FUNCTIONAL SERVICING STUDY

PROPOSED SUBDIVISION

ZAVARELLA & KULMATYCKY LANDS
PART OF LOT 31 & 32
CONCESSION 2

FORMER TOWNSHIP OF BRANTFORD
GEOGRAPHIC AREA OF PARIS
COUNTY OF BRANT

Prepared By:

J.H. Cohoon Engineering Limited
440 Hardy Road
Brantford, Ontario
N3T 5L8
Phone (519) 753-2656
Fax (519) 753-4263

Submitted:

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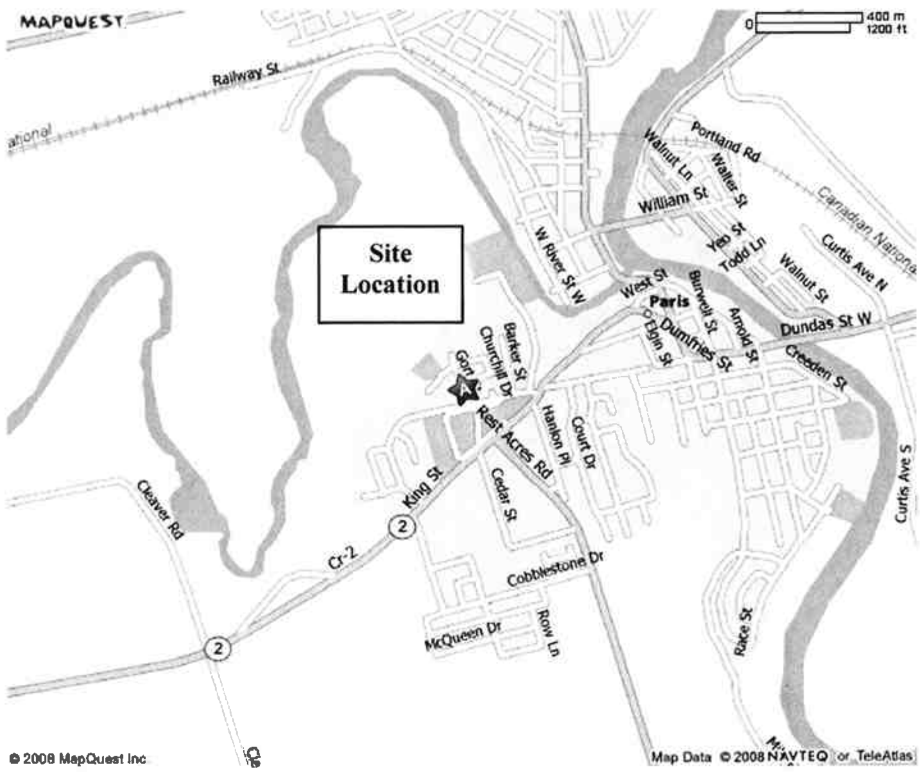
1.0 INTRODUCTION

This Functional Servicing Study was prepared by J.H. Cohoon Engineering Limited, in support of an Area Plan Application by Steve Kulmatycky and Zavarella Construction for the proposed development of approximately 34.9 hectares (86.4acres) of vacant land into a total of 369 lots comprised of 264 single family, 6 semi-detached, 90 medium density street townhouse units and 3 multiple density blocks. The projected total population is approximately **1,280 persons** or 52 people per developable hectare.

The lands, situated north of Dundas St. West, are legally referred to as Part of Lots 31 & 32, Concession 2, Former Township of Brantford, Geographic Area of Paris, County of Brant.

At the time of this report, it is anticipated that full municipal services will be installed. The following sections outline the servicing strategy to be employed for the development.



2.0 EXISTING SITE TOPOGRAPHY

The existing site is bordered by the Nith River along the west, north and east. The existing topography slopes out towards the river bank(s) around the perimeter of the property. Elevations vary from 269.0 along the south development limit to 234.0 at the north east site limit. There is a surface depression or "sinkhole" located on the north side of Dundas St. just south of the proposed cul-de-sac at the west limit of Street "A". The sinkhole (elev. 261.5) does not have a drainage outlet, however, the soils are highly permeable and surface runoff infiltrates quickly into the ground. The Kulmatycky lands situated north of Street "C" drain in a north-easterly direction to a low point near the existing wetland. Average slopes on the site generally range between 5%-15% with slopes of 3:1 and 4:1 also at some locations.

3.0 ROAD GEOMETRICS

The proposed road pattern for the development provides for three access points to Dundas St. West. The westerly access location (Extension of Street "C") will link up with Churchill Drive. The easterly access will be a connection to Barker St. The central access will be a northerly extension of Gort Avenue. A total of nine (9) public roads are proposed for this development, each with a 20.0 metre right-of-way. Preliminary road profiles were developed to provide for a minimum 2.4 metres of cover over the proposed storm sewer, 2.75m cover over the proposed sanitary sewer and 1.85m cover over the proposed watermains. The geometric design criteria used for this development conforms with County of Brant standards for local and minor collector roads as follows:

- Minimum K value (sag) = 12
- Minimum K value (crest) = 8
- Minimum Horizontal Radius = 80m
- Pavement Width = 9.2m
- Pavement Crossfall = 2%
- Minimum Grade = 0.50%
- Maximum Grade = 8.0%
- Intersection Angle = 70-90 degrees
- Minimum Asphalt Radius At Intersections = 9.0m
- Minimum Tangent Length Between Reverse Curves = 30m

4.0 PAVEMENT

A proposed pavement design for this development is outlined in the geotechnical investigation report prepared by Naylor Engineering Associates Ltd. dated August 2006. The recommended pavement component thicknesses based on pavement useage, frost susceptibility and strength of subgrade soils are as follows:

Pavement Component	Residential Streets
HL-4 Base Course Asphalt HL-3 Surface Course Asphalt	55 mm 35 mm
Granular 'A' Base Course	150 mm
Granular 'B' Sub-base Course	300-450 mm

It is recommended that Granular 'B' sub-base and Granular 'A' base courses be compacted to 100% Standard Proctor Maximum Dry Density. Due to the good natural drainage characteristics of the granular subgrade soils, pavement subdrains will not be required.

Concrete curb and gutter shall be constructed as per OPSD 600.10 (narrow gutter mountable). Sidewalks, 1.4 metres in width, shall be provided in accordance with County Standards.

5.0 SANITARY SEWERS & APPURTENANCES

5.1 Design Flows

The total design flow rate from the proposed development using current County of Brant design criteria is as follows:

1,280 persons x 450 litres/pers/day / 86,400 sec x 3.6 Harmon Peaking Factor + 0.2 L / Ha.s x 24.5 Ha.= 29.0 litres/sec total design flow.

A 250mm diameter gravity sewer @ 0.40% grade would be required to convey this flow. In accordance with County of Brant Engineering Standards, the minimum sewer size will be 200mm diameter with 150mm being considered for top sections of sewer runs in order to attain minimum flows with reduced depth. The minimum cover to the crown of the sanitary sewer will be 2.4m. The minimum size of sanitary sewer lateral will be 100mm diameter. Manholes will be spaced maximum 100 metres.

5.2 Sanitary Outlets

There are two (2) existing sanitary sewer outlets in the vicinity of the proposed development. The first is the existing 300mm diameter sanitary sewer on King St. discharging to the sewage treatment plant via the Church St. trunk sanitary sewer system. The second is the existing siphon under the Nith River downstream of the existing footbridge below Laurel St. discharging to the Mechanic St. sewer and ultimately the Willow St. Pumping Station.

5.2.1 Church Street Trunk Sewer

The existing sanitary sewer elevation at King and Dundas St. is approximately 265.00 (T/C elev. 267.80m). The sewer invert at the lowest point in the proposed development is 233.30. Therefore a pumping station will be required to deliver 29.0 litres/sec @ 31.7m of head.

The location of the required sanitary pumping station will be influenced by the following considerations:

- pumping stations are typically situated at the lowest point in the development to facilitate gravity flow into the station. Therefore, the permanent station should be located on Gort Ave. in the Kulmatycky lands. A sanitary forcemain would be constructed on Gort Ave. southerly a distance of approximately 765m to Dundas St. W.
- the Public Works Department has indicated that the County would like to abandon the siphon under the Nith River due to ongoing maintenance problems, age and number of existing river crossings (3). There is an opportunity to situate the proposed pumping station outside the development limits in a location that would facilitate interception of the existing sanitary flows from the Victoria Street Subdivision at Laurel Street. This option would entail constructing the sanitary pumping station north of the Lion's Park swimming pool. A trunk sanitary sewer would be constructed down the embankment along the east side of the Kulmatycky lands to service the proposed development. A second trunk sanitary sewer would be constructed from the point of interception with the existing sanitary sewer at Laurel St. (approx. 225m north of Dundas St.) down Laurel Street to the proposed pumping station location. A sanitary forcemain would then be constructed from the pumping station location, up the Laurel Street hill to King Street. This option would involve cost sharing between the Developer and the County for the pumping station and forcemain in proportion of the existing flows to the overall flows. The County would also be required to install the gravity sanitary sewer on Laurel Street.

The most recent analysis provided by the County indicates that there are capacity deficiencies in certain sections of the Church/Ball/Race St. trunk sanitary sewers. A summary of the capacity deficiencies is provided as follows:

Street Name	Pipe No.	Diameter (mm)	Slope (%)	Design Flow (l/sec)	Capacity (l/sec)	Sewer Length (m)
Dundas St. W.	PRS0157	250	0.11	67.9	20	10.7
King Edward St.	PRS0152	300	0.30	66.2	53	107.1
King Edward St.	PRS0151	300	0.35	67.4	58	105.2
Church St.	PRS0150	300	0.50	68.2	68	23.8
Church St.	PRS0149	300	0.50	68.9	68	114.3
Church St.	PRS0148	300	0.50	69.3	68	112.8
Church St.	PRS0142	300	0.37	72.3	59	45.6

Church St.	PRS0141	300	0.52	72.6	70	99.1
Burwell St.	PRS0140	300	0.40	73.0	61	62.5
Ball St.	PRS0039	675	0.18	385.6	357	30.5
Ball St.	PRS0038	675	0.15	391.1	326	82.0
Ball St.	PRS0035	675	0.16	406.5	336	146.6
Ball St.	PRS0034	675	0.19	408.4	366	89.3
Ball St.	PRS0033	675	0.21	408.8	385	96.65

Note: The above figures include approximately 22 l/sec from the Grandville sewage pumping station. This flow is to be re-directed to the Mile Hill system once it is constructed.

The analysis was based on a theoretical per capita average daily flow rate of 450 l/pers.day. Flow monitoring in the Grandville sanitary sewer system and at the Paris Wastewater Treatment Plant indicate that actual flows are likely in the order of 270-350 l/pers.day. Secondly, as noted above, the construction of the Mile Hill trunk sanitary sewer / forcemain will allow the 22 l/sec from the Grandville Subdivision currently being discharged into the Church St. system to be re-directed. Once this has been done, there should be sufficient capacity in the existing system to accommodate the design flows from the Zavarella and Kulmatycky developments.

5.2.2 Nith River Siphon

The existing siphon invert is 222.76 metres on the west side, 221.896 on the east side and 200mm diameter (length = 55.5m slope 1.57%). Since the lowest sanitary sewer invert in the proposed development is 233.30 it is possible to gravity drain the sewer to the siphon and eliminate the need for a pumping station. This option would be more economical than the pumping station option. **The theoretical capacity of the siphon is approximately 40 litres per second. The required capacity is 29.0 litres per second for the new development plus 8.7 litres per second for the existing Victoria Survey residential for a total of 37.7 litres per second.**

5.2.3 Alternate Sanitary Servicing of Zavarella Lands

The County of Brant is presently reconstructing existing sanitary sewers in Victoria Survey and there is an opportunity to lower the existing invert at the north limit of Barker Street to elevation 263.130. Should the Kulmatycky development not proceed prior to or simultaneously with the Zavarella development, there is an opportunity to gravity drain sanitary sewage from Street A. This would allow about 40 units to develop in advance of the installation of the sewage pumping station.

5.3 Paris Wastewater Treatment Plant Capacity

5.3.1 Wastewater Design Criteria

The design criteria used in the theoretical analysis of existing and proposed infrastructure capacities is based on Ontario Ministry of the Environment Guidelines and County of Brant Engineering Standards. The average daily per capita flow rate presently employed by the County for residential flows is 450 litres/pers/day. This is the upper limit of the 225 to 450 litres/person/day range provided in the 1985 MOE Guidelines and is considered very conservative. Existing flow records measured at the County's Paris and St. George Wastewater Pollution Control Plants indicate a significant discrepancy between theoretical flows and actual flows with measured actual flows being as much as 50% lower than theoretical flows based on the County's design standards[i]. This indicates that the existing infrastructure is not being used efficiently. Other Ontario based municipalities have recognized this and have correspondingly adopted a lower per capita design flow rate of approximately 350 litres/person/day as noted in the following table;

Location	**Per Capita Design Flow Rate (l/per/day)**
City of Barrie	225
City of London	295
Regional Municipality of Niagara (City of Niagara Falls, Town of Fort Erie, City of St. Catharines, Town of Grimsby)	320
Regional Municipality of Waterloo (City of Kitchener, City of Cambridge, City of Waterloo, City of Guelph)	350
Halton Region	365
City of Ottawa	350
City of Kingston	350
City of Brantford	450
Haldimand County	450

Theoretical flows are a function of assumed per capita flow generation rates, contributing area and population density. Current County Standards stipulate a population density of 3.5 persons per single family residential unit. Recent census data and Official Plan numbers suggest that actual population density ranges between 2.5 and 2.8 persons per unit[ii]. The Official Plan further stipulates the maximum number of units per hectare to be 19 for low density. This translates to approximately 53 persons per gross hectare. Although these numbers are appropriate for new developments, they do not reflect actual populations in existing residential areas. Census data again shows that the actual population density is approximately 38 persons per hectare (13 units per hectare).

Given that current Provincial Policy (ie. Places To Grow) recommends a minimum of 50 combined residential persons and employment spaces and; the desire to increase the efficiency of the existing infrastructure, the following is recommended:

1. Reduce the per capita average daily design flow generation rate from 450 L/person/day to 370 L/person/day for residential development. This figure was obtained by analysing existing treatment plant flow records over a 5 year period
2. Reduce the average day design flow from industrial areas from 45 m^3/hectare to 26 m^3/hectare
3. Analyze the existing system assuming 38 persons/hectare for existing built up residential areas as agreed to by the County Public Works Department
4. Adopt a proposed density of 60 persons/hectare for new development or as proposed in the new Official Plan. Design flow for new residential developments should be based on the maximum allowable densities in the new Official Plan or as amended from time to time

   The above revisions to the County infrastructure design criteria will provide an efficient use of the existing system; provide a reasonable design basis for new infrastructure and; thereby facilitate additional development without occurring additional cost.

5.3.2 Paris Wastewater Pollution Control Plant Capactiy

The rated capacity of the existing plant is 7,056 m^3/day (per C of A #3-0216-80-006 dated October 27,1980). Using the proposed per capita flow rate of 370 L/pers/day translates to a theoretical population of 19,070 (vs. 15,680 using 450 L/pers/day). In the year 2000, measured flow data at the plant indicated that the plant was servicing a population base of 9,000 at an average daily flow rate of 3,339 m^3/day (57.6% capacity, equivalent to 370 L/pers/day (1 m^3 = 1,000 L)).

The 2006 census population for the Paris Urban Area was 11,177. Using 370 L/pers/day translates to a theoretical average day flow of 4,135 m^3/day (vs. 5,030 m^3/day using 450 L/pers/day). The maximum available plant reserve capacity based on 370 L/pers/day is therefore (7,056-4,135) = 2,921 m^3/day (2,026 m^3/day @ 450 L/pers/day). This is equivalent to an average day flow of between 23.4 to 33.8 L/sec. Therefore there is sufficient capacity to treat the estimated 8.5 L/sec (736 m^3/day) average day flow from the proposed Kulmatycky and Zavarella developments.

5.3.3 Willow St. Sewage Pumping Station

Upgrades to the Willow Street Sewage Pumping Station were recently completed in 2008. the rated capacity per C of A 4631-6ZQREV is 295 L/sec (three pumps each with a capacity of 98.3 L/ssec, the fourth pump is a standby and not counted when establishing the firm capacity of the station). Sewer design sheets provided by the County indicate a theoretical flow rate of 330 l/sec at the station. This is based on County Design Standards of 450 L/cap.day, 38 persons/hectare residential density, 45 m^3/ha. Industrial average daily flow rate and 100 pers/Ha. industrial population density. Most recent flow monitoring in the system indicates actual peak flows approximately 40% of theoretical design flows or 132 L/sec. Therefore available capacity in the pumping station is estimated to be 295-132 = 163 L/sec.

6.0 STORM SEWERS & APPURTENANCES

The proposed storm drainage system for this development has been designed using the major/minor dual drainage system concept. The minor system consists of storm sewers, manholes and catchbasins designed to collect and convey surface flows from the road up to and including the 5 year storm event. Foundation drains will be connected to sump pumps and discharged to grade. Stormwater runoff generated by storms greater than the 5 year event will be contained within the proposed road right of way and conveyed along the proposed overland flow route (road surface). The storm sewers will be designed in accordance with County of Brant Engineering Standards as follows:

- Minimum pipe size 300mm diameter
- City of Cambridge I-D-F Curve Parameters – 5 year design storm event
- Minimum 1.2 metres cover from the outside of the pipe barrel to the ground surface
- Minimum allowable velocity 0.8 m/s and Maximum allowable velocity 6 m/s
- 100m maximum manhole spacing
- 90m maximum catchbasin spacing

7.0 WATERMAINS & APPURTENANCES

7.1 Water Supply Capacity

The current state of the Paris Water Supply System is described in detail in the April 2, 2008 Corporate Development Committee Report prepared by Alex Davidson, Water Division Manager[iii]. The report discusses additional system capacity recently gained through upgrades to the Gilbert Well Field and potential new supply from investigations being completed on the proposed Bethel well location. Ongoing projects to construct an elevated tank in north Paris and expand the Sharpe Reservoir in southern Paris will be completed to be available for peak seasonal demand in 2010.

Prior to completing the upgrades in early 2008, the County had placed a moratorium on new development through PWE-2003-05 'Paris Interim Water Servicing Policy'. Completion of the system upgrades has provided an additional 7.3 L/sec (630 m^3/day) available capacity. to service approximately 486 equivalent residential units @ 370 L/pers/day (400 equivalent residential units using current County of Brant Standards)

Calculations provided in the CDC Report are based on the following assumptions:

1. The total optimized well operating capacity of 194 L/sec cannot be provided at all times due to the possibility of unscheduled maintenance. Therefore, the firm well production capacity has been reduced to 159 L/sec to account for the largest pump being out of service

2. The currently committed capacity is based on the maximum measured maximum day demand between 2003 and 2007 (119 L/sec) and 754 residential units at 450 L/pers/day and 3.5 persons/unit per current County Standards (13.7 L/sec)
3. The applied design consumption of 1.575 cubic metres per day per residential unit is based on historical data and may be conservative in the current culture (staff are preparing to analyze this. As this requires analysis of historical data from equipment that is currently being put in place, a reliable evaluation is approximately 5 years out.)
4. The proposed new elevated storage tank in Zone 1 (north Paris) and increased pumping and storage at the Sharpe Reservoir (southern Paris) will provide the required equalization and fire storage to optimize the supply from the wellfields and mitigate potential concerns related to fire supply capability. These projects are expected to be completed for peak demand season of 2010. Until additional storage is constructed, fire water storage compared to industry standards may be compromised during periods of peak demand. Nothwithstanding, an operational plan is in place to increase flow in the distribution system toward a fire if and when necessary.
5. The CDC report states "any industrial development in the near future will include dry industries only". This is appropriate given the supply limitations of the Paris groundwater system and anticipated demands.

Based on the above, there is water available to service a portion of the subject development(s). However, a new well source is required to achieve full build out. Phasing of the proposed development(s) is required.

The September 10, 2007 "Project Update for the Bethel Road Water Exploration Program" presented to CDC by KMK Consultants indicates a target of 30-60 L/sec from the Bethel well system. There is also potential for attaining up to 50 L/sec from the Airport Area if required. The current Airport well capacity of 26.5 L/sec is only being partially utilized[iv].

7.2 Distribution System

Municipal water is available to service the proposed development via the existing 200mm diameter watermain on Dundas St. W. and 250mm diameter watermain on Hwy. 2. The existing 150mm diameter watermains on Gort Ave. and Barker St. will need to be upgraded and a looped system provided. Hydrants will be provided at maximum 150 metre spacings to supply fire flows to the development. Individual (19mm diameter) domestic water service connections will be provided to the property line for each new residence. A 200mm diameter watermain will be provided on Streets A, C, F, and Gort Ave. Extension. 150mm diameter watermain will be provided elsewhere.

8.0 STORMWATER MANAGEMENT

8.1 Introduction

The proposed stormwater management plan described in this document was prepared in conformance with the current County of Brant Development Engineering Standards and the 2003 Ministry of the Environment Stormwater Practices Planning & Design Manual. The plan addresses surface water quality and quantity impacts of the proposed development on the surrounding environment and outlines mitigation / control measures to be incorporated in the construction of the development.

8.2 Existing Pre-Development Conditions

8.2.1 Land Use and Topography

The existing land use is considered to be agricultural with corn the predominant crop. As stated in Section 2.0 the existing site topography slopes towards the Nith River around the site perimeter. Elevations vary from 269.0 along the south development limit (Street "A" between Gort Ave and Street "C" extension) to 234.0 at lot 308 (Gort Ave. Sta. 0+765). There is a surface depression or "sinkhole" located on the north side of Dundas St. just south of the proposed cul-de-sac at the west limit of Street "A". The sinkhole (elev. 261.5) does not have a drainage outlet, however, the soils are highly permeable and surface runoff infiltrates quickly into the ground. The Kulmatycky lands situated north of Street "C" drain in a north-easterly direction to a low point near lot 307 (Gort Av. Sta. 0+765). Average slopes on the site generally range between 5%-15% with slopes of 3:1 and 4:1 also at some locations.

8.2.2 Site Specific Soil Characteristics

On-site soil conditions are described in the August 2006 Preliminary Geotechnical Investigation Report by Naylor Engineering Associates. Topsoil was encountered at the ground surface in all the boreholes and test pits across the site. The topsoil ranges in thickness from 200mm to 700mm and typically comprises dark brown silt that was moist at the time of the fieldwork.

Silt was encountered in boreholes 1,3,4,6,10 and 11, below the sand in boreholes 8 and 9, and below the topsoil in test pits 1,3,6,7,26 to 29, 31 and 33. Silt was encountered inter-layered with sand in boreholes 2,3 and 4. The silt ranged in thickness from 0.3 to 11.7m. Results of a particle size distribution analysis carried out on a sample of silt shows the sample contains 45% sand, 49% silt and 6% clay. The silt is typically loose to compact in the upper 1.0m of the deposits and has a compact ot dense relative density below 1.0m. The moisture content of the silt ranges from 12 to 25% indicating moist to wet conditions.

Sand deposits were encountered throughout the Zavarella property and into the south central part of the Kulmatycky (north) property. A major deposit of sand was encountered in Boreholes 1 to 5, 8 & 9. The sand was 2.0 and 4.3m thick respectively in Boreholes 8 and 9 and extends between 10.2 and 19.0m below ground surface in boreholes 1 to 5. Sand was also encountered in 19 of the 33 shallow test pits, and ranged in thickness from 0.4 to 1.4m and extended below the termination depths of sixteen test pits. The texture of the sand deposits ranges from silty sand to gravelly sand with trace silt. A grain size distribution analysis carried out on a sample of the sand shows that the sample contains 31% gravel, 63% sand and 6% silt.

Sand and gravel was contacted at boreholes 3,4,5,10 and 11 and half of the test pits excavated at the site. The sand and gravel was typically encountered in the southwest part of the Zavarella property and throughout the north part of the Kulmatycky property. The sand and gravel ranges in thickness from 0.5 to 2.6m at boreholes 3, 4 and 10, and test pits 2,5,6 and 25. The sand and gravel extends to depths of 6.0 and 7.9 m respectively at Boreholes 5 and 11 and extends below the termination depths of test pits 3,4,9,21,22,26 and 28 to 33.

Groundwater observations and measurements carried out in the boreholes indicate that the stabilized groundwater table is generally 7 to 18 m below existing grade or between elevation 228 and 249m. The groundwater is typically perched in the lower portion of the sand, and sand and gravel deposits above the glacial till. The horizontal hydraulic gradients is generally from south to north.

8.2.3 Pre-Development Hydrologic Modeling Parameters

MIDUSS modeling software was used to establish pre-development runoff rates for the site. For modeling purposes, Soil/Land Use Curve Numbers were used to determine runoff potential from the site during pre-development conditions. The overlying soils are classified as Type B soils with moderate to good drainage characteristics and moderate to low potential for runoff. The corresponding Soil Conservation Service (SCS) curve number was taken to be 81 (straight row crops, contoured) (Average Anticedent Moisture Condition AMC II). Initial rainfall abstractions were estimated to be 0.1 times the available soil storage (0.1*S). The Manning's roughness value for shallow overland flow was taken to be 0.10. The existing topography slope is approximately 5 to 10% (say 8% average).

8.3 Post Development Conditions

The proposed concept plan includes a total of 369 lots comprised of 264 single family, 6 semi-detached, 90 medium density street townhouse units and 3 multiple density blocks. Roads are to be paved with asphalt a width of 9.14 metres and curb and gutter provided to facilitate storm drainage. Sidewalks will also be provided on at least one side of the right of way. The total road right of way is 20 metres. A maximum imperviousness ratio of 55% has been assumed in the hydrologic modeling for single family residential development and 65% for medium density development as calculated below:

<u>Calculate Percent Impervious Surfaces</u>

Area of Lot + ROW for 12.2m frontage

= 12.2m x 42m
= 512 m^2

Impervious Areas:

Road	= 4.57m x 12.2m	= 56 m^2
Sidewalk	= 1.40m x 12.2m	= 17 m^2
Curb	= 0.44m x 12.2m	= 5.5 m^2
House (roof)		= 150 m^2
Driveway	= 11.0m x 6.0m	= 66.0 m^2

Total **= 295 m^2**

∴ % Imperviousness = 295/512 = 0.576 Say 55%

8.4 Stormwater Management Design

Grading and phasing considerations will require that a combination of stormwater management solutions be implemented for the proposed development(s).

The south half of the Zavarella property drains east and west to low points at each end of Street 'A'. The north half of the Zavarella property drains north towards the Kulmatycky lands. The east end of Street 'A' drains to a low point in the road. Quantity (flood) control storage can be provided within the street right of way or outside the road right of way in a detention pond. Given the small drainage area, a water quality inlet (ie. Stormceptor or equivalent) can provide the required water quality treatment control. Surface runoff draining to the west end of Street 'A' will be directed to the existing sink hole(s) to the south and allowed to infiltrate into the ground. Pre-treatment for water quality will be provided by a water quality inlet (ie. Stormceptor or equivalent). Surface water drainage flowing north along Gort Ave. will be directed to a proposed stormwater management pond facility to be built on the Kulmatycky property.

8.4.1 Modeling Results - Quantity Control

Stormwater flows were calculated using MIDUSS modeling software. City of Cambridge IDF parameters were used to generate rainfall for sizing of the SWM facility in accordance with County of Brant Development Engineering Standards.

Peak flow reduction will be achieved through on site detention in an effort minimize the potential for downstream flooding and erosion. Post development surface water runoff will be controlled to existing pre-development levels for the 2, 5, 10, 25, 50 and 100 year storm events. An emergency overflow will be provided in the system for the regional storm event. The results of the Miduss modeling are as follows:

Table 8.1 – Peak Flow Rates At Temporary SWM Pond Outlet (Location 2) North Limit of Zavarella Lands

Storm Event	Pre-Development Peak Flow (m^3/sec)	Post Development Peak Flow No SWM (m^3/sec)	Storage Volume Required (m^3)
100 Year	1.033	1.414	454
50 Year	0.843	1.252	
25 Year	0.654	1.106	
10 Year	0.443	0.947	

5 Year	0.290	0.765	
2 Year	0.118	0.569	
25mm	0.036	0.346	

Table 8.2 – Peak Flow Rates At Outlet Location 4 (Exist. Sinkhole @ Southwest Limit of Zavarella Lands)

Storm Event	Pre-Development Peak Flow (m^3/sec)	Post Development Peak Flow No SWM (m^3/sec)	Post Development Peak Flow No SWM (m^3/sec)	Storage Volume Required (m^3)
100 Year	0.165	0.764	0.047	1,479
50 Year	0.131	0.730	0.047	
25 Year	0.108	0.597	0.047	
10 Year	0.069	0.511	0.047	
5 Year	0.046	0.413	0.047	
2 Year	0.020	0.295	0.047	
25mm	0.006	0.187		

Provide orifice plate in catchbasin to control post development flows to existing 5 year event pre-development levels.

$$Q = C_d A\sqrt{2gh} = 0.60(\frac{\pi D^2}{4})\sqrt{2x9.81x1.5m}$$

$$\text{Required diameter } D_{req'd} = \sqrt{\frac{4Q}{(0.60x\pi\sqrt{29.43}}} = \sqrt{\frac{4(0.047)}{(0.60x\pi\sqrt{29.43}}} = 135mm$$

Table 8.3 – Peak Flow Rates At Outlet Location 3 (Existing Gully @ East Limit of Zavarella Lands)

Storm Event	Pre-Development Peak Flow (m^3/sec)	Post Development Peak Flow No SWM (m^3/sec)	Post Development Peak Flow With SWM (m^3/sec)	Storage Volume Required (m^3)
100 Year	1.114	0.531	0.108	595
50 Year	0.772	0.508	0.092	
25 Year	0.705	0.415	0.108	

10 Year	0.479	0.356	0.108	
5 Year	0.267	0.301	0.108	
2 Year	0.108	0.288	0.108	
25mm	0.033	0.130		

Provide orifice plate in catchbasin to control post development flows to existing 2 year event pre-development levels.

$$Q = C_d A\sqrt{2gh} = 0.60(\frac{\pi D^2}{4})\sqrt{2x9.81x1.5m}$$

$$\text{Required diameter } D_{req'd} = \sqrt{\frac{4Q}{(0.60x\pi\sqrt{29.43}}} = \sqrt{\frac{4(0.108)}{(0.60x\pi\sqrt{29.43}}} = 205mm$$

Table 8.4 – Peak Flow Rates At Permanent Pond Outlet (Location 1)

Storm Event	Pre-Development Peak Flow (m^3/sec)	Post Development Peak Flow No SWM (m^3/sec)	Post Development Peak Flow With SWM (m^3/sec)	Storage Volume Required (m^3)
100 Year	4.67	7.29	4.39	5,856
50 Year	3.78	6.46	3.37	5,623
25 Year	2.97	5.70	2.43	5,409
10 Year	1.99	4.88	1.07	5,083
5 Year	1.33	3.95	0.42	4,743
2 Year	0.58	2.94	0.08	3,863
25mm	0.15	1.79		

8.4.2 Quality Control

Water quality control will be achieved for the Zavarella development via the installation of water quality inlets at each end of Street A and a temporary *Wet Pond* in the linear park at the north subdivision limit. The pond will discharge down the bank to the river. A permanent wet pond will be installed on the Kulmatycky development.

There are presently no subwatershed plans available for the area to provide guidance on water quality protection requirements. Therefore the minimum design criteria are proposed as follows: