ 0.454^2}{4}} = \ 0.99\frac{m}{s}$$

To get the actual velocity flowing in the pipe, the three ratios of $\frac{q}{Q}, \frac{d}{D}, \frac{v}{V}$ needed to be found. In this case, q represents the actual flow in our pipes, Q_{peak}.

$$Q_{peak} = \ 2.778 \ \times Cumulative \ Area \ \times Intensity$$

$$Q_{peak} = 2.778 \ \times 0.534 \ ha \ \times \ 91.21\frac{mm}{hr}$$

$$Q_{peak} = \ 135.30\frac{L}{s}$$

$$\frac{q}{Q} = \frac{Q \ peak}{Q \ full} = \frac{135.30\frac{L}{s}}{162.72\frac{L}{s}} = \ 0.83$$

In this case, the ratio of $\frac{q}{Q}$ was 0.83 m/s. This ratio was then used to find the remaining three ratios using the hydraulic elements chart which was digitalized into a polynomial function.

$$\frac{d}{D} = 0.77$$

$$\frac{v}{V} = 0.99$$

The ratio $\frac{v}{V}$ can then be used to calculate the actual velocity in the pipe

$$v = V_{full} \times \frac{v}{V} = 0.99 \times 0.99 = 0.99 \ m/s$$

Appendix D.1.4: Sample Time in Section Calculations for Maintenance Hole 1 to 2

To find the time in section, we must divide the length of the pipe by the speed of the water flowing in the pipe (spanning from M.H.1 to M.H.2)

$$Time \ in \ section = \frac{length \ of \ pipe}{actual \ speed}$$

$$Time \ in \ section = \frac{79.2 \ m}{0.99 \ m/s} \times \frac{1 \ min}{60s} = \ 1.338 \ min$$

Appendix D.2- Data Tables

Appendix D.2.1: Invert Elevation, Ground Elevation and Depth of Cover above Crown

From MH	To MH	Invert Elev (m)		Ground Elev (m)		Depth of Cover above Crown (m)	
		Upper End	Lower End	Upper End	Lower End	Upper End	Lower End
Rear A	0	202.996	202.698	206.25	207	3	4.048
Rear B	0	202.996	202.698	206.25	207	3	4.048
0	1	202.571	202.3334	207	207.9	4.048	5.1856
1	2	202.2574	202.0198	207.9	207.97	5.1856	5.4932
2	3	201.9438	201.687	207.97	207.73	5.4932	5.51
3	4	201.657	201.4854	207.73	206.45	5.54	4.4316
4	5	201.4084	201.146	206.45	205.58	4.4316	3.824
5	6	201.116	200.882	205.58	205.26	3.854	3.768
6	7	200.5012	200.3572	205.26	205.09	3.9208	3.8948
7	8	200.3072	200.0684	205.09	204.98	3.9448	4.0736
9	10	204.919	204.7326	208.3	207.888	3	2.7744
10	11	204.6576	204.5644	207.888	207.6	2.8494	2.6546
11	12	204.4884	203.7928	207.6	207.01	2.6546	2.7602
12	13	203.4866	203.3642	207.01	206.7	2.9904	2.8028
Rear M	13	203.236	202.828	206.49	206.7	3	3.618
13	14	202.549	202.4929	206.7	206.55	3.618	3.5241
Rear E	14	202.766	202.366	206.02	206.55	3	3.93
14	15	202.087	202.0582	206.55	206.5	3.93	3.9088
Rear F	15	202.546	202.142	205.8	206.8	3	4.404
15	16	201.863	201.7706	206.5	206.27	4.104	3.9664
Rear G	16	202.446	202.041	205.7	206.27	3	3.975
16	17	201.685	201.666	206.27	206.2	3.975	3.924
Rear H	17	202.226	201.823	205.48	206.2	3	4.123
17	18	201.467	201.405	206.2	206.01	4.123	3.995
Rear I	18	202.096	201.693	205.35	206.01	3	4.063
18	19	201.337	201.163	206.01	205.55	4.063	3.777
Rea K	19	202.096	201.719	205.35	205.55	3	3.577
19	6	201.088	200.7292	205.55	205.26	3.852	3.9208
20	12	203.895	203.7146	207.2	207.01	3	2.9904
Rear C	21	202.996	202.018	205.25	206.23	2	3.958
Rear D	21	203.246	202.754	205.5	206.23	2	3.222
21	22	201.891	201.8386	206.23	205.9	3.958	3.6804
22	23	201.7636	201.6024	205.9	205.65	3.7554	3.6666
23	24	201.5264	201.2936	205.65	205.17	3.6666	3.4194
24	25	201.2176	200.9107	205.17	204.45	3.4194	3.0063
25	26	200.8607	200.7086	204.45	204.35	3.0563	3.1084
26	40	200.6336	200.5118	204.35	204.3	3.1834	3.2552
Rear J	40	201.746	201.239	205	204.3	3	2.807
40	27	200.4618	200.2778	204.3	204.53	3.3052	3.7192

Rear L	27	202.096	201.664	205.35	204.53	3	2.612
27	8	200.2008	199.941	204.53	204.98	3.7192	4.429
28	31	203.619	203.3014	207	207.37	3	3.6876
29	31	204.219	204.1638	207.6	207.37	3	2.8252
30	31	204.319	204.0498	207.7	207.37	3	2.9392
31	32	203.2254	203.0946	207.37	206.3	3.6876	2.7484
32	33	202.7446	202.6	206.3	205.45	3.0984	2.393
33	34	202.3632	202.0876	205.45	204.78	2.5538	2.1594
34	35	202.0106	201.8525	204.78	204.53	2.1594	2.0675
35	36	201.8025	201.6264	204.53	204.5	2.1175	2.2636
37	39	203.5	203.4284	206.3	205.51	2.419	1.7006
38	39	203.7	203.6264	206.4	205.51	2.319	1.5026
39	33	203.3784	202.5152	205.51	205.45	1.7506	2.5538
8	36	199.637	199.3158	204.98	204.5	4.429	4.2702
36	EXIT	199		204.5		4.433	

Appendix D.3– Hydraulic Drop Computations for Largest Diameter M.H

Maintenance hole 36 has the largest outlet flow pipe in the system. It is the only maintenance hole with an inlet/outlet pipe of 1050 mm or greater. The process to determine the invert elevation for the outlet pipe is shown below.

Table D.3.1summarizes the properties and elevations of the two inlet pipes of MH.36.

Table D.3.1. Properties of MH.36 Inlet Pipes

From MH	To MH	Pipe Diameter (m)	Inlet Angle to Outlet Pipe	Invert Drop Required (mm)	Inlet Invert Elevation	Inlet Obvert Elevation
8	36	0.914	0	30	199.3158	200.23
35	36	0.610	90	75	201.6264	202.236

Outlet pipe (MH.36 – exit) has a diameter of 1.067 m.

Outlet invert elevation due to minimum drop applied to invert:

$$Inlet\ Invert\ Elevation - Drop\ Required = \ Outlet\ Invert\ Elevation$$

Outlet invert elevation due to aligning the inlet and outlet obverts:

$$Inlet\ Invert\ Elevation - (Outlet\ Diameter - Inlet\ Diameter) \\ = \ Outlet\ Invert\ Elevation$$

Pipe spanning MH.8 – MH.36

Outlet invert elevation due to minimum invert drop: $199.3158 - 0.03 = \mathbf{199.286\ m}$

Outlet invert elevation due to matching crowns = $199.3158 - (1.067 - 0.914) = \mathbf{199.163\ m}$

<u>Pipe spanning MH.35 – MH.36</u>

Outlet invert elevation due to minimum invert drop: $201.6264 - 0.075 = \mathbf{201.55\ m}$

Outlet invert elevation due to matching crowns = $201.6264 - (1.067 - 0.610) = \mathbf{201.169\ m}$

The lowest invert elevation of these four values was taken as the invert elevation of the outlet pipe to ensure that the outlet obvert was either aligned with or below all inlet obverts and minimum invert drop requirements were met.

This method was used to determine all outlet pipe invert elevations at all maintenance holes in the system.

The inlet elevation for MH.36 is 199 m. If this value was not given, the elevation would be taken as 199.163 m which is the smallest of the four calculated values.

Actual Invert Drops = $Inlet\ Invert\ Elevation - (Outlet\ Inlet\ Elevation) = \ Invert\ Drop$

Actual Invert drop from Pipe MH.8 – MH.36:

$$199.3158 - 199 = \ \mathbf{0.3158\ m}$$

Actual Invert drop from Pipe MH.35 – MH.36:

$$201.6264 - 199 = \ \mathbf{2.66\ m}$$

Appendix E - Design Sheets

Appendix E.1 - Sanitary Design Sheet

Appendix E.2 - Storm Design Sheet

SANITARY SEWER DESIGN SHEET

SUBDIVISION/PROJECT NAME ADAMSVILLE VILLAGE

TOWN OF ADAMSVILLE

Min. Sewer Diameter	=	200	mm
Manning's "n"	=	0.013	
Min. Velocity	=	0.75	m/s
Max. Velocity	=	3	m/s
Min. K. Peaking Factor	=	2	
Max. K. Peaking Factor	=	4	

Harmon Peaking Factor $K = 1 + \frac{14}{4 + p^{0.5}}$

Design Flow $Q = K \times (Q_{avg,residential} + Q_{avg,commercial}) + IA$

LOCATION			SECTION		CUMULATIVE		K	CUMM. AVG. RESIDENTIAL (L/s)	CUMM. AVG. PARK (L/s)	CUMM. INFILTRATION (L/s)	CUMM. DESIGN FLOW (Qd) (L/s)	ELEVATION			PIPE DATA		FULL FLOW (Qf) (L/s)	FULL VEL. (m/s)	MIN VELOCITY (m/s)	MAX VELOCITY (m/s)	SUPER OR SUB CRITICAL	% FULL (Qd/Qf)
STREET	MANHOLE FROM	MANHOLE TO	POP. (no)	AREA (ha)	POP. (no)	AREA (ha)						MH FROM INVERT	M.H. TO SURFACE	M.H. TO INVERT	SIZE (mm)	SLOPE (%)						
Zuccaro Court	1	2	28	1.202	28	1.20	4.00	0.146	0.000	0.276	0.86	204.68	207.80	203.96	250	1.00	62.039	1.22	0.347	0.347	Sub	1.39
Mokin Street	2	3	56	0.696	84	1.90	4.00	0.438	0.000	0.437	2.19	203.87	207.70	202.97	250	1.00	62.039	1.22	0.480	0.480	Sub	3.52
Mokin Street	3	4	32	0.500	116	2.40	4.00	0.604	0.000	0.552	2.97	202.94	206.50	202.35	250	1.00	62.039	1.22	0.534	0.534	Sub	4.78
Mokin Street	4	5	52	0.584	168	2.98	4.00	0.875	0.000	0.686	4.19	202.32	205.50	201.67	250	1.00	62.039	1.22	0.599	0.599	Sub	6.75
Mokin Street	5	6	24	0.307	192	3.29	4.00	1.000	0.000	0.756	4.76	201.64	200.02	200.08	250	1.00	62.039	1.22	0.624	0.624	Sub	7.67
Mokin Street	6	7	28	0.361	420	7.35	4.00	2.188	0.000	1.691	10.44	200.02	205.00	199.60	250	0.92	59.505	1.17	0.753	0.753	Sub	17.55
Mokin Street	7	8	28	0.386	448	7.74	4.00	2.333	0.000	1.780	11.11	199.57	204.97	198.13	250	0.90	58.855	1.16	0.758	0.758	Sub	18.88
Zuccaro Court	9	10	12	0.642	12	0.64	4.00	0.063	0.000	0.148	0.40	204.80	207.90	204.09	250	1.00	62.039	1.22	0.281	0.281	Super	0.64
Zuccaro Court	10	11	16	1.162	28	1.80	4.00	0.146	0.000	0.415	1.00	204.00	207.05	203.13	250	1.00	62.039	1.22	0.364	0.364	Sub	1.61
Selak Place	11	12	20	0.381	48	2.19	4.00	0.250	0.000	0.503	1.50	203.10	206.60	202.65	250	1.00	62.039	1.22	0.419	0.419	Sub	2.42
Selak Place	12	13	56	0.619	104	2.80	4.00	0.542	0.000	0.645	2.81	202.62	206.20	201.93	250	1.00	62.039	1.22	0.524	0.524	Sub	4.53
Selak Place	13	14	52	0.678	156	3.48	4.00	0.813	0.000	0.801	4.05	201.90	205.56	201.09	250	1.00	62.039	1.22	0.593	0.593	Sub	6.53
Selak Place	14	6	44	0.583	200	4.07	4.00	1.042	0.000	0.935	5.10	201.00	205.25	200.11	250	1.00	62.039	1.22	0.637	0.637	Sub	8.22
Fabian Papa Way	15	17	0	0.439	0	0.44	4.00	0.000	0.116	0.101	0.57	204.50	207.39	203.85	250	1.00	62.039	1.22	0.307	0.307	Super	0.91
Fabian Papa Way	18	17	8	0.321	8	0.32	4.00	0.042	0.000	0.074	0.24	203.96	207.39	203.63	250	1.00	62.039	1.22	0.256	0.256	Super	0.39
Karney Street	17	19	16	0.260	24	1.02	4.00	0.125	0.116	0.235	1.20	203.54	206.38	202.91	250	1.30	70.735	1.40	0.422	0.422	Sub	1.69
Karney Street	19	21	28	0.336	52	1.36	4.00	0.271	0.116	0.312	1.86	202.88	205.55	202.29	250	1.38	72.879	1.44	0.502	0.502	Sub	2.55
Nault Drive	20	21	0	0.464	0	0.46	4.00	0.000	0.096	0.107	0.49	204.15	205.55	202.87	250	1.00	62.039	1.22	0.296	0.296	Super	0.79
Karney Street	21	22	52	0.600	104	2.42	4.00	0.542	0.212	0.557	3.57	202.26	204.77	201.51	250	1.00	62.039	1.22	0.569	0.569	Sub	5.75
Karney Street	22	23	36	0.447	140	2.87	4.00	0.729	0.212	0.660	4.42	201.48	204.50	200.94	250	1.00	62.039	1.22	0.610	0.610	Sub	7.13
Karney Street	23	24	20	0.326	160	3.19	3.99	0.833	0.212	0.735	4.91	200.91	204.50	197.16	250	1.00	62.039	1.22	0.630	0.630	Sub	7.92
Meyer Street	25	26	12	0.276	12	0.28	4.00	0.063	0.000	0.064	0.31	202.71	205.71	202.22	250	1.00	62.039	1.22	0.268	0.268	Super	0.51
Meyer Street	26	27	16	0.240	28	0.52	4.00	0.146	0.000	0.119	0.70	202.19	205.40	201.70	250	1.00	62.039	1.22	0.326	0.326	Sub	1.13
Meyer Street	27	28	32	0.463	60	0.98	4.00	0.313	0.000	0.225	1.48	201.67	204.87	200.79	250	1.00	62.039	1.22	0.417	0.417	Sub	2.38
Meyer Street	28	29	28	0.499	88	1.48	4.00	0.458	0.000	0.340	2.17	200.76	199.68	199.77	250	1.00	62.039	1.22	0.478	0.478	Sub	3.50
Meyer Street	29	30	8	0.211	96	1.69	4.00	0.500	0.000	0.389	2.39	199.68	204.43	198.95	250	1.00	62.039	1.22	0.495	0.495	Sub	3.85
Adams Avenue	30	8	24	0.391	120	2.08	4.00	0.625	0.000	0.479	2.98	198.92	197.87	197.90	250	1.00	62.039	1.22	0.535	0.535	Sub	4.80
Adams Avenue	8	24	0	0.129	568	9.95	3.95	2.958	0.000	2.289	13.96	197.87	204.50	197.25	250	0.75	53.727	1.06	0.750	0.750	Sub	25.98
Adams Avenue	24	Outflow	12	0.906	740	14.05	3.75	3.854	0.368	3.232	19.62	197.00	-	-	250	1.00	62.039	1.22	0.914	0.914	Sub	31.62

STORM SEWER DESIGN SHEET

SUBDIVISION/PROJECT NAME ADAMSVILLE VILLAGE **TOWN OF ADAMSVILLE**

Min. Diameter Commercial	=	375	mm
Min. Diameter Residential	=	300	mm
Manning's "n"	=	0.013	
Max Velocity	=	4.5	m/s

Nominal Capacity $Q_{capacity} = \frac{1}{n} * A * (D/4)^{\frac{2}{3}} * S^{\frac{1}{2}}$

Design Flow $Q_{design} = 2.778 * C * I * A$

Street	Location		Section			Cummulative													
	Maintenance hole																		
	From	To	Area (ha)	Cavg	Area x C	Area x C	Tc (min)	Rainfall Intensity I (mm)	Qact (L/s)	Size of Pipe (mm)	Slope (%)	Mannings 'n'	Nominal Capacity Qcap (l/s)	Full Flow Velocity (m/s)	Actual Flow Velocity (m/s)	Length (m)	Time in Sect. (min)	Total Time (min)	Qact/Qcap
	RLCB A	0	0.10	0.40	0.04	0.04	10.00	98.11	11.16	254	1.00	0.013	62.04	1.22	0.79	29.80	0.629	10.63	0.18
	RLCB B	0	0.09	0.40	0.03	0.03	10.00	98.11	9.40	254	1.00	0.013	62.04	1.22	0.76	29.80	0.657	10.66	0.15
	0	1	0.50	0.43	0.21	0.29	10.66	95.33	75.82	381	0.40	0.013	115.68	1.01	0.94	59.40	1.052	11.71	0.66
Zuccaro Court	1	2	0.55	0.45	0.25	0.53	11.71	91.21	135.30	457	0.30	0.013	162.72	0.99	0.99	79.20	1.339	13.05	0.83
Mokin Street	2	3	0.56	0.47	0.26	0.80	13.05	86.50	191.84	533	0.30	0.013	245.25	1.10	1.07	85.60	1.331	14.38	0.78
Mokin Street	3	4	0.35	0.48	0.17	0.97	14.38	82.32	221.39	533	0.30	0.013	245.25	1.10	1.12	57.20	0.850	15.23	0.90
Mokin Street	4	5	0.50	0.47	0.24	1.20	15.23	79.87	267.34	610	0.40	0.013	405.83	1.39	1.29	65.60	0.847	16.08	0.66
Mokin Street	5	6	0.41	0.47	0.19	1.40	16.08	77.59	300.99	610	0.40	0.013	405.83	1.39	1.33	58.50	0.732	16.81	0.74
Mokin Street	6	7	0.22	0.48	0.10	3.14	16.81	75.73	659.70	838	0.40	0.013	946.50	1.72	1.62	36.00	0.371	17.18	0.70
Mokin Street	7	8	0.42	0.47	0.20	3.33	17.18	74.83	692.43	838	0.40	0.013	946.50	1.72	1.64	59.70	0.606	17.78	0.73
Zuccaro Court	9	10	0.59	0.44	0.26	0.26	10.00	98.11	70.57	381	0.40	0.013	115.68	1.01	0.92	46.60	0.841	10.84	0.61
Zuccaro Court	10	11	0.30	0.41	0.12	0.38	10.84	94.57	100.28	381	0.40	0.013	115.68	1.01	1.02	23.30	0.380	11.22	0.87
Zuccaro Court	11	12	0.40	0.41	0.17	0.55	11.22	93.06	141.53	457	0.30	0.013	162.72	0.99	1.00	65.20	1.086	12.31	0.87
Selak Place	12	13	0.10	0.49	0.05	0.75	12.31	89.03	184.83	533	0.40	0.013	283.19	1.27	1.18	30.60	0.434	12.74	0.65
	RLCB M	13	0.04	0.45	0.02	0.02	10.00	98.11	5.10	254	1.00	0.013	62.04	1.22	0.64	40.80	1.067	11.07	0.08
Selak Place	13	14	0.13	0.48	0.06	0.83	12.74	87.53	201.78	533	0.30	0.013	245.25	1.10	1.09	18.70	0.286	13.03	0.82
	RLCB E	14	0.08	0.45	0.04	0.04	10.00	98.11	9.66	254	1.00	0.013	62.04	1.22	0.76	40.00	0.875	10.87	0.16
Selak Place	14	15	0.06	0.46	0.03	0.89	13.03	86.57	215.17	533	0.30	0.013	245.25	1.10	1.11	9.60	0.144	13.17	0.88
	RLCB F	15	0.07	0.45	0.03	0.03	10.00	98.11	8.50	254	1.00	0.013	62.04	1.22	0.74	40.40	0.914	10.91	0.14
Selak Place	15	16	0.19	0.47	0.09	1.02	13.17	86.09	242.85	533	0.30	0.013	245.25	1.10	1.15	30.80	0.448	13.62	0.99
	RLCB G	16	0.07	0.45	0.03	0.03	10.00	98.11	8.77	254	1.00	0.013	62.04	1.22	0.74	40.50	0.908	10.91	0.14
Selak Place	16	17	0.06	0.46	0.03	1.08	13.62	84.65	253.31	610	0.20	0.013	286.97	0.98	1.00	9.50	0.159	13.78	0.88
	RLCB H	17	0.08	0.45	0.04	0.04	10.00	98.11	10.09	254	1.00	0.013	62.04	1.22	0.77	40.30	0.872	10.87	0.16
Selak Place	17	18	0.20	0.47	0.09	1.21	13.78	84.15	282.37	610	0.20	0.013	286.97	0.98	1.02	31.00	0.505	14.28	0.98
	RLCB I	18	0.07	0.45	0.03	0.03	10.00	98.11	9.17	254	1.00	0.013	62.04	1.22	0.75	40.30	0.893	10.89	0.15
Selak Place	18	19	0.30	0.48	0.14	1.38	14.28	82.60	317.76	610	0.30	0.013	351.46	1.20	1.23	58.00	0.787	15.07	0.90
	Rea K	19	0.04	0.45	0.02	0.02	10.00	98.11	5.07	254	1.00	0.013	62.04	1.22	0.64	37.70	0.988	10.99	0.08
Selak Place	19	6	0.48	0.48	0.23	1.63	15.07	80.32	364.79	610	0.40	0.013	405.83	1.39	1.42	89.70	1.056	16.13	0.90
	20	12	0.25	0.60	0.15	0.15	10.00	98.11	41.14	305	0.40	0.013	63.91	0.87	0.81	45.10	0.931	10.93	0.64
	RLCB C	21	0.29	0.40	0.12	0.12	10.00	98.11	32.13	254	1.00	0.013	62.04	1.22	1.06	97.80	1.534	11.53	0.52
	RLCB D	21	0.14	0.40	0.06	0.06	10.00	98.11	15.45	254	1.00	0.013	62.04	1.22	0.86	49.20	0.957	10.96	0.25
Fabian Papa Way	21	22	0.23	0.61	0.14	0.32	11.53	91.86	80.99	381	0.40	0.013	115.68	1.01	0.96	13.10	0.228	11.76	0.70
Meyer Street	22	23	0.18	0.51	0.09	0.41	11.76	91.01	102.93	381	0.40	0.013	115.68	1.01	1.03	40.30	0.651	12.41	0.89
Meyer Street	23	24	0.33	0.51	0.17	0.57	12.41	88.66	141.46	457	0.30	0.013	162.72	0.99	1.00	77.60	1.293	13.71	0.87
Meyer Street	24	25	0.57	0.49	0.28	0.85	13.71	84.37	199.66	533	0.30	0.013	245.25	1.10	1.08	102.30	1.572	15.28	0.81
Meyer Street	25	26	0.21	0.55	0.11	0.97	15.28	79.74	214.02	533	0.30	0.013	245.25	1.10	1.11	50.70	0.761	16.04	0.87
Adams Avenue	26	40	0.09	0.59	0.05	1.02	16.04	77.69	219.50	533	0.30	0.013	245.25	1.10	1.12	40.60	0.605	16.64	0.90
	RLCB J	40	0.13	0.45	0.06	0.06	10.00	98.11	15.44	254	1.00	0.013	62.04	1.22	0.86	50.70	0.986	10.99	0.25
Adams Avenue	40	27	0.29	0.49	0.14	1.22	16.64	76.14	257.63	533	0.40	0.013	283.19	1.27	1.30	46.00	0.590	17.23	0.91
	RLCB L	27	0.09	0.45	0.04	0.04	10.00	98.11	11.14	254	1.00	0.013	62.04	1.22	0.79	43.20	0.912	10.91	0.18
Adams Avenue	27	8	0.32	0.49	0.16	1.42	16.64	76.14	299.78	610	0.30	0.013	351.46	1.20	1.21	86.60	1.198	17.84	0.85
Fabian Papa Way	28	31	0.51	0.40	0.20	0.20	10.00	98.11	55.26	381	0.40	0.013	115.68	1.01	0.86	79.40	1.540	11.54	0.48
Fabian Papa Way	29	31	0.00	0.00	0.00	0.00	10.00	98.11	0.00	381	0.40	0.013	0.00	0.00	0.00	13.80	0.000	10.00	0.00
Fabian Papa Way	30	31	0.33	0.43	0.14	0.14	10.00	98.11	38.59	381	0.40	0.013	115.68	1.01	0.77	67.30	1.459	11.46	0.33
Karney Street	31	32	0.24	0.49	0.12	0.46	11.54	91.84	117.25	457	0.30	0.013	162.72	0.99	0.94	43.60	0.769	12.31	0.72
Karney Street	32	33	0.32	0.48	0.15	0.61	12.31	89.02	151.51	457	0.30	0.013	162.72	0.99	1.02	48.20	0.786	13.10	0.93
Karney Street	33	34	0.58	0.47	0.27	1.10	13.10	86.34	263.20	533	0.40	0.013	283.19	1.27	1.31	68.90	0.879	13.97	0.93
Karney Street	34	35	0.45	0.47	0.21	1.31	13.97	83.54	303.30	610	0.30	0.013	351.46	1.20	1.21	52.70	0.726	14.70	0.86
Karney Street	35	36	0.35	0.48	0.17	1.47	14.70	81.37	333.19	610	0.30	0.013	351.46	1.20	1.24	58.70	0.786	15.49	0.95
Nault Drive	37	39	0.43	0.32	0.14	0.14	10.00	98.11	37.49	381	0.40	0.013	115.68	1.01	0.76	17.90	0.391	10.39	0.32
Nault Drive	38	39	0.06	0.35	0.02	0.02	10.00	98.11	5.88	381	0.40	0.013	115.68	1.01	0.45	18.40	0.679	10.68	0.05
Nault Drive	39	33	0.11	0.49	0.05	0.21	10.68	95.23	56.31	381	0.40	0.013	115.68	1.01	0.86	40.80	0.787	11.47	0.49
Adams Avenue	8	36	0.19	0.52	0.10	4.85	17.84	73.27	986.55	914	0.40	0.013	1193.05	1.82	1.80	80.30	0.742	18.58	0.83
Adams Avenue	36	EXIT	0.78	0.31	0.24	6.56	18.58	71.60	1305.31	1067	0.25	0.013	1425.11	1.59	1.63	98.10	1.000	19.58	0.92