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7582 County Road 9, Creemore Township of Clearview

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

2826878 Ontario Inc.

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1 Introduction

Tatham Engineering Limited (Tatham) has been retained by 2826878 Ontario Inc. to complete a Functional Servicing & Stormwater Management Report (FSSWMR) in support of a Zoning By-law Amendment (ZBA) application for a proposed residential development located at 7582 County Road 9 in Creemore, Township of Clearview (Township).

1.1 OBJECTIVES

The objective of this report is to demonstrate the feasibility of the proposed development with respect to servicing including site grading, stormwater management (SWM), sewage collection and treatment, water supply and distribution, site grading, and utility distribution.

1.2 GUIELINES AND BACKGROUND INFORMATION

This report has been prepared in accordance with local and provincial guidelines, including the following publications:

- The Ministry of the Environment, Conservation, and Parks (MECP, formerly known as Ministry of Environment), Stormwater Management Practices Planning and Design Manual (March 2003);
- The MECP, Design Guidelines for Drinking-Water Systems, (2003);
- The Ministry of the Environment, Conservation, and Parks (MECP, formerly known as Ministry of Environment), Source Protection Information Atlas (accessed August 30, 2024);
- Ministry of Transportation (MTO), IDF Curve Lookup (accessed August 30, 2024);
- Nottawasaga Valley Conservation Authority (NVCA), NVCA Stormwater Technical Guide (2013);
- R.J. Burnside & Associates Limited, Technical Memorandum: Mary Street Sanitary Sewer Capacity (December 2021);
- R.J. Burnside & Associates Limited, Creemore Water and Wastewater Master Servicing Plan (May 2024); and
- R.J. Burnside & Associates Limited, East Creemore Drainage Study, Community of Creemore, Clearview Township Municipal Class Environmental Assessment (Schedule B) (September 2024).

This report was prepared in consideration of the following site-specific studies and reports:

- *Geotechnical Investigation Report – 7582 County Road 9, Creemore*, Cambium Inc., (June 4, 2025);
- *Natural Hazard Study,* Tatham Engineering Limited (April 2024); and
- *Hydrogeological Assessment,* Tatham Engineering Limited (June 2025).

2 Site Description

2.1 SITE DESCRIPTION

The proposed development (referred to as 'the site') is located on the southern half of the subject property at 7582 County Road 9. The subject property is 6.04 ha with 3.02 ha proposed for development under the current ZBA application.

The site is bound by Fairgrounds Road South to the east, vacant land to the north (north half of subject property), existing residential fronting County Road 9 to the south, and existing residential fronting Collingwood Street to the west.

The subject property is legally described as:

PART OF THE SOUTH HALF
OF LOT 10
CONCESSION 4
(GEOGRAPHIC TOWNSHIP OF NOTTAWAGA, VILLAGE OF CREEMORE)
TOWNSHIP OF CLEARVIEW
COUNTY OF SIMCOE

Refer to Figure 1 for the Site Location Map.

2.2 LAND USE & ZONING

The subject property is located within the Settlement Area of Creemore and is currently designated 'Residential' land use in the Township's Official Plan.

The site is currently zoned 'Development Area' (DA) in the Township's comprehensive Zoning By-law. The site is also partially subject to the Hazard Land Overlay (FP) and regulated by the Nottawasaga Valley Conservation Authority (NVCA).

This report has been prepared to support the ZBA application to re-zone the site to 'Residential Multiple Low Density (RS3)' Zone and a site-specific 'Residential Multiple Medium Density Exception (RS5-X)' Zone.

2.3 TOPOGRAPHY

Information related to existing topography, ground cover, and drainage was obtained through a review of the topographic survey and base mapping, site visits and a review of relevant background studies and plans.

A topographic survey was completed by Tatham in January 2024.

The subject property is currently developed with a residential dwelling, a detached barn and an existing driveway access is provided to County Road 9. The remainder of the subject property is moderately to heavily treed with maintained turf on the south half and naturalized vegetation in the north.

The subject property generally slopes from north to south at a moderate grade (8%+ slope) on the north half to a flat grade (0 - 3% slope) on the south half. A berm (approximately 3 to 8 m high) traverses the north central area of the subject property. Drainage appears to gradually sheet flow south towards an existing roadside ditch on County Road 9.

Based on Ontario Soil Survey Complex the existing soils are classified as Osprey Loam.

2.4 GEOTECHNICAL INVESTIGATION

The Geotechnical Investigation completed by Cambium Inc. identified subsurface conditions and determined engineering properties of in-situ soils to support design and construction of the proposed development.

The subsurface stratigraphy consisted of a surficial topsoil layer (0.14 to 0.23 metres below ground surface (mbgs)) underlain by non-cohesive soils (predominantly sand and silt mixtures) and cohesive soils (ranging from sandy silt to silty clay). Bedrock was not encountered in boreholes terminated at depths between 6.2 and 6.7 mbgs.

Monitoring wells installed in two boreholes observed groundwater levels at depths of 2.64 to 5.97 mbgs. Tatham completed a Hydrogeological Investigation for further groundwater table analysis.

2.5 HYDROGEOLOGICAL INVESTIGATION

The Hydrogeological Investigation was completed to assess the presence and impact of groundwater on the design and construction of the proposed development.

Five monitoring wells were installed to facilitate measurement of stabilized groundwater levels. To date, the highest groundwater levels were measured in April 2025 and ranged between 0.3 and 3.84 mbgs. Long-term groundwater monitoring is underway to establish the seasonal high groundwater table.

The estimated construction dewatering volume is 126,000 L/day based on available information. This volume requires an EASR registration but will not require a PTTW.

2.6 NATURAL HAZARD STUDY

The west side Fairgrounds Road South roadside ditch is a tributary of the Mad River. A Natural Hazard Study was completed by Tatham to delineate the flood and erosion hazard limits associated with the tributary. The study found the regional flood limit is contained to the banks

of the existing ditch. Erosion hazards were not applicable as long-term erosion or meandering will be prevented through Municipal maintenance of the ditch.

2.7 NATURAL HERITAGE CONSTRAINTS ANALYSIS

A Preliminary Natural Heritage Constraints Analysis was completed by Cambium Inc. to identify natural heritage features potentially limiting the development potential of the site.

No areas of high constraints were identified in the study. Moderate constraints identified on site include Wetland, Woodland, Watercourse and Anthropogenic Structures (Barn). An Environmental Impact Study (EIS) is required to confirm if the moderate constraint features are considered significant. The EIS is underway and expected to be completed in 2025.

3 Proposed Development

3.1 CONCEPTUAL SITE PLAN

The Conceptual Site Plan, prepared by IPS, dated March 6, 2025, includes the following:

- 2 Single Detached Residential Lots fronting Collingwood Street;
- 6 Single Detached Residential Lots fronting County Road 9;
- 6.25 m Drainage Block adjacent to Fairgrounds Road South; and
- A condominium development block including:
  - 116 Townhouse Dwellings;
  - an amenity / Stormwater Management (SWM) facility area;
  - private laneway access to Fairgrounds Road South;
  - private laneway access to County Road 9; and
  - private laneways, accessible parking and pedestrian walkways.

Following ZBA approval, the proposed development will proceed through the Draft Plan of Subdivision and Site Plan Control approval processes.

Refer to Appendix A for the Conceptual Site Plan.

3.2 CREEMORE MASTER SERVICING PLAN UPDATE

The Creemore Master Servicing Plan (MSP) has recently been completed to provide a recommended servicing strategy for future growth in Creemore. Supporting technical memos prepared by R.J. Burnside & Associates Ltd. (RJB) are referenced and appended to the MSP. When this report references the MSP, it includes references to the associated RJB technical memos.

The preferred servicing strategies identified in the MSP are presented in the relevant sections of this report.

3.2.1 Future Growth and Servicing Needs

The MSP defines the overall study area and delineates sub-areas based on anticipated growth projections to provide targeted servicing recommendations.

Refer to Figure 2 for an illustration of the Creemore Study Area / Sub-Areas.

The site is located within the 'Creemore Core' sub-area. The MSP forecasts Creemore's total population growth to approximately 5,817 people by 2041, comprised of:

- Existing population: 2,051 people
- Growth in 'Creemore East': 3,417 people
- Growth in 'Creemore Core': 349 people (infill)

Projected growth within the 'Creemore North' is beyond the 2041 horizon, and no further anticipated servicing needs are considered for 'Creemore West' and 'Creemore Southeast'.

The infill population growth projection (349 people) for 'Creemore Core' is based both on remaining capacity within the Mary Street sanitary trunk sewer, as well as remaining capacity at the Creemore Wastewater Treatment Plant (WWTP). Further to the MSP, the 'Creemore Meadows' development (72 residential units x 2.8 ppu = 202 people) – located within 'Creemore Core', effectively reduces the remaining WWTP capacity to 147 equivalent people for the proposed infill allocation.

The MSP assigned a projected equivalent population of 132 people (45 units) for the site, significantly lower than the current Concept Site Plan (348 people (124 units) using 2.8 ppu).

4 Water Supply and Distribution

4.1 EXISTING WATER SYSTEM

The Creemore Drinking Water System is owned and operated by the Township and consists of a water treatment plant supplied by groundwater, a distribution system, and a water storage reservoir. The existing water system services an estimated 2,190 equivalent people.

The drinking water system currently meets the water demands of the existing population in Creemore, however water system upgrades will be required to support demands based on the projected population growth to 2042.

Refer to Table 1 for a summary of the increased demand for the water system.

Table 1: Water System Demands

YEAR	SERVICE POPULATION	WATER SUPPLY	WATER STORAGE
Existing (MSP)	2,190	15.0 L/s	1,570 m^3
2042	5,956	40.1 L/s	2,379 m^3
Beyond 2042	9,676	70.8 L/s	4,059 m^3

4.1.1 Water Supply & Treatment

The Creemore Water Treatment Plant (WTP) is located on the east side of Mary Street, north of Caroline Street East. The plant has a rated treatment capacity of 2,688 m^3/d (31.1 L/s) supplied by two groundwater wells. Plant controls limit water taking from each well to 15 L/s to maintain compliance with the Municipal Drinking Water License, resulting in a maximum treated water discharge of 2,592 m^3/d (30 L/s). With one well required as standby, the firm supply capacity is 1,296 m^3/d (15 L/s). As noted in the MSP, the remaining available operational capacity of the WTP is 442 m^3/d (5.1 L/s).

Following construction of the 'Creemore Meadows' development, the current remaining available capacity is further reduced to 284 m^3/d (3.3 L/s).

A new municipal well was recently constructed and successfully tested, confirming a yield of 864 m^3/d (10.0 L/s). The additional well theoretically increases the available operational capacity from 3.3 L/s to 13.3 L/s (1,149 m^3/d). Further additional water supply is required to meet the projected demand of 3,465 m^3/day (40.1 L/s) for the 2042 growth horizon.

It is understood the Township has retained a consultant to advance the necessary investigations required to identify appropriate sites within the community to support the installation of

additional supply wells. Further discussion with the Township will be required to confirm timeline and ultimate supply capacity.

4.1.2 Water Storage

An existing water reservoir is located at the north end of Collingwood Street, approximately 375 m north of the site. The reservoir provides a total storage volume of 1,570 m^3 with 424 m^3 of available storage capacity, according to the MSP.

Following construction of the 'Creemore Meadows' development, the current available storage capacity is further reduced to 338 m^3.

An expansion to the municipal water storage system will be required to meet the projected demand of 2,379 m^3 for the 2042 growth horizon. The preferred solution identified in the MSP involves twinning the existing reservoir to provide a total storage volume of 3,140 m^3.

4.1.3 Water Distribution

An existing 300 mm dia. watermain is located on Collingwood Street and County Road 9. The existing dwelling located on the site and is currently serviced by an existing 25 mm water service, extending from the 300 mm dia. watermain within County Road 9.

Refer to Appendix B for MSP figures of Creemore's existing water distribution system.

4.2 PROPOSED WATER SYSTEM

The proposed development will be serviced by the Creemore Drinking Water System via connections to the existing 300 mm dia. watermains on Collingwood Street and County Road 9.

4.2.1 Water Demand

Water demand for the proposed development has been calculated in accordance with Township standards, including the following:

- per capita Average Day Demand (ADD) = 350 L/person/day
- person per unit (ppu) density = 2.8

A Maximum Day Factor (MDF) of 2.25 and a Peak Rate Factor (PHF) of 3.38 were applied based on Table 3-1 of MECP guidelines for service populations between 2,000 – 3,000 persons.

Summary of water demands are provided below in Table 2. Refer to Appendix C for Water Demand Calculations.

Table 2: Summary of Water Demand

SCENARIO	FLOW (m^3/d)	FLOW (L/s)
Average Day Demand	121.80 m^3/d	1.41 L/s
Maximum Day Demand (MDD)	274.05 m^3/d	3.17 L/s
Peak Hour Demand		4.77 L/s

Based on the MDD for the proposed development, the municipal supply system has the available operational capacity to service the proposed development.

4.2.2 Fire Protection

In support of the MSP, R.J. Burnside & Associates Ltd. prepared a technical memorandum (*Flows and Demands, May 2024*), for the purpose of estimating the water and wastewater demands associated with the projected future growth of Creemore. A required fire flow value of 95 L/s was used for the 'Creemore Core' area in these calculations. For consistency, 95 L/s was also applied for the subject development in this FSR. This assumption will be refined at detailed design upon confirmation of anticipated building design.

As part of the study, the technical memo notes based on the required MDD of 40.1 L/s for the overall 2042 horizon, fire flow was estimated in order to quantify required water storage capacity. A fire flow rate of 144 L/s (for a duration of 2 hours) was included in the MSP for the purpose of storage sizing for the community.

An existing conditions model of the Creemore water distribution system was prepared in support of R.J. Burnside's MSP updates. Excerpt figures of the model results are provided in Appendix B of this report for reference, with Figure 3 providing an illustration of the 'Existing Fire Flow + Maximum Day Demand' model results from R.J. Burnside's report. The figure shows modelled flows at junctions across the distribution network. The junctions of the Collingwood Street/County Road 9 intersection, the existing fire hydrant on the south side of County Road 9, and the Fairgrounds Road/County Road 9 intersection (connected to the existing 300 mm dia. watermain within County Road 9) list available flows of 200 L/s; satisfying the 144 L/s fire flow requirement outlined in the *Flows and Demands* technical memo as well as the 95 L/s assumed for the development.

Modelled available fire flows at these three junctions increase to 200.10 L/s under all future scenarios (refer to excerpt Figures 5 to 10 in Appendix B).

Fire protection water demands will be required to be calculated in accordance with the Fire Underwriters Survey (FUS) method to support of detailed design for the proposed development.

4.2.3 Water Storage

The total required storage volume for the proposed development was calculated to be 49 m^3.

Refer to Appendix C for Water Storage Calculations.

The existing reservoir currently has the available operational capacity to service the proposed development.

4.2.4 Water Distribution

The condominium block will be serviced by a 150 mm dia. watermain looped internally and connected to the municipal system at County Road 9. A backflow preventor, meter and enclosure will be provided at the connection to the municipal distribution system. Individual water services will be provided for each townhouse unit.

The single detached lots fronting Collingwood Street and County Road 9 will be serviced with individual water services connected to the existing 300 mm dia. watermain.

The existing water service to the property will be decommissioned in accordance with Township standards.

Watermain sizing, hydrant and valve locations will be confirmed in support of detailed design.

5 Sewage Collection & Treatment

5.1 EXISTING SEWAGE SYSTEM

The Creemore Wastewater System is owned by the Township. Local existing sewage collection is provided within the 'Creemore Core' sub-area and discharged to the Mary Street Trunk Sewer. The Mary Street Trunk Sewer conveys sewage to the Creemore Wastewater Treatment Plant (WWTP).

5.1.1 Creemore Wastewater Treatment Plant

The WWTP was designed to be constructed in two phases with an ultimate rated capacity of 1,400 m^3/d. The first phase of the WTTP was constructed to a rated ADF capacity of 860 m^3/d. Due to operational bottlenecks within the system, the MSP established the available capacity of the WWTP as 114 m^3/d. Further to construction of the Creemore Meadows development, the available capacity is reduced to 38 m^3/d.

The MSP notes the existing sludge digestion mechanism is undersized to handle any additional flow.

The preferred solution in the MSP includes phased upgrades to the WWTP: first, an expansion to the approved capacity of 1,400 m^3/d, followed by a Schedule 'C' Municipal Class Environmental Assessment (MCEA) and additional studies to increase capacity further to a target capacity of 2,100 m^3/d to service the 2042 growth horizon.

Appendix B of the MSP includes a *Wastewater Flow Projections* table which determined the sewage demands of the future growth populations for the various sub-areas of Creemore. The cumulative ADF for the 2042 horizon (comprised of existing and 'Creemore Core' infill, and the 'Creemore East' lands) equates to 2,055 m^3/d. It should be noted the recently approved Creemore Meadows development is included within the 'Creemore Core' infill lands in this calculation.

An Assimilative Capacity Study (ACS) of the Mad River (receiving waterbody for the Creemore WWTP) was completed by Hutchinson Environmental Services Ltd. (Hutchinson) in support of the plant expansion, to quantify the extent the treatment plant could be expanded to. Section 5 of the ACS (Water Quality Modelling) mentions mass-balance modelling was undertaken to predict the effects of expansion to the WWTP on the water quality of the Mad River. It should be noted this type of modelling does not assume any mixing zone, and as a result provides a conservative calculation of fully mixed concentration in the receiver (Mad River). The study's

conservative mass-balancing approach suggests there may be potential for further expansion, provided additional studies confirm adequate assimilative capacity in the Mad River.

The total future ADF of 2,055 m^3/d from the MSP was used to inform the phase two WWTP plant expansion capacity target of 2,100 m^3/d, to generally align with the findings of the ACS.

5.1.2 Mary Street Trunk Sewer Capacity

The Mary Street trunk sewer has a full flow capacity of 110.92 L/s. The MSP includes a sewer capacity analysis to confirm the available sewer capacity for the 2042 growth horizon. The analysis assigned 45 units for the site and assigned 75 units for the 'Creemore Meadows' development. The estimated total peak flow in the Mary Street trunk sewer is 102 L/s, representing 92% of the theoretical full flow capacity at the 2042 growth horizon.

The sewer design sheet was revised to reflect the updated unit count for the site based on the current Conceptual Site Plan in Section 5.2.3.

5.1.3 Sewage Collection

An existing 200 mm dia. sanitary sewer is located on County Road 9. The existing dwelling located on the site is currently serviced by an existing sanitary service connected to the existing sanitary sewer. The downstream sanitary sewers connect to the Mary Street trunk sewer from Wellington Street East.

5.2 PROPOSED SEWAGE SYSTEM

5.2.1 Sewage Flow

The sewage flow for the proposed development has been calculated in accordance with Township standards (varying slightly from the MSP design parameter values), including the following:

- Per capita Average Daily Flow (ADF) = 350 L/person/day (in accordance with the MSP)
- Person per unit (ppu) = 2.8
- Extraneous flow (infiltration) = 0.23 L/s/ha

Refer to Appendix D for Sewage Flow Calculations and Table 3 for a summary.

Table 3: Summary of Sewage Flow

SCENARIO	FLOW (m^3/d)	FLOW (L/s)
Average Daily Flow	121.80 m^3/d	1.41 L/s
Extraneous Flow		0.67 L/s
Peak Flow (incl. Extraneous)		6.38 L/s

5.2.2 Downstream Sanitary Sewer Capacity

The most hydraulically constrained local sewer upstream of the Mary Street trunk sewer is a 200 mm dia. pipe with a 0.42% slope, located on County Road 9, west of the Fairgrounds Road/Mill Street intersection. This sewer segment has a calculated full flow capacity of 21.3 L/s.

The existing upstream estimated flow contributing to this sewer segment is 7.29 L/s. This flow combined with the estimated sewage flow for the site (6.38 L/s) provides a residual capacity of 7.73 L/s (36%). The downstream local sewers have sufficient capacity to service the proposed development.

5.2.3 Mary Street Trunk Sewer Capacity Analysis

The design spreadsheet prepared to estimate the available capacity of the Mary Street trunk sewer in the MSP was replicated for this report and revised to include an additional 79 units from the proposed development not previously accounted for.

The total flow required to service to the 2042 growth horizon and the additional units is 105 L/s or 95% of the full flow capacity.

Note, the ADF rate used in the previous design sheet is 450 L/cap/day, greater than the ADF rate used for WWTP calculations in the MSP (350 L/cap/day). Adjusting the rate to 350 L/cap/day and calculating for a total of 227 infill units results in a total flow to the Mary Street trunk of 87.60 L/s (reflecting 79% of the theoretical full-flow capacity).

Refer to Appendix D for the updated Mary Street Trunk Sewer Design Sheet.

5.2.4 WWTP Capacity Allocation

Currently the WWTP lacks sufficient capacity to accommodate wastewater flows from the proposed development. The WWTP currently has a rated ADF capacity of 860 m^3/d with 38 m^3/d available, while the proposed development will generate an estimated ADF of 122 m^3/d.

Capacity is expected to be available once the expansion to the approved capacity of 1,400 m^3/d has been completed.

The unit count based on the current Conceptual Site Plan exceeds the planned capacity allocation of the MSP by 79 units, increasing the projected sewage flows from 44 m^3/d to 122 m^3/d for the site, resulting in an approximate 2.5% increase above the overall planned capacity expansion of 2,100 m^3/d. As the WWTP upgrades are sized based on the MSP 2042 growth horizon, this will reduce the available capacity for future planned developments. However, a review of the design criteria used in the MSP suggests the equivalent population calculations may have incorporated conservative assumptions refined based on industry standards and recent data.

These design criteria include:

- Population Density: The MSP applies a ppu of 3.0 for future development. Using a blanket ppu of 3.0 is overly conservative for modern developments as it fails to account for substantial variation in household composition across different housing types, with medium density (i.e. townhouses) typically averaging approximately 2.3 ppu. The current Township standard is slightly lower applying a 2.8 ppu for all densities.
- Demand per Capita: The MSP applies a water demand of 350 L/cap/day, while the MSP estimates a water ADD of 216 L/cap/d, and a wastewater ADF of 188 L/cap/d based on historically observed data. Based on the density of the development concept, per capita flows/demands would be expected to be similar to urbanized communities. The per capita flows/demands for surrounding urban municipalities include 225 L/p/day (City of Barrie), 325 L/p/d (Town of Innisfil) and 260 L/p/d (Town of Collingwood).
- Inflow & Infiltration: The MSP applies an average day I&I flow rate of 90 L/p/d and a peak I&I rate of 227 L/p/d. Expressing I&I rates on a per capita basis is an uncommon metric as infiltration is directly related to infrastructure characteristics (pipe length, material condition, joint integrity, and groundwater levels), rather than population served. Further, I&I should be applied as a static baseline addition to the calculated peak flow rather than being subjected to peaking factors, as infiltration occurs relatively consistently throughout the system regardless of usage patterns, aligning with standard engineering practices treating it as a separate, additive component in sanitary sewer design calculations. In addition, the average day I&I for new developments would be expected to be negligible due to modern construction methods and materials

We have recalculated the sewage flow and equivalent population using the following adjusted design criteria to demonstrate the magnitude of change these adjustments may produce:

- Population Per Unit criteria: Medium Density = 2.3



- Demand per Capita: 300 L/p/d
- Eliminate Average Day Infiltration
- Peak Infiltration Rate = 20,000 L/ha/d (per Township standard)

The resulting ADF for the proposed development with 124 units is 86 m^3/d compared to the 153 m^3/d calculated in accordance with the criteria established in the MSP. For further comparison, the adjusted design criteria have been applied to the allocated equivalent population for the 2042 growth horizon in Table 4.

Table 4: Wastewater Flow Projections to 2042 (Adjusted Design Criteria)

AREA	EQUIV. POP	ADF (DRY) (m^3/day)	AVG. I/I (m^3/day)	ADF (WET) (m^3/day)
Ex. Creemore	2,051	398	-	398
'Creemore Core'				
Creemore Meadows	166	50	15	65
Subject Development	286	86	26	112
Remaining Creemore Infill	79	24	7	31
'Creemore East'	2,620	786	236	1,022
			Total:	**1,627**
			WWTP Target Capacity (MSP):	2,100

Applying the adjusted design criteria to the 2042 growth horizon frees up additional capacity and results in a total required capacity less than the WWTP Target capacity of 2,100 m^3/d. The adjusted design criteria has been presented in this report for consideration only.

Alternatively, the Township could explore opportunity to expand the plant beyond the current target capacity to allow for increased densification. An updated ACS would be required to support a new target.

5.2.5 Sewage Collection

The condominium block will be serviced by a 200 mm dia. sanitary sewer connected to the municipal system at County Road 9. Individual sanitary services will be provided for each townhouse unit.

The single detached lots fronting County Road 9 will be serviced with individual sanitary services connected to the existing 200 mm dia. sanitary sewer. The single detached lots fronting Collingwood Street will be serviced with individual sanitary services connected to a future sanitary sewer.

Sanitary sewer sizing will be confirmed in support of detailed design.

6 Stormwater Management

6.1 EXISTING CONDITIONS

Under existing condition the site predominantly consists of undeveloped pasture and woodland.

Site runoff ultimately drains to Outlet #1, a Mad River tributary located north of the County Road 9 intersection at the intersection with Fairgrounds Road South. The site is delineated into two drainage catchments. Runoff from Catchment 102 sheet flows easterly towards the Mad River tributary (west roadside ditch for Fairgrounds Road South), and runoff from Catchment 101 sheet flows south to the County Road 9 road allowance.

Catchment 101 encompasses the west half of the property, including development lands (2.22 ha), undeveloped areas (2.35 ha), external lands to the north (0.75 ha), and existing residential lots on Collingwood Street (0.37 ha). The total catchment area is 5.69 ha with a runoff coefficient of 0.40. Catchment 102 encompasses the east half of the property, including a development lands (0.80 ha) and undeveloped areas (0.74 ha). The total catchment area is 1.54 ha with a runoff coefficient of 0.31.

Refer to Appendix E for the Pre-Development Drainage Plan (Drawing DP.1).

6.2 EXISTING HYDROLOGY

The site has been modelled in accordance with the MECP *Stormwater Management Planning and Design Manual* and the NVCA *Stormwater Technical Guide*. The rational method was used to estimate the pre-development peak flows for the 1:2-year through 1:100-year return frequency design storms. Site-specific Intensity-Duration Frequency (IDF) rainfall data was generated using the IDF Curve values obtained from the MTO IDF look-up tool in accordance with the Township of Clearview's Engineering Standards.

The existing condition peak flows from the site at Outlet #1 are summarized in Table 5.

Refer to Appendix G for supporting calculations.

Table 5: Existing Condition Peak Flow Summary

DESIGN STORMS	OUTLET #1 PEAK FLOW (m^3/s)
1:2-year	0.312
1:5-year	0.413
1:10-year	0.481

DESIGN STORMS	OUTLET #1 PEAK FLOW (m^3/s)
1:25-year	0.622
1:50-year	0.753
1:100-year	0.864

6.3 POST-DEVELOPMENT CONDITIONS

Under post-development condition, the site is delineated in four drainage catchments (Catchments 201, 202, 203, and Catchment 204).

Catchment 201 encompasses of the majority of the proposed condominium development, the two proposed single detached lots fronting Collingwood Street, and the existing lots fronting Collingwood Street which will continue to drain into the site under proposed conditions. The total catchment area is 2.79 ha with a runoff coefficient of 0.73.

Catchment 202 encompasses the 6 proposed single detached lots fronting County Road 9 as well as a portion of condominium Street 'D'. The total catchment area is 0.34 ha with a runoff coefficient of 0.71.

Catchment 203 encompasses a small east area of the proposed condominium development. The total catchment area is 0.92 ha with a runoff coefficient of 0.34.

Catchment 204 encompasses the west area of the undeveloped lands to the north. Drainage from this catchment will be conveyed through the proposed development to Outlet #1. The total catchment area is 3.14 ha with a runoff coefficient of 0.31.

Refer to Appendix E for the Post-Development Drainage Plan (Drawing DP.1).

6.4 PROPOSED HYDROLOGY

The site has been modelled as per the *Stormwater Management Planning and Design Manual* in accordance with MECP standards, and the NVCA *Stormwater Technical Guide*. The modified-rational method was used to estimate the post-development peak flows for the 1:2-year through 1:100-year return frequency design storms.

The post-development condition peak flows from the site at Outlet #1 are summarized in Table 6.

Refer to Appendix G for supporting calculations.

6.5 QUANTITY CONTROL

Post-development peak flow rates are to be controlled to pre-development levels for the 1:2-year to 1:100-year design storms at any given outlet location to ensure the proposed development does not have adverse impacts on drainage systems downstream of the site.

Water quantity control for the proposed development will be provided by an underground storage tank located within the proposed amenity area. Based on the modified rational method 1,450 m^3 of active storage is required to reduce the post-development peak flows to pre-development peak flows.

The underground storage facility will outlet to a proposed storm sewer. The proposed storm sewer will be extended externally from the site along County Road 9 to an outlet at the Mad River tributary (County Road 9 north roadside ditch, east of the Fairgrounds Road South intersection) and sized to convey the 1:5-year design storm.

The tank has been sized to overcontrol post-development peak flow rates to the pre-development 5-year peak flow rate (0.413 m^3/s) or less.

The preliminary design of the tank is based on a hydrostar HS290 stormwater chamber system complete with 0.6 m arches and minimum 0.15 m of stone above the arches for additional storage. The tank will be placed on a minimum of 0.33 m of stone. Peak flow attenuation will be provided by an orifice control installed downstream of the storage system. The size of the orifice will be confirmed in support of detailed design.

The estimated groundwater depth at the tank location is 4.83 mbgs allowing for and open bottom design to promote exfiltration. The storage volume required to control peak flows to the desired rate will be provided within the system above the invert of the outlet pipe. Volume within the 0.33 m layer of stone beneath the unit has not been considered as quantity storage.

The MECP recognizes for infill developments up to 6 lots, stormwater management controls may not be feasible due to space constraints and small development areas, and therefore, may be exempt from controls. The single detached lots fronting County Road 9 will be graded rear-to-front, and uncontrolled runoff will drain uncontrolled to County Road 9. However, the underground storage tank proposed within the condominium development has been oversized to account for this uncontrolled drainage area and provides peak flow attenuation and volume control such the downstream outlet experiences no increase in peak flow rate under post-development conditions.

Table 6: Post-Development Conditions Peak Flow Summary

DESIGN STORMS	OUTLET #1 PEAK FLOW (m^3/s)
1:2-year	0.312 (*0.312*)
1:5-year	0.413 (*0.413*)
1:10-year	0.413 (*0.481)*
1:25-year	0.413 (0.622)
1:50-year	0.413 (0.753)
1:100-year	0.413 (0.864)

Note: Values in *(italics)* denote existing conditions

Detailed design of the orifice control and underground tank will be completed in support of detailed design.

Refer to Appendix F for detailed calculations.

6.6 WATER QUALITY

"Enhanced" Level 1 water quality control (corresponding to 80% TSS removal) is required for the proposed development. Water quality controls are not required for rooftop areas as runoff generated from these surfaces are generally considered to be clean.

Water quality will be controlled via two DM-OS OGS units located directly upstream of the two inlets to the underground storage tank, and a sediment debris row located downstream of each inlet within the underground storage tank. Each OGS unit will be sized to a treatment efficiency of 60% TSS removal for the associated drainage area. The sediment debris rows will have a TSS removal efficiency of 84%. This treatment train approach will have a combined TSS removal efficiency of 94%; satisfying the "Enhanced" Level 1 water quality requirement.

Refer to Appendix G OGS sizing and treatment train calculations.

6.7 VOLUMETRIC CONTROL

In accordance with the *NVCA Stormwater Technical Guide*, the first 5 mm of runoff shall be infiltrated on site. Based on the current Conceptual Site Plan 151 m^3 of runoff will be required to be retained. Volume retention will be provided within 0.33 m layer of stone beneath the underground storage tank. The clear stone layer will provide a volume of 152.3 m^3.

Detailed drawings of the underground tank and volume controls will be prepared in support of detailed design.



6.8 MAJOR AND MINOR FLOW CONVEYANCE

Major and minor flows from Catchments 202, and 203 will be conveyed to Outlet #1 via overland flow routes.

Minor flows from Catchment 201 and 204 will be conveyed to the proposed underground storage tank via a network of storm sewers. Major flows from Catchments 201 and 204 will be conveyed to the proposed underground storage tank via overland flow routes. From the tank, runoff will be conveyed through a run of storm sewers ultimately discharging to Outlet #1.

Refer to Appendix E for the Preliminary Site Servicing Plan (Drawing SS.1).

The detailed design of the external County Road 9 storm sewer will be completed in support of the detailed design.

6.9 WATER BALANCE

A preliminary water balance has been completed by Tatham as part of a Hydrogeological Assessment. The purpose of the water balance is to assess the potential impact of the development on local groundwater and surface water resources. The annual pre-development infiltration volume for the site was calculated to be 6,074 m^3/year. The annual post-development infiltration volume for the site was calculated to be 2,220 m^3/year, resulting in an infiltration deficit of approximately 3,854 m^3/year, which is equivalent to the 12.8 mm storm event across the site. Additional details regarding the water balance are provided in the *Hydrogeological Assessment* (Tatham, July 2025).

The post-development condition infiltration deficit is to be offset via infiltration from the underground storage tank as discussed above in Section 5.8. The underground tank is to incorporate an open-bottom design and be placed on a minimum 0.33 m depth clear stone bed. The volume within the clear stone has been sized to the equivalent of the 8.9 mm storm event across the LID's 1.72 ha impervious drainage area (Catchment 201) and provides a total annual infiltration volume of 8,105 m^3. Therefore, under post-development conditions there will be an annual increase (surplus) in infiltration of 4,250 m^3.

Further review of the proposed infiltration measures will occur at detailed design.

6.10 PHOSPHORUS TREATMENT AND MITIGATION

In accordance with NVCA requirements, post development phosphorous is required to be mitigated to pre-development levels.

6.10.1 Phosphorous Loading – Existing Condition

Under existing conditions, the site has been modelled as woodland, hay-pasture, and low intensity development with phosphorous loading rates of 0.06 kg/ha/year, 0.08 kg/ha/year, and 0.13 kg/ha/year respectively.

Under existing conditions, 3.02 ha of area drains to the Mad River tributary (Outlet #1), resulting in a phosphorous loading rate of 0.22 kg/year.

6.10.2 Phosphorous Loading – Post-development Condition Without Mitigation

Under post-development conditions, the central condominium development has been modelled as high intensity development – residential, and the infill lots on County Road 9 and Collingwood Street have been modelled as low intensity development with phosphorous loading rates of 0.41 kg/ha/year, and 0.13 kg/ha/year respectively.

Under post-development conditions a total of 3.02 ha of area drains to a tributary of the Mad River (Outlet #1), resulting in a loading rate of 1.06 kg/year with no mitigation.

6.10.3 Phosphorous Loading – Post-Development Condition with Mitigation

A treatment train approach is proposed to mitigate the post-development phosphorous loading. This is proposed to occur via two DM-OS OGS units located upstream of the underground storage tank as well as via the sediment debris rows and runoff infiltration within the storage tank itself. The following controls and their respective phosphorous removal rates utilized are:

- DM-OS OGS Unit = 20% removal efficiency; and
- HS290 (underground storage tank) Infiltration = 60% removal efficiency.

The proposed treatment train results in a total removal efficiency of 68% of the phosphorous under post-development conditions, reducing the total post-development load to 0.40 kg/year. Relevant certifications for the proposed phosphorus controls are included in Appendix F for reference.

A summary of the completed phosphorous assessment is shown below in Table 7.

Table 7: Phosphorous Loading Summary

SCENARIO	AREA (ha)	PHOSPHORUS LOADING (kg/year)
Pre-Development	3.02	0.22
Post-Development (Without Controls)	3.02	1.06

SCENARIO	AREA (ha)	PHOSPHORUS LOADING (kg/year)
Post-Development (With Controls)	3.02	0.40
Overall Increase in Phosphorous Load	**3.02**	**0.18**

Review of the proposed design, and further assessment of additional phosphorous treatment controls will be completed in support of detailed design.

7 Grading

7.1 GRADING

7.1.1 Road Grading

Internal road grading has been designed to ensure stormwater runoff is collected by the proposed storm sewers and will be conveyed to sufficient outlets via overland flow routes. The proposed road grading will also provide sufficient overland flow routes contained within the road allowances.

7.1.2 Lot Grading

Lot grading is designed to provide positive drainage away from the dwellings and will be in accordance with Township engineering standards.

Due to the variable topography across the site, and steep conditions at the northern property, retaining walls are proposed along the rear of the existing lots along County Road 9, and adjacent to the proposed lots along Collingwood Street in the north-west corner of the site.

The preliminary site grading design is depicted on the Preliminary Site Grading Plan (Drawing SG.1) enclosed in Appendix E for reference.

8 Erosion and Sediment Control

Erosion and sediment control measures will be implemented for all construction activities within the development site including vegetation clearing, topsoil stripping, stockpiling of materials, site access construction, grading and servicing. The basic principles considered to minimize erosion and sedimentation and the potential negative environmental impacts include:

- minimize disturbance activities where possible;
- expose the smallest possible land area to erosion for the shortest amount of time;
- institute erosion control measures as required immediately;
- implement sediment control measures before the outset of construction activities; and
- carry out regular inspection of erosion/sediment control measures and repair or maintain them, as necessary.

Erosion and sediment control measures shall be implemented in accordance with the *Erosion & Sediment Control Best Management Practices Guide* and are to include the following:

- sediment control fence;
- construction access mat(s);
- heavy-duty silt fence surrounding stripping and material stockpile areas;
- catch basin filter screens; and
- sediment traps placed in all existing and proposed catch basins adjacent to the site.

Regular inspection of control measures will be completed through a monitoring and mitigation plan, with regular repairs made as necessary. An erosion and sediment control plan will be developed during the detailed design stage.

9 Utilities

The following utility agencies provide services to the proposed development:

- HydroOne/Elexicon Energy;
- Enbridge;
- Bell Canada; and
- Rogers Communication Inc.

All utilities (electrical, gas and telecommunications) are expected to be available to service the proposed development. Utility coordination will be initiated at the detailed design stage.

10 Summary

10.1 WATER SERVICING

All proposed single-detached lots fronting Collingwood Street and County Road 9 would be serviced by new individual 25 mm dia. watermain services. Within the condominium development, each townhouse unit will be provided with an individual 25 mm dia. water service. The preliminary sizing of the watermain to service the condominium site is estimated to be a 200 mm dia. watermain extended from the existing 300 mm dia. watermain within County Road 9, feeding a looped 150 mm dia. main within the site. Ultimate sizing will be confirmed through computational modelling during detailed design.

10.2 SANITARY SERVICING

The proposed single detached lots along County Road 9 will be serviced by new individual service laterals to the existing 200 mm dia. sanitary sewer within County Road 9; with the exception of Lot #7, which proposes to re-use the existing 125 mm dia. sanitary connection to the property.

The proposed single detached lots along Collingwood Street will be serviced by new individual service laterals once a sanitary sewer is constructed along Collingwood Street.

The proposed condominium development will be serviced via a new 150 mm dia. sanitary sewer extending from the existing main within County Road 9 to service the development. Each townhouse unit is proposed to have an individual service connection from the new sewer.

10.3 DRAINAGE & STORMWATER MANAGEMENT

The preliminary SWM plan has been designed to mitigate negative impacts with respect to the SWM objectives and guidelines of the approving agencies. Water quantity control for the proposed development will be provided by an underground storage tank. Water quality and phosphorous is to be controlled via two OGS units as well as through the sediment debris rows and infiltration within the underground storage tank. Volume control requirements and water balance mitigation is to be provided by infiltration within the underground storage tank. The proposed hydrology and SWM design will be refined during the detailed design stage. A new storm sewer within County Road 9 will be required to convey flows from the development and is proposed to discharge into the north roadside ditch within County Road 9, approximately 230 m east of the site.

10.4 SILTATION & EROSION CONTROL PLAN

Siltation and erosion controls will be provided with the proper construction mitigation efforts.



10.5 GRADING & LANDSCAPING

The grading of the proposed development will match existing perimeter grades along the limits of the development and generally reflect pre-existing drainage splits. Retaining walls are required in some areas to promote desirable lot grades as well as avoid large slopes. Review of the grading design will occur during detailed design.



7582 County Road 9, Creemore

Figure 1: Site Location Plan

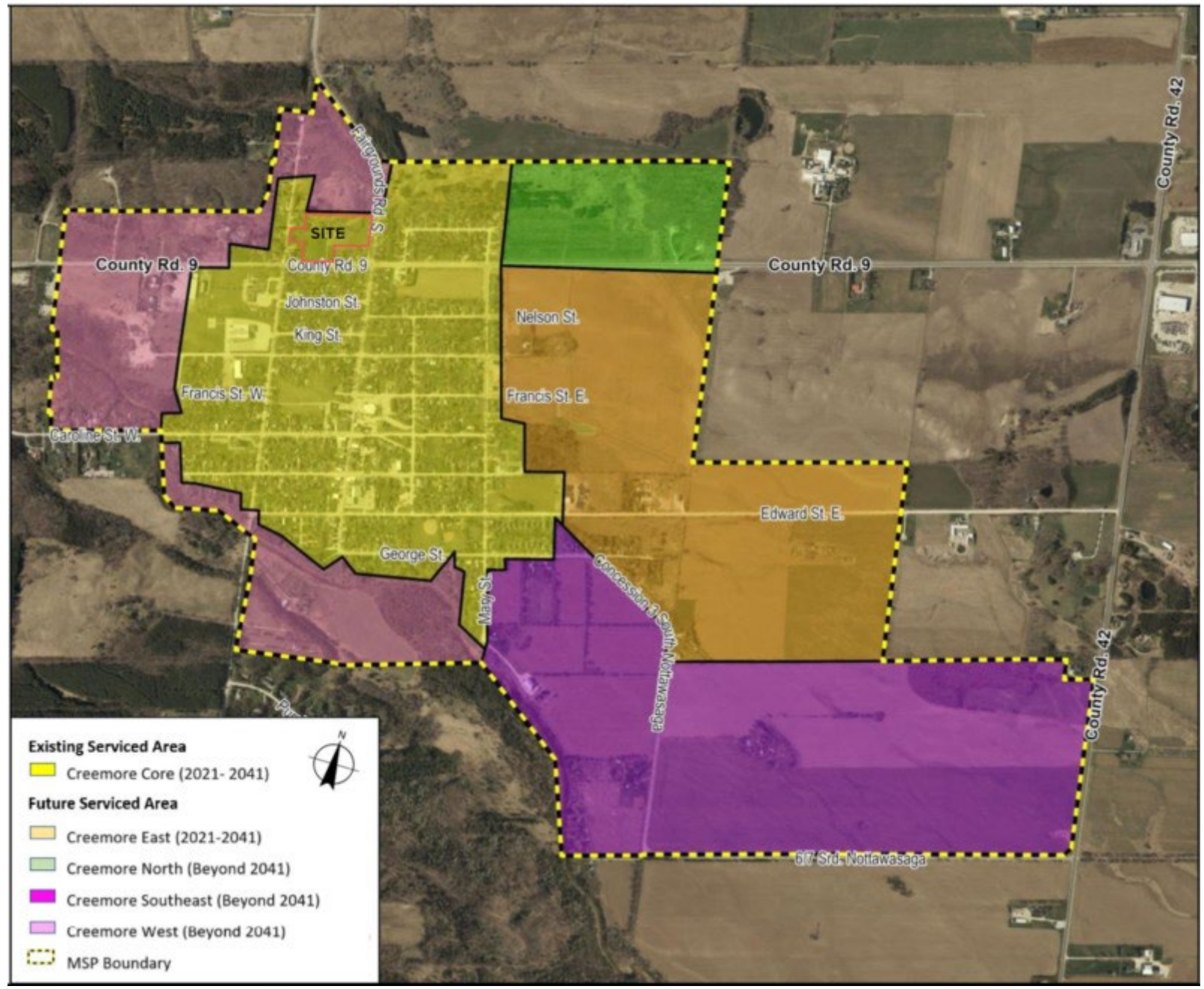
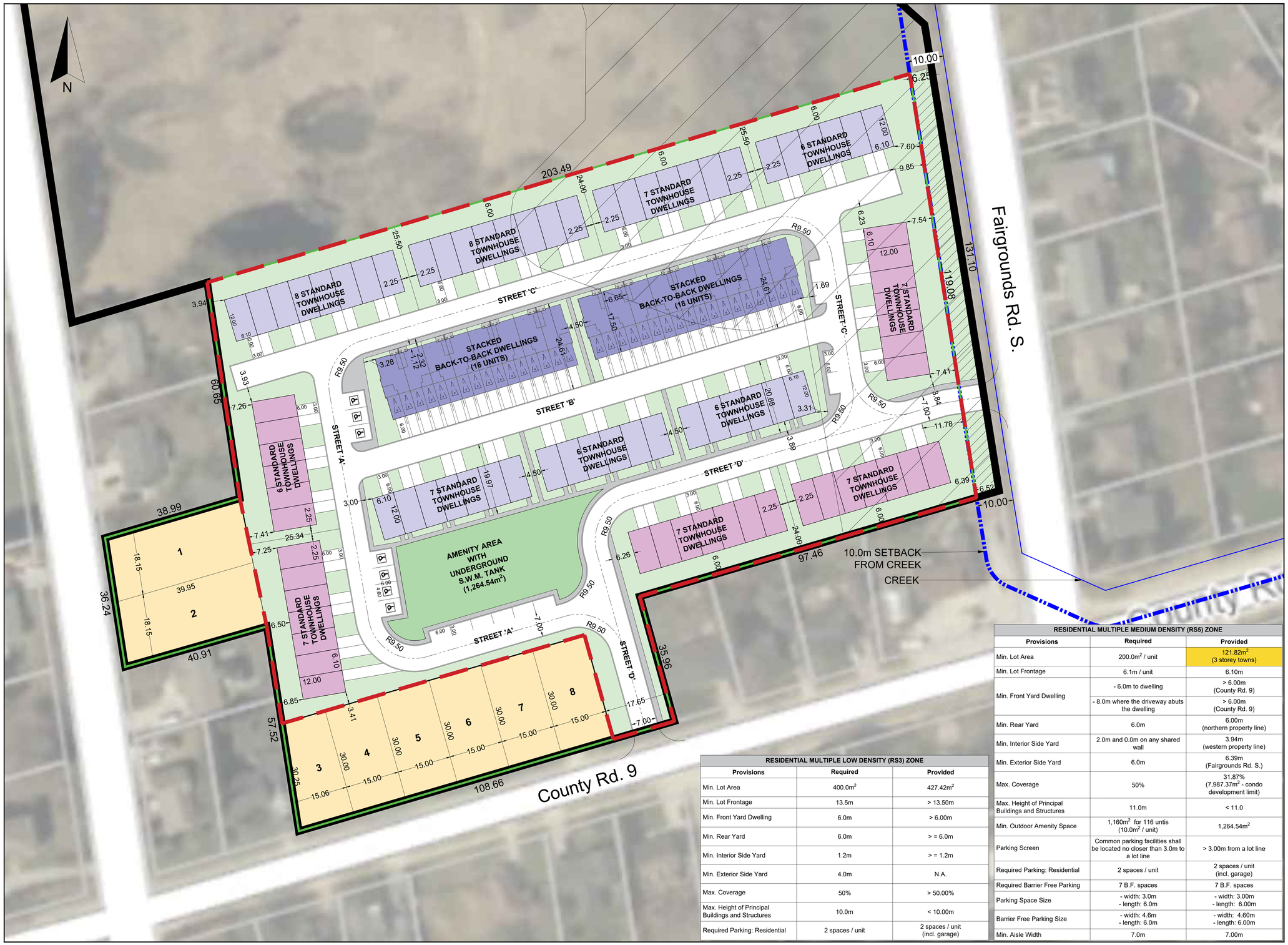
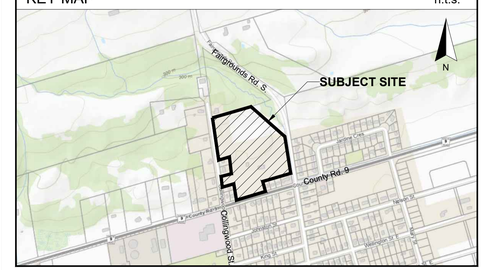


Figure 2: Creemore Service Area Map and MSP Boundary (Excerpt from Creemore Water and Wastewater Master Servicing Plan (May 2024))

Appendix A:
Conceptual Site Plan



KEY MAP n.t.s.



CONCEPTUAL SITE PLAN

Scale

0 10 20 30 40 50m

LEGEND

Ownership Area:
- Area: 60,497.60m² / 6.05ha (County of Simcoe Interactive Mapping)

Development Limit
- Area: 29,185.29m² / 2.91ha
- Density: 42.61 units / ha

Condo Development Limit
- Area: 25,066.15m²/ 2.51ha
- Density: 46.22 units / ha

Lands to be be Conveyed to N.V.C.A.
Area: 760.00m²

Single Detached - 8 lots

2 Storey Standard Townhouse Dwellings - 34 units
- Unit Size: 6.10m x 12.00m
- Unit G.F.A.: 128.40m² (excl. garage)

3 Storey Standard Townhouse Dwellings - 48 units
- Unit Size: 6.10m x 12.0m
- Unit G.F.A.: 201.60m² (excl. garage)

3 Storey Stacked Back-To-Back Townhouse Dwellings - 34 units
- Dwelling Size: 6.65m x 17.50m
- Unit G.F.A.: 155.57m² / unit (excl. garage)

Amenity Area (1,264.54m²)

Fire Route

N.V.C.A. Reg. Limit

Total Parking: 249 spaces (excl. single detached)
- 17 visitor parking spaces incl. 7 B.F. spaces
- 232 spaces (incl. garage)

RESIDENTIAL MULTIPLE LOW DENSITY (RS3) ZONE

Provisions	Required	Provided
Min. Lot Area	400.0m²	427.42m²
Min. Lot Frontage	13.5m	> 13.50m
Min. Front Yard Dwelling	6.0m	> 6.00m
Min. Rear Yard	6.0m	> = 6.0m
Min. Interior Side Yard	1.2m	> = 1.2m
Min. Exterior Side Yard	4.0m	N.A.
Max. Coverage	50%	> 50.00%
Max. Height of Principal Buildings and Structures	10.0m	< 10.00m
Required Parking: Residential	2 spaces / unit	2 spaces / unit (incl. garage)

RESIDENTIAL MULTIPLE MEDIUM DENSITY (RS6) ZONE

Provisions	Required	Provided
Min. Lot Area	200.0m² / unit	121.82m² (3 storey towns)
Min. Lot Frontage	6.1m / unit	6.10m
Min. Front Yard Dwelling	- 6.0m to dwelling	> 6.00m (County Rd. 9)
	- 8.0m where the driveway abuts the dwelling	> 6.00m (County Rd. 9)
Min. Rear Yard	6.0m	6.00m (northern property line)
Min. Interior Side Yard	2.0m and 0.0m on any shared wall	3.94m (western property line)
Min. Exterior Side Yard	6.0m	6.39m (Fairgrounds Rd. S.)
Max. Coverage	50%	31.87% (7,987.37m² - condo development limit)
Max. Height of Principal Buildings and Structures	11.0m	< 11.0
Min. Outdoor Amenity Space	1,160m² for 116 untis (10.0m² / unit)	1,264.54m²
Parking Screen	Common parking facilities shall be located no closer than 3.0m to a lot line	> 3.00m from a lot line
Required Parking: Residential	2 spaces / unit	2 spaces / unit (incl. garage)
Required Barrier Free Parking	7 B.F. spaces	7 B.F. spaces
Parking Space Size	- width: 3.0m - length: 6.0m	- width: 3.00m - length: 6.00m
Barrier Free Parking Size	- width: 4.6m - length: 6.0m	- width: 4.60m - length: 6.00m
Min. Aisle Width	7.0m	7.00m

Source: Township of Clearview Zoning By-Law
County of Simcoe Interactive Mapping

Note: Information shown is approximate and subject to change.

CONCEPTUAL SITE PLAN - 124 UNITS

267 COLLINGWOOD ST., CREEMORE

SCHEDULE OF REVISIONS

No.	Date	Description	By

IPS INNOVATIVE PLANNING SOLUTIONS
PLANNERS • PROJECT MANAGERS • LAND DEVELOPERS
647 WELHAM ROAD, UNIT 9, BARRIE, ON, L4N 0B7
tel: 705 • 812 • 3281 fax: 705 • 812 • 3438 e: info@ipsconsultinginc.com www.ipsconsultinginc.com

Date:	March 6, 2025	Drawn By:	A.S.
File:	23 - 1297	Checked:	K.B.

Appendix B:
Excerpts from MSP

TATHAM ENGINEERING	PROJECT 7582 County Road 9	FILE	424350
		DATE	3/20/2025
	SUBJECT Water Demand Calculations	NAME	DB
		PAGE	1 OF 2

1.1 Demand Criteria

Design Criteria as per Township of Clearview *Engineering Standards* (2016)

Per Capita Demand = 350 L/cap/day

Residential Population Density = 2.8 PPU

Site Information as per *Conceptual Site Plan* by IPS (dated March 6, 2025)

[1]Fire Flow value carried from MSP - refer to Section 3.2.3 of FSR.

Peaking Factors

Residential Maximum Day Factor* = 2.25

Peak Hour Factor = 3.38

*From Table 3-1 of MECP's Design Guidelines for Drinking-Water Systems

1.2 Entire Development - Water Demands

Standard & Stacked Towns = 124 units

Population (rounded up) = 348 ppl

Average Day Demand (ADD)	=	121,800	L/day	=	121.80	m^3/day	=	1.41 L/s
Maximum Day Demand (MDD)	=	121,800	L/day	x	2.25			
	=	274,050	L/day	=	274.05	m^3/day	=	3.17 L/s
MDD + Fire Flow (FF)[1]	=	3.17	L/s	+	95.00	L/s	=	98.17 L/s
Peak Hour Demand (PHD)	=	121,800	L/day	x	3.38			
	=	411,684	L/day	=	411.68	m^3/day	=	4.77 L/s

1.3 Proposed Condominium Development - Water Demands

Stacked/Standard Townhouses = 116 units Population (rounded up) = 325 ppl

Average Day Demand (ADD)	=	113,750	L/day	=	113.75	m^3/day	=	1.32	L/s
Maximum Day Demand (MDD)	=	113,750	L/day	x	2.25				
	=	255,938	L/day	=	255.94	m^3/day	=	2.96	L/s
MDD + Fire Flow (FF)[1]	=	2.96	L/s	+	95.00	L/s	=	97.96	L/s
Peak Hour Demand (PHD)	=	113,750	L/day	x	3.38				
	=	384,475	L/day	=	384.48	m^3/day	=	4.45	L/s

1.3 Proposed County Road 9 Lots - Water Demands

Single Detached Units = 6 units Population (rounded up) = 17 ppl

Average Day Demand (ADD)	=	5,950	L/day	=	5.95	m^3/day	=	0.07	L/s
Maximum Day Demand (MDD)	=	5,950	L/day	x	2.25				
	=	13,388	L/day	=	13.39	m^3/day	=	0.15	L/s
MDD + Fire Flow (FF)[1]	=	0.15	L/s	+	95.00	L/s	=	95.15	L/s
Peak Hour Demand (PHD)	=	5,950	L/day	x	3.38				
	=	20,111	L/day	=	20.11	m^3/day	=	0.24	L/s

1.3 Proposed Collingwood Street Lots - Water Demands

Single Detached Units = 2 units Population (rounded up) = 6 ppl

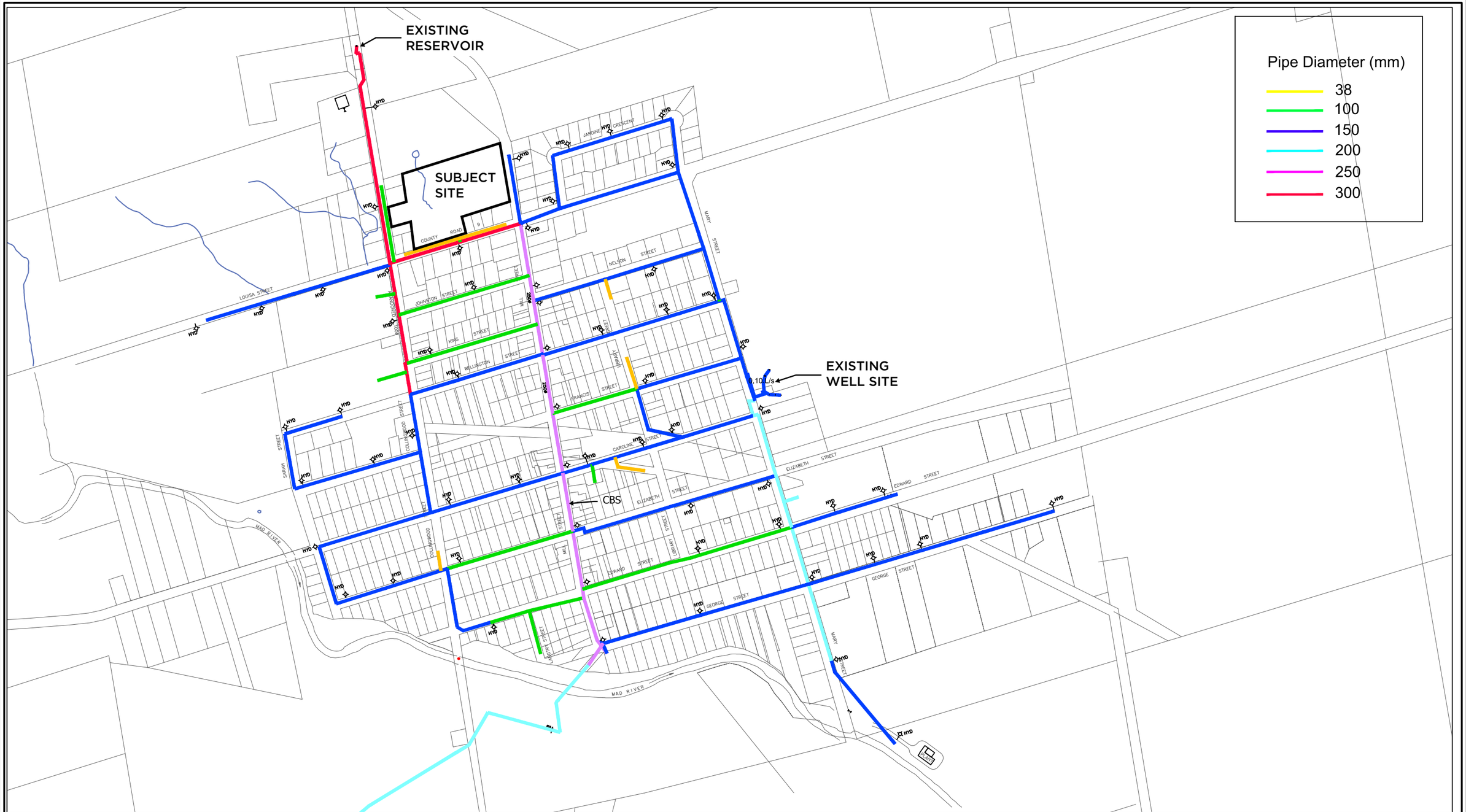
Average Day Demand (ADD)	=	2,100	L/day	=	2.10	m^3/day	=	0.03	L/s
Maximum Day Demand (MDD)	=	2,100	L/day	x	2.25				
	=	4,725	L/day	=	4.73	m^3/day	=	0.05	L/s
MDD + Fire Flow (FF)[1]	=	0.05	L/s	+	95.00	L/s	=	95.05	L/s
Peak Hour Demand (PHD)	=	2,100	L/day	x	3.38				
	=	7,098	L/day	=	7.10	m^3/day	=	0.09	L/s

BURNSIDE

[THE DIFFERENCE IS OUR PEOPLE]

Appendix A

Figures



BURNSIDE

Figure Title

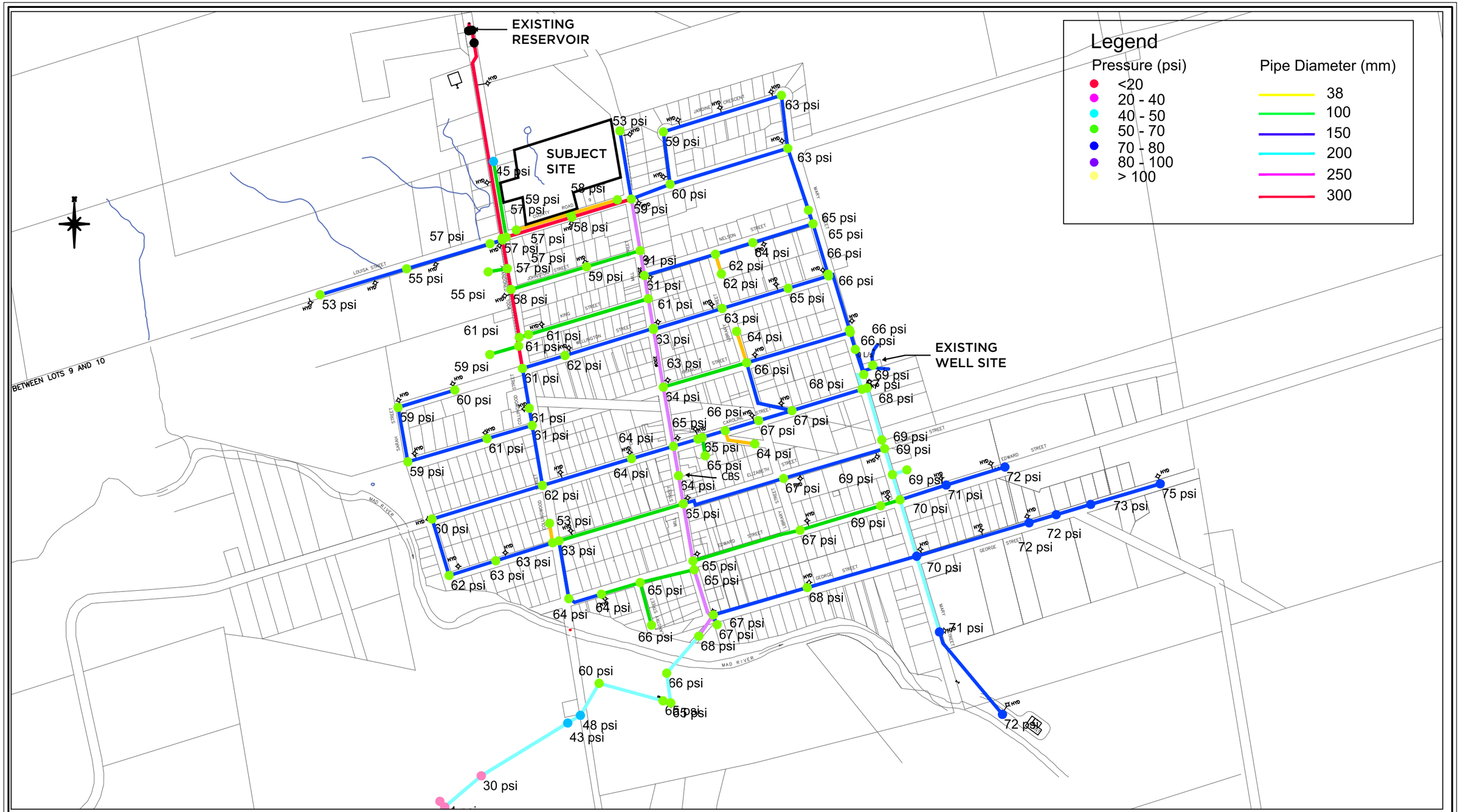
CREEMORE
WATER DISTRIBUTION MODEL
EXISTING SYSTEM

Client

TOWNSHIP OF CLEARVIEW

Drawn	Checked	Date	Figure No.
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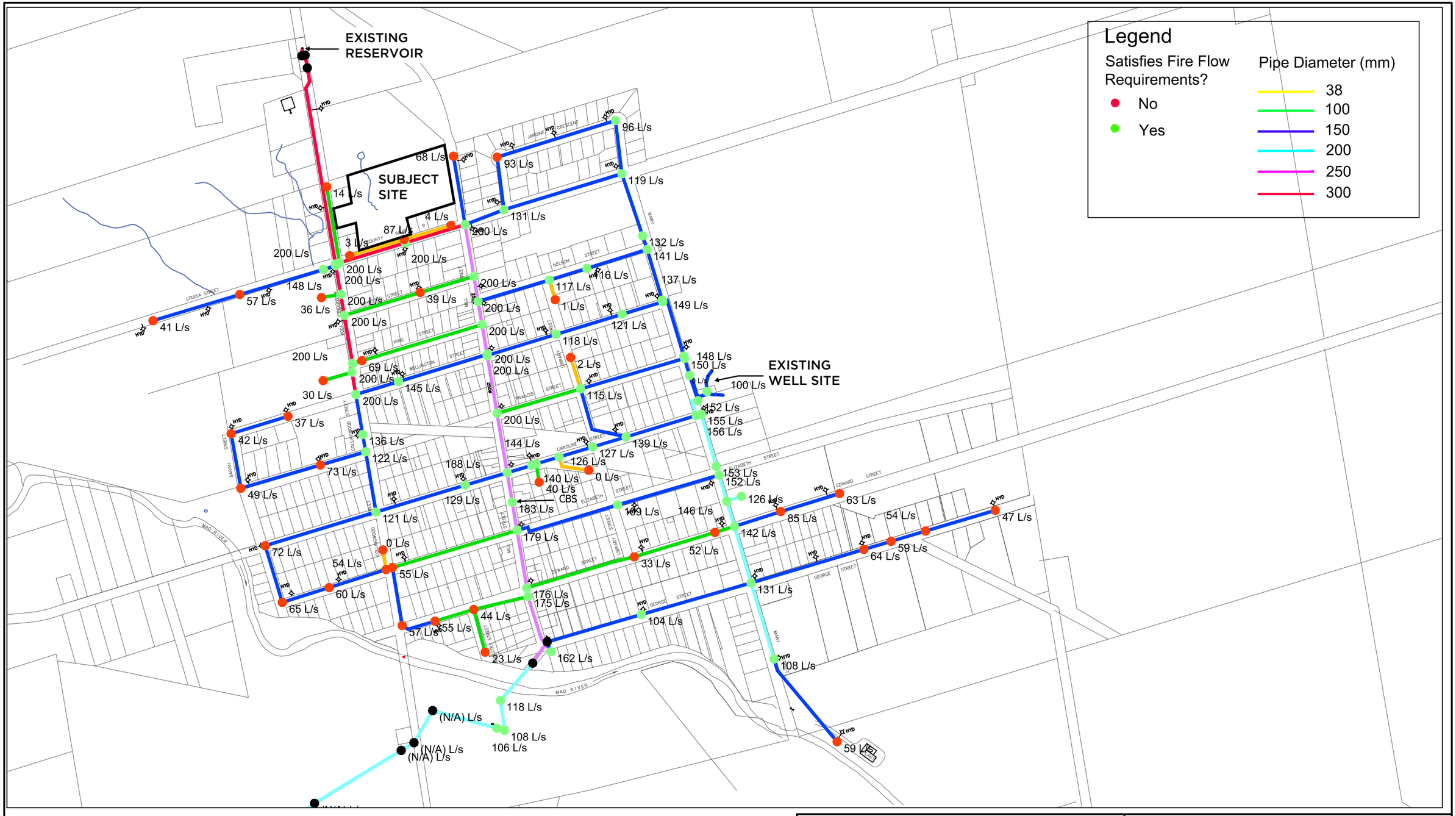
CREEMORE
WATER DISTRIBUTION MODEL
EXISTING MAXIMUM DAY DEMAND

Client

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Figure Title

CREEMORE WATER DISTRIBUTION SYSTEM MODEL

EXISTING FIRE FLOW + MDD

Client

TOWNSHIP OF CLEARVIEW

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EXISTING RESERVOIR

SUBJECT SITE

CE-1

CE-2

CE-3

EXISTING WELL SITE

Legend

Pressure (psi)

- <20
- 20 - 40
- 40 - 50
- 50 - 70
- 70 - 80
- 80 - 100
- > 100
- PROPOSED UPGRADES

Pipe Diameter (mm)

- 38
- 100
- 150
- 200
- 250
- 300

N

0 100 200 400 600

Metres

BURNSIDE

Figure Title

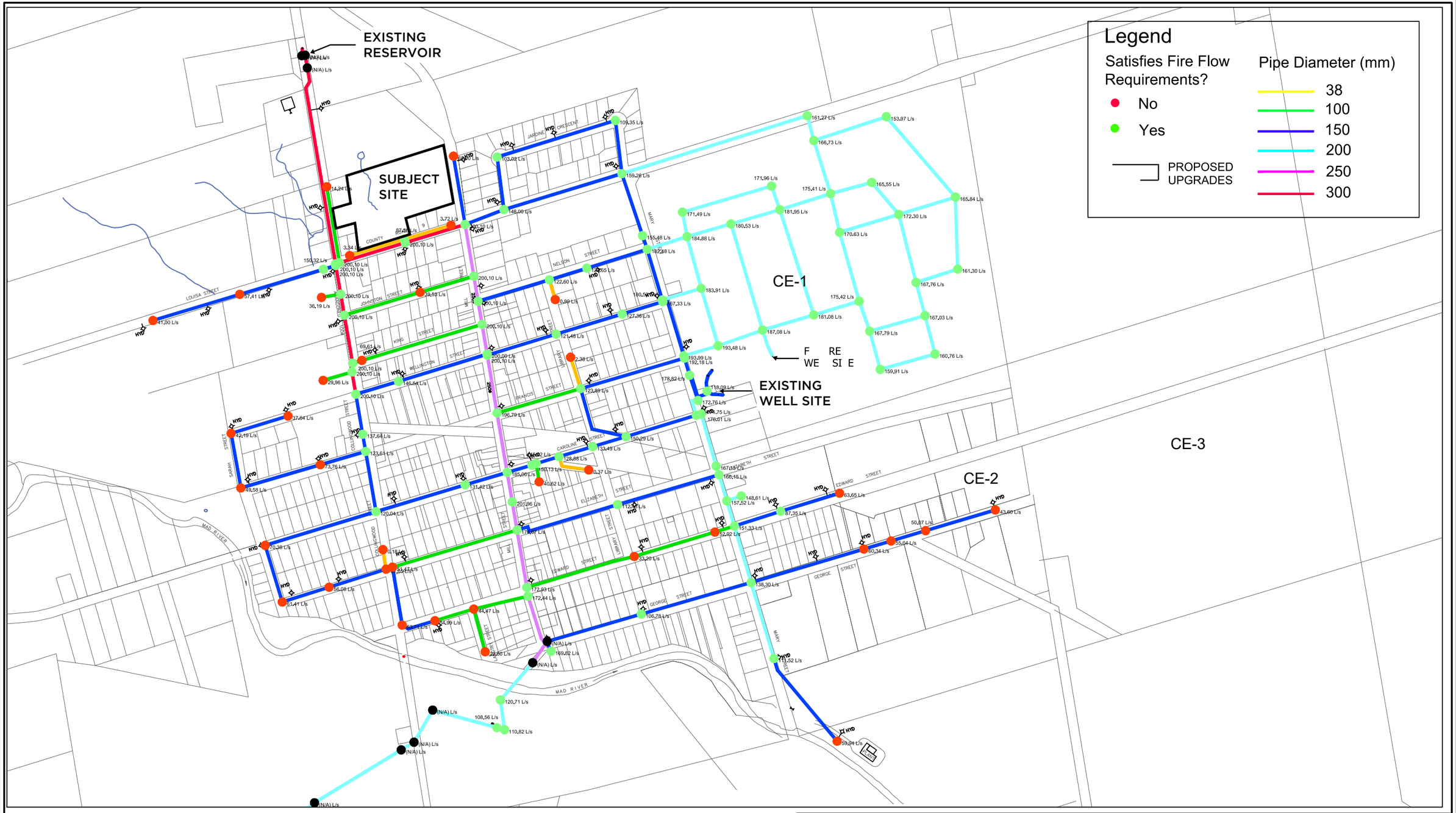
CREEMORE
WATER DISTRIBUTION MODEL
PROPOSED MAXIMUM DAY DEMAND
CE-1

Client

TOWNSHIP OF CLEARVIEW

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File: \\collingwood\Shared Work Areas\water and sewer modeling\WaterCADD-Municipal\Creemore\240328 Creemore Water Model.dwg Date Plotted: April 10, 2024 - 2:22 PM



BURNSIDE

Client

TOWNSHIP OF CLEARVIEW

Figure Title

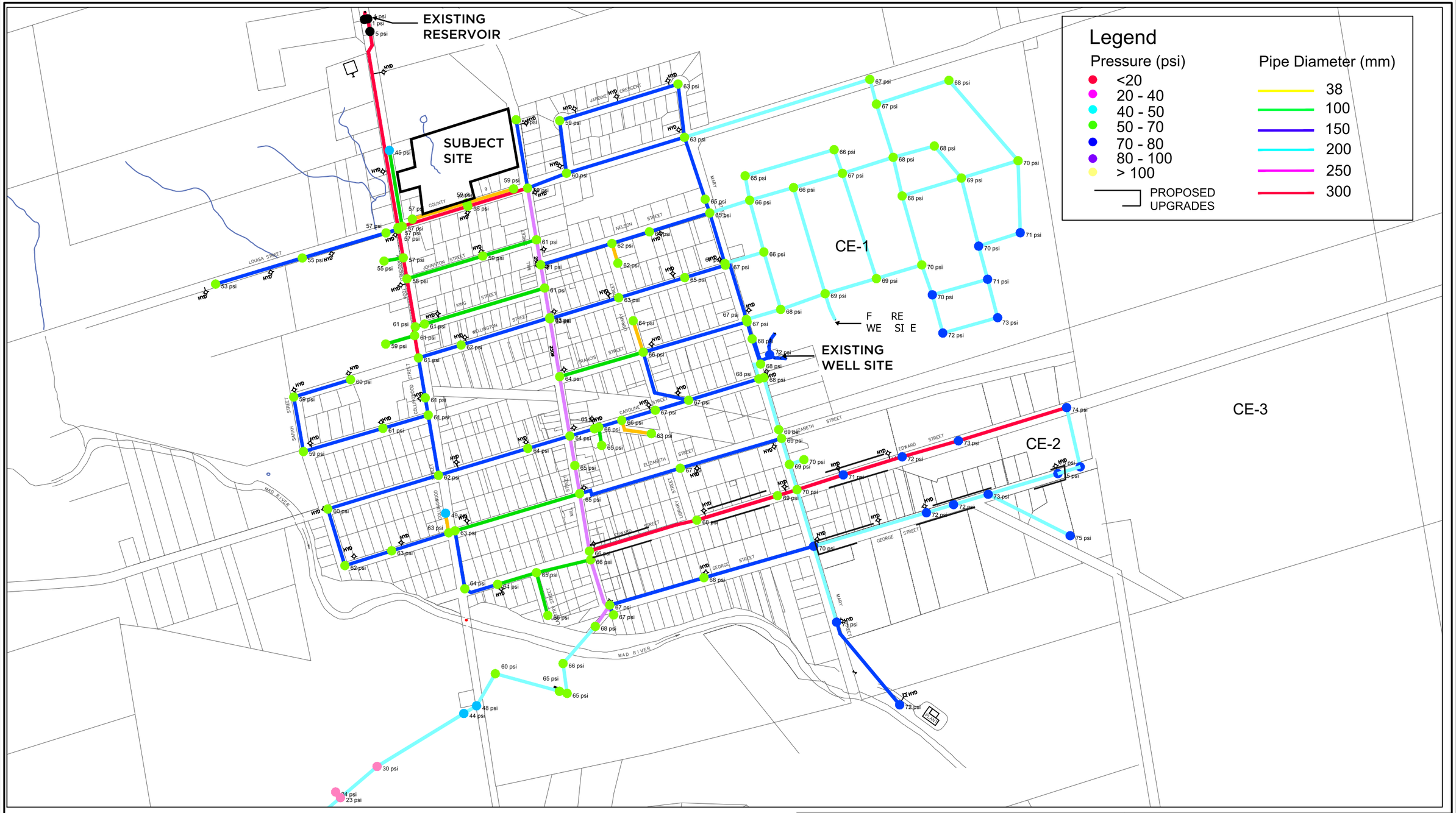
CREEMORE WATER DISTRIBUTION SYSTEM MODEL

PROPOSED FIRE FLOW + MDD

CE-1

Drawn	Checked	Date	Figure No.
RW	JLL	APRIL 2024	5
Scale		Project No.	
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BURNSIDE

Client

TOWNSHIP OF CLEARVIEW

Figure Title

CREEMORE
WATER DISTRIBUTION MODEL
PROPOSED MAXIMUM DAY DEMAND
CE-1 & CE-2

Drawn	Checked	Date	Figure No.
RW	JLL	APRIL 2024	6
Scale		Project No.	
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EXISTING RESERVOIR

SUBJECT SITE

CE-1

CE-2

CE-3

EXISTING WELL SITE

Legend

Satisfies Fire Flow Requirements?

- No
- Yes

PROPOSED UPGRADES

Pipe Diameter (mm)

- 38
- 100
- 150
- 200
- 250
- 300

N

0 100 200 400 600

Metres

BURNSIDE

Figure Title

CREEMORE WATER DISTRIBUTION SYSTEM MODEL

PROPOSED FIRE FLOW + MDD

CE-1 & CE-2

Client

TOWNSHIP OF CLEARVIEW

Drawn	Checked	Date	Figure No.
RW	JLL	APRIL 2024	7
Scale		Project No.	
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File: \\collingwood\Shared Work Areas\water and sewer modeling\WaterCADD-Municipal\Creemore\240328 Creemore Water Model.dwg Date Plotted: April 10, 2024 - 2:29 PM

EXISTING RESERVOIR

SUBJECT SITE

EXISTING WELL SITE

CE-1

CE-2

CE-3

Legend

Pressure (psi)

- <20
- 20 - 40
- 40 - 50
- 50 - 70
- 70 - 80
- 80 - 100
- > 100
- PROPOSED UPGRADES

Pipe Diameter (mm)

- 38
- 100
- 150
- 200
- 250
- 300

N

0 100 200 400 600

Metres

BURNSIDE

Client

TOWNSHIP OF CLEARVIEW

Figure Title

CREEMORE
WATER DISTRIBUTION MODEL
PROPOSED MAXIMUM DAY DEMAND
CE-1, CE-2 & CE-3

Drawn	Checked	Date	Figure No.
RW	JLL	APRIL 2024	8
Scale		Project No.	
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File: \\collingwood\Shared Work Areas\water and sewer modeling\WaterCADD-Municipal\Creemore\240328 Creemore Water Model.dwg Date Plotted: April 10, 2024 - 2:44 PM

EXISTING RESERVOIR

SUBJECT SITE

CE-1

CE-2

CE-3

EXISTING WELL SITE

Legend

Satisfies Fire Flow Requirements?

- No
- Yes

PROPOSED UPGRADES

Pipe Diameter (mm)

- 38
- 100
- 150
- 200
- 250
- 300

N

0 100 200 400 600

Metres

BURNSIDE

Client

TOWNSHIP OF CLEARVIEW

Figure Title

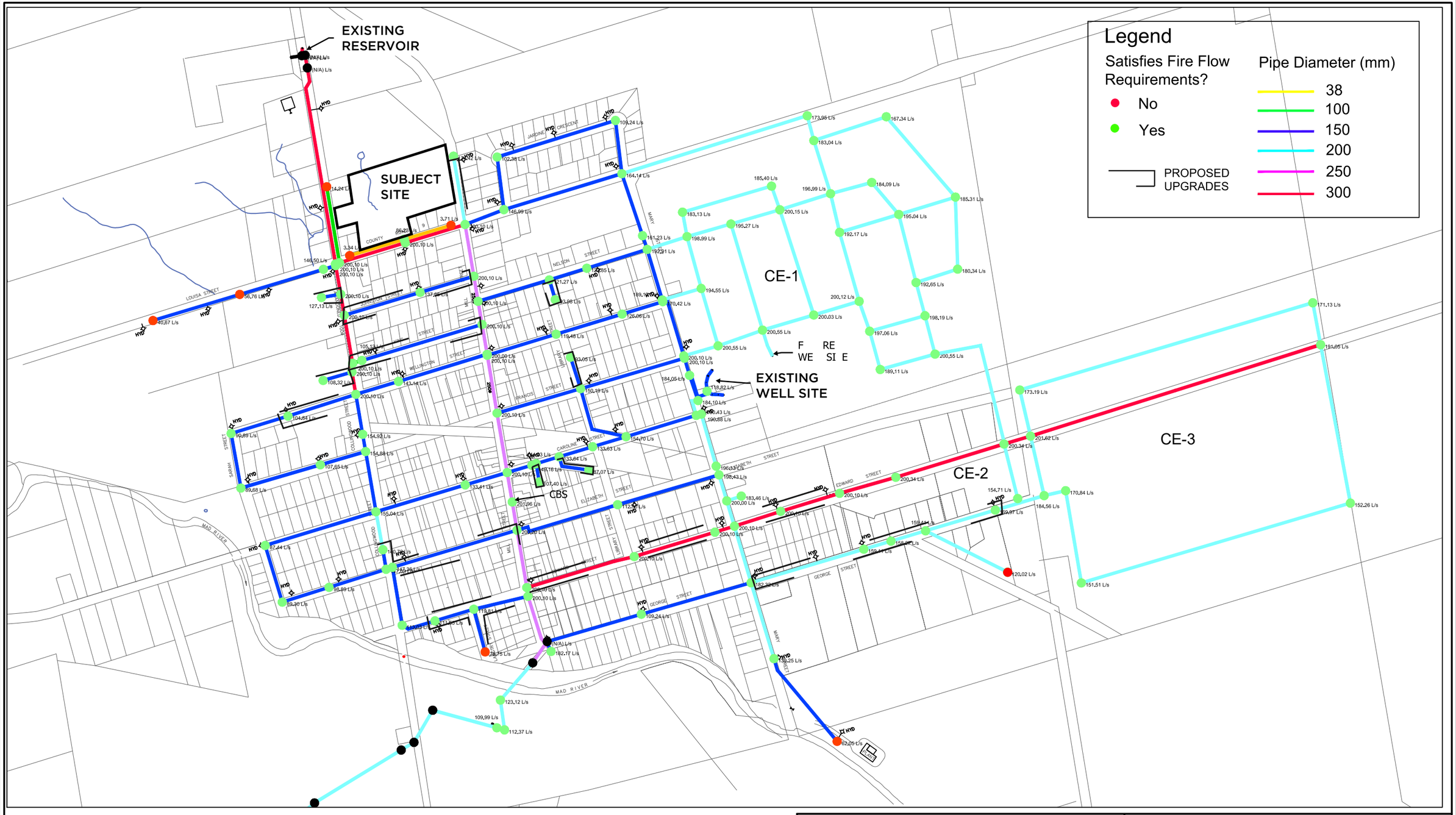
CREEMORE WATER DISTRIBUTION SYSTEM MODEL

PROPOSED FIRE FLOW + MDD

CE-1, CE-2 & CE-3

Drawn	Checked	Date	Figure No.
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Scale		Project No.	
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File: \\collingwood\Shared Work Areas\water and sewer modeling\WaterCADD-Municipal\Creemore\240328 Creemore Water Model.dwg Date Plotted: April 10, 2024 - 2:43 PM



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Client

TOWNSHIP OF CLEARVIEW

Figure Title

CREEMORE WATER DISTRIBUTION SYSTEM MODEL

PROPOSED FIRE FLOW + MDD UPGRADES TO EXISTING SYSTEM

Drawn	Checked	Date	Figure No.
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Scale		Project No.	
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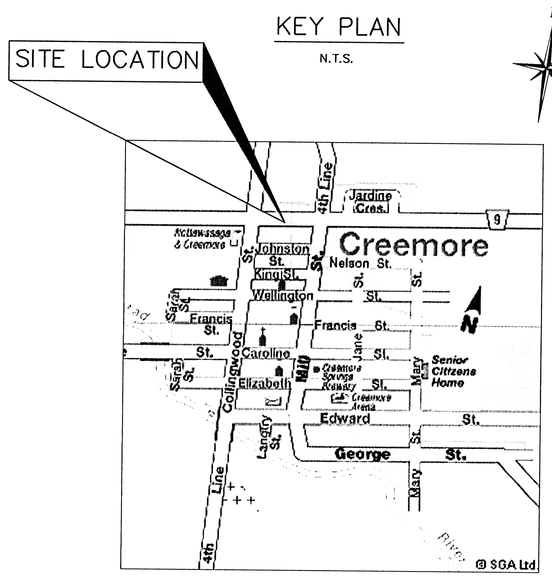
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R.J. Burnside & Associates Limited

TOWNSHIP OF CLEARVIEW
COUNTY of SIMCOE
COUNTY ROAD 9, CREEMORE
WATERMAIN REPLACEMENT

KEY PLAN

N.T.S.



MUNICIPALITY

CLEARVIEW TOWNSHIP
Established 1994

TOWNSHIP of CLEARVIEW
217 GIDEON ST.
STAYNER, ONTARIO
LOM 1S0

PROJECT ENGINEER

BURNSIDE

R. J. Burnside & Associates Limited
3 Ronell Crescent, Collingwood, Ontario
telephone (705) 446-0515 **fax** (705) 446-2399
web www.rjburnside.com

DRAWING #	DRAWING TITLE
MG-04-6450-PP1-AC	PLAN AND PROFILE
MG-04-6450-SD1	DETAIL SHEET

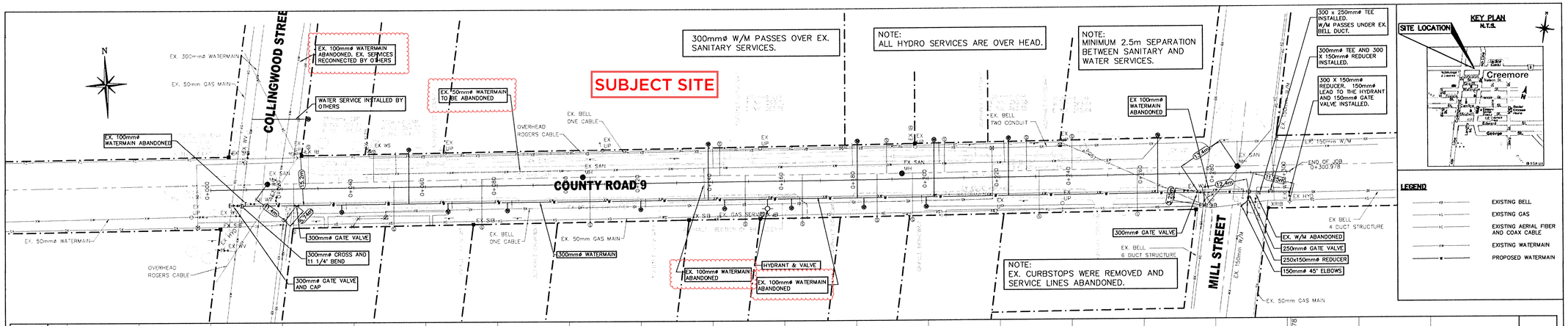
(XXX.XX) DENOTES – CONTRUCTION RECORD INFORMATION

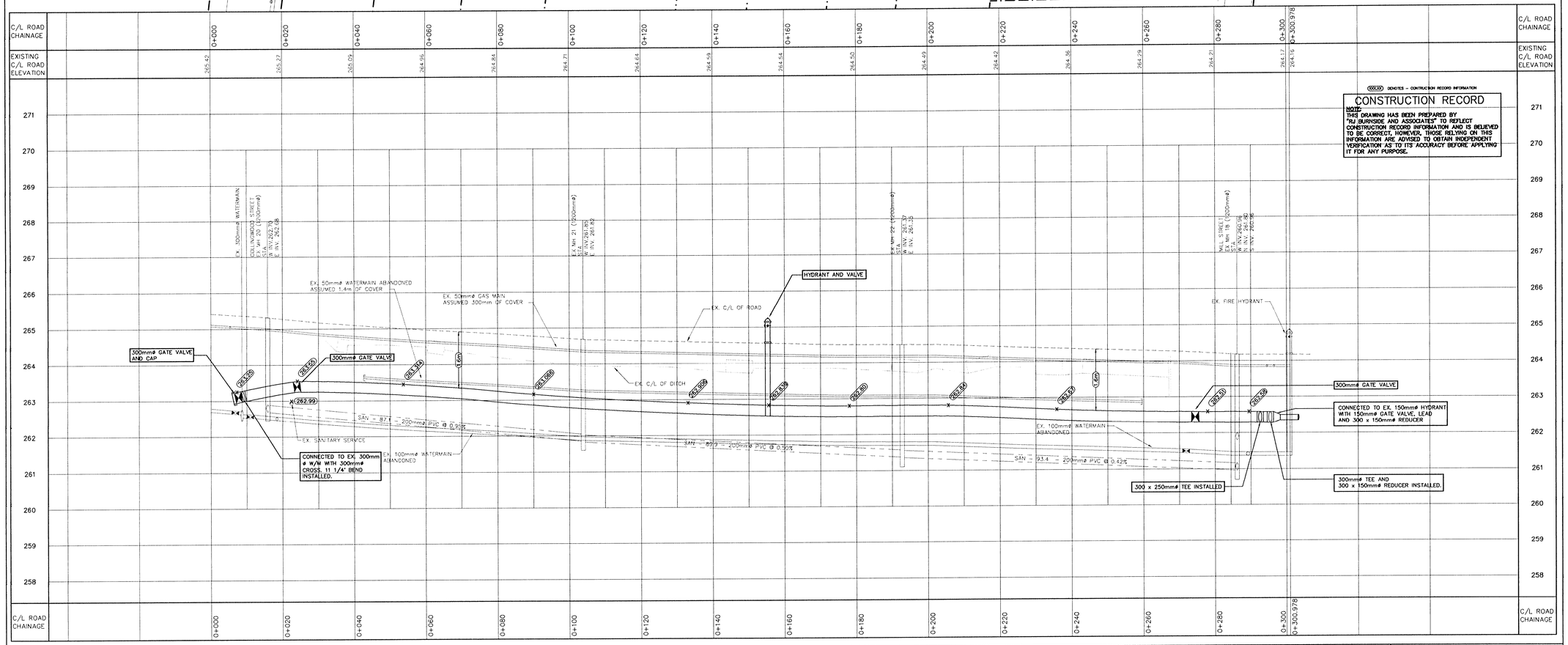
CONSTRUCTION RECORD

NOTE:
THIS DRAWING HAS BEEN PREPARED BY "RJ BURNSIDE AND ASSOCIATES" TO REFLECT CONSTRUCTION RECORD INFORMATION AND IS BELIEVED TO BE CORRECT, HOWEVER, THOSE RELYING ON THIS INFORMATION ARE ADVISED TO OBTAIN INDEPENDENT VERIFICATION AS TO ITS ACCURACY BEFORE APPLYING IT FOR ANY PURPOSE.

PROJECT No. MG 04 6450
CONSTRUCTION RECORD

County Rd 9 Mr. Built





1. This drawing is the exclusive property of R. J. Burnside & Associates Limited and the reproduction of any part without prior written consent of this office is strictly prohibited.

2. The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to this office prior to construction.

3. This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

4. Do not scale the drawings.

No.	Issue / Revision	Date
1.	MOE APPROVAL	FEB. 2005
2.	FOR TENDER	APRIL 2005
3.	CONSTRUCTION RECORD	MARCH 2006

BENCHMARK:

TOP OF BOLT ON FIRE HYDRANT ON SE CORNER OF MILL ST. AND CTY. RD. 9 = 264.482

SPIKE ON E FACE OF HPLS AT SW CORNER OF CTY. RD. 9 AND MILL ST.

CLEARVIEW TOWNSHIP
Established 1994

BURNSIDE
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web www.rjburnside.com

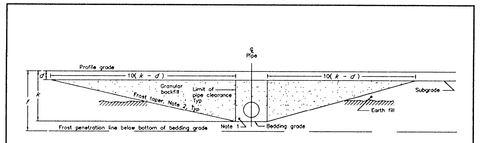
Client
TOWNSHIP OF CLEARVIEW
COUNTY OF SIMCOE

P.O. BOX 200, 217 GIDEON STREET
STAYNER, ONTARIO
L0M 1S0

Drawing Title
COUNTY ROAD 9, CREEMORE
WATERMAIN REPLACEMENT

PLAN AND PROFILE

Drawn By	Checked By	Drawing No.
K.B./J.R.	DLM/PGS	PP1-AC
Scale H: 1:500 V: 1:50	Date 08/13/04	Project No. MG 04 6450

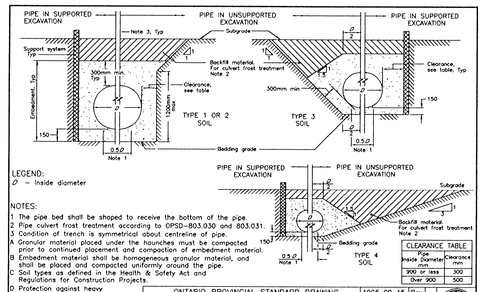


FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:
1 Pipe embedment according to:
  a) Flexible – OPSD–802.010, 802.014, 802.020 and 802.024.
  b) Rigid – OPSD–802.030 to 802.032, 802.034, 802.050 to 802.052 and 802.054.
2 Frost tapers start at bedding grade.
A Protection against heavy construction equipment according to OPSD–808.010.
B Frost tapers are not required in rock embankment.
C Frost tapers not required when frost line is above the top of pipe.
D All dimensions are in millimetres or metres unless otherwise shown.

LEGEND:
d –depth of roadbed granular
A –depth of frost treatment
f –depth of frost penetration

ONTARIO PROVINCIAL STANDARD DRAWING	1996 09 15	Rev
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BELOW BEDDING GRADE	Date	
	OPSD – 803.030	

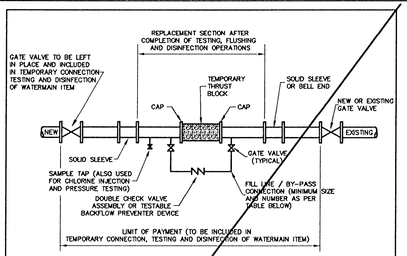


LEGEND:
D – Inside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
300 or less	300
Over 900	500

NOTES:
1 The pipe bed shall be shaped to receive the bottom of the pipe.
2 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.
3 Condition of trench is symmetrical about centreline of pipe.
A Granular material placed under the haunches must be compacted prior to continued placement and compaction of embedment material.
B Embedment material shall be homogeneous granular material, and shall be placed and compacted uniformly around the pipe.
C Soil types as defined in the Health & Safety Act and Regulations for Construction Projects.
D Protection against heavy construction equipment according to OPSD–808.010.
E All dimensions are in millimetres or metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	1996 09 15	Rev
FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION	Date	
	OPSD – 802.010	

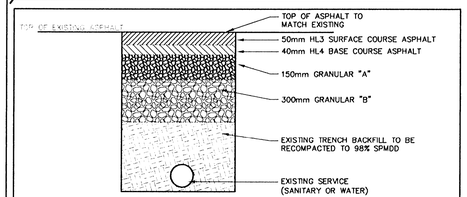


TYPICAL TEMPORARY CONNECTION FOR WATERMAINS

NTS

PIPE DIAMETER	FLOW REQUIRED TO PRODUCE 0.76m/s (APPROX) VELOCITY IN MAIN	SIZE OF TAP (mm)			NUMBER OF OPEN 64mm HYDRANT OUTLETS
		25	38	51	
mm	l/s	NUMBER OF TAPS ON PIPE			
100	6.3	1	–	–	1
150	12.6	–	1	–	1
200	25.2	–	2	1	1
250	37.9	–	3	2	1
300	56.8	–	–	2	2
400	109.9	–	–	4	2

REQUIRED FLOW AND OPENINGS TO FLUSH PIPELINES (276 kPa/40 PSI RESIDUAL PRESSURE IN WATERMAIN)



NOTE: 1. GRANULARS TO BE COMPACTED TO 100% SPMDD

REPAIR OF ROAD SURFACE AT EXISTING SERVICE CONNECTIONS

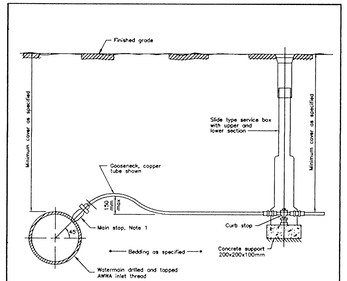
N.T.S.

GENERAL NOTES:

1. DO NOT SCALE DRAWNGS.
2. ALL DISTURBED AREAS SHALL BE RESTORED TO ORIGINAL OR BETTER CONDITIONS. ALL AREAS IN FRONT OF EXISTING RESIDENCES SHALL BE RESTORED WITH 100MM OF TOPSOIL AND SODDED. ALL OTHER AREAS SHALL BE RESTORED WITH 100MM OF TOPSOIL AND HYDROSEEDED.
3. ALL DITCHES DISTURBED SHALL BE REGRADED TO PROVIDE POSITIVE DRAINAGE, TO THE SATISFACTION OF THE ENGINEER.
4. FOR ALL WORK NEAR WATERCOURSES, USE SILT FENCE AND MAINTAIN A 3.0m MIN. UNDISTURBED BUFFER STRIP BETWEEN WATERCOURSE AND WORK AREA.
5. ALL DIMENSION UNITS ARE MILLIMETERS AND ALL ELEVATION UNITS ARE METERS UNLESS OTHERWISE SPECIFIED.
6. EXISTING UTILITIES HAVE BEEN SHOWN FOR DESIGN PURPOSES ONLY. THE CONTRACTOR SHALL HAVE ALL UTILITIES LOCATED IN THE FIELD AND ANY CONFLICTS SHALL BE REPORTED TO THE ENGINEER PRIOR TO EXCAVATION. THE CONTRACTOR SHALL PRESERVE AND PROTECT ALL EXISTING UTILITIES.
7. LOCATION OF EXISTING SERVICES AND UTILITIES ARE APPROXIMATE ONLY. ADDITIONAL UTILITIES MAY EXIST WHICH ARE NOT SHOWN. CONTRACTOR SHALL HAVE ALL UTILITIES LOCATED BEFORE CONSTRUCTION/EXCAVATION.
8. SUPPORT ALL SERVICES AT ALL CROSSINGS. CONCRETE ENCASEMENT IS REQUIRED IF CLEARANCE IS LESS THAN 300mm.
9. DO NOT OPERATE ANY VALVES WITHOUT THE WRITTEN PERMISSION OF THE OPERATING AUTHORITY.
10. THE CONTRACTOR SHALL AS PART OF HIS WORK CHECK AND VERIFY ALL DIMENSIONS THROUGH MEASUREMENT OR CALCULATION AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH CONSTRUCTION.
11. THE CONTRACTOR SHALL VIEW ALL CONTRACT DRAWINGS AND SPECIFICATIONS AS ONE CONTRACT AND SHALL FULLY COORDINATE THE WORK WITH ALL THE FORCES.
12. ALL OPEN EXCAVATIONS SHALL HAVE ADEQUATE PROTECTION BARRIERS AROUND THEM AT ALL TIMES.
13. ALL ROADS AND DRIVEWAYS DISTURBED SHALL BE RESTORED AS PER MINIMUM RESTORATION DETAILS AS SHOWN OR AS OTHERWISE SPECIFIED IN THE DOCUMENTS. ALL ASPHALT REMOVED WITHIN THE LIMITS OF CONSTRUCTION SHALL BE DISPOSED IN AN AREA APPROVED BY THE ENGINEER.
14. ALL MATERIALS, INSTALLATIONS, RESTORATION AND TESTING SHALL BE IN ACCORDANCE WITH CURRENT MUNICIPAL AND OPSS SPECIFICATION.
15. OPSD'S PROVIDED ON DRAWNGS ARE PROVIDED FOR CONVENIENCE, AND ARE NOT INTENDED TO BE EXHAUSTIVE. ALL WORKS SHALL BE COMPLETED IN CONFORMANCE WITH APPLICABLE OPSD'S UNLESS OTHERWISE NOTED.
16. ALL NATIVE BACKFILL TO BE APPROVED BY THE ENGINEER. BACKFILL TO BE COMPACTED TO 95% SPMDD.
17. GRANULAR MATERIAL AS APPROVED BY THE ENGINEER SHALL BE COMPACTED TO A DENSITY OF 100% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY IN ACCORDANCE TO OPSS 501.
18. A MINIMUM 48 HOURS WRITTEN NOTICE TO ADJACENT PROPERTY OWNERS, THE OWNER AND THE ENGINEER IS REQUIRED PRIOR TO ANY DISRUPTIONS TO WATER, GAS, POWER AND TELEPHONE.
19. EXISTING TREES AND SHRUBS LOCATED ADJACENT TO THE CONTRACTORS WORKING AREA SHALL BE PROTECTED DURING CONSTRUCTION. TREES AND SHRUBS DAMAGED AS A RESULT OF THE CONTRACTOR'S NEGLIGENCE SHALL BE REPAIRED AT THE CONTRACTOR'S EXPENSE TO THE SATISFACTION OF THE ENGINEER.
20. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE SUPPORT OF HYDRO, BELL, AND ALL OTHER POLES NOT INDICATED TO BE RELOCATED.
21. NO GRANULAR ROAD BASE MATERIAL IS TO BE PLACED UNTIL THE ENGINEER HAS INSPECTED AND APPROVED BACKFILL.
22. THE CONTRACTOR IS RESPONSIBLE FOR CONFIRMING ACCURACY OF ALL TEMPORARY BENCHMARKS ESTABLISHED FOR DESIGN PURPOSES PRIOR TO CONSTRUCTION.
23. THE CONTRACTOR SHALL ENSURE THAT ALL EXISTING PIPES THAT ARE TO REMAIN DURING CONSTRUCTION PERIOD SHALL BE PROTECTED FROM ANY POTENTIAL DAMAGE, DUE TO CONSTRUCTION ACTIVITY.
24. A MINIMUM OF ONE LANE ON ALL STREETS MUST REMAIN TO TRAFFIC AT ALL TIMES.
25. THE CONTRACTOR SHALL COORDINATE AND COMPLY WITH THE REQUIREMENTS OF ALL UTILITIES WHEN CROSSING OR WORKING NEAR THEIR PLANT.

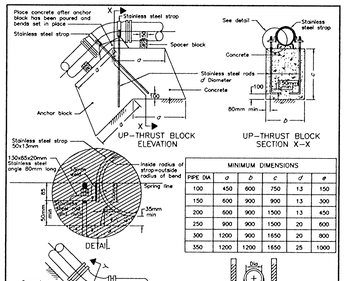
WATERMAIN NOTES:

1. WATERMAIN PIPE AND FITTINGS SHALL BE POLYVINYL CHLORIDE (PVC) PIPE AWWA C900 CLASS 150 (DR–18) WITH RUBBER GASKET JOINT FITTINGS AND TRACING WIRE (TW) #12 GUAGE). ALL VERTICAL CHANGES IN ELEVATION ARE TO BE JOINT DEFLECTION NOT TO EXCEED MANUFACTURER'S SPECIFICATIONS. ALL HORIZONTAL CHANGES IN ALIGNMENT BY PIPE DEFLECTIONS ARE NOT TO EXCEED MANUFACTURER'S SPECIFICATIONS.
2. FITTING CONNECTIONS SHOWN ON DRAWNGS ARE BASED ON AVAILABLE INFORMATION. AT ALL LOCATIONS WHERE THE CONTRACTOR IS TO CONNECT TO EXISTING WATERMAINS, THE CONTRACTOR SHALL PRE–EXCAVATE TO CONFIRM SIZE, MATERIAL AND LOCATION PRIOR TO ORDERING FITTINGS. THE CONTRACTOR IS THEN TO BACKFILL EXCAVATIONS UNTIL FITTINGS ARRIVE, AND PROVIDE THE ENGINEER WITH A SKETCH SHOWING THE WATERMAIN SIZE, MATERIAL, AND LOCATION PROVIDING TIES TO EXISTING FEATURES AND A DEPTH FROM TOP OF ROAD. IF A FITTING CHANGE IS REQUIRED, CONTRACTOR TO INFORM ENGINEER. EXISTING PIPE MATERIAL IS EXPECTED TO BE DUCTILE IRON.
3. BEDDING TO BE GRANULAR A AND COVER SHALL BE SAND (CLASS "B" WITH NO PARTICLES GREATER THAN 19mm) INSTALLED AS PER OPSD 802.010 (FOR FLEXIBLE PIPE) OR 802.030 (FOR RIGID PIPE).
4. ALL BURIED VALVES, FITTINGS AND PIPE CONNECTIONS SHALL HAVE MECHANICAL JOINTS AND CONCRETE THRUST BLOCKS. ALL MECHANICAL JOINT PIPE TO BE RESTRAINED WITH SIGMA ONE–LOK (FOR DI PIPE) OR SIGMA PV–LOK (FOR PVC PIPE) OR ROMAC GRIPRING. CORROSION PROTECTION SHALL BE PROVIDED THROUGH THE USE OF 'PROTECTO–CAPS #90P190 (10mm)' SACRIFICIAL ANODES. THE CAPS SHALL BE INSTALLED ON EVERY OTHER BOLT OF EVERY FITTING.
5. THE MINIMUM COVER ON WATERMAINS, SERVICES AND HYDRANT LEADS SHALL BE 1.7m AT ALL POINTS (UNLESS OTHERWISE NOTED). INSTALL INSULATION AS PER DETAIL WHERE WATERMAIN COVER IS LESS THAN 1.7 m AS NOTED ON DRAWNGS OR AS DIRECTED BY ENGINEER.
6. WHERE WATERMAIN VALVES ARE LOCATED UNDER TRAVELED ROAD SURFACES, THE TOP OF THE OPERATION BOX SHALL BE SET FLUSH WITH THE TRAVELED SURFACE.
7. AS DETAILED IN THE SPECIFICATION, PRESSURE AND LEAKAGE TESTS SHALL BE PERFORMED ON THE COMPLETE WATERMAIN INSTALLATION.
8. THE CONTRACTOR SHALL ARRANGE THE TESTS FOR SECTIONS OF WATERMAINS AND SHALL INFORM THE CONTRACT ADMINISTRATOR WHEN A SECTION IS ON TEST AND READY FOR INSPECTION.
9. ANY SECTIONS FAILING THE TEST SHALL BE REPAIRED AND RETESTED AT THE CONTRACTOR'S EXPENSE.
10. TRENCH BACKFILL FOR WATERMAIN AND SERVICE ACROSS ROADWAYS SHALL BE GRANULAR MATERIAL COMPACTED TO 100 % SPMDD OR SUITABLE NATIVE MATERIAL, COMPACTED TO 98% SPMDD AND SUBJECT TO APPROVAL BY THE CONTRACT ADMINISTRATOR.
11. CONTRACTOR TO USE ADEQUATE TRENCH SUPPORT AS REQUIRED TO ENSURE THAT EXISTING FACILITIES AND PRIVATE PROPERTY ARE NOT DISTURBED.
12. ALL FITTINGS TO BE MECHANICAL JOINT CEMENT LINED DUCTILE IRON.
13. ALL WATER SERVICES, HYDRANTS AND OTHER WORKS REQUIRED ON ANY SECTION OF WATERMAIN SHALL BE COMPLETED PRIOR TO CONSTRUCTION OF FINISHED ROADS AND DITCHES.
14. BEFORE BEING PLACED IN SERVICE, THE WATER SYSTEM SHALL BE TESTED IN ACCORDANCE WITH OPSS 701.07.22 AND DISINFECTED BY THE CONTRACTOR IN ACCORDANCE WITH THE REQUIREMENTS OF THE AWWA C651–99, FOR DISINFECTION AND CONNECTION OF POTABLE WATER SUPPLIES AND TO THE SATISFACTION OF THE TOWNSHIP OF CLEARVIEW.
15. THE CONTRACTOR SHALL BE RESPONSIBLE TO ENSURE THERE IS NO DISRUPTION OF WATER SERVICE TO PRIVATE PROPERTIES WHICH ARE CONNECTED TO THE EXISTING WATER SUPPLY SYSTEM, UNLESS OTHERWISE APPROVED BY THE TOWNSHIP OF CLEARVIEW.
16. WHERE FLEXIBLE COUPLINGS ARE SPECIFIED, RESTRAINED JOINTS ARE TO BE PROVIDED.
17. THE WATER SERVICE BOX SHALL BE SET 150mm ABOVE FINISHED GRADE AND PAINTED BLUE.
18. WATERMAIN SHALL BE INSTALLED SO THAT HIGH AND LOW POINTS ONLY OCCUR AT COMBINATION AIR VALVE OR DRAIN VALVE CHAMBERS, AS SHOWN ON THE PROFILE DRAWNGS. TO ACHIEVE THIS STRICT ADHERENCE TO PROFILES AS SHOWN ON PLAN AND PROFILES IS REQUIRED BY THE CONTRACTOR.



NOTES:
1 For plastic service pipes, install main stop at 15° above horizontal with a minimum 1.2m long gooseneck.
A Service connections to plastic watermains to be made using compression saddles or factory made tees.
B For any junction made in the service pipe, between main stop and curb stop, couplings will not be permitted unless the service length exceeds 20m.
C All water services to be installed 90° to the longitudinal axis of the watermain.
D All dimensions are in millimetres unless otherwise shown.

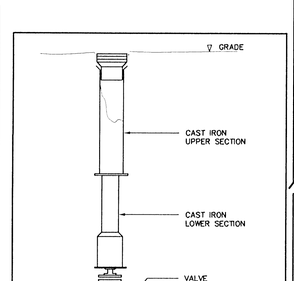
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2003	Rev	1
WATER SERVICE CONNECTION DETAIL 20 and 25mm DIAMETER SIZES			
	OPSD – 1104.010		



MINIMUM DIMENSIONS					
PIPE DIA	a^2	b	c	a^2	e
100	450	600	750	13	150
150	600	900	900	13	300
200	600	900	1500	13	450
250	900	900	1500	20	600
300	1200	900	1650	20	800
350	1200	1200	1650	25	1000

NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking is for bends up to 45° only.
E This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
F All dimensions are in millimetres unless otherwise shown.

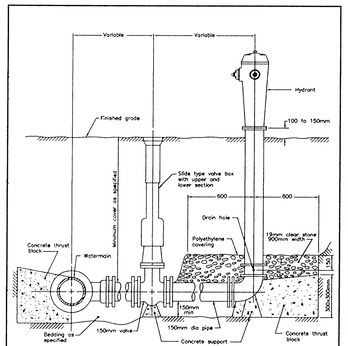
ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev	1
CONCRETE THRUST BLOCKS FOR VERTICAL BENDS			
	OPSD – 1103.020		



VALVE AND BOX DETAIL

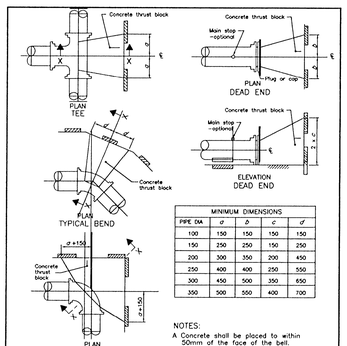
N.T.S.

WM–10



NOTES:
A All concrete thrust blocks to be poured against undisturbed ground.
B Bond breaker to be used between the concrete and the fittings and appurtenances.
C Bolts and nuts for buried flange to flange connections are to be stainless steel.
D Flange of standpipe extensions not to be in frost zone.
E All dimensions are in millimetres unless otherwise shown.

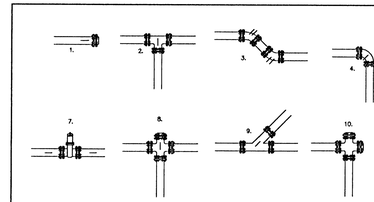
ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
HYDRANT INSTALLATION		
	OPSD – 1105.010	



MINIMUM DIMENSIONS					
PIPE DIA	a	b	c	d	e
100	150	150	150	150	150
150	250	250	150	250	250
200	300	350	200	450	
250	400	400	250	550	
300	450	500	350	650	
350	500	550	400	700	

NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
CONCRETE THRUST BLOCKS FOR TEES, PLUGS AND HORIZONTAL BENDS		
	OPSD – 1103.010	

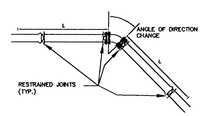


FITTING ARRANGEMENTS
1. DEAD END (CAP/PLUG)
2. TEES
3. HYDRANT RUNOUT
4. VERTICAL OFFSET
5. HORIZONTAL BEND
6. REDUCER
7. INLINE VALVE
8. THRU LINE CONNECTION, CROSS USED AS TEE
9. THRU LINE CONNECTION, WYE
10. DIRECTION CHANGE, CROSS USED AS ELBOW

TABLE 1.0

PIPE DIAMETER	L – MIN. LENGTH TO BE RESTRAINED ON EACH SIDE OF FITTING ARRANGEMENTS (M)						
	ELBOWS (DEG.) FITTING ARRANGEMENT No. 3 & 4				HYDRANT VALVES, "x" & TEES, FITTING ARRANGEMENTS No. 2, 7 & 9	REDUCERS FITTING ARRANGEMENTS No. 1, 5, 8 & 10	REDUCER FITTING ARRANGEMENTS No. 6
	11 1/4	22 1/2	45	90			
mm							
100	8.0	8.0	8.0	8.0	8.0	12.4	8.0
150	8.0	8.0	8.0	8.0	8.0	17.8	9.2
200	8.0	8.0	8.0	8.0	8.7	23.1	9.5
250	8.0	8.0	8.0	8.7	13.6	28.3	9.7
300	8.0	8.0	8.0	10.2	18.0	33.5	9.8
350	8.0	8.0	8.0	11.7	23.7	38.7	9.9
400	8.0	8.0	8.0	13.3	28.6	43.8	10

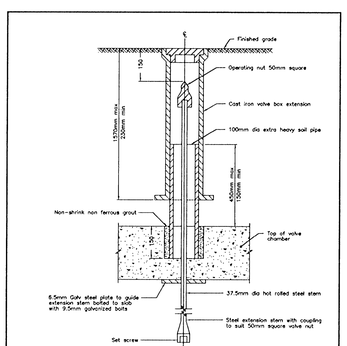
* NOTE THE MINIMUM LENGTH OF PIPE ALLOWED ON EACH SIDE OF THE FITTING (BEFORE NEXT FITTING) TO BE 1.5m



NOTE:
– ALL JOINTS MUST BE RESTRAINED IN ALL APPLICATIONS. TABLE 1 SHOWS RESTRAINED JOINTS REQUIRED ON BOTH SIDE OF THE RESTRAINED FITTINGS
– WHERE CONNECTING TO EXISTING WATERMAIN, EXCAVATE AND INSTALL RESTRAINTS ON EXISTING WATERMAIN FOR LENGTHS IDENTIFIED IN TABLE 1.0

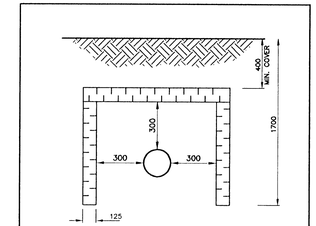
Drawing Title			
RESTRAINED JOINTS DETAIL			
Drawn By	Checked By	Scale	Date
B.G.G.	S.J.	N.T.S.	07/04/04

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NOTES:
A Valve stem and valve shall be placed plumb.
B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
VALVE OPERATOR DETAIL		
	OPSD – 1101.020	



NOTES:
– TO BE USED WHEN DIRECTED BY THE ENGINEER OR WHERE SHOWN ON THE DRAWNGS.
– WHERE IDENTIFIED AT CULVERTS, INSULATION TO EXTEND A MIN. 1.2m ON EITHER SIDE OF CULVERT OR UNTIL 1.7m DIAGONAL DISTANCE FROM CULVERT TO EDGE OF INSULATION IS ACHIEVED, WHICHEVER IS GREATER.
– RIGIDBOARD TO BE STYROFOAM S.M.
– 150mm OF BEDDING TO BE PLACED ON ALL SIDES OF INSULATION.
– INSULATION LAYERS TO BE PLACED IN ALTERNATE DIRECTIONS.
– FOR EVERY 300mm INCREASE IN DEPTH OF COVER BEYOND 400mm, THE THICKNESS OF INSULATION MAY BE DECREASED BY 25mm.

RIGIDBOARD INSULATION DETAIL

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2. The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to this office prior to construction.
3. This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.
4. Do not scale the drawings.

No.	Issue / Revision	Date
1.	MOE APPROVAL	FEB. 2005
2.	FOR TENDER	APRIL 2005
3.	CONSTRUCTION RECORD	MARCH 2006

(XXXX) DENOTES – CONTRUCTION RECORD INFORMATION

CONSTRUCTION RECORD

NOTE:
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CLEARVIEW TOWNSHIP
Established 1994

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COUNTY OF SIMCOE
P.O. BOX 200, 217 GIDEON STREET,
STAYNER, ONTARIO
L0M 1S0

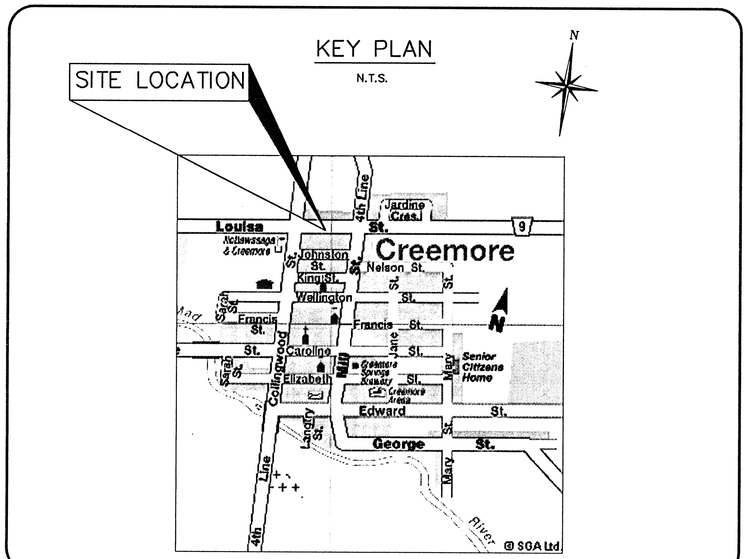
Drawing Title
COUNTY ROAD 9, CREEMORE
WATERMAIN REPLACEMENT

STANDARD DETAILS

Drawn By	Checked By	Drawing No.
KB/JR	DLM / PGS	SD1
Scale	Date / Project No.	
AS SHOWN	08/13/04 / MG 04 6450	

County 9

TOWN OF CREEMORE
COUNTY ROAD 9
(LOUISA STREET)
WATERMAIN REPLACEMENT



TOWNSHIP OF CLEARVIEW
COUNTY of SIMCOE

DRAWING #	DRAWING TITLE
MG-04-6450-PP1	PLAN AND PROFILE
MG-04-6450-SD1	DETAIL SHEET
MG-04-6450-SD2	DETAIL SHEET

MUNICIPALITY

CLEARVIEW TOWNSHIP
Established 1994

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217 GIDEON ST.
STAYNER, ONTARIO
LOM 1S0

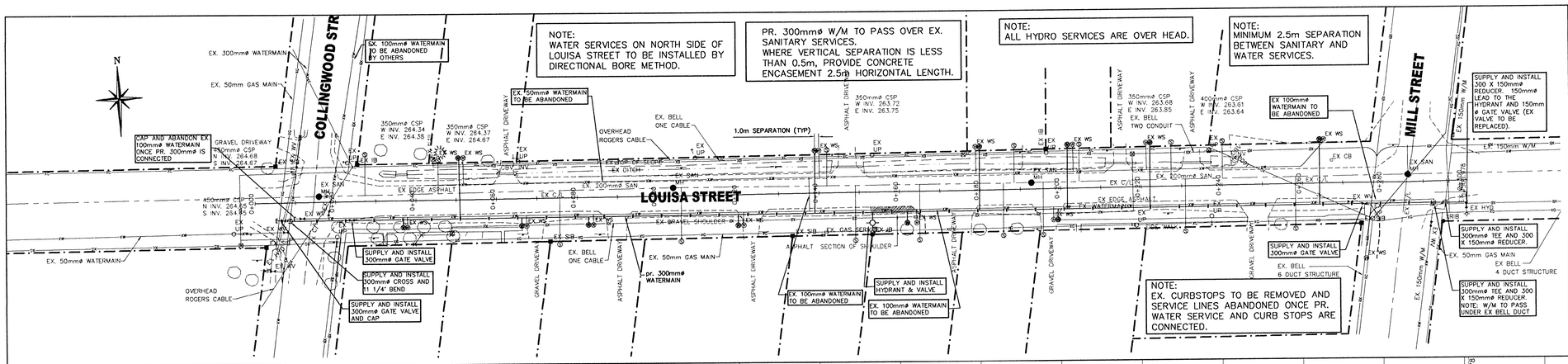
For Information Only

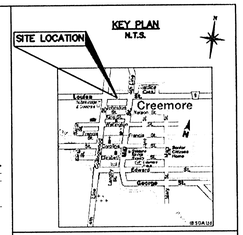
PROJECT ENGINEER

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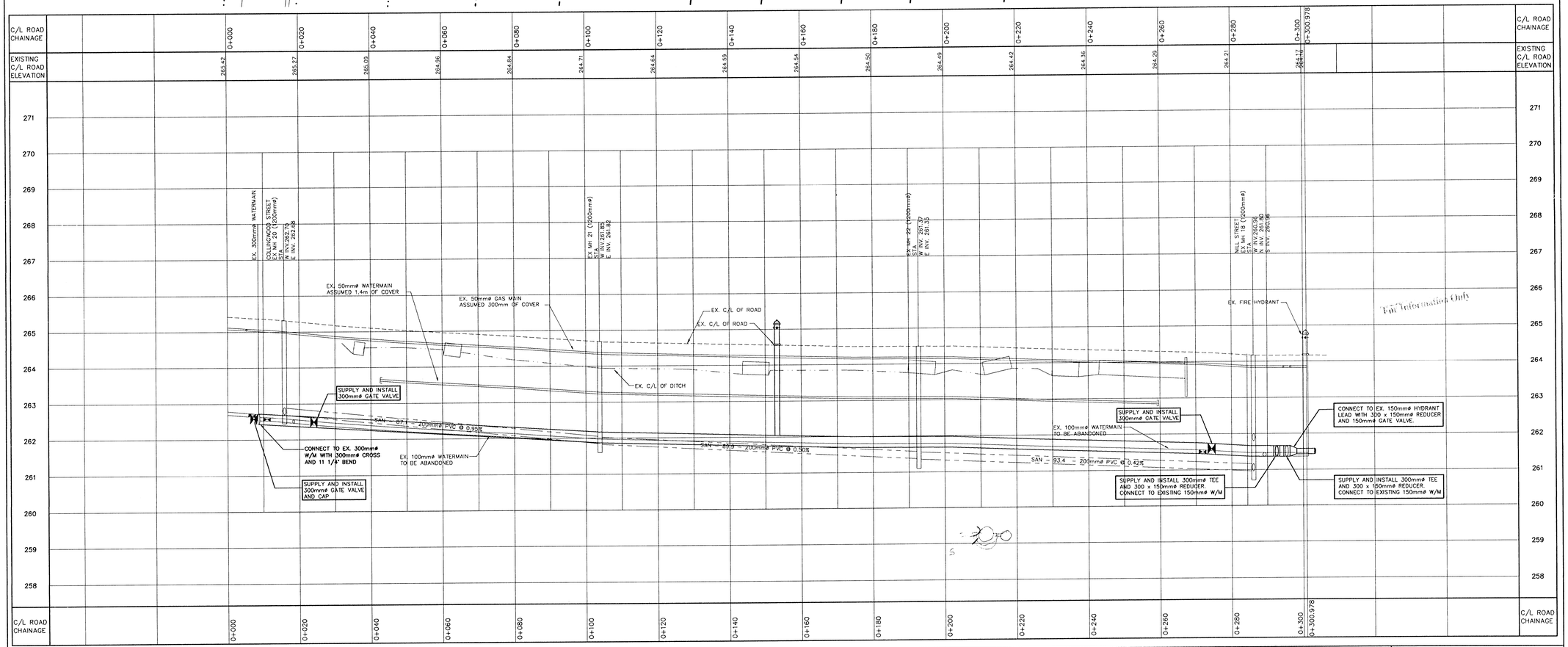
PROJECT No. MG 04 6450





LEGEND

- EXISTING BELL
- EXISTING GAS
- EXISTING AERIAL FIBER AND COAX CABLE
- EXISTING WATERMAIN
- PROPOSED WATERMAIN



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No.	Issue / Revision	Date
1	MOE APPROVAL	FEB. 2005

LICENSED PROFESSIONAL ENGINEER — P. G. SLAMA — Feb 16, 21 — PROVINCE OF ONTARIO

CLEARVIEW TOWNSHIP Established 1994

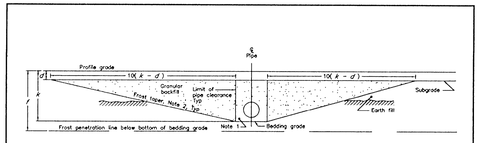
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STAYNER, ONTARIO
L0M 1S0

Drawing Title
COUNTY ROAD 9 (LOUISA STREET)
WATERMAIN REPLACEMENT
TOWN OF CREEMORE

PLAN AND PROFILE

Drawn By	Checked By	Drawing No.
K.B.	DLM/PGS	PP1
Scale H: 1: 500 V: 1:50	Date 08/13/04	Project No. MG 04 6450

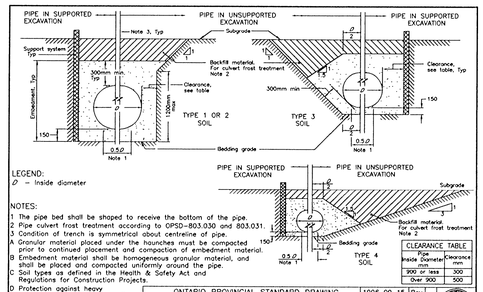


FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:
1 Pipe embedment according to:
  a) Flexible – OPSD–802.010, 802.014, 802.020 and 802.024.
  b) Rigid – OPSD–802.030 to 802.032, 802.034, 802.050 to 802.052 and 802.054.
2 Frost tapers start at bedding grade.
A Protection against heavy construction equipment according to OPSD–808.010.
B Frost tapers are not required in rock embankment.
C Frost tapers not required when frost line is above the top of pipe.
D All dimensions are in millimetres or metres unless otherwise shown.

LEGEND:
d –depth of roadbed granular
A –depth of frost treatment
f –depth of frost penetration

ONTARIO PROVINCIAL STANDARD DRAWING	1996 09 15	Rev
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BELOW BEDDING GRADE	Date	
	OPSD – 803.030	

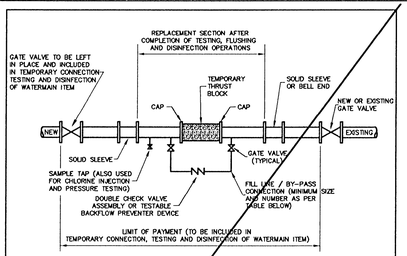


LEGEND:
D – Inside diameter

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
300 or less	300
Over 900	500

NOTES:
1 The pipe bed shall be shaped to receive the bottom of the pipe.
2 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.
3 Condition of trench is symmetrical about centreline of pipe.
A Granular material placed under the haunches must be compacted prior to continued placement and compaction of embedment material.
B Embedment material shall be homogeneous granular material, and shall be placed and compacted uniformly around the pipe.
C Soil types as defined in the Health & Safety Act and Regulations for Construction Projects.
D Protection against heavy construction equipment according to OPSD–808.010.
E All dimensions are in millimetres or metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	1996 09 15	Rev
FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION	Date	
	OPSD – 802.010	

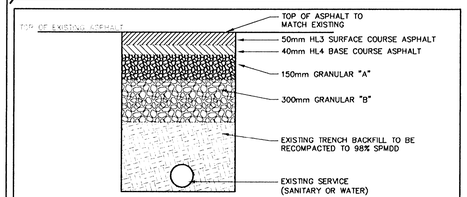


TYPICAL TEMPORARY CONNECTION FOR WATERMAINS

NTS

PIPE DIAMETER	FLOW REQUIRED TO PRODUCE 0.76m/s (APPROX) VELOCITY IN MAIN	SIZE OF TAP (mm)			NUMBER OF OPEN 64mm HYDRANT OUTLETS
		25	38	51	
mm	l/s	NUMBER OF TAPS ON PIPE			
100	6.3	1	–	–	1
150	12.6	–	1	–	1
200	25.2	–	2	1	1
250	37.9	–	3	2	1
300	56.8	–	–	2	2
400	109.9	–	–	4	2

REQUIRED FLOW AND OPENINGS TO FLUSH PIPELINES (276 kPa/40 PSI RESIDUAL PRESSURE IN WATERMAIN)



NOTE: 1. GRANULARS TO BE COMPACTED TO 100% SPMDD

REPAIR OF ROAD SURFACE AT EXISTING SERVICE CONNECTIONS

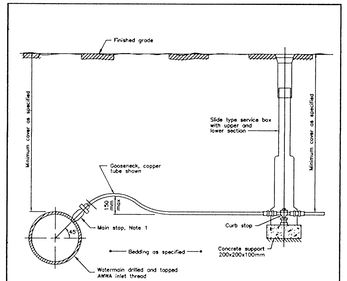
N.T.S.

GENERAL NOTES:

1. DO NOT SCALE DRAWNGS.
2. ALL DISTURBED AREAS SHALL BE RESTORED TO ORIGINAL OR BETTER CONDITIONS. ALL AREAS IN FRONT OF EXISTING RESIDENCES SHALL BE RESTORED WITH 100MM OF TOPSOIL AND SODDED. ALL OTHER AREAS SHALL BE RESTORED WITH 100MM OF TOPSOIL AND HYDROSEEDED.
3. ALL DITCHES DISTURBED SHALL BE REGRADED TO PROVIDE POSITIVE DRAINAGE, TO THE SATISFACTION OF THE ENGINEER.
4. FOR ALL WORK NEAR WATERCOURSES, USE SILT FENCE AND MAINTAIN A 3.0m MIN. UNDISTURBED BUFFER STRIP BETWEEN WATERCOURSE AND WORK AREA.
5. ALL DIMENSION UNITS ARE MILLIMETERS AND ALL ELEVATION UNITS ARE METERS UNLESS OTHERWISE SPECIFIED.
6. EXISTING UTILITIES HAVE BEEN SHOWN FOR DESIGN PURPOSES ONLY. THE CONTRACTOR SHALL HAVE ALL UTILITIES LOCATED IN THE FIELD AND ANY CONFLICTS SHALL BE REPORTED TO THE ENGINEER PRIOR TO EXCAVATION. THE CONTRACTOR SHALL PRESERVE AND PROTECT ALL EXISTING UTILITIES.
7. LOCATION OF EXISTING SERVICES AND UTILITIES ARE APPROXIMATE ONLY. ADDITIONAL UTILITIES MAY EXIST WHICH ARE NOT SHOWN. CONTRACTOR SHALL HAVE ALL UTILITIES LOCATED BEFORE CONSTRUCTION/EXCAVATION.
8. SUPPORT ALL SERVICES AT ALL CROSSINGS. CONCRETE ENCASEMENT IS REQUIRED IF CLEARANCE IS LESS THAN 300mm.
9. DO NOT OPERATE ANY VALVES WITHOUT THE WRITTEN PERMISSION OF THE OPERATING AUTHORITY.
10. THE CONTRACTOR SHALL AS PART OF HIS WORK CHECK AND VERIFY ALL DIMENSIONS THROUGH MEASUREMENT OR CALCULATION AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH CONSTRUCTION.
11. THE CONTRACTOR SHALL VIEW ALL CONTRACT DRAWINGS AND SPECIFICATIONS AS ONE CONTRACT AND SHALL FULLY COORDINATE THE WORK WITH ALL THE FORCES.
12. ALL OPEN EXCAVATIONS SHALL HAVE ADEQUATE PROTECTION BARRIERS AROUND THEM AT ALL TIMES.
13. ALL ROADS AND DRIVEWAYS DISTURBED SHALL BE RESTORED AS PER MINIMUM RESTORATION DETAILS AS SHOWN OR AS OTHERWISE SPECIFIED IN THE DOCUMENTS. ALL ASPHALT REMOVED WITHIN THE LIMITS OF CONSTRUCTION SHALL BE DISPOSED IN AN AREA APPROVED BY THE ENGINEER.
14. ALL MATERIALS, INSTALLATIONS, RESTORATION AND TESTING SHALL BE IN ACCORDANCE WITH CURRENT MUNICIPAL AND OPSS SPECIFICATION.
15. OPSD'S PROVIDED ON DRAWNGS ARE PROVIDED FOR CONVENIENCE, AND ARE NOT INTENDED TO BE EXHAUSTIVE. ALL WORKS SHALL BE COMPLETED IN CONFORMANCE WITH APPLICABLE OPSD'S UNLESS OTHERWISE NOTED.
16. ALL NATIVE BACKFILL TO BE APPROVED BY THE ENGINEER. BACKFILL TO BE COMPACTED TO 95% SPMDD.
17. GRANULAR MATERIAL AS APPROVED BY THE ENGINEER SHALL BE COMPACTED TO A DENSITY OF 100% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY IN ACCORDANCE TO OPSS 501.
18. A MINIMUM 48 HOURS WRITTEN NOTICE TO ADJACENT PROPERTY OWNERS, THE OWNER AND THE ENGINEER IS REQUIRED PRIOR TO ANY DISRUPTIONS TO WATER, GAS, POWER AND TELEPHONE.
19. EXISTING TREES AND SHRUBS LOCATED ADJACENT TO THE CONTRACTORS WORKING AREA SHALL BE PROTECTED DURING CONSTRUCTION. TREES AND SHRUBS DAMAGED AS A RESULT OF THE CONTRACTOR'S NEGLIGENCE SHALL BE REPAIRED AT THE CONTRACTOR'S EXPENSE TO THE SATISFACTION OF THE ENGINEER.
20. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE SUPPORT OF HYDRO, BELL, AND ALL OTHER POLES NOT INDICATED TO BE RELOCATED.
21. NO GRANULAR ROAD BASE MATERIAL IS TO BE PLACED UNTIL THE ENGINEER HAS INSPECTED AND APPROVED BACKFILL.
22. THE CONTRACTOR IS RESPONSIBLE FOR CONFIRMING ACCURACY OF ALL TEMPORARY BENCHMARKS ESTABLISHED FOR DESIGN PURPOSES PRIOR TO CONSTRUCTION.
23. THE CONTRACTOR SHALL ENSURE THAT ALL EXISTING PIPES THAT ARE TO REMAIN DURING CONSTRUCTION PERIOD SHALL BE PROTECTED FROM ANY POTENTIAL DAMAGE, DUE TO CONSTRUCTION ACTIVITY.
24. A MINIMUM OF ONE LANE ON ALL STREETS MUST REMAIN TO TRAFFIC AT ALL TIMES.
25. THE CONTRACTOR SHALL COORDINATE AND COMPLY WITH THE REQUIREMENTS OF ALL UTILITIES WHEN CROSSING OR WORKING NEAR THEIR PLANT.

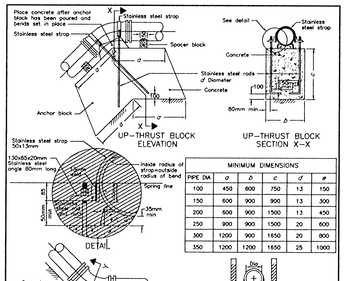
WATERMAIN NOTES:

1. WATERMAIN PIPE AND FITTINGS SHALL BE POLYVINYL CHLORIDE (PVC) PIPE AWWA C900 CLASS 150 (DR–18) WITH RUBBER GASKET JOINT FITTINGS AND TRACING WIRE (TW) #12 GUAGE). ALL VERTICAL CHANGES IN ELEVATION ARE TO BE JOINT DEFLECTION NOT TO EXCEED MANUFACTURER'S SPECIFICATIONS. ALL HORIZONTAL CHANGES IN ALIGNMENT BY PIPE DEFLECTIONS ARE NOT TO EXCEED MANUFACTURER'S SPECIFICATIONS.
2. FITTING CONNECTIONS SHOWN ON DRAWNGS ARE BASED ON AVAILABLE INFORMATION. AT ALL LOCATIONS WHERE THE CONTRACTOR IS TO CONNECT TO EXISTING WATERMAINS, THE CONTRACTOR SHALL PRE–EXCAVATE TO CONFIRM SIZE, MATERIAL AND LOCATION PRIOR TO ORDERING FITTINGS. THE CONTRACTOR IS THEN TO BACKFILL EXCAVATIONS UNTIL FITTINGS ARRIVE, AND PROVIDE THE ENGINEER WITH A SKETCH SHOWING THE WATERMAIN SIZE, MATERIAL, AND LOCATION PROVIDING TIES TO EXISTING FEATURES AND A DEPTH FROM TOP OF ROAD. IF A FITTING CHANGE IS REQUIRED, CONTRACTOR TO INFORM ENGINEER. EXISTING PIPE MATERIAL IS EXPECTED TO BE DUCTILE IRON.
3. BEDDING TO BE GRANULAR A AND COVER SHALL BE SAND (CLASS "B" WITH NO PARTICLES GREATER THAN 19mm) INSTALLED AS PER OPSD 802.010 (FOR FLEXIBLE PIPE) OR 802.030 (FOR RIGID PIPE).
4. ALL BURIED VALVES, FITTINGS AND PIPE CONNECTIONS SHALL HAVE MECHANICAL JOINTS AND CONCRETE THRUST BLOCKS. ALL MECHANICAL JOINT PIPE TO BE RESTRAINED WITH SIGMA ONE–LOK (FOR DI PIPE) OR SIGMA PV–LOK (FOR PVC PIPE) OR ROMAC GRIPRING. CORROSION PROTECTION SHALL BE PROVIDED THROUGH THE USE OF 'PROTECTO–CAPS #90P190 (10mm)' SACRIFICIAL ANODES. THE CAPS SHALL BE INSTALLED ON EVERY OTHER BOLT OF EVERY FITTING.
5. THE MINIMUM COVER ON WATERMAINS, SERVICES AND HYDRANT LEADS SHALL BE 1.7m AT ALL POINTS (UNLESS OTHERWISE NOTED). INSTALL INSULATION AS PER DETAIL WHERE WATERMAIN COVER IS LESS THAN 1.7 m AS NOTED ON DRAWNGS OR AS DIRECTED BY ENGINEER.
6. WHERE WATERMAIN VALVES ARE LOCATED UNDER TRAVELED ROAD SURFACES, THE TOP OF THE OPERATION BOX SHALL BE SET FLUSH WITH THE TRAVELED SURFACE.
7. AS DETAILED IN THE SPECIFICATION, PRESSURE AND LEAKAGE TESTS SHALL BE PERFORMED ON THE COMPLETE WATERMAIN INSTALLATION.
8. THE CONTRACTOR SHALL ARRANGE THE TESTS FOR SECTIONS OF WATERMAINS AND SHALL INFORM THE CONTRACT ADMINISTRATOR WHEN A SECTION IS ON TEST AND READY FOR INSPECTION.
9. ANY SECTIONS FAILING THE TEST SHALL BE REPAIRED AND RETESTED AT THE CONTRACTOR'S EXPENSE.
10. TRENCH BACKFILL FOR WATERMAIN AND SERVICE ACROSS ROADWAYS SHALL BE GRANULAR MATERIAL COMPACTED TO 100 % SPMDD OR SUITABLE NATIVE MATERIAL, COMPACTED TO 98% SPMDD AND SUBJECT TO APPROVAL BY THE CONTRACT ADMINISTRATOR.
11. CONTRACTOR TO USE ADEQUATE TRENCH SUPPORT AS REQUIRED TO ENSURE THAT EXISTING FACILITIES AND PRIVATE PROPERTY ARE NOT DISTURBED.
12. ALL FITTINGS TO BE MECHANICAL JOINT CEMENT LINED DUCTILE IRON.
13. ALL WATER SERVICES, HYDRANTS AND OTHER WORKS REQUIRED ON ANY SECTION OF WATERMAIN SHALL BE COMPLETED PRIOR TO CONSTRUCTION OF FINISHED ROADS AND DITCHES.
14. BEFORE BEING PLACED IN SERVICE, THE WATER SYSTEM SHALL BE TESTED IN ACCORDANCE WITH OPSS 701.07.22 AND DISINFECTED BY THE CONTRACTOR IN ACCORDANCE WITH THE REQUIREMENTS OF THE AWWA C651–99, FOR DISINFECTION AND CONNECTION OF POTABLE WATER SUPPLIES AND TO THE SATISFACTION OF THE TOWNSHIP OF CLEARVIEW.
15. THE CONTRACTOR SHALL BE RESPONSIBLE TO ENSURE THERE IS NO DISRUPTION OF WATER SERVICE TO PRIVATE PROPERTIES WHICH ARE CONNECTED TO THE EXISTING WATER SUPPLY SYSTEM, UNLESS OTHERWISE APPROVED BY THE TOWNSHIP OF CLEARVIEW.
16. WHERE FLEXIBLE COUPLINGS ARE SPECIFIED, RESTRAINED JOINTS ARE TO BE PROVIDED.
17. THE WATER SERVICE BOX SHALL BE SET 150mm ABOVE FINISHED GRADE AND PAINTED BLUE.
18. WATERMAIN SHALL BE INSTALLED SO THAT HIGH AND LOW POINTS ONLY OCCUR AT COMBINATION AIR VALVE OR DRAIN VALVE CHAMBERS, AS SHOWN ON THE PROFILE DRAWNGS. TO ACHIEVE THIS STRICT ADHERENCE TO PROFILES AS SHOWN ON PLAN AND PROFILES IS REQUIRED BY THE CONTRACTOR.



NOTES:
1 For plastic service pipes, install main stop at 15° above horizontal with a minimum 1.2m long gooseneck.
A Service connections to plastic watermains to be made using compression saddles or factory made tees.
B For any junction made in the service pipe, between main stop and curb stop, couplings will not be permitted unless the service length exceeds 20m.
C All water services to be installed 90° to the longitudinal axis of the watermain.
D All dimensions are in millimetres unless otherwise shown.

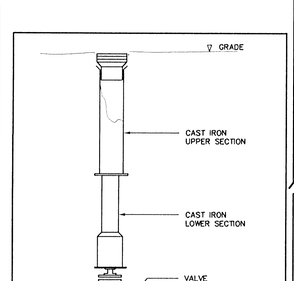
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2003	Rev	1
WATER SERVICE CONNECTION DETAIL 20 and 25mm DIAMETER SIZES			
	OPSD – 1104.010		



MINIMUM DIMENSIONS					
PIPE DIA	a^2	b	c	a^2	e
100	450	600	750	13	150
150	600	900	900	13	300
200	600	900	1500	13	450
250	900	900	1500	20	600
300	1200	900	1650	20	800
350	1200	1200	1650	25	1000

NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking is for bends up to 45° only.
E This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
F All dimensions are in millimetres unless otherwise shown.

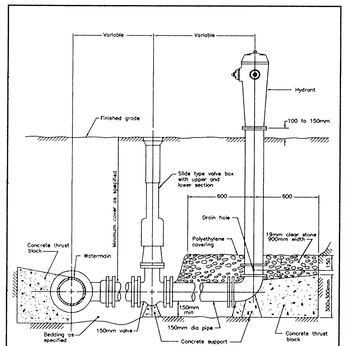
ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev	1
CONCRETE THRUST BLOCKS FOR VERTICAL BENDS			
	OPSD – 1103.020		



VALVE AND BOX DETAIL

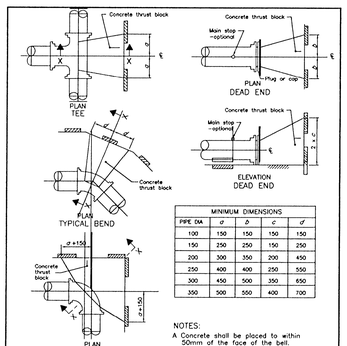
N.T.S.

WM–10



NOTES:
A All concrete thrust blocks to be poured against undisturbed ground.
B Bond breaker to be used between the concrete and the fittings and appurtenances.
C Bolts and nuts for buried flange to flange connections are to be stainless steel.
D Flange of standpipe extensions not to be in frost zone.
E All dimensions are in millimetres unless otherwise shown.

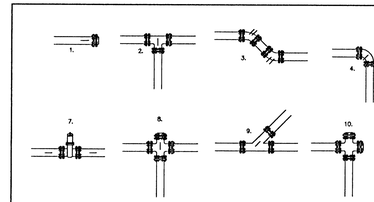
ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
HYDRANT INSTALLATION		
	OPSD – 1105.010	



MINIMUM DIMENSIONS					
PIPE DIA	a	b	c	d	e
100	150	150	150	150	150
150	250	250	150	250	250
200	300	350	200	450	
250	400	400	250	550	
300	450	500	350	650	
350	500	550	400	700	

NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
CONCRETE THRUST BLOCKS FOR TEES, PLUGS AND HORIZONTAL BENDS		
	OPSD – 1103.010	

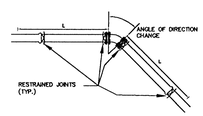


FITTING ARRANGEMENTS
1. DEAD END (CAP/PLUG)
2. TEES
3. HYDRANT RUNOUT
4. VERTICAL OFFSET
5. HORIZONTAL BEND
6. REDUCER
7. INLINE VALVE
8. THRU LINE CONNECTION, CROSS USED AS TEE
9. THRU LINE CONNECTION, WYE
10. DIRECTION CHANGE, CROSS USED AS ELBOW

TABLE 1.0

PIPE DIAMETER	L – MIN. LENGTH TO BE RESTRAINED ON EACH SIDE OF FITTING ARRANGEMENTS (M)						
	ELBOWS (DEG.) FITTING ARRANGEMENT No. 3 & 4				HYDRANT VALVES, "x" & TEES, FITTING ARRANGEMENTS No. 2, 7 & 9	REDUCERS FITTING ARRANGEMENTS No. 1, 5, 8 & 10	REDUCER FITTING ARRANGEMENTS No. 6
	11 1/4	22 1/2	45	90			
mm							
100	8.0	8.0	8.0	8.0	8.0	12.4	8.0
150	8.0	8.0	8.0	8.0	8.0	17.8	9.2
200	8.0	8.0	8.0	8.0	8.7	23.1	9.5
250	8.0	8.0	8.0	8.7	13.6	28.3	9.7
300	8.0	8.0	8.0	10.2	18.0	33.5	9.8
350	8.0	8.0	8.0	11.7	23.7	38.7	9.9
400	8.0	8.0	8.0	13.3	28.6	43.8	10

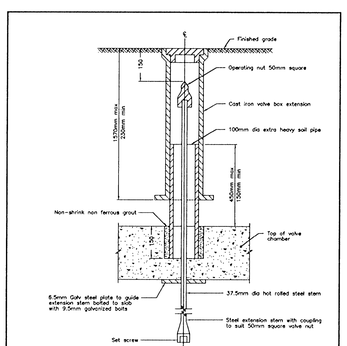
* NOTE THE MINIMUM LENGTH OF PIPE ALLOWED ON EACH SIDE OF THE FITTING (BEFORE NEXT FITTING) TO BE 1.5m



NOTE:
– ALL JOINTS MUST BE RESTRAINED IN ALL APPLICATIONS. TABLE 1 SHOWS RESTRAINED JOINTS REQUIRED ON BOTH SIDE OF THE RESTRAINED FITTINGS
– WHERE CONNECTING TO EXISTING WATERMAIN, EXCAVATE AND INSTALL RESTRAINTS ON EXISTING WATERMAIN FOR LENGTHS IDENTIFIED IN TABLE 1.0

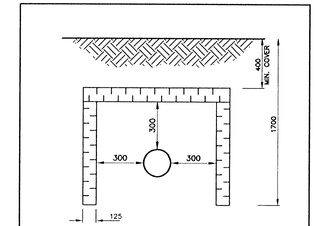
Drawing Title			
RESTRAINED JOINTS DETAIL			
Drawn By	Checked By	Scale	Date
B.G.G.	S.J.	N.T.S.	07/04/04

BURNSIDE
R. J. Burnside & Associates Limited
3 Ronell Crescent, Collingwood, Ontario
telephone (705) 445-0515 fax (705) 445-2399



NOTES:
A Valve stem and valve shall be placed plumb.
B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
VALVE OPERATOR DETAIL		
	OPSD – 1101.020	



NOTES:
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– WHERE IDENTIFIED AT CULVERTS, INSULATION TO EXTEND A MIN. 1.2m ON EITHER SIDE OF CULVERT OR UNTIL 1.7m DIAGONAL DISTANCE FROM CULVERT TO EDGE OF INSULATION IS ACHIEVED, WHICHEVER IS GREATER.
– RIGIDBOARD TO BE STYROFOAM S.M.
– 150mm OF BEDDING TO BE PLACED ON ALL SIDES OF INSULATION.
– INSULATION LAYERS TO BE PLACED IN ALTERNATE DIRECTIONS.
– FOR EVERY 300mm INCREASE IN DEPTH OF COVER BEYOND 400mm, THE THICKNESS OF INSULATION MAY BE DECREASED BY 25mm.

RIGIDBOARD INSULATION DETAIL

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3. This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.
4. Do not scale the drawings.

No.	Issue / Revision	Date
1.	MOE APPROVAL	FEB. 2005
2.	FOR TENDER	APRIL 2005
3.	CONSTRUCTION RECORD	MARCH 2006

(XXXX) DENOTES – CONTRUCTION RECORD INFORMATION

CONSTRUCTION RECORD

NOTE:
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CLEARVIEW TOWNSHIP
Established 1994

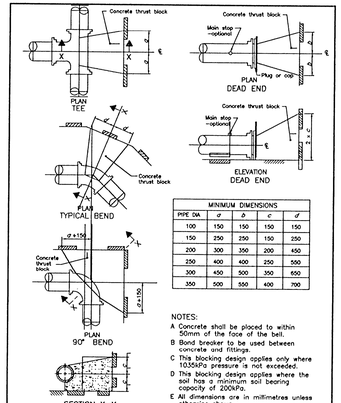
BURNSIDE
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3 Ronell Crescent, Collingwood, Ontario
telephone (705) 445-0515 fax (705) 445-2399
web www.rjburnside.com

Client
TOWNSHIP OF CLEARVIEW
COUNTY OF SIMCOE
P.O. BOX 200, 217 GIDEON STREET,
STAYNER, ONTARIO
L0M 1S0

Drawing Title
COUNTY ROAD 9, CREEMORE
WATERMAIN REPLACEMENT

STANDARD DETAILS

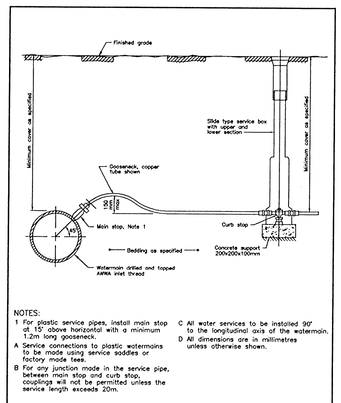
Drawn By	Checked By	Drawing No.
KB/JR	DLM / PGS	SD1
Scale	Date / Project No.	
AS SHOWN	08/13/04 / MG 04 6450	



MINIMUM DIMENSIONS				
PIPE DIA	a'	b	c	d'
100	150	150	150	150
150	250	250	150	250
200	300	350	200	450
250	400	400	250	550
300	450	500	350	650
350	500	550	400	700

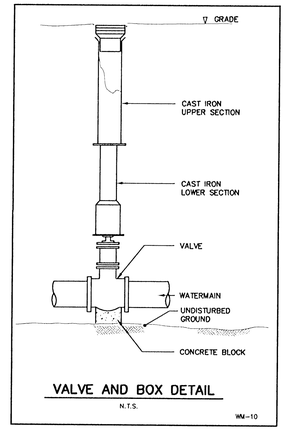
NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
CONCRETE THRUST BLOCKS FOR TEES, PLUGS AND HORIZONTAL BENDS		
OPSD – 1103.010		



NOTES:
1 For plastic service pipes, install main stop at 15° above horizontal with a minimum 1.2m long gooseneck.
A Service connections to plastic watermains to be made using service saddles or factory made tees.
B For any junction made in the service pipe, between main stop and curb stop, couplings will not be permitted unless the service length exceeds 20m.
C All water services to be installed 90° to the longitudinal axis of the watermain.
D All dimensions are in millimetres unless otherwise shown.

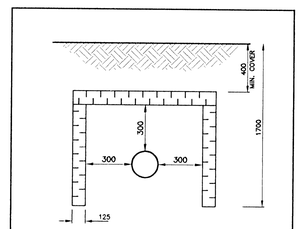
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2003	Rev	1
WATER SERVICE CONNECTION DETAIL 20 and 25mm DIAMETER SIZES			
OPSD – 1104.010			



VALVE AND BOX DETAIL

N.T.S.

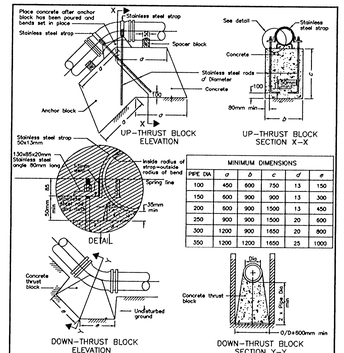
WM–10



NOTES:
- TO BE USED WHEN DIRECTED BY THE ENGINEER OR WHERE SHOWN ON THE DRAWINGS.
- WHERE IDENTIFIED AT CULVERTS, INSULATION TO EXTEND A MIN. 1.2m ON EITHER SIDE OF CULVERT OR UNTIL 1.7m DIAGONAL DISTANCE FROM CULVERT TO EDGE OF INSULATION IS ACHIEVED, WHICHEVER IS GREATER.
- RIGIDBOARD TO BE STYROFOAM S.M.
- 150mm OF BEDDING TO BE PLACED ON ALL SIDES OF INSULATION.
- INSULATION LAYERS TO BE PLACED IN ALTERNATE DIRECTIONS.
- FOR EVERY 300mm INCREASE IN DEPTH OF COVER BEYOND 400mm, THE THICKNESS OF INSULATION MAY BE DECREASED BY 25mm.

RIGIDBOARD INSULATION DETAIL

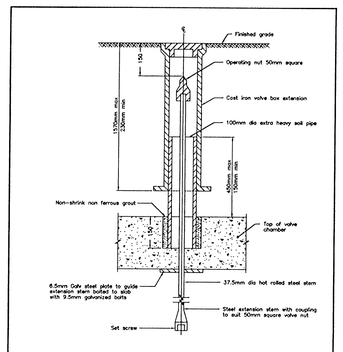
N.T.S.



MINIMUM DIMENSIONS					
PIPE DIA	a'	b	c	d'	e
100	450	600	750	13	150
150	600	900	900	13	300
200	600	900	1500	13	450
250	900	900	1500	20	600
300	1200	900	1650	20	900
350	1200	1200	1650	25	1000

NOTES:
A Concrete shall be placed to within 50mm of the face of the bell.
B Bond breaker to be used between concrete and fittings.
C This blocking design applies only where 1035kPa pressure is not exceeded.
D This blocking is for bends up to 45° only.
E This blocking design applies where the soil has a minimum soil bearing capacity of 200kPa.
F All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev	1
CONCRETE THRUST BLOCKS FOR VERTICAL BENDS			
OPSD – 1103.020			



NOTES:
A Valve stem and valve shall be placed plumb.
B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	April 1999	Rev
VALVE OPERATOR DETAIL		
OPSD – 1101.020		

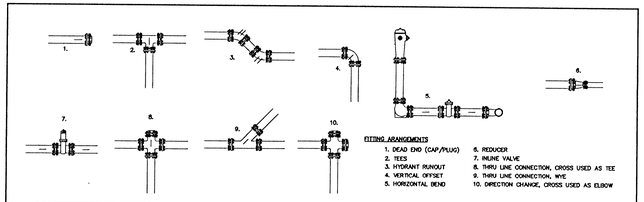
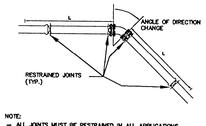


TABLE 1.0							
	L – MIN. LENGTH TO BE RESTRAINED ON EACH SIDE OF FITTING ARRANGEMENTS (M)						
PIPE DIAMETER	ELBOWS (DEG.) FITTING ARRANGEMENT No. 3 & 4				INLINE VALVES, Y's & TEES, FITTING ARRANGEMENTS No. 2, 7 & 9	HEADWORK FITTING ARRANGEMENTS No. 1, 5, 6 & 10	REDUCER FITTING ARRANGEMENTS No. 8
	11 1/4	22 1/2	45	90			
mm							
100	8.0	8.0	8.0	8.0	8.0	12.4	8.0
150	8.0	8.0	8.0	8.0	8.0	17.8	9.2
200	8.0	8.0	8.0	8.0	8.7	23.1	9.5
250	8.0	8.0	8.0	8.7	13.6	28.3	9.7
300	8.0	8.0	8.0	10.2	18.0	33.5	9.8
350	8.0	8.0	8.0	11.7	23.7	38.7	9.9
400	8.0	8.0	8.0	13.3	28.6	43.8	10

* NOTE THE MINIMUM LENGTH OF PIPE ALLOWED ON EACH SIDE OF THE FITTING (BEFORE NEXT FITTING) TO BE 1.5m



NOTE:
- ALL JOINTS MUST BE RESTRAINED IN ALL APPLICATIONS. TABLE 1 SHOWS RESTRAINED JOINTS REQUIRED ON BOTH SIDE OF THE RESTRAINED FITTINGS
- WHERE CONNECTING TO EXISTING WATERMAIN, EXCAVATE AND INSTALL RESTRAINTS ON EXISTING WATERMAIN FOR LENGTHS IDENTIFIED IN TABLE 1.0

Drawing Title				
RESTRAINED JOINTS DETAIL				
Drawn By	Checked By	Scale	Date	BURNSIDE
B.G.G.	S.J.	N.T.S.	07/04/04	R. J. Burnside & Associates Limited 3 Floneil Crescent, Collingwood, Ontario telephone (705) 446-0515 fax (705) 446-2369

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No.	Issue / Revision	Date
1	MOE APPROVAL	FEB. 2005

LICENSED PROFESSIONAL ENGINEER P. G. SLAMA Feb 18 2005 PROVINCE OF ONTARIO

CLEARVIEW TOWNSHIP Incorporated 1994

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web www.rjburnside.com

Client
TOWNSHIP OF CLEARVIEW
COUNTY OF SIMCOE
P.O. BOX 200, 217 GIDEON STREET
STAYNER, ONTARIO
L0M 1S0

Drawing Title
COUNTY ROAD 9 (LOUISA STREET)
WATERMAIN REPLACEMENT
TOWN OF CREEMORE
STANDARD DETAILS

Drawn By	Checked By	Drawing No.
KB	DLM / PGS	SD2
Scale: AS SHOWN	Date: 08/13/04	Project No.: MO 04 6450

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Appendix A

Water Demand and Water Storage

Water Supply and Storage Forecast

Creemore MSP

Flow Per Person		350	L/p/d
Persons Per Unit		3	ppu
Actual Residential	1203		
Ex. Commercial/Ind as Equivelent	987		
Total Ex. Equivelent Persons	2190		

			Water Demands (m3/d)							Required Fire Flow and Water Storgae Requirements					
			Persons	Cumul. Pop.	ADD (m3/d)	Cumil. ADD (m3/d)	MDF	Cumulative MDF (m3/d)	Cumulative MDF (L/s)	Flow Rate (L/s)	Duration (hrs)	A = Recommended Fire Storage (m3)	B = Eq. Storage (m3)	C = Emergency (m3)	Total Storage (m3)
2021 Ex. ADD			2190	2190		414	2.25	932	10.8	95.00	2.00	684	233	229	1146
Creemore Core			349	2539	122.2	536.2	2.25	1206	14.0	95.00	2.00	684	302	246	1232
Creemore East Area CE-1	546	3	1638	4177	573.3	1109.5	2.00	2219	25.7	125.00	2.00	900	555	364	1818
Cremore East Area CE-2	57	3	171	4348	59.9	1169.3	2.00	2339	27.1	125.00	2.00	900	585	371	1856
Creemore East (Area CE-3)	536	3	1608	5956	562.8	1732.1	2.00	3464	40.1	144.00	2.00	1037	866	476	2379
Creemore Core			2953	8909	1033.6	2765.7	2.00	5531	64.0	159.00	3.00	1717	1383	775	3875
Creemore North	281	3	843	9752	295.1	3060.7	2.00	6121	70.8	159.00	3.00	1717	1530	812	4059

Scenario	Maximum Day Demand		Water Storage	
To Satisfy 20 year Forecast (2042)	3464 m3/d	40.1 L/s	2379 m3	
To Satisfy Build Out Year Forecast	6121 m3/d	70.8 L/s	4059 m3	

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Appendix A.08

Flows and Demands

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Creemore MWWSP Technical Memorandum – Flows and Demands

Township of Clearview
217 Gideon Street
Stayner ON L0M 1S0

Creemore MWWSP Technical Memorandum – Flows and Demands

Township of Clearview
217 Gideon Street
Stayner ON L0M 1S0

R.J. Burnside & Associates Limited
3 Ronell Crescent
Collingwood ON L9Y 4J6 CANADA

May 2024
300050306.0000

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Record of Revisions

Revision	Date	Description
-	April 19, 2024	Initial Submission to Township of Clearview
1	May 29, 2024	Final Submission to Township of Clearview

R.J. Burnside & Associates Limited

Report Prepared By:



Jeff Langlois, P.Eng., MBA
Senior Vice President Public Sector Water and Wastewater
JLL:tc

Table of Contents

1.0 Introduction 1
1.1 Purpose of Technical Memorandum 1
2.0 Water Requirements 1
2.1 Water Supply 1
2.2 Water Storage Requirements 2
3.0 Wastewater Requirements 3
3.1 Wastewater Treatment Requirements 3

Appendices

Appendix A Water Demand and Water Storage
Appendix B Wastewater Flow

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1.0 Introduction

The Township of Clearview (Township) has retained R.J. Burnside & Associates Limited (Burnside) to provide engineering consulting services to complete a Master Water and Wastewater Servicing Plan (MSP) for the Community of Creemore (Creemore).

Creemore is the second largest population center in the Township and includes a mix of residential, commercial, institutional, and industrial land uses. The majority of residents are served by the existing municipal water and wastewater systems. The Official Plan for the Township was last updated in 2019. The Township is in the process of updating the Official Plan. The Master Servicing Plan is being undertaken to identify a preferred strategy to provide water and wastewater servicing to address the needs of the community.

1.1 Purpose of Technical Memorandum

The purpose of the Technical Memorandum is to estimate the water demands and wastewater flows associated with the projected service population for the Community of Creemore for the water and wastewater systems. The derivation of the projected service population is detailed in the technical memo on population and growth.

Identifying the estimated water demands and wastewater flows allows for an assessment of the ability of the existing infrastructure to accommodate the projected needs and as necessary to quantity any gaps between servicing needs and servicing capacity.

2.0 Water Requirements

2.1 Water Supply

For the derivation of water system demands the following assumptions were used.

For the existing served water users past use is assumed to be reflective of future needs. The average daily demand (ADD) for water for the existing users is therefore assumed to 414 m^3/d.

New users have been assumed to need 350 L/person/day as detailed in the technical memo on population and growth.

It is estimated that there are 1203 persons served by the existing water system. By considering the volume of water billed to existing non-residential customers, the equivalent service population of the existing user base is estimated to be 2190 equivalent persons.

At the design year of 2042, the projected service population is expected to have increased to 5956 equivalent persons. This includes allowances for additional users in the Creemore Core Area and Creemore East Area.

The assumed ratio of maximum day demand (MDD) to average day demand in 2042 is assumed to 2.0.

A municipal drinking water system with water storage in the distribution is required to be able to have a firm supply capacity equal to or greater than the MDD for the system.

The required MDD in 2042 is a firm capacity 40.1 L/s. This is significantly in excess of the existing DWS's current firm capacity of 15 L/s.

The required MDD at buildout is a firm capacity of 70.8 L/s.

An additional water supply is required to meet 2042 water demands. Further additional supply is required to meet the demands at build out.

Water demand calculations are presented in Appendix A.

2.2 Water Storage Requirements

The estimated water storage requirements associated with the required water supply MDD of 40.1 L/s have been calculated in accordance with the MECP guidelines. The water storage volume included recommended volumes for equalization, firefighting, and emergencies. Based on the projected equivalent service population in 2042 in accordance with MECP guidelines, the required fire flow is assumed to be 144 L/s for two hours.

The total required water storage volume associated with the 2042 servicing forecast is 2,379 m^3. This exceeds the current water storage volume of 1,570 m^3. Additional water storage is therefore required to address the 2042 servicing needs.

The total required water storage volume associated with the build out servicing forecast is 4,059 m^3. Further additional storage is required to address the servicing needs at build out.

Water storage calculations are presented in Appendix A.

3.0 Wastewater Requirements

3.1 Wastewater Treatment Requirements

For the derivation of wastewater system flows the following assumptions were used.

For the existing served wastewater users past flows are assumed to be reflective of future needs. The average daily flow (ADD) for water for the existing users is therefore assumed to 398 m^3/d.

New users have been assumed to need 350 L/person/day as detailed in TM4. In addition, an infiltration allowance of 90 L/person day at average flow has been included.

Peak wastewater flows are estimated by multiplying the average flow (without infiltration) by the Harmon Peaking factor, which varies with population and then adding an infiltration allowance of 227 L/person/day under peak flows.

It is estimated that there are 1128 persons served by the existing wastewater system. By considering the volume of water billed to existing non-residential customers, the equivalent service population of the existing user base is estimated to be 2051 equivalent persons

At the design year of 2042, the projected service population is expected to have increased to 5817 equivalent persons. This includes allowances for additional users in the Creemore Core Area and Creemore East Area.

Wastewater treatment facilities are generally rated based on the ADF they can handle. It is also important to ensure that they can handle the associated peak flows.

The required ADF of the wastewater treatment facility in 2042 is 2055 m^3/d or approximately 2,100 m^3/d. The associated peak flow is 6,784 m^3/d or approximately 6,800 m^3/d.

This ADF is significantly larger than the current rating of 1,400 m^3/d. The current rating is also significantly larger than the flow that the facility can actually currently handle.

Additional wastewater capacity is therefore required to address the needs forecasted for 2042. Even more capacity is required to address the needs at build out.

Wastewater flow calculations are presented in Appendix B

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Technical Memorandum

Date: December 14, 2021 **Project No.:** 300032944.0000

Project Name: Mary St. Sanitary Sewer Capacity

Client Name: Township of Clearview

Submitted To: Mike Rawn, C.E.T., Director of Public Works

Submitted By: Don McNalty, P.Eng.

Reviewed By: Jennifer Georgas, P.Eng.

This Technical Memorandum provides a summary of the historic review and analysis of the capacity of the Mary Street sanitary sewer as well as the current situation with respect to reserve capacity for this part of the Creemore sewage collection system.

1.0 Background

We have included in Appendix A the original Sewer design spread sheets for the Creemore Sewage system as prepared by Marshall Macklin and Monaghan (MMM) in May of 1999. Also included in Appendix A are the associated catchment area plans for the sewage system also prepared by MMM in 1999.

For the sake of context, we have included excerpts from an email we forwarded to the Township on May 31, 2015 regarding the Mary Street sewer capacity review. A copy of that complete email as well as the associated spreadsheets are included in Appendix B.

Any analysis of the Mary Street sewer requires an analysis of the expected flows from existing users as well as a determination of the future anticipated development and servicing strategies over the whole settlement community. Future development would be in the form of new green field development as well as infilling and intensification as required by the Provincial Policy Statement and Growth Plans. The expansion of the Creemore Springs Brewery and the associated hydraulic flows need to be accommodated as well. This required consideration of land use designations, development constraints such as flood limits and topography and servicing alternatives.

When MMM originally designed the system in the late 1990's and established the sewer catchment areas they used four persons per unit (PPU) and 450 l/cap/day ADF in accordance with the MOE guidelines for the design of the sanitary sewer collection systems. Determining the exact current PPU is difficult, at best, and it will be continually changing. We (Burnside and Clearview) believe that the four PPU is very conservative for Creemore. The Stats Canada number for Creemore was 2.48 at the time of previous analysis and for planning purposes we have established consensus that 2.8 PPU is an appropriate number for this community.

Although MMM used a very conservative number for PPU it is noted that they did not include lands that were designated for development on the west and north edges of the settlement area. In addition, the Provincial Policy Statement (PPS) which requires intensification of the settlement area as well as the expansion of the Creemore Springs Brewery came after the MMM design. In summary, although the original design was very conservative on the PPU and the ADF daily flows per capita, the impacts of intensification, the potential addition of additional designated lands and the brewery expansion would consume most of the reserve capacity created by the conservative initial design. As previously noted, a copy of the original design sheets and the catchment area maps as prepared by MMM are included in Appendix A of this memorandum.

We have included in Appendix C a sewer design spreadsheet (Excel format) for the Mary Street sewer which duplicates the MMM design, and which became the base for subsequent versions of the Mary Street sewer capacity analysis.

The capacity of the Mary St sewer was a significant issue relative to the McIntosh Development throughout the development process. It was one of the primary issues that was addressed before the Ontario Municipal Board (OMB) and within the resulting Minutes of Settlement dated in 2007. The development went back to the OMB and the capacity of the Mary Street sewer was again an issue. Although there was agreement that there was capacity in the Mary Street sewer at the time the proposed development was being proposed (2007), it was the Township's position that if all the additional designated lands on the west side and north edge of Creemore (not included in the MMM design) were developed and serviced in the future along with intensification/infilling within the community, that there would not be reserve capacity for the McIntosh development. This would then require either an upsizing of the Mary St sewer from Edward St to the sewage plant or the flows from McIntosh would need to discharge to a future pumping station on the east side of the settlement area.

We believe that there was agreement for the McIntosh Development to connect to the Mary Street sewer with security placed with the Township for the upgrading of the Mary Street sewer if ever required and if determined that an upgrade was not required through the de-designation of development lands, changes in development policies, reduced actual flows, etc. that the security would be released. The terms of this agreement should be included in the Minutes of Settlement and/or draft plan conditions.

It should also be noted that the earlier debate over the capacity of the Mary Street sewer occurred during a period where there were significant infiltration and extraneous flows entering the sewer system and consequently there was a legitimate level of caution regarding the flows in the system.

A Sanitary Sewer design sheet which was prepared in May of 2015 and which accompanied the May 31, 2015 email (see Appendix B) was based on the MOE guidelines for per capita flows and infiltration and allowed for the brewery expansion, higher density development as per the PPS, the development of the all designated lands west of Mary Street and 148 units of intensification. It can be noted that under this scenario the total flow exceeds the capacity of the existing Mary St. sewer once complete build out occurs. The attachment dated 05282010 Mary Street illustrates the future built out flow based on design guidelines at 116.36 l/s compared to a pipe capacity of 110.92 l/s.

In 2015 we also prepared a spreadsheet (also in Appendix B) with a per capita ADF of 350 l/cap/day which represented a conservative flow based on current metered flows at the time. The rest of the parameters remained the same. The attached spread sheet titled 05302015 which reduced the per capita flow to 350 l/cap/day produced a flow of 96.97 l/sec which is less than the pipe capacity (110.92 l/sec)

2.0 Where Are We Now?

Determination of future flows is not an exact science as there are numerous variables and there is a need for some engineering judgement. This is supported by the significant variation in the flows per capita, persons per unit, infiltration parameters and peaking factors. We have run an endless number of scenarios over the years with respect to the Mary St. sewer and some have shown that there is no reserve capacity; and some have shown there is some capacity. We would agree that the flow per capita used in the past is conservative (450 l/cap/day ADF). With an annual ADF of 370 cu m/day to the plant and an assumed population of around 1500, the flow per person is more like 250 l/cap/day including infiltration. If you considered the Max day flow of around 600 cu m/day (March 2012), the flow per person would be 400 l/cap/day. There would still be a peaking factor applied to determine the anticipated peak flow in the collection system. Of course, the demographics of the Creemore residents may change over time and the maintenance of the system to minimize extraneous flows will be ongoing.

Based on the past analysis, it appears that there are currently 573 units (not including any connections in Alliance) connected to the system in the Creemore settlement area. If one uses the 2.8 ppu which provides a moderate cushion for change in the demographics over the 2.48 PPU from the past Stats Canada numbers, there is just over 1600 equivalent residential population in Creemore. Adding all the designated Industrial and residential lands west of Mary St., the brewery expansion, increasing the density of development from 45 persons per hectare to 50 to comply to the PPS and allowing for 148 units of intensification for the lands situated west of Mary St. will generate a total residential equivalent population of almost 5200 people or over three times the current population. Again, this is only for the lands situated west of Mary Street but does include the additional designated lands.

From relatively recent discussions as well as review of the future development of Creemore through the OP update, it has been determined that much of the designated vacant land on the west side of Creemore will have development constraints due to the flood line and further, and more definite, is it is not physically practical to extend sewers further than the lands that are already serviced with sanitary sewers. This would also be consistent with the original design of the Creemore sewer system by MMM as most of those lands were not included in any of the original sewer design calculations. As noted previously we have included a spreadsheet file Mary St MMM Design July 12, 2021 (Appendix C) which is a duplication of the original design sheets prepared by MMM with the resulting flows being almost identical. This is provided for reference. Note that the MMM design used 4.0 persons per unit which we would consider very conservative but as is often the case this has worked in our favour.

So as per our discussions we have amended the analysis for the Mary St sewer to not include any of the westerly lands except for the lands already serviced being where the Fire Hall and the County Works Building are situated. Consequently, the sewer catchment area is more or less the same as the MMM design on the west side with this minor exception. This added 2.35 ha of land and at the industrial equivalent factor produced an addition 176 equivalent persons. We have also included the Quanbury Subdivision lots on the east side of Mary Street as well as an anticipated population number for the Creemore Meadows (McIntosh) site. We have increased the future residential from 45 persons/ha to 50 persons/ha as directed by growth policies and as done in previous calculations. In addition, the Creemore Springs Expansion (hydraulic flows) is included and 148 units of intensification. We note that the intensification would likely be spread over the whole community. We added the additional 414 equivalent persons to the sewer section between Node 11 and 15 which does result in a sewer surcharge on that one length of sewer but this is a very conservative approach as the intensification will not be isolated to that one sewer section. We used 2.8 persons per unit which was a blended PPU that was used previously and continued to use the MECP Guideline of 450 l/cap.

A catchment area plan as prepared by the Township's GIS department is included in Appendix D. A spreadsheet showing the 2021 analysis of the Mary St. sewer is in Appendix E. The resulting flow as can be noted on the spreadsheet is 101.98 l/s compared to the pipe capacity of 110.92 l/s.

It is also worth noting that the spreadsheet continues to include the land north of the Jardine Sub'd which produces a population of 250 persons. This land was included in the MMM design but it is anticipated that the topography will constrain the amount of development on this land, if any. Removal of development designation on that land would provide some additional capacity.

3.0 Next Steps

In summary, there was agreement in the past that there was currently capacity in the Mary St. sewer. It is the future development parameters and the build out of lands west of Mary St. in accordance with those parameters which resulted in the capacity constraints. Additional capacity could be achieved on Mary Street by upgrading the Mary Street sewer which would allow additional lands to be discharged into that sewer. It would make sense to bring as much land as possible by gravity into the Mary St. outlet to defer the need for a pump station east of Mary St. as long as possible, as well as the maintenance and operation of that SPS. However, financing for that future pumping station and the possible upgrading of the Mary St. outlet needs to be considered and planned for. As discussed, the possible future upgrading of the Mary St. outlet and even the future SPS should be included in the Development Charge for sanitary servicing rather than assigned to the lands situated east of Mary St.

R.J. Burnside & Associates Limited



Don McNalty, P.Eng.
DMcN:sp

Enclosure(s) Appendix A – Original 1999 Sewer Design
Appendix B – Email Correspondence dated May 31, 2015
Appendix C – Mary Street Sanitary Sewer Design Sheet Duplication of 1999 Design (Digital)
Appendix D – Catchment Area Plan
Appendix E – Mary Street Sewer Analysis Sheet 2021 (Digital)

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Appendix A

Original 1999 Sewer Design

LOCATION				POPULATION						FLOW								SEWER DESIGN				
(1) STREET	(2) MANHOLE FROM	(3) MANHOLE TO	(4) NO. OF UNITS	(5) P.P.U.	(6) POP. (4)x(5)	(7) POP. DENSITY	(8) EQUIV. POP. (7)x(12)	(9) ACCUM. POP.	(10) PEAKING FACTOR	(11) PEAK DAY DRY FLOW (L/s)	(12) SECTION AREA (ha)	(13) ACCUM. AREA (ha)	(14) INFILTR (L/s) (13)x0.23	(15) TOTAL FLOW (L/s) (11)+(14)	(16) PIPE LENGTH (m)	(17) PIPE DIA. (mm)	(18) SLOPE (%)	(20) Q FULL FLOW CAP. (L/s)	(21) FULL FLOW VEL. (m/s)	(22) TOTAL FLOW/ Q FULL % (15)/(20)x100	(23) V/ V FULL % (m/s)	(24) ACTUAL FLOW VEL. (m/s) (21)*(23)/100
JARDINE CRESCENT	1	2	9	4	36			36	4.34	0.8	1.60	1.60	0.4	1.2	111.2	200	0.40	20.7	0.66	6	46	0.30
POSSIBLE FUTURE RESIDENTIAL	2		40	4	160						5.00											
		3	11	4	44			240	4.12	5.1	1.40	8.00	1.8	7.0	123.8	200	0.50	23.2	0.74	30	86	0.63
EASEMENT	4	5	3	4	12			12	4.41	0.3	0.32	0.32	0.1	0.3	54.0	200	2.35	50.3	1.60	1	35	0.56
EASEMENT	6	5	3	4	12			12	4.41	0.3	0.32	0.32	0.1	0.3	32.0	200	0.40	20.7	0.66	2	35	0.23
	5	3	0		0			24	4.37	0.5	0.46	1.10	0.3	0.8	53.0	200	0.50	23.2	0.74	3	39	0.29
	3	7	12	4	48			312	4.07	6.6	1.58	10.68	2.5	9.1	133.1	200	0.40	20.7	0.66	44	95	0.63
	7	8	6	4	24			336	4.06	7.1	0.74	11.42	2.6	9.7	91.7	200	0.40	20.7	0.66	47	97	0.64
MARY STREET																						
COUNTY RD 9 R.O.W.	8	9	0		0			336	4.06	7.1	0.54	11.96	2.8	9.8	29.5	200	0.40	20.7	0.66	47	98	0.65
POSSIBLE FUTURE RESIDENTIAL	9	10				45/ha	108	444	4.00	9.2	2.40	14.36	3.3	12.5	56.0	200	0.46	22.2	0.71	56	103	0.73
POSSIBLE FUTURE RESIDENTIAL	10					45/ha	90				0.99											
MARY STREET R.O.W		11	0		0			534	3.96	11.0	0.17	15.52	3.6	14.6	87.5	200	0.40	20.7	0.66	70	109	0.72
NELSON STREET	12	13	9	4	36			36	4.34	0.8	1.22	1.22	0.3	1.1	96.0	200	1.20	35.9	1.14	3	39	0.45
	13	14	9	4	36			72	4.28	1.6	0.96	2.18	0.5	2.1	96.0	200	1.00	32.8	1.04	6	49	0.51
	14	11	6	4	24			96	4.25	2.1	0.72	2.90	0.7	2.8	93.2	200	1.00	32.8	1.04	8	55	0.57
MARY STREET	11	15	3	4	12			642	3.92	13.1	0.47	18.89	4.3	17.4	123.7	200	0.40	20.7	0.66	84	113	0.75
COUNTY ROAD 9																						
SCHOOL	19					75/ha	100				1.33											
		20	4	4	16			116	4.23	2.5	1.04	2.37	0.5	3.1	122.6	200	0.50	23.2	0.74	13	64	0.47
POSSIBLE FUTURE DEVELOPMENT	20		20	4	80						2.63											
		21	6	4	24			220	4.13	4.7	0.75	5.75	1.3	6.1	87.1	200	0.95	32.0	1.02	19	73	0.74
POSSIBLE FUTURE DEVELOPMENT	21					45/ha	132				2.93											
		22	6	4	24			376	4.03	7.9	0.71	9.39	2.2	10.0	89.9	200	0.50	23.2	0.74	43	95	0.70
RETAIL	22					75/ha	12				0.15											
		18	4	4	16			404	4.02	8.4	0.45	9.99	2.3	10.7	90.2	200	0.40	20.7	0.66	52	100	0.66
MILL STREET	16	17	5	4	20			20	4.38	0.5	0.77	0.77	0.2	0.6	94.5	200	2.91	55.9	1.78	1	35	0.62
	17	18	1	4	4			24	4.37	0.5	0.15	0.92	0.2	0.8	50.5	200	1.84	44.5	1.42	2	35	0.50
	18	23	8	4	32			460	3.99	9.6	1.29	12.20	2.8	12.4	118.7	200	0.62	25.8	0.82	48	98	0.81
COLLINGWOOD STREET	24	25	1	4	4			4	4.45	0.1	0.38	0.38	0.1	0.2	33.5	200	0.40	20.7	0.66	1	35	0.23
JOHNSTON STREET	25	26	5	4	20			24	4.37	0.5	0.71	1.09	0.3	0.8	95.5	200	0.55	24.3	0.77	3	39	0.30
	26	27	9	4	36			60	4.30	1.3	0.99	2.08	0.5	1.8	86.0	200	0.75	28.4	0.90	6	49	0.44
	27	23	2	4	8			68	4.29	1.5	0.36	2.44	0.6	2.1	86.9	200	0.80	29.3	0.93	7	51	0.48

NOTES :

PEAKING FACTOR $= 1 + \dfrac{14}{4 + P^{1/2}}$ where P = POP. IN 1000'S

$n = 0.013$

WET WEATHER INFILT. = 0.23 L/s/ha

CONSULTANT : MARSHALL MACKLIN MONAGHAN

PROJECT : CREEMORE SANITARY SEWERS

PROJECT No : 14-99004

LOCATION : CREEMORE

DESIGNED BY : J.E.

CHECKED BY : A.S.

DATE : MAY 21, 1999

DATE REVISED:

TOWNSHIP OF CLEARWATER

SANITARY SEWER DESIGN

SHEET 1 OF4

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LOCATION (1) STREET	(2) MANHOLE FROM	(3) MANHOLE TO	(4) NO OF UNITS	POPULATION (5) PPU	(6) POP. (4)x(5)	(7) POP DENSITY	(8) EQUIV POP. (7)x(12)	(9) ACCUM POP.	(10) PEAKING FACTOR	FLOW (11) PEAK DAY DRY FLOW (L/s)	(12) SECTION AREA (ha)	(13) ACCUM. AREA (ha)	(14) INFILTR (L/s) (13)x0.23	(15) TOTAL FLOW (L/s) (11)+(14)	(16) PIPE LENGTH (m)	(17) PIPE DIA. (mm)	(18) SLOPE (%)	SEWER DESIGN (20) Q FULL FLOW CAP. (L/s)	(21) FULL FLOW VEL. (m/s)	(22) TOTAL FLOW/ Q FULL % (15)/(20)x100	(23) V/ V FULL % (m/s)	(24) ACTUAL FLOW VEL. (m/s) (21)*(23)/100
MILL STREET	23	28	9	4	36			564	3.95	11.6	1.19	15.83	3.6	15.2	99.7	200	0.50	23.2	0.74	66	107	0.79
KING STREET	29	30	8	4	32			32	4.35	0.7	0.81	0.81	0.2	0.9	115.0	200	0.40	20.7	0.66	4	43	0.28
	30	28	5	4	20			52	4.31	1.2	0.63	1.44	0.3	1.5	116.1	200	0.40	20.7	0.66	7	51	0.34
MILL STREET	28	31	5	4	20			636	3.92	13.0	0.45	17.72	4.1	17.0	65.2	200	1.30	37.4	1.19	46	97	1.15
WELLINGTON STREET																						
POSSIBLE FUTURE RESIDENTIAL	32					45/ha	61				1.35											
		33	6	4	24			85	4.26	1.9	0.64	1.99	0.5	2.3	118.0	200	0.50	23.2	0.74	10	59	0.44
ARENA AND LEGION	33	34				75/ha	216	301	4.08	6.4	2.88	4.87	1.1	7.5	116.5	200	0.50	23.2	0.74	32	87	0.64
	35	34	1	4	4			305	4.08	6.5	0.27	5.14	1.2	7.6	45.5	200	0.40	20.7	0.66	37	90	0.59
COLLINGWOOD STREET R.O.W.	34		0		0						0.24											
		36	8	4	32			333	4.06	7.0	0.70	6.08	1.4	8.4	63.5	200	0.40	20.7	0.66	41	94	0.62
CHURCH	36					75/ha	6				0.07											
		37	10	4	40			379	4.03	7.9	1.41	7.56	1.7	9.7	103.0	200	0.50	23.2	0.74	42	94	0.69
	37	31	8	4	32			411	4.02	8.6	0.76	8.32	1.9	10.5	105.1	200	0.53	23.9	0.76	44	95	0.72
MILL STREET CONNECTION	31	38	6	4	24			1071	3.78	21.1	0.60	26.64	6.1	27.2	81.3	300	0.40	61.2	0.87	44	96	0.83
	38	39	7	4	28			1099	3.77	21.6	0.82	27.46	6.3	27.9	84.0	300	0.40	61.2	0.87	46	97	0.84
LIBRARY STREET	40	39	1	4	4			4	4.45	0.1	0.30	0.30	0.1	0.2	71.5	200	0.50	23.2	0.74	1	35	0.26
WELLINGTON STREET	39	41	10	4	40			1143	3.76	22.4	1.13	28.89	6.6	29.0	111.4	300	0.40	61.2	0.87	47	98	0.85
	41	15	10	4	40			1183	3.75	23.1	1.27	30.16	6.9	30.0	110.4	300	0.70	80.9	1.14	37	91	1.04
MARY STREET	15	42	1	4	4	45/ha		1829	3.62	34.4	0.35	49.40	11.4	45.7	123.0	300	0.40	61.2	0.87	75	110	0.95
FRANCIS STREET	45	46				75/ha	193	193	4.15	4.2	2.57	2.57	0.6	4.8	54.2	200	0.50	23.2	0.74	21	75	0.55
MILL ST. (SECTION N OF FRANCIS)	47	46				75/ha	58	58	4.30	1.3	0.77	0.77	0.2	1.5	78.9	200	0.60	25.4	0.81	6	46	0.37
FRANCIS STREET																						
	46		3	4	12						0.38											
SUPERMARKET		48				75/ha	51	314	4.07	6.6	0.30	4.02	0.9	7.6	90.9	200	0.50	23.2	0.74	33	87	0.64
	48	49	9	4	36			350	4.05	7.4	1.13	5.15	1.2	8.6	90.4	200	0.57	24.8	0.79	35	89	0.70
LIBRARY STREET (N OF FRANCIS)	50	49	4	4	16			16	4.39	0.4	0.52	0.52	0.1	0.5	61.7	200	1.86	44.7	1.42	1	35	0.50
LIBRARY STREET (S OF FRANCIS)	51	49	4	4	16			16	4.39	0.4	0.47	0.47	0.1	0.5	66.9	200	0.40	20.7	0.66	2	35	0.23
FRANCIS STREET	49	52	10	4	40			422	4.01	8.8	1.05	7.19	1.7	10.5	114.2	200	0.40	20.7	0.66	50	100	0.66
	52	42	8	4	32			454	4.00	9.4	1.03	8.22	1.9	11.3	106.2	200	0.40	20.7	0.66	55	102	0.67
MARY STREET	42	53	2	4	8			2291	3.54	42.2	0.65	58.27	13.4	55.6	120.7	375	0.40	110.9	1.00	50	100	1.00
CAROLINE STREET																						
LIBRARY	54					75/ha	40				0.53											
		55	7	4	28			68	4.29	1.5	1.15	1.68	0.4	1.9	78.7	200	0.60	25.4	0.81	7	51	0.41

NOTES :

PEAKING FACTOR $= 1 + \dfrac{14}{4 + P^{1/2}}$ (n = 0.013)

WHERE P = POP. IN 1000'S

WET WEATHER INFILT. = 0.23 L/s/ha

CONSULTANT : MARSHALL MACKLIN MONAGHAN

PROJECT : CREEMORE SANITARY SEWERS

PROJECT No : 14-99004

LOCATION : CREEMORE

DESIGNED BY : J.E.

CHECKED BY : A.S.

DATE : MAY 21, 1999

DATE REVISED:

TOWNSHIP OF CLEARWATER

SANITARY SEWER DESIGN

SHEET 2 OF 4

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LOCATION (1) STREET	(2) MANHOLE FROM	(3) MANHOLE TO	(4) NO. OF UNITS	POPULATION (5) P.P.U	(6) POP. (4)x(5)	(7) POP. DENSITY	(8) EQUIV POP. (7)x(12)	(9) ACCUM. POP.	(10) PEAKING FACTOR	FLOW (11) PEAK DAY DRY FLOW (L/s)	(12) SECTION AREA (ha)	(13) ACCUM. AREA (ha)	(14) INFILTR (L/s) (13)x0.23	(15) TOTAL FLOW (L/s) (11)+(14)	(16) PIPE LENGTH (m)	(17) PIPE DIA. (mm)	(18) SLOPE (%)	SEWER DESIGN (20) Q FULL FLOW CAP. (L/s)	(21) FULL FLOW VEL. (m/s)	(22) TOTAL FLOW/ Q FULL % (15)/(20)x100	(23) V/ V FULL % (m/s)	(24) ACTUAL FLOW VEL. (m/s) (21)*(23)/100
	55	53	3	4	12			80	4.27	1.8	0.44	2.12	0.5	2.3	73.9	200	0.50	23.2	0.74	10	57	0.42
MARY STREET																						
CREEMORE NURSING HOME	53					75/ha	81				1.07											
		56	4	4	16			2468	3.51	45.1	0.81	62.27	14.3	59.4	117.5	375	0.42	113.6	1.03	52	101	1.04
FRANCIS ST. (W OF COLLINGWOOD)	57	58	12	4	48			48	4.32	1.1	1.77	1.77	0.4	1.5	118.0	200	0.80	29.3	0.93	5	46	0.43
	58	59	9	4	36			84	4.26	1.9	1.02	2.79	0.6	2.5	119.8	200	0.50	23.2	0.74	11	59	0.44
COLLINGWOOD STREET	59	60	6	4	24			108	4.23	2.4	0.73	3.52	0.8	3.2	65.5	200	0.50	23.2	0.74	14	64	0.47
	60	61	2	4	8			116	4.23	2.5	0.29	3.81	0.9	3.4	62.0	200	0.50	23.2	0.74	15	67	0.49
CAROLINE (W OF COLLINGWOOD)	103	104	9	4	36			36	4.34	0.8	1.21	1.21	0.3	1.1	89.0	200	0.70	27.4	0.87	4	39	0.34
	104	105	9	4	36			72	4.28	1.6	0.69	1.90	0.4	2.0	90.2	200	0.40	20.7	0.66	10	57	0.38
SCHOOL	105					75/ha	62				0.82											
		61	4	4	16			150	4.19	3.3	0.56	3.15	0.7	4.0	88.7	200	0.61	25.6	0.82	16	68	0.55
CAROLINE STREET																						
CHURCH	61					75/ha	16				0.21											
		62	6	4	24			306	4.07	6.5	0.88	8.05	1.9	8.3	114.6	200	0.50	23.2	0.74	36	90	0.66
	62	63	9	4	36			342	4.05	7.2	1.38	9.43	2.2	9.4	116.0	200	0.50	23.2	0.74	40	94	0.69
MILL STREET RETAIL	63	65				75/ha	44	386	4.03	8.1	0.58	10.01	2.3	10.4	63.5	200	0.49	23.0	0.73	45	97	0.71
MILL STREET RETAIL	65	66				75/ha	29	415	4.01	8.7	0.38	10.39	2.4	11.1	65.5	200	0.65	26.4	0.84	42	94	0.79
MILL STREET RETAIL	66					75/ha	58				0.77											
		68	4	4	16			489	3.98	10.1	0.73	11.89	2.7	12.9	115.0	200	0.40	20.7	0.66	62	105	0.69
LIBRARY ST. (N OF ELIZABETH)	68	69	0		0			489	3.98	10.1	0.27	12.16	2.8	12.9	67.7	200	0.41	21.0	0.67	61	105	0.70
ELIZABETH ST.	70	71	12	4	48			48	4.32	1.1	1.32	1.32	0.3	1.4	100.1	200	0.70	27.4	0.87	5	46	0.40
	71	72	9	4	36			84	4.26	1.9	1.08	2.40	0.6	2.4	107.5	200	0.40	20.7	0.66	12	61	0.40
COLLINGWOOD ST. (N OF ELIZABETH	73	72	3	4	12			12	4.41	0.3	0.36	0.36	0.1	0.4	46.5	200	0.40	20.7	0.66	2	35	0.23
ELIZABETH STREET	72	74	11	4	44			140	4.20	3.1	1.35	4.11	0.9	4.0	120.0	200	0.40	20.7	0.66	19	74	0.49
	74	75	11	4	44			184	4.16	4.0	1.38	5.49	1.3	5.2	125.7	200	0.40	20.7	0.66	25	81	0.53
CREEMORE SPRINGS BREWERY	75									1.5	0.26											
MILL STREET RETAIL		78				75/ha	79	263	4.10	7.1	1.05	6.80	1.6	8.7	82.3	200	0.40	20.7	0.66	42	94	0.62
	78	81	7	4	28			291	4.08	7.7	0.86	7.66	1.8	9.4	73.5	200	0.56	24.5	0.78	38	91	0.71

NOTES :

PEAKING FACTOR $= 1 + \dfrac{n = 0.013}{\dfrac{14}{4 + P^{1/2}}}$

WHERE P = POP. IN 1000'S

WET WEATHER INFILT. = 0.23 L/s/ha

CONSULTANT : MARSHALL MACKLIN MONAGHAN

PROJECT : CREEMORE SANITARY SEWERS

PROJECT No : 14-99004

LOCATION : CREEMORE

DESIGNED BY : J.E.

CHECKED BY : A.S.

DATE : MAY 21, 1999

DATE REVISED:

TOWNSHIP OF CLEARWATER

SANITARY SEWER DESIGN

SHEET 3 OF 4

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LOCATION (1) STREET	(2) MANHOLE FROM	(3) MANHOLE TO	(4) NO. OF UNITS	POPULATION (5) P P U.	(6) POP (4)x(5)	(7) POP DENSITY	(8) EQUIV POP (7)x(12)	(9) ACCUM. POP	(10) PEAKING FACTOR	FLOW (11) PEAK DAY DRY FLOW (L/s)	(12) SECTION AREA (ha)	(13) ACCUM AREA (ha)	(14) INFILTR (L/s) (13)x0.23	(15) TOTAL FLOW (L/s) (11)+(14)	(16) PIPE LENGTH (m)	(17) PIPE DIA. (mm)	(18) SLOPE (%)	SEWER DESIGN (20) Q FULL FLOW CAP (L/s)	(21) FULL FLOW VEL. (m/s)	(22) TOTAL FLOW/ Q FULL % (15)/(20)x100	(23) V/ V FULL % (m/s)	(24) ACTUAL FLOW VEL. (m/s) (21)*(23)/100
MEDICAL CENTER						75/ha	15				0.20											
	81	69	3	4	12			318	4.07	8.2	0.53	8.39	1.9	10.2	77.2	200	0.54	24.1	0.77	42	95	0.73
	69	82	8	4	32			839	3.85	18.3	0.89	21.44	4.9	23.2	113.0	300	0.47	66.3	0.94	35	90	0.84
	82	56	12	4	48			887	3.83	19.2	1.53	22.97	5.3	24.5	108.5	300	0.89	91.2	1.29	27	82	1.06
MARY STREET	56	83	1	4	4			3359	3.40	60.9	0.32	85.56	19.7	80.6	124.7	375	0.40	110.9	1.00	73	109	1.09
EDWARD STREET	84	85	12	4	48			48	4.32	1.1	1.58	1.58	0.4	1.4	95.0	200	0.58	25.0	0.80	6	46	0.37
	85	86	9	4	36			84	4.26	1.9	1.03	2.61	0.6	2.5	97.5	200	0.53	23.9	0.76	10	59	0.45
	86	87A	16	2.5	40			124	4.22	2.7	0.73	3.34	0.8	3.5	95.0	200	0.53	23.9	0.76	15	67	0.51
	87A	88	11	4	44			128	4.21	2.8	1.16	3.77	0.9	3.7	88.0	200	0.50	23.2	0.74	16	68	0.50
	88	89	12	4	48			176	4.17	3.8	1.48	5.25	1.2	5.0	88.3	200	0.48	22.7	0.72	22	78	0.56
PARK	89					0/ha	0				0.63											
		90	5	4	20			196	4.15	4.2	0.95	6.83	1.6	5.8	115.3	200	0.50	23.2	0.74	25	81	0.60
PARK	90					0/ha	0				0.54											
		83	10	4	40			236	4.12	5.1	0.73	8.10	1.9	6.9	114.2	200	0.68	27.0	0.86	26	81	0.70
MARY STREET																						
POSSIBLE FUTURE RESIDENTIAL	83					45/ha	13				0.29											
		91	2	4	8			3616	3.37	64.9	0.44	94.39	21.7	86.6	125.1	375	0.40	110.9	1.00	78	111	1.11
LANGTRY STREET	92	93	4	4	16			16	4.39	0.4	0.45	0.45	0.1	0.5	34.0	200	0.40	20.7	0.66	2	35	0.23
GEORGE STREET (W OF MILL)	93	94	3	4	12			28	4.36	0.6	0.46	0.91	0.2	0.8	113.0	200	0.40	20.7	0.66	4	43	0.28
MILL STREET	95	94	7	4	28			28	4.36	0.6	0.58	0.58	0.1	0.8	65.0	200	0.40	20.7	0.66	4	39	0.26
GEORGE STREET (E OF MILL)	94	96	9	4	36			92	4.25	2.0	1.17	2.66	0.6	2.6	111.6	200	0.42	21.3	0.68	12	63	0.43
	96	97	10	4	40			132	4.21	2.9	1.35	4.01	0.9	3.8	113.0	200	0.40	20.7	0.66	18	73	0.48
PARK	97					0/ha	0				0.71											
		98	5	4	20			152	4.19	3.3	1.11	5.83	1.3	4.7	110.0	200	0.40	20.7	0.66	22	78	0.52
PARK	98					0/ha	0				0.21											
		91	5	4	20			172	4.17	3.7	0.54	6.58	1.5	5.2	111.1	200	0.40	20.7	0.66	25	81	0.53
MARY STREET																						
POSSIBLE FUTURE RESIDENTIAL	91					45/ha	20				0.43											
		99	4	4	16			3824	3.35	68.1	0.62	102.02	23.5	91.6	86.0	375	0.40	110.9	1.00	83	112	1.12
POSSIBLE FUTURE RESIDENTIAL	99					45/ha	41				0.91											
		106	7	4	28			3893	3.34	69.2	1.30	104.23	24.0	93.2	86.0	375	0.40	110.9	1.00	84	113	1.13
	106	101						3893	3.34	69.2	0.56	104.79	24.1	93.3	88.5	375	0.40	110.9	1.00	84	113	1.13
	101	102						3893	3.34	69.2		104.79	24.1	93.3	55.0	375	0.40	110.9	1.00	84	113	1.13

NOTES :

PEAKING FACTOR $= 1 + \dfrac{14}{4 + P^{1/2}}$ where $n = 0.013$

WHERE P = POP. IN 1000'S

WET WEATHER INFILT. = 0.23 L/s/ha

CONSULTANT : MARSHALL MACKLIN MONAGHAN

PROJECT : CREEMORE SANITARY SEWERS

PROJECT No : 14-99004

LOCATION : CREEMORE

DESIGNED BY : J.E.

CHECKED BY : A.S.

DATE : MAY 21, 1999

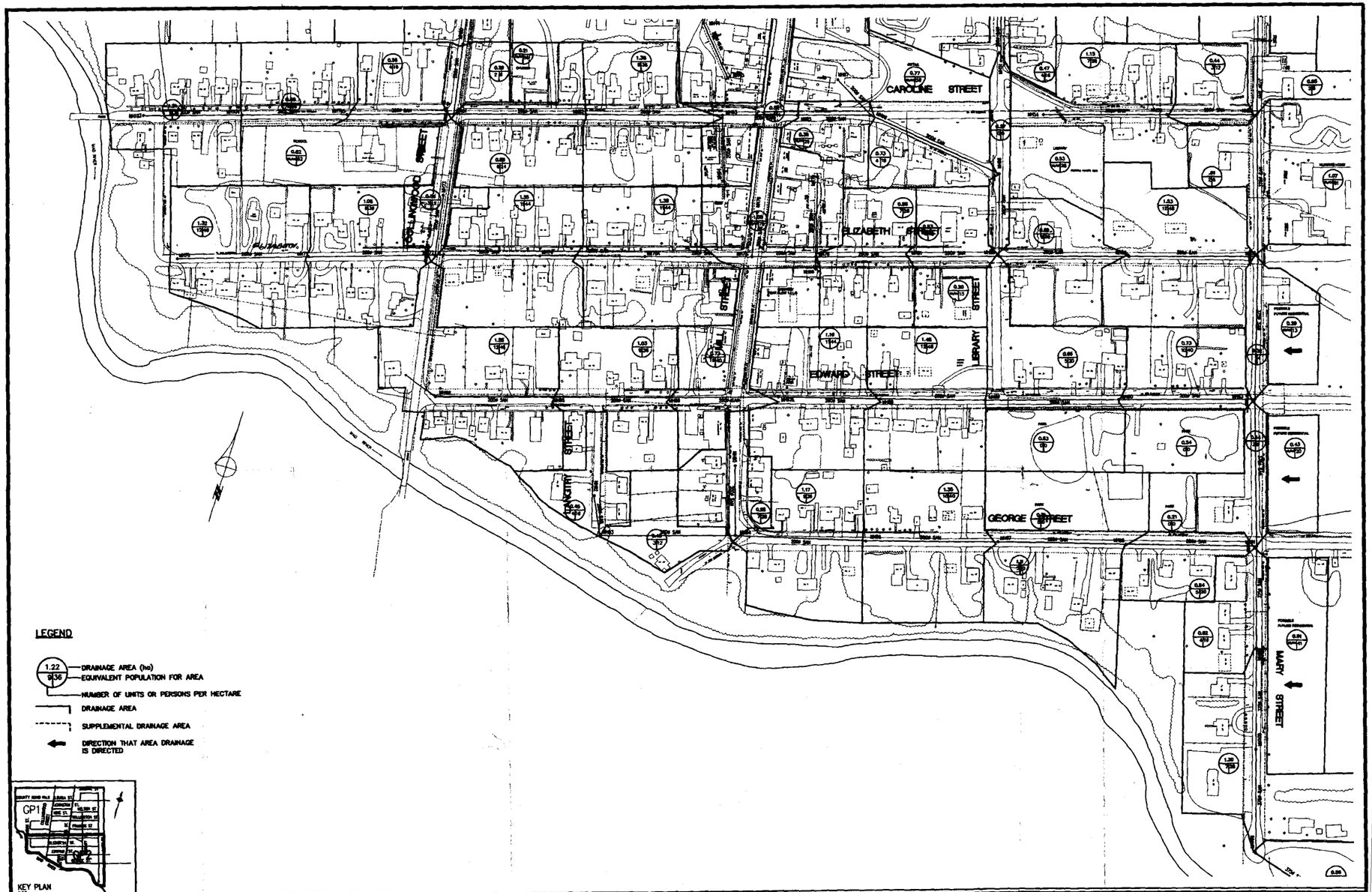
DATE REVISED:

TOWNSHIP OF CLEARWATER

SANITARY SEWER DESIGN

SHEET 4 OF 4

M:\1999\149904\design\sanitary\SAN2.xls



No.	REVISIONS	BY	DATE	APPR.
3	AS CONSTRUCTED	BTB	DEC/00	
2	JAIME PARK SEWER LOCATION ADJUSTED		JULY 30/99	
1	ISSUED FOR TENDERING		JUNE 1/99	

ALL PREVIOUS ISSUES OF THIS DRAWING ARE SUPERSEDED

STAMP

Original Stamped by Alex Strynadka P. Eng. Dated Aug 5,1999

THE CORPORATION OF THE TOWNSHIP OF CLEARVIEW
(COMMUNITY OF CREEMORE)

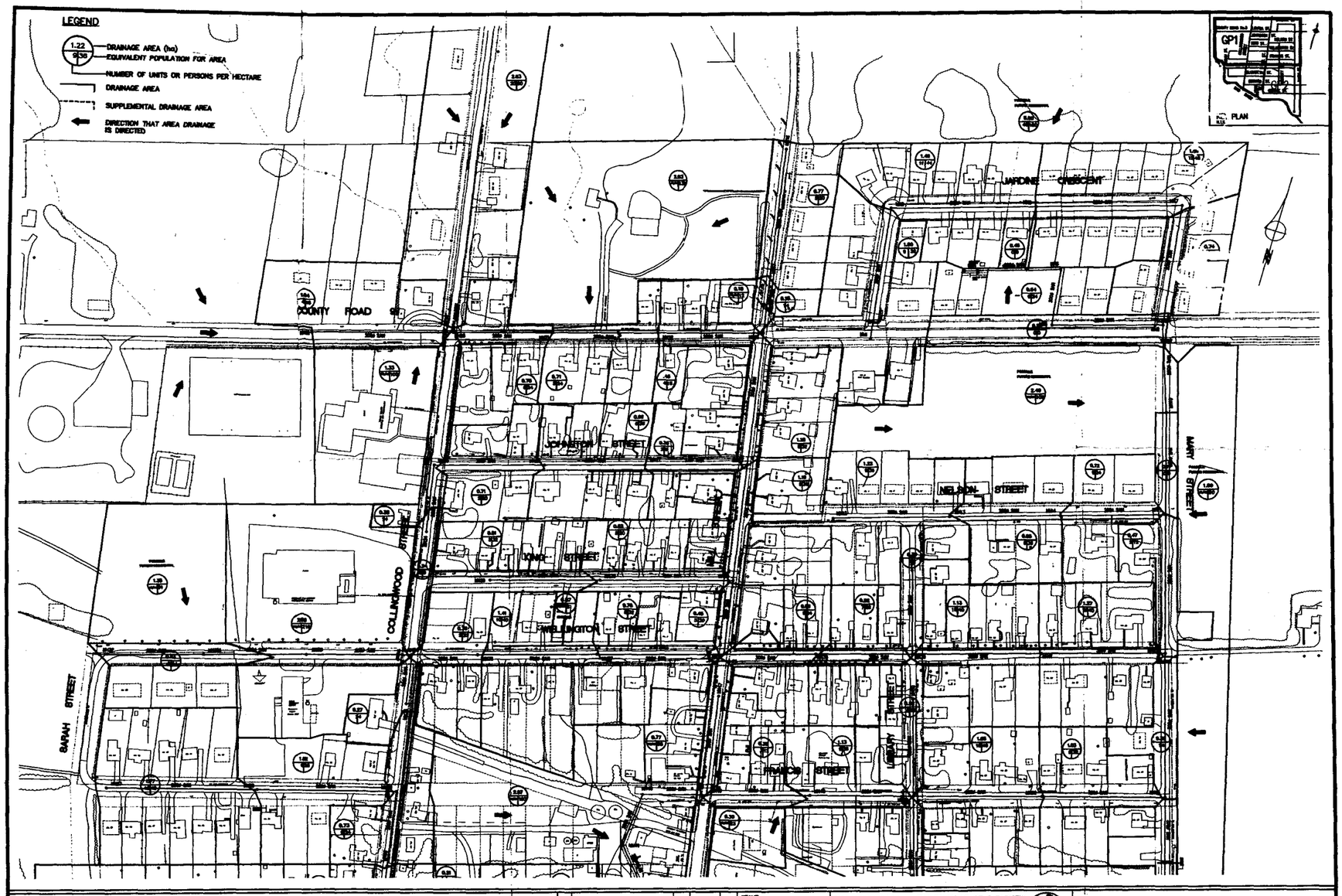
CREEMORE SANITARY SEWER SYSTEM

GENERAL PLAN SOUTH AND SANITARY DRAINAGE PLAN

Marshall Macklin Monaghan
CONSULTING ENGINEERS · SURVEYORS · PLANNERS
80 Commerce Valley Dr. East
Thornhill, Ont. L3T 7N4
Tel (905)882-1100
Fax (905)882-0055

DATE: MAY 1999 | PROJECT No. 14-99004 | CONTRACT No.
DESIGN: J.E. | CHECKED: A.S. | SCALE: HORIZ. 1:1500 | DWG No. GP2A
DRAWN: 10/12 CAD

WWG 094659



No.	REVISIONS	BY	DATE	APPR.
3	AS CONSTRUCTED	BTB	DEC/00	
2	JARDINE PARK SEWER LOCATION ADJUSTED		JULY 30/99	
1	ISSUED FOR TENDERING		JUNE 1/99	

ALL PREVIOUS ISSUES OF THIS DRAWING ARE SUPERSEDED

STAMP

Original Stamped by Alex Szymotiw P. Eng. Dated Aug 5/1999

THE CORPORATION OF THE TOWNSHIP OF CLEARVIEW
(COMMUNITY OF CREEMORE)

CREEMORE SANITARY SEWER SYSTEM

GENERAL PLAN NORTH
AND SANITARY DRAINAGE PLAN

Marshall Macklin Monaghan
CONSULTING ENGINEERS · SURVEYORS · PLANNERS

80 Commerce Valley Dr. East
Thornhill, Ont. L3T 7N4
Tel (905)882-1100
Fax (905)882-0055

DATE: MAY 1999		PROJECT No. 14-99004	CONTRACT No.
DESIGN: J.E.	CHECKED: A.S.	SCALE: HORIZ. 1:1500	DWG No. GP1A
DRAWN: 10/12 CAD			

BURNSIDE

[THE DIFFERENCE IS OUR PEOPLE]

Appendix B

Email Correspondence dated May 31, 2015

RE: McIntosh Mary St Sanitary Sewer Capacity ##MCG094590.0001

Don McNalty <Don.McNalty@rjburnside.com>

Sun 2015-05-31 11:50 PM

To: Mike Rawn <mrawn@clearview.ca>

Cc: Steve Sage <ssage@clearview.ca>; Rossalyn Workman <rworkman@clearview.ca>; Ron Kerr <Ron.Kerr@rjburnside.com>

2 attachments (74 KB)

05282010Mary St 2.8+Add Lands+50vs45+Brewery+148 Units.xls; 05302015Mary St 2.8+Add Lands+50vs45+Brewery+148 Units Revised per capita_ flow.xls;

Mike:

As noted on Friday I apologise for not having a report to you earlier in regard to the history and possible amended analysis of the Mary St sewer capacity. Any complete analysis requires a determination of the future anticipated development over the whole settlement community and as you know we have not been able to coordinate such an in depth review with Planning staff. It also may require a review and/or confirmation of the policies regarding the cost of future infrastructure and whether it is financed by particular development(s) or whether it becomes part of the development charges for sanitary servicing. It has been the Township's position in the past that additional infrastructure required to service the lands east of Mary St would be financed by the development lands situated east of Mary St..This position may be amended or not in the future.

When MMM originally designed the system and established the Sewer catchment area they used 4 persons per unit (PPU) and 450 l/cap/day ADF. Determining the exact PPU is not easy and it will be continually changing. We believe that the 4 PPU is very conservative for Creemore. I think the Stats CAnada number for Creemore was 2.48 and for planning purposes 2.8 PPU seemed to be the number of choice. Although MMM used a very conservative number for PPU they neglected to include some lands that are designated for development on the west and north edges of the settlement area. Of course the new Provincial Policy Statement (PPS) which requires intensification of the settlement area came after the MMM design as well. Further the expansion of the Creemore Springs Brewery was not anticipated in their design either. So in summary although they were very conservative on the PPU and the ADF daily flows per capita, the intensification numbers, addition of missed designated lands and the brewery expansion more than exceeded the conservative initial design.

The capacity of the Mary St sewer has been an issue with the McIntosh Development throughout the development process. It was one of the issues that was addressed before the OMB and within the Minutes of Settlement back in 2007 and then again when it went back before the OMB. Rossalyn had sent me the OMB decision and Minutes of Settlement from 2007. Although references to the agreements made around the sewer capacity issues are found in the Minutes of settlement and the attached subdivision draft plan conditions from 2007, I believe there were further concessions made by the Municipality when the matter went back to the OMB in 2010-2012 (?????)
My recollection is that there was agreement for the McIntosh Development to connect to the MAry St Sewer with security placed with the Township for the upgrading of the MAry St Sewer if ever required and if determined that an upgrade was not required through the de-designation of development lands, changes in development policies, reduced actual flows, etc that the security would be released. My understanding is that there was agreement reached in this regard which should be included in a later Minutes of Settlement and/or draft plan conditions.

The Township has never taken the position that there was not currently capacity in the Mary St sewer for the McIntosh Development. It is the possible future addition of the lands not accounted for in the MMM design, the infilling of lots, intensification of existing development that will consume considerable capacity in that sewer. It should also be noted

that the debate over the Mary St sewer occurred during a period where there were significant infiltration and extraneous flows entering the sewer system to the point where even the plant capacity was exceeded so there was a legitimate level of caution regarding the flows in the system.

Determination of future flows is not an exact science as there are a number of variables and there is a need for some engineering judgement. This is supported by the significant variation in the flows per capita, persons per unit infiltration parameters and peaking factors. We have run an endless number of scenarios with respect to the MAry St sewer over the years and some have shown that there is no reserve capacity and some have shown there is some capacity. We believe and agree with you that the flow per capita we have used in the past is conservative (450 l/cap/day ADF) With an annual ADF of 370 cu m/day to the plant and an assumed population of around 1500, the flow per person is more like 250 l/cap/day including infiltration. If you considered the Max day flow of around 600 cu m/day (MArch 2012), the flow per person would be 400 l/cap/day. There would still be a peaking factor applied to determine the anticipated peak flow in the collection system. Of course the demographics of the Creemore residents may change over time and the maintenance of the system to minimize extraneous flows will be ongoing.

Based on the past analysis, it appears that there are currently 573 units connected to the system in the Creemore settlement area. If one uses the 2.8 ppu which provides a little cushion for change in the demographics over the 2.48 PPU from the past Stats Canada numbers there is just over 1600 equivalent residential population in Creemore. Adding the designated Industrial and residential lands, the brewery expansion, increasing the density of development from 45 persons per hectare to 50 to comply to the PPS and allowing for 148 units of intensification for the lands situated west of Mary St will generate a total residential equivalent population of almost 5200 people or over three times the current population. Again this is only for the lands situated west of Mary St and therefore does not include the McIntosh development or the Alliance development or any other development situated east of Mary St.

We have attached the Sanitary Sewer design sheet which was prepared in May of 2010 based on the MOE guidelines for per capita flows and infiltration and allowing for the brewery expansion, higher density development as per the PPS, the development of the designated lands and 148 units of intensification. . It can be noted that under this scenario the total flow exceeds the capacity of the existing Mary St Sewer once complete build out occurs. There is no question that there is ample capacity in the sewer today .
(See attached file: 05282010Mary St 2.8+Add Lands+50vs45+Brewery+148 Units.xls)

From my discussion with Rossalyn on Friday it would be difficult to anticipate at this time the down designation of any of the designated lands. We could possibly have further discussions with the Planning department in this regard which may influence the capacity calculation but without those discussions it would be premature.

We have run the spreadsheet with a per capita ADF of 350 l/cap/day. This would be somewhere between the flow anticipated based on the Annual ADF of 250 and 400 from above which included infiltration and is likely representative of current flows. We have run this only for demonstration and establishing any revised per capita flow would require additional review and analysis.

(See attached file: 05302015Mary St 2.8+Add Lands+50vs45+Brewery+148 Units Revised per capita flow.xls)

It can be noted that the reduction of daily per capita flows results in reserve capacity being available in the Mary St Sewer. Of course if any of the development parameters were reduced there would be a further reduction in the anticipated flows.

In summary, there was agreement in the past that there was currently capacity in the Mary St sewer. It is the future development parameters and the build out of lands west of Mary St in accordance with those parameters which

SANITARY SEWER DESIGN SHEET — **Creemore Residential/McIntosh** **MCG09459**

MARY STREET - 2.8 PPU DESIGN + Additional Industrial/Residential Lands+Fut Res at 50 vs 45+Brewery Exp+148 units Intens.

PROJ. NAME: MARY STREET
COMMUNITY OF CREEMORE

SHEET: A
DATE: May 28, 2010

FLOWS
RESIDENTIAL-AVG DAILY PER CAP: 450
INDUST/COMM-AVG DAILY PER AREA: 75 Equiv persons/ha 33.75
RESIDENTIAL-AVG DAILY PER AREA: 50 Equiv.persons/ha 22.5

Qtotal=(P*q*M/86.4)+Qi
M=1+(14/4+P^0.5)
P=POPULATION/1000
Qi = 0.23 l/s/ha

PIPE CAPACITY DETERMINED BY MANNINGS
MANNING'S "n"= 0.013

LOCATION DEV.	FROM MH	TO MH	SAN D.A. NO.	UNITS (No)	DENS. (ppu)	EX. RES. EQUIV. POP.	FUT. & ADD. RES EQUIV. POP 50/ha	MMM I/C EQUIV. POP. 75/ha	ADD. I/C EQUIV. POP. 75/ha	ACCUM POP.		PEAK'NG FACTOR	RESIDENT.S EW.FLOW (l/s)	COMM. AREA (ha)	COMM.SEW. FLOW (l/s)	AREA* (ha)	ACC. A (ha)	INFILT. (l/s)	TOT FLOW (l/s)	LENGTH (m)	SLOPE (m/m)	PIPE DIAM (mm)	CAPACITY (l/s)	VEL. (m/s)
Mary Street	upst end	8	1	**44**	**2.8**	123	**250**			373	4.03628295	4.00	7.78	0.00	0.00	11.96	11.96	2.75	10.53	29.5	0.004	200	20.75	0.66
Mary Street	8	10	2	**0**	**2.8**	0	120			493	3.97727784	3.98	10.22	0.00	0.00	2.4	14.36	3.30	13.52	56	0.0046	200	22.25	0.71
Mary Street	10	11	3		**2.8**	0	99			592	3.93529028	3.94	12.14	0.00	0.00	2.16	16.52	3.80	15.94	87.5	0.004	200	20.75	0.66
Mary Street	11	15	4	**27**	**2.8**	90				682	3.90098083	3.90	13.86	0.00	0.00	3.37	19.89	4.57	18.44	123.7	0.004	200	20.75	0.66
Mary Street	15	42	5	**165**	**2.8**	462	**590**	334	703	2771	3.47144939	3.47	50.10	0.00	0.00	47.38	67.27	15.47	65.58	123	0.004	300	61.18	0.87
Mary Street	42	53	6	**40**	**2.8**	112		302		3185	3.42017198	3.42	56.74	0.00	0.00	8.87	76.14	17.51	74.25	120.7	0.004	375	110.92	1.00
Mary Street	53	56	7	**14**	**2.8**	39		121		3345	3.40176627	3.40	59.27	0.00	0.00	4	80.14	18.43	77.70	117.5	0.0042	375	113.66	1.03
Mary Street	56	83	8	**147**	**2.8**	412	**46**	303	**240**	4346	3.30085061	3.30	74.72	0.00	0.00	24.33	104.47	24.03	98.74	124.7	0.004	375	110.92	1.00
Mary Street	83	91	9	**72**	**2.8**	202	**414**			4962	3.2481058	3.25	83.94	0.00	0.00	9.27	113.74	26.16	110.10	125.1	0.004	375	110.92	1.00
Mary Street	91	99	10	**57**	**2.8**	160				5121	3.23534803	3.24	86.30	0.00	0.00	10	123.74	28.46	114.76	86	0.004	375	110.92	1.00
Mary Street	99	106	11	**7**	**2.8**	20	46			5187	3.23020326	3.23	87.26	0.00	0.00	2.21	125.95	28.97	116.23	86	0.004	375	110.92	1.00
Mary Street	106	102	12	**0**	**2.8**	0				5187	3.23020326	3.23	87.26	0.00	0.00	0.56	126.51	29.10	116.36	143.5	0.004	375	110.92	1.00
TOTAL				**573**		1619	1565	1060	943	5186.8									**116.36**					

SANITARY SEWER DESIGN SHEET — Creemore Residential/McIntosh MCG09459

MARY STREET - 2.8 PPU DESIGN + Additional Industrial/Residential Lands+Fut Res at 50 vs 45+Brewery Exp+148 units Intens.

PROJ. NAME: MARY STREET
COMMUNITY OF CREEMORE

SHEET: A
DATE: May 28, 2010

FLOWS
RESIDENTIAL-AVG DAILY PER CAP: 350
INDUST/COMM-AVG DAILY PER AREA: 75 Equiv persons/ha 33.75
RESIDENTIAL-AVG DAILY PER AREA: 50 Equiv.persons/ha 22.5

Qtotal=(P*q*M/86.4)+Qi
M=1+(14/4+P^0.5)
P=POPULATION/1000
Qi = 0.23 l/s/ha

PIPE CAPACITY DETERMINED BY MANNINGS
MANNING'S "n"= 0.013

LOCATION DEV.	FROM MH	TO MH	SAN D.A. NO.	UNITS (No)	DENS. (ppu)	EX. RES. EQUIV. POP.	FUT. & ADD. RES EQUIV. POP 50/ha	MMM I/C EQUIV. POP. 75/ha	ADD. I/C EQUIV. POP. 75/ha	ACCUM POP.		PEAK'NG FACTOR	RESIDENT.SEW.FLOW (l/s)	COMM. AREA (ha)	COMM.SEW. FLOW (l/s)	AREA* (ha)	ACC. A (ha)	INFILT. (l/s)	TOT FLOW (l/s)	LENGTH (m)	SLOPE (m/m)	PIPE DIAM (mm)	CAPACITY (l/s)	VEL. (m/s)
Mary Street	upst end	8	1	**44**	**2.8**	123	**250**			373	4.03628295	4.00	6.05	0.00	0.00	11.96	11.96	2.75	8.80	29.5	0.004	200	20.75	0.66
Mary Street	8	10	2	**0**	**2.8**	0	120			493	3.97727784	3.98	7.95	0.00	0.00	2.4	14.36	3.30	11.25	56	0.0046	200	22.25	0.71
Mary Street	10	11	3		**2.8**	0	99			592	3.93529028	3.94	9.44	0.00	0.00	2.16	16.52	3.80	13.24	87.5	0.004	200	20.75	0.66
Mary Street	11	15	4	**27**	**2.8**	90				682	3.90098083	3.90	10.78	0.00	0.00	3.37	19.89	4.57	15.36	123.7	0.004	200	20.75	0.66
Mary Street	15	42	5	**165**	**2.8**	462	**590**	334	703	2771	3.47144939	3.47	38.97	0.00	0.00	47.38	67.27	15.47	54.44	123	0.004	300	61.18	0.87
Mary Street	42	53	6	**40**	**2.8**	112		302		3185	3.42017198	3.42	44.13	0.00	0.00	8.87	76.14	17.51	61.64	120.7	0.004	375	110.92	1.00
Mary Street	53	56	7	**14**	**2.8**	39		121		3345	3.40176627	3.40	46.10	0.00	0.00	4	80.14	18.43	64.53	117.5	0.0042	375	113.66	1.03
Mary Street	56	83	8	**147**	**2.8**	412	**46**	303	**240**	4346	3.30085061	3.30	58.11	0.00	0.00	24.33	104.47	24.03	82.14	124.7	0.004	375	110.92	1.00
Mary Street	83	91	9	**72**	**2.8**	202	**414**			4962	3.2481058	3.25	65.28	0.00	0.00	9.27	113.74	26.16	91.44	125.1	0.004	375	110.92	1.00
Mary Street	91	99	10	**57**	**2.8**	160				5121	3.23534803	3.24	67.12	0.00	0.00	10	123.74	28.46	95.58	86	0.004	375	110.92	1.00
Mary Street	99	106	11	**7**	**2.8**	20	46			5187	3.23020326	3.23	67.87	0.00	0.00	2.21	125.95	28.97	96.84	86	0.004	375	110.92	1.00
Mary Street	106	102	12	**0**	**2.8**	0				5187	3.23020326	3.23	67.87	0.00	0.00	0.56	126.51	29.10	96.97	143.5	0.004	375	**110.92**	1.00
TOTAL				**573**		1619	1565	1060	943	5186.8									**96.97**					

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Appendix C

Mary Street Sanitary Sewer Design Sheet
Duplication of 1999 Design

SANITARY SEWER DESIGN SHEET #1 **Creemore Residential/McIntosh MCG09459**

MARY STREET - DUPLICATION OF MMM DESIGN (1999)

PROJ. NAME: MARY STREET
COMMUNITY OF CREEMORE
FLOWS

SHEET: A
DATE: May 25, 2010

RESIDENTIAL-AVG DAILY PER CAP: 450
INDUST/COMM-AVG DAILY PER AREA: 75 Equiv persons/ha 33.75
RESIDENTIAL-AVG DAILY PER AREA: 45 Equiv persons/ha 20.25

Qtotal=(P*q*M/86.4)+Qi
M=1+(14/4+P^0.5)
P=POPULATION/1000
Qi = 0.23 l/s/ha

PIPE CAPACITY DETERMINED BY MANNINGS
MANNING'S "n"= 0.013

LOCATION DEV.	FROM MH	TO MH	SAN D.A. NO.	UNITS (No)	DENS. (ppu)	EX. RES. POP.	FUT. RES. EQUIV. POP. 45/ha	IND/COM EQUIV. POP. 75/ha	FUT. I/C EQUIV.POP.	ACCUM POP.		PEAK'NG FACTOR	RESIDENT.SEW.FLOW (l/s)	COMM. AREA (ha)	COMM.SEW. FLOW (l/s)	AREA* (ha)	ACC. A (ha)	INFILT. (l/s)	TOT FLOW (l/s)	LENGTH (m)	SLOPE (m/m)	PIPE DIAM (mm)	CAPACITY (l/s)	VEL. (m/s)
Mary Street	upst end	8	1	44	4	176	160			336	4.05699879	4.00	7.00	0.00	0.00	11.96	11.96	2.75	9.75	29.5	0.004	200	20.75	0.66
Mary Street	8	10	2	0	4	0	108			444	4.00021435	4.00	9.25	0.00	0.00	2.4	14.36	3.30	12.55	56	0.0046	200	22.25	0.71
Mary Street	10	11	3		4	0	90			534	3.95935972	3.96	11.01	0.00	0.00	2.16	16.52	3.80	14.81	87.5	0.004	200	20.75	0.66
Mary Street	11	15	4	27	4	108				642	3.91590791	3.92	13.09	0.00	0.00	3.37	19.89	4.57	17.67	123.7	0.004	200	20.75	0.66
Mary Street	15	42	5	165	4	660	193	334		1829	3.61564648	3.62	34.44	0.00	0.00	30.51	50.40	11.59	46.03	123	0.004	300	61.18	0.87
Mary Street	42	53	6	40	4	160		302		2291	3.53917357	3.54	42.23	0.00	0.00	8.87	59.27	13.63	55.86	120.7	0.004	375	110.92	1.00
Mary Street	53	56	7	14	4	56		121		2468	3.51301971	3.51	45.16	0.00	0.00	4	63.27	14.55	59.71	117.5	0.0042	375	113.66	1.03
Mary Street	56	83	8	147	4	588		303		3359	3.40023694	3.40	59.49	0.00	0.00	23.42	86.69	19.94	79.43	124.7	0.004	375	110.92	1.00
Mary Street	83	91	9	61	4	244	13			3616	3.37224676	3.37	63.51	0.00	0.00	9.56	96.25	22.14	85.65	125.1	0.004	375	110.92	1.00
Mary Street	91	99	10	47	4	188	20			3824	3.3507662	3.35	66.74	0.00	0.00	7.63	103.88	23.89	90.63	86	0.004	375	110.92	1.00
Mary Street	99	106	11	7	4	28	41			3893	3.34385385	3.34	67.80	0.00	0.00	2.21	106.09	24.40	92.20	86	0.004	375	110.92	1.00
Mary Street	106	102	12	0	4	0				3893	3.34385385	3.34	67.80	0.00	0.00	0.56	106.65	24.53	92.33	143.5	0.004	375	110.92	1.00
TOTAL				552		2208	625	1060		3893									92.33					

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Appendix D

Catchment Area Plan
Prepared By Township



Creemore Sanitary Drainage
Mary Street Sewer
October 2021

Catchment Areas & Sub Areas

Base plan referenced from sanitary drainage plan prepared by Marshall, Macklin, Monaghan (May, 1999).

1:2,000

Document Name: WSI_Creemore_Sanitary_Drainage_36x36_2021

Date Saved: 2021-12-07 9:12:29 AM

- Sewer Manholes
- Sewer Gravity Mains
- County Road
- Local Road
- Local - Seasonal
- Private Road
- Unassumed Road
- Unopened Road
- Catchment Areas
- Catchment Sub Areas
- Additional Service Areas 2021
- Building Footprints
- Assessment Parcels

CLEARVIEW

The information presented on this map is provided as a public resource of general information by the Corporation of the Township of Clearview. The information is provided as a convenience only, and the Township of Clearview disclaims any responsibility for content, accuracy, and/or completeness. This is not a plan of survey, nor should this map be used for navigational, tracking or other purposes requiring precise measurements of distance or direction.

SIMCOE COUNTY LAND INFORMATION NETWORK COOPERATIVE LINC

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Appendix E

Mary Street Sewer Analysis Sheet 2021

SANITARY SEWER DESIGN SHEET — **Clearview Sewer and Water 300032944**

MARY STREET - 2.8 PPU DESIGN + Additional Industrial/Residential Lands+Fut Res at 50 vs 45+Brewery Exp+148 units Intens.

PROJ. NAME: MARY STREET
COMMUNITY OF CREEMORE

FLOWS

RESIDENTIAL-AVG DAILY PER CAP: 450
INDUST/COMM-AVG DAILY PER AREA: 75 Equiv persons/ha 33.75
RESIDENTIAL-AVG DAILY PER AREA: 50 Equiv.persons/ha 22.5

SHEET: A
DATE: July 12, 2021

Qtotal=(P*q*M/86.4)+Qi
M=1+(14/4+P^0.5)
P=POPULATION/1000
Qi = 0.23 l/s/ha

PIPE CAPACITY DETERMINED BY MANNINGS
MANNING'S "n"= 0.013

LOCATION DEV.	FROM	TO	SAN D.A.	UNITS	DENS.	EX. RES.	FUT & ADD RES	MMM I/C	ADD. I/C	ACCUM		PEAK'NG	RESIDENT.SEW.FLOW	COMM. AREA	COMM.SEW. FLOW	AREA*	ACC. A	INFILT.	TOT FLOW	LENGTH	SLOPE	PIPE DIAM	CAPACITY	VEL.
	MH	MH	NO.	**(No)**	(ppu)	EQUIV. POP.	EQUIV. POP	EQUIV. POP.	EQUIV. POP.	POP.		FACTOR	(l/s)	(ha)	(l/s)	(ha)	(ha)	(l/s)	(l/s)	(m)	(m/m)	(mm)	(l/s)	(m/s)
							50/ha	75/ha	75/ha															
Mary Street	upst end	8	1	**44**	**2.8**	123	**250**			373	4.03628295	4.00	7.78	0.00	0.00	11.96	11.96	2.75	10.53	29.5	0.004	200	20.75	0.66
Mary Street	8	10	2	**0**	**2.8**	0	120			493	3.97727784	3.98	10.22	0.00	0.00	2.4	14.36	3.30	13.52	56	0.0046	200	22.25	0.71
Mary Street	10	11	3		**2.8**	0	99			592	3.93529028	3.94	12.14	0.00	0.00	2.16	16.52	3.80	15.94	87.5	0.004	200	20.75	0.66
Mary Street	11	15	4	**27**	**2.8**	90	414			1096	3.77392747	3.77	21.55	0.00	0.00	3.37	19.89	4.57	26.12	123.7	0.004	200	20.75	0.66
Mary Street	15	42	5	**165**	**2.8**	462	**215**	334	176	2283	3.54036175	3.54	42.10	0.00	0.00	32.86	52.75	12.13	54.23	123	0.004	300	61.18	0.87
Mary Street	42	53	6	**40**	**2.8**	112		302		2697	3.48125085	3.48	48.90	0.00	0.00	8.87	61.62	14.17	63.08	120.7	0.004	375	110.92	1.00
Mary Street	53	56	7	**14**	**2.8**	39		121		2857	3.46029066	3.46	51.50	0.00	0.00	4	65.62	15.09	66.59	117.5	0.0042	375	113.66	1.03
Mary Street	56	83	8	**147**	**2.8**	412	**168**	303	**240**	3980	3.33528184	3.34	69.14	0.00	0.00	23.42	89.04	20.48	89.62	124.7	0.004	375	110.92	1.00
Mary Street	83	91	9	**74**	**2.8**	207				4187	3.3154791	3.32	72.31	0.00	0.00	11.21	100.25	23.06	95.36	125.1	0.004	375	110.92	1.00
Mary Street	91	99	10	**58**	**2.8**	162	20			4370	3.29871514	3.30	75.07	0.00	0.00	10	110.25	25.36	100.43	86	0.004	375	110.92	1.00
Mary Street	99	106	11	**7**	**2.8**	20	41			4430	3.29327591	3.29	75.99	0.00	0.00	2.21	112.46	25.87	101.85	86	0.004	375	110.92	1.00
Mary Street	106	102	12	**0**	**2.8**	0				4430	3.29327591	3.29	75.99	0.00	0.00	0.56	113.02	25.99	101.98	143.5	0.004	375	**110.92**	1.00
TOTAL				**576**		1627	1327	1060	416	4430.2									**101.98**					

Figure 2: Creemore East Sub Areas



2.3.5 Creemore Southeast

The Creemore Southeast Lands are located east and south of the Creemore East Lands and are comprised of lands identified as industrial (42.3 ha) and Hazard Lands in the Official Plan. Based on discussion with the Township, it is anticipated that the development of these lands is well into the future and that the nature of the development is such that it would be more readily serviced using individual private servicing and as such does not affect the projections for the municipal systems serving the Community of Creemore.

2.4 Summary of Projected Service Populations

A table detailing the development of the water service population is included in Appendix A. A table detailing the development of the wastewater service population is included in Appendix B. These tables identify the various assumptions used to develop the projected water and wastewater service populations discussed above. The water and wastewater service populations are summarized in Table 2 and Table 3 respectively. Within the 20-year planning horizon, it is anticipated that an additional 3,811 equivalent persons need to be accommodated.

Table 2: Historical and Projected Equivalent Service Populations - Water

	Creemore Core	Creemore West	Creemore North	Creemore East	Creemore Southeast	Total Additional Population	Cumulative Total Projected Population
Existing (2021)	2,190	-	-	-	-	-	2,190
2022-2042	349	-	-	3,417	-	3,766	5,956
2042-Build Out	2,878	-	842	-	-	3,720	9,676
Total	5,417	-	842	3,417	-	-	9,676

Table 3: Historical and Projected Equivalent Service Populations - Wastewater

	Creemore Core	Creemore West	Creemore North	Creemore East	Creemore Southeast	Total Additional Population	Cumulative Total Projected Population
Existing (2021)	2,051	-	-	-	-	-	2,051
2022-2042	349	-	-	3,417	-	3,766	5,817
2042-Build Out	2,953	-	842	-	-	3,795	9,612
Total	5,353	-	842	3,417	-	-	9,612

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Appendix B

Wastewater Flow

Creemore MSP - Wastewater Flow Projections

Design Criteria

Flow Per Person	350	L/p/d
Avg i/i	90	L/p/d
Peak i/i	227	L/p/d
Persons Per Unit	3	ppu
Actual Residential Service Pop	1128	persons
Eq. Residential Service Poplation associted with I/C/I	923	persons
Include Eq. Population in Exsiting Population	Y	
Ex. Equivalent Service Population	2051	persons

			Average Flows (m3/d)							Peak Flows (m3/d)			
	Units	PPU	Persons	Cumul. Pop.	ADF(dry) m3/d	Cumil. Dry ADF	Avg. Infil	Cumil. Avg Inf m3/d	Cumul. ADF m3/d	Peaking Factor	Cumul. Peak Dry m3/d	Cumul. Peak I/I m3/d	Peak Flow m3/d
2021 Ex. ADF			2051	2051		398			398	3.58	1423.7	465.6	1889
Allocations			349	2400	122	520	31	31	552	3.52	1832.4	544.8	2377
Area CE-1	546	3	1638	4038	573	1093	147	179	1272	3.33	3640.8	916.6	4557
AreaCE-2	57	3	171	4209	60	1153	15	194	1348	3.31	3821.4	955.4	4777
Area CE-3	536	3	1608	5817	563	1716	145	339	2055	3.18	5463.1	1320.5	6784
Village Core			2953	8770	1034	2750	266	605	3354	3.01	8279.4	1990.8	10270
Village North	281	3	843	9613	295	3045	76	681	3725	2.97	9047.9	2182.2	11230

Appendix C:
Site Water Calculations

TATHAM ENGINEERING	PROJECT	7582 County Road 9	FILE 424350
			DATE 3/20/2025
	SUBJECT	Water Demand Calculations	NAME DB
			PAGE 1 OF 2

1.1 Demand Criteria

Design Criteria as per Township of Clearview *Engineering Standards* (2016)

Per Capita Demand	=	350	L/cap/day
Residential Population Density	=	2.8	PPU

Site Information as per *Conceptual Site Plan* by IPS (dated March 6, 2025)

[1]Fire Flow value carried from MSP - refer to Section 3.2.3 of FSR.

Peaking Factors

Residential	Maximum Day Factor*	=	2.25
	Peak Hour Factor	=	3.38

*From Table 3-1 of MECP's Design Guidelines for Drinking-Water Systems

1.2 Entire Development - Water Demands

Total Units = 124 units Population (rounded up) = 348 ppl

Average Day Demand (ADD)	=	121,800	L/day	=	121.80	m^3/day	=	1.41	L/s
Maximum Day Demand (MDD)	=	121,800	L/day	x	2.25				
	=	274,050	L/day	=	274.05	m^3/day	=	3.17	L/s
MDD + Fire Flow (FF)[1]	=	3.17	L/s	+	95.00	L/s	=	98.17	L/s
Peak Hour Demand (PHD)	=	121,800	L/day	x	3.38				
	=	411,684	L/day	=	411.68	m^3/day	=	4.77	L/s

1.3 Proposed Condominium Development - Water Demands

Stacked/Standard Townhouses = 116 units Population (rounded up) = 325 ppl

Average Day Demand (ADD)	=	113,750	L/day	=	113.75	m^3/day	=	1.32	L/s
Maximum Day Demand (MDD)	=	113,750	L/day	x	2.25				
	=	255,938	L/day	=	255.94	m^3/day	=	2.96	L/s
MDD + Fire Flow (FF)[1]	=	2.96	L/s	+	95.00	L/s	=	97.96	L/s
Peak Hour Demand (PHD)	=	113,750	L/day	x	3.38				
	=	384,475	L/day	=	384.48	m^3/day	=	4.45	L/s

1.3 Proposed County Road 9 Lots - Water Demands

Single Detached Units	=	6	units				Population (rounded up)	=	17	ppl	
Average Day Demand (ADD)	=	5,950	L/day	=	5.95	m^3/day	=	0.07	L/s		
Maximum Day Demand (MDD)	=	5,950	L/day	x	2.25						
	=	13,388	L/day	=	13.39	m^3/day	=	0.15	L/s		
MDD + Fire Flow (FF)[1]	=	0.15	L/s	+	95.00	L/s	=	95.15	L/s		
Peak Hour Demand (PHD)	=	5,950	L/day	x	3.38						
	=	20,111	L/day	=	20.11	m^3/day	=	0.24	L/s		

1.3 Proposed Collingwood Street Lots - Water Demands

Single Detached Units	=	2	units				Population (rounded up)	=	6	ppl	
Average Day Demand (ADD)	=	2,100	L/day	=	2.10	m^3/day	=	0.03	L/s		
Maximum Day Demand (MDD)	=	2,100	L/day	x	2.25						
	=	4,725	L/day	=	4.73	m^3/day	=	0.05	L/s		
MDD + Fire Flow (FF)[1]	=	0.05	L/s	+	95.00	L/s	=	95.05	L/s		
Peak Hour Demand (PHD)	=	2,100	L/day	x	3.38						
	=	7,098	L/day	=	7.10	m^3/day	=	0.09	L/s		

Appendix D:
Site Wastewater Calculations

TATHAM ENGINEERING

PROJECT	7582 County Road 9, Creemore	FILE	424350
		DATE	3/20/2025
SUBJECT	Sanitary Flow Calculations	NAME	DB
		PAGE	1 OF 2

1.1 Demand Criteria

Design Criteria as per Township of Clearview's *Engineering Standards* (March 2017)

Population Densities and Average Day Flows from Creemore Water and Wastewater Master Servicing Plan's Wastewater Flow Projections, prepared by R.J. Burnside & Associates Ltd. (May 2024)

Per Capita Flow	=	350	L/cap/day	Peaking Factor	
Extraneous Flow (Infiltration)	=	0.23	L/s/ha	Residential	= Harmon
Residential Population Density	=	2.8	PPU (all types)		= 1+14/(4+0.399^0.5)
Site Information as per *Conceptual Site Plan* by IPS (dated March 6, 2025)					= 4.05

1.2 Entire Development - Sewage Generation

Total Units	=	124	units			Population (rounded up)	=	348	ppl	
Development Area	=	2.91	ha							
Average Day Flow (ADF)	=	121,800	L/day	=	121.80	m^3/day	=	1.41	L/s	
Peak Flow (PF)	=	121,800	L/day	x	4.05		=	5.71	L/s	
Total Extraneous Flow	=	57,828	L/day	=	57.83	m^3/day	=	0.67	L/s	
PF (incl. extraneous)	=	551,138	L/day	=	551.14	m^3/day	=	6.38	L/s	

1.3 Proposed Condominium Development - Sewage Generation

Stacked/Standard Townhouses	=	116	units			Population	=	325	ppl	
Development Area	=	2.51	ha							
Average Day Flow (ADF)	=	113,680	L/day	=	113.68	m^3/day	=	1.32	L/s	
Peak Flow (PF)	=	113,680	L/day	x	4.05		=	5.33	L/s	
Total Extraneous Flow	=	49,879	L/day	=	49.88	m^3/day	=	0.58	L/s	
PF (incl. extraneous)	=	510,302	L/day	=	510.30	m^3/day	=	5.91	L/s	

1.3 Proposed County Road 9 Lots - Sewage Generation

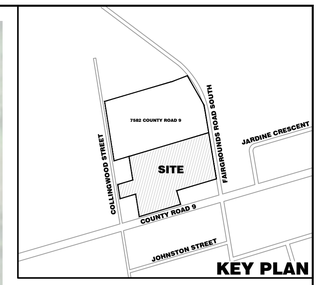
Stacked/Standard Townhouses	=	6	units			Population	=	17	ppl	
Development Area	=	0.27	ha							
Average Day Flow (ADF)	=	5,880	L/day	=	5.88	m^3/day	=	0.07	L/s	
Peak Flow (PF)	=	5,880	L/day	x	4.05		=	0.28	L/s	
Total Extraneous Flow	=	5,330	L/day	=	5.33	m^3/day	=	0.06	L/s	
PF (incl. extraneous)	=	29,145	L/day	=	29.14	m^3/day	=	0.34	L/s	

1.4 Proposed Collingwood Street Lots - Sewage Generation

Stacked/Standard Townhouses	=	2	units			Population		=	6	ppl
Development Area	=	0.14	ha							
Average Day Flow (ADF)	=	1,960	L/day	=	1.96	m^3/day	=	0.02	L/s	
Peak Flow (PF)	=	1,960	L/day	x	4.05		=	0.09	L/s	
Total Extraneous Flow	=	2,868	L/day	=	2.87	m^3/day	=	0.03	L/s	
PF (incl. extraneous)	=	10,806	L/day	=	10.81	m^3/day	=	0.13	L/s	

Appendix E:
Preliminary Drawings





LEGEND

- 270.00 — EXISTING CONTOUR
- CATCHMENT BOUNDARY
- PRE-DEVELOPMENT OVERLAND FLOW DIRECTION
- 101 / 2.21 | 0.35 — CATCHMENT ID / AREA (ha) | RUNOFF COEFFICIENT

DISCLAIMER AND COPYRIGHT

CONTRACTOR MUST VERIFY ALL DIMENSIONS AND BE RESPONSIBLE FOR SAME. ANY DISCREPANCIES MUST BE REPORTED TO THE ENGINEER BEFORE COMMENCING WORK. DRAWINGS ARE NOT TO BE SCALED.

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BENCHMARK

TBM-1 NAIL SPIKE IN HYDROPOLE ON NORTH SIDE OF COUNTY ROAD 9 (CR9), EAST OF GRAVEL DRIVEWAY ENTRANCE TO #7582 CR9 (264.63 m).

TBM-2 NAIL SPIKE IN HYDROPOLE ON NORTH SIDE OF COUNTY ROAD 9 (CR9), WEST OF GRAVEL DRIVEWAY ENTRANCE TO #7582 CR9 (264.67 m).

NOTES

SITE PLAN COMPLETED BY INNOVATIVE PLANNING SOLUTIONS INC., DATED MARCH 20, 2025.

LOT LINE FABRIC FROM LEGAL SURVEY COMPLETED BY RUDY MAK ON MAY 21, 2025.

TOPOGRAPHIC SURVEY COMPLETED BY TATHAM ENGINEERING LIMITED, DATED JANUARY 26, 2024.

EXISTING INFRASTRUCTURE AS-BUILT INFORMATION PROVIDED BY TOWNSHIP OF CLEARVIEW.

No.	REVISION DESCRIPTION	DATE
1.	ISSUED FOR ZBA – FIRST SUBMISSION	JULY 18, 2025

ENGINEER STAMP

LICENSED PROFESSIONAL ENGINEER
D. BRITO
100521347
July 18th, 2025
PROVINCE OF ONTARIO

7582 COUNTY ROAD 9
CREEMORE, TOWNSHIP OF CLEARVIEW

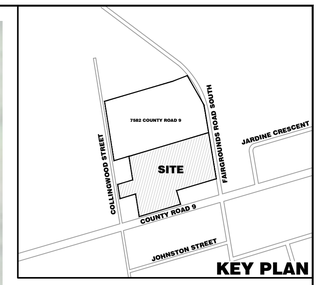
TATHAM ENGINEERING

PRE-DEVELOPMENT DRAINAGE PLAN

DESIGN: JRL	FILE: 424350	DWG: DP.1
DRAWN: JRL	DATE: MAY 2024	
CHECK: DB/JG	SCALE: 1:1000	

Drawing Name: 424350-DP01.dwg, Plotted: Jul 17, 2025





LEGEND

- 270.00 — EXISTING CONTOUR
- CATCHMENT BOUNDARY
- PRE-DEVELOPMENT OVERLAND FLOW DIRECTION
- POST-DEVELOPMENT OVERLAND FLOW DIRECTION
- 201 / 2.75 | 0.70 — CATCHMENT ID, RUNOFF COEFFICIENT, AREA (ha)

DISCLAIMER AND COPYRIGHT

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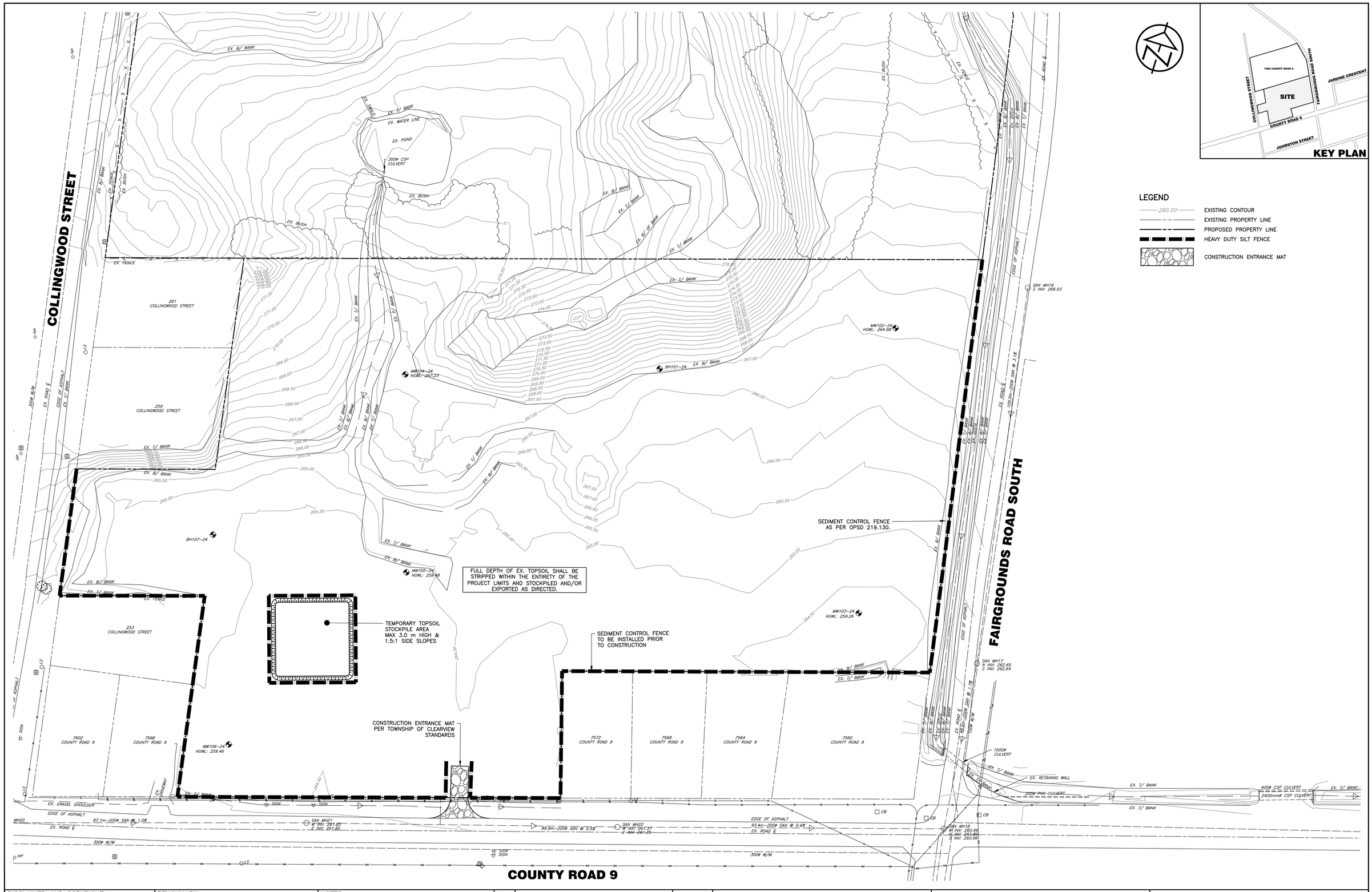
7582 COUNTY ROAD 9
CREEMORE, TOWNSHIP OF CLEARVIEW

TATHAM ENGINEERING

POST-DEVELOPMENT DRAINAGE PLAN

DESIGN: JRL	FILE: 424350	DWG:	
DRAWN: JRL	DATE: MAY 2024	DP.2	
CHECK: DB	SCALE: 1:1000		

Drawing Name: 424350-DP02.dwg, Plotted: Jul 17, 2025



LEGEND

- 280.00 — EXISTING CONTOUR
- EXISTING PROPERTY LINE
- PROPOSED PROPERTY LINE
- HEAVY DUTY SILT FENCE
- CONSTRUCTION ENTRANCE MAT

DISCLAIMER AND COPYRIGHT

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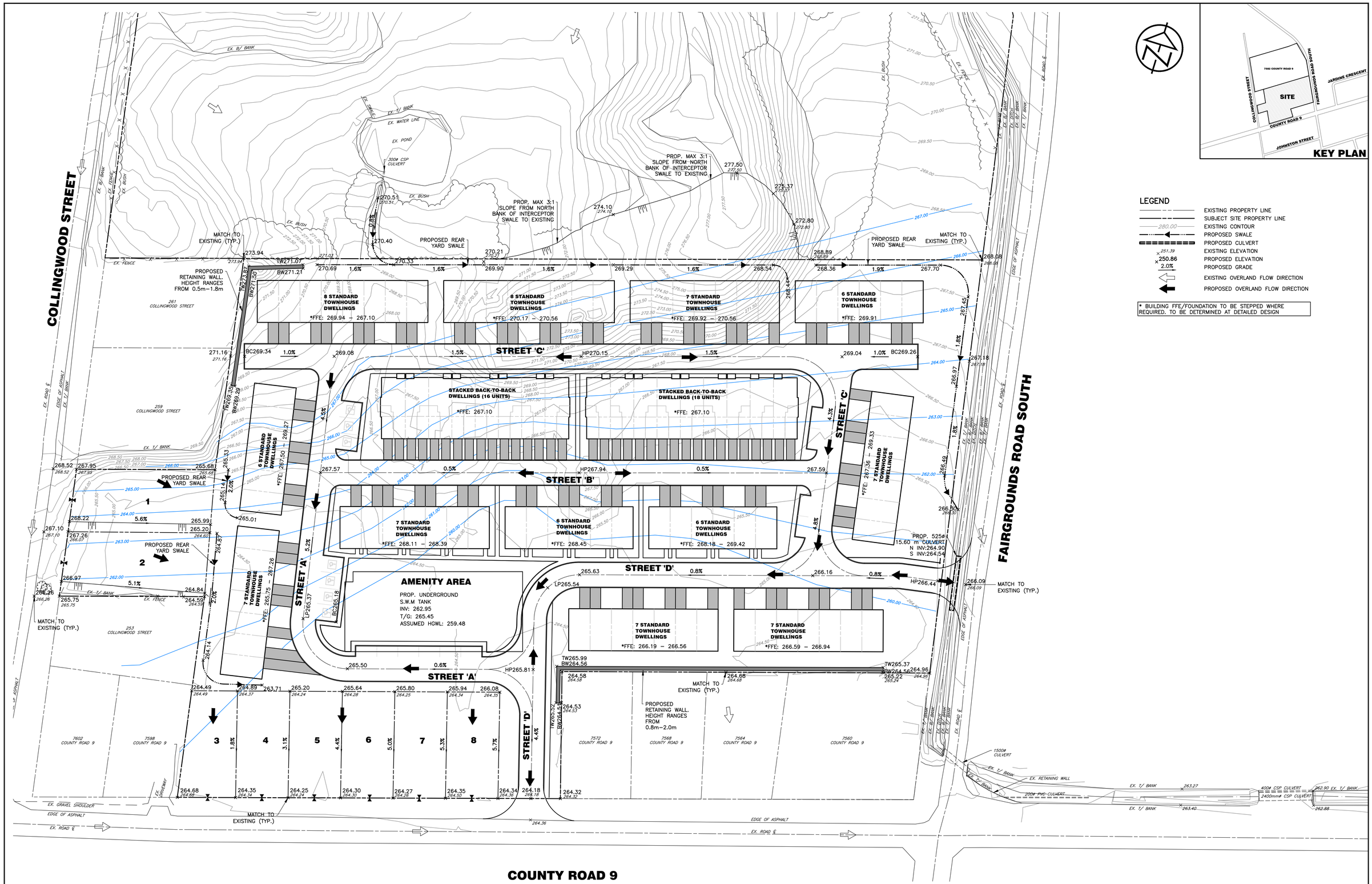
7582 COUNTY ROAD 9
CREEMORE, TOWNSHIP OF CLEARVIEW

TATHAM ENGINEERING

PRELIMINARY EROSION & SEDIMENT CONTROL PLAN

DESIGN: JRL	FILE: 424350	DWG: SC.1
DRAWN: JRL	DATE: MAY 2024	
CHECK: DB	SCALE: 1:500	

Drawing Name: 424350–SC01.dwg, Plotted: Jul 17, 2025



LEGEND

- EXISTING PROPERTY LINE
- SUBJECT SITE PROPERTY LINE
- 280.00 EXISTING CONTOUR
- PROPOSED SWALE
- PROPOSED CULVERT
- 251.39 EXISTING ELEVATION
- 250.86 PROPOSED ELEVATION
- 2.0% PROPOSED GRADE
- EXISTING OVERLAND FLOW DIRECTION
- PROPOSED OVERLAND FLOW DIRECTION

* BUILDING FFE/FOUNDATION TO BE STEPPED WHERE REQUIRED. TO BE DETERMINED AT DETAILED DESIGN

DISCLAIMER AND COPYRIGHT

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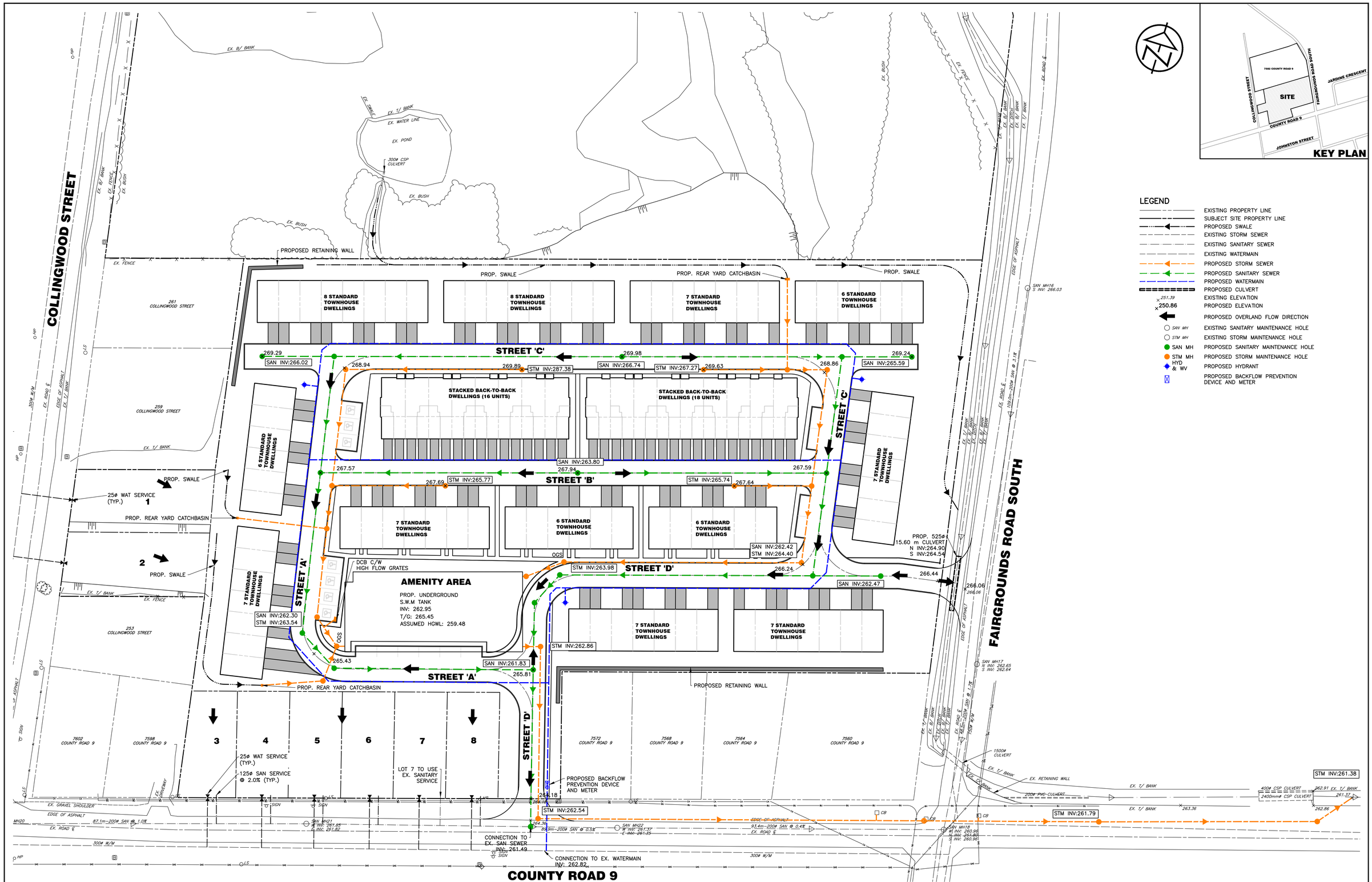
7582 COUNTY ROAD 9
CREEMORE, TOWNSHIP OF CLEARVIEW

TATHAM ENGINEERING

PRELIMINARY
SITE GRADING PLAN

DESIGN: JRL	FILE: 424350	DWG: SG.1
DRAWN: JRL	DATE: MAY 2024	
CHECK: DB	SCALE: 1:500	

Drawing Name: 424350–SG01.dwg, Plotted: Jul 17, 2025



LEGEND

- EXISTING PROPERTY LINE
- SUBJECT SITE PROPERTY LINE
- PROPOSED SWALE
- EXISTING STORM SEWER
- EXISTING SANITARY SEWER
- EXISTING WATERMAIN
- PROPOSED STORM SEWER
- PROPOSED SANITARY SEWER
- PROPOSED WATERMAIN
- PROPOSED CULVERT
- EXISTING ELEVATION
- PROPOSED ELEVATION
- PROPOSED OVERLAND FLOW DIRECTION
- EXISTING SANITARY MAINTENANCE HOLE
- EXISTING STORM MAINTENANCE HOLE
- PROPOSED SANITARY MAINTENANCE HOLE
- PROPOSED STORM MAINTENANCE HOLE
- PROPOSED HYDRANT
- PROPOSED BACKFLOW PREVENTION DEVICE AND METER

DISCLAIMER AND COPYRIGHT

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TOPOGRAPHIC SURVEY COMPLETED BY TATHAM ENGINEERING LIMITED, DATED JANUARY 26, 2024.

EXISTING INFRASTRUCTURE AS–BUILT INFORMATION PROVIDED BY TOWNSHIP OF CLEARVIEW.

No.	REVISION DESCRIPTION	DATE
1.	ISSUED FOR ZBA – FIRST SUBMISSION	JULY 18, 2025

ENGINEER STAMP

LICENSED PROFESSIONAL ENGINEER
D. BRITO
100521347
July 18th, 2025
PROVINCE OF ONTARIO

7582 COUNTY ROAD 9
CREEMORE, TOWNSHIP OF CLEARVIEW

TATHAM ENGINEERING

PRELIMINARY
SITE SERVICING PLAN

DESIGN: JRL	FILE: 424350	DWG: SS.1
DRAWN: JRL	DATE: MAY 2024	
CHECK: DB	SCALE: 1:500	

Drawing Name: 424350–SS01.dwg, Plotted: Jul 17, 2025

Appendix F:
Site Stormwater Management Calculations

Rational Method Parameter Calculations

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	101
Catchment Area (ha)	5.69
Impervious %	4%

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	5.69			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2	0.21	100	0.95									
Gravel	3		89	0.50									
Woodland	10	4.77	60	0.30									
Pasture/Lawns	5		69	0.35									
Meadows	8	1.92	65	0.33									
Cultivated	7		74	0.45									
Waterbody	12		50	0.05									
Average CN			75.92										
Average C			0.40										
Average IA			11.16										

Time to Peak Calculations

Max Catchment Elevation (m)	305.00
Min Catchment Elevation (m)	264.18
Catchment Length (m)	428
Catchment Slope (%)	9.54%
Method Airport Method	
Time of Concentration (mins)	22.55

Summary

Catchment CN	75.9
Catchment C	0.40
Catchment IA (mm)	11.16
Time of Concentration (hrs)	0.38
Catchment Time to Peak (hrs)	0.25
Catchment Time Step (mins)	3.01

Rational Method Parameter Calculations

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	102
Catchment Area (ha)	1.54
Impervious %	

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	1.54			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2		100	0.95									
Gravel	3		89	0.50									
Woodland	10	0.71	60	0.30									
Pasture/Lawns	5		69	0.35									
Meadows	8	0.83	65	0.33									
Cultivated	7		74	0.45									
Waterbody	12		50	0.05									
Average CN			62.69										
Average C			0.31										
Average IA			8.92										

Time to Peak Calculations

Max Catchment Elevation (m)	284.20
Min Catchment Elevation (m)	264.64
Catchment Length (m)	293
Catchment Slope (%)	6.68%
Method Airport Method	
Time of Concentration (mins)	23.46

Summary

Catchment CN	62.7
Catchment C	0.31
Catchment IA (mm)	8.92
Time of Concentration (hrs)	0.39
Catchment Time to Peak (hrs)	0.26
Catchment Time Step (mins)	3.13

TATHAM ENGINEERING

Visual OTTHYMO Model Parameter Calculations (NasHYD)

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	201
Catchment Area (ha)	2.79
Impervious %	63%

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	2.79			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2	1.77	100	0.95									
Gravel	3		89	0.50									
Woodland	10	0.19	60	0.30									
Pasture/Lawns	5	0.76	69	0.35									
Meadows	8	0.07	65	0.33									
Cultivated	7		74	0.45									
Waterbody	12		50	0.05									
Average CN		87.95											
Average C		0.73											
Average IA		3.51											

Time to Peak Calculations

Max Catchment Elevation (m)	276.71
Min Catchment Elevation (m)	265.50
Catchment Length (m)	166
Catchment Slope (%)	6.75%
Method Bransby-Williams Formula	
Time of Concentration (mins)	5.83

Summary

Catchment CN	88.0
Catchment C	0.73
Catchment IA (mm)	3.51
Time of Concentration (hrs)	0.10
Catchment Time to Peak (hrs)	0.06
Catchment Time Step (mins)	0.78

TATHAM ENGINEERING

Visual OTTHYMO Model Parameter Calculations (NasHYD)

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	202
Catchment Area (ha)	0.33
Impervious %	64%

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	0.33			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2	0.21	100	0.95									
Gravel	3		89	0.50									
Woodland	10		60	0.25									
Pasture/Lawns	5	0.12	69	0.28									
Meadows	8		65	0.27									
Cultivated	7		74	0.35									
Waterbody	12		50	0.05									
Average CN		88.73											
Average C		0.71											
Average IA		3.09											

Time to Peak Calculations

Max Catchment Elevation (m)	265.81
Min Catchment Elevation (m)	264.20
Catchment Length (m)	36.6
Catchment Slope (%)	4.40%
Method Bransby-Williams Formula	
Time of Concentration (mins)	1.73

Summary

Catchment CN	88.7
Catchment C	0.71
Catchment IA (mm)	3.09
Time of Concentration (hrs)	0.03
Catchment Time to Peak (hrs)	0.02
Catchment Time Step (mins)	0.23

TATHAM ENGINEERING

Visual OTTHYMO Model Parameter Calculations (NasHYD)

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	203
Catchment Area (ha)	0.91
Impervious %	3%

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	0.91			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2	0.03	100	0.95									
Gravel	3		89	0.50									
Woodland	10	0.50	60	0.30									
Pasture/Lawns	5	0.18	69	0.35									
Meadows	8	0.20	65	0.33									
Cultivated	7		74	0.45									
Waterbody	12		50	0.05									
Average CN		64.20											
Average C		0.34											
Average IA		8.31											

Time to Peak Calculations

Max Catchment Elevation (m)	284.20
Min Catchment Elevation (m)	264.68
Catchment Length (m)	255
Catchment Slope (%)	7.65%
Method Airport Method	
Time of Concentration (mins)	20.30

Summary

Catchment CN	64.2
Catchment C	0.34
Catchment IA (mm)	8.31
Time of Concentration (hrs)	0.34
Catchment Time to Peak (hrs)	0.23
Catchment Time Step (mins)	2.71

Rational Method Parameter Calculations

Project Details

7582 County Road 9, Creemore	424350

Prepared By

JRL	July 2, 2025

Data Sources

Detailed Soil Survey Reports for Ontario, NVCA Stormwater Technical Guide (2013), MTO Drainage Management Manual (1997)

Pre-Development Condition

Watershed	NVCA
Catchment ID	204
Catchment Area (ha)	3.14
Impervious %	1%

Average Curve Number (CN), Runoff Coefficient (C) & Initial Abstraction (IA)

Soil Symbol	**Opl**			
Soil Series	Osprey			
Hydrologic Soils Group	B			
Soil Texture	Loam or Silt Loam			
Runoff Coefficient Type	2			
Area (ha)	3.14			
Percentage of Catchment	100%			

Land Cover Category	**IA**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**	**A (ha)**	**CN**	**C**
Impervious	2	0.02	100	0.95									
Gravel	3		89	0.50									
Woodland	10	2.34	60	0.30									
Pasture/Lawns	5		69	0.35									
Meadows	8	0.78	65	0.33									
Cultivated	7		74	0.45									
Waterbody	12		50	0.05									
Average CN		61.50											
Average C		0.31											
Average IA		9.45											

Time to Peak Calculations

Max Catchment Elevation (m)	305.00
Min Catchment Elevation (m)	265.50
Catchment Length (m)	528
Catchment Slope (%)	7.48%
Method Airport Method	
Time of Concentration (mins)	30.45

Summary

Catchment CN	61.5
Catchment C	0.31
Catchment IA (mm)	9.45
Time of Concentration (hrs)	0.51
Catchment Time to Peak (hrs)	0.34
Catchment Time Step (mins)	4.06

TATHAM ENGINEERING

Rational Method Calculation

Project Details

7582 County Road 9, Creemore	424350

Municipality

Township of Clearview

Prepared By

JRL	7/2/2025

IDF Parameters

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
a	20.700	27.400	31.900	37.500	41.600	45.800
b	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (m^3/s)$$

where; C = Runoff Coefficient
I = Rainfall Intensity (mm/hr)
A = Drainage Area (ha)

Rainfall Intensity Formula

$$I = A \cdot \left(\frac{T_C}{60}\right)^B$$

where; I = Rainfall Intensity for 2-100 year storms (mm/hr)
A = IDF Parameter
B = IDF Parameter
T_c = Time of Concentration (min)

Rainfall Intensity Formula (25mm storm event)

I_{25mm} = (43 x C) + 5.9 (mm/hr)

where; C = Runoff Coefficient

Runoff Coefficient Factors

2-yr	C_2 =	1.00
5-yr	C_5 =	1.00
10-yr	C_{10} =	1.00
25-yr	C_{25} =	1.10
50-yr	C_{50} =	1.20
100-yr	C_{100} =	1.25

		PRE-DEVELOPMENT					POST-DEVELOPMENT								
Catchment ID		102	**(EAST)**	101	**(SOUTH)**	**(SITE)**	201	204	**CONTROLLED (SOUTH)**	202	**UNCONTROLLED (SOUTH)**	203	**UNCONTROLLED (EAST)**	**UNCONTROLLED (SITE)**	**TOTAL (SITE)**
A	(ha)	1.54	**1.54**	5.69	**5.69**	**7.23**	2.79	3.14	**5.93**	0.34	**0.34**	0.92	**0.92**	**1.26**	**7.19**
Tc	(min)	23.5	**23.46**	22.6	**22.55**	**-**	10.0	30.5	**10.00**	10.0	**10.0**	20.4	**20.4**	**-**	**-**
C		0.31	**0.31**	0.40	**0.40**	**0.38**	0.73	0.31	**0.51**	0.71	**0.71**	0.34	**0.34**	**0.44**	**0.50**
Q_{25mm}	(m^3/s)	0.026	**0.026**	0.146	**0.146**	**0.172**	0.211	0.052	**0.232**	0.024	**0.024**	0.018	**0.018**	**0.038**	**0.269**
Q_2	(m^3/s)	0.053	**0.053**	0.259	**0.259**	**0.312**	0.410	0.090	**0.606**	0.049	**0.049**	0.038	**0.038**	**0.087**	**0.692**
Q_5	(m^3/s)	0.070	**0.070**	0.343	**0.343**	**0.413**	0.542	0.119	**0.802**	0.064	**0.064**	0.051	**0.051**	**0.115**	**0.917**
Q_{10}	(m^3/s)	0.082	**0.082**	0.400	**0.400**	**0.481**	0.631	0.139	**0.933**	0.075	**0.075**	0.059	**0.059**	**0.134**	**1.067**
Q_{25}	(m^3/s)	0.105	**0.105**	0.517	**0.517**	**0.622**	0.817	0.179	**1.207**	0.097	**0.097**	0.076	**0.076**	**0.173**	**1.380**
Q_{50}	(m^3/s)	0.128	**0.128**	0.625	**0.625**	**0.753**	0.988	0.217	**1.460**	0.117	**0.117**	0.092	**0.092**	**0.209**	**1.670**
Q_{100}	(m^3/s)	0.146	**0.146**	0.717	**0.717**	**0.864**	1.133	0.249	**1.675**	0.134	**0.134**	0.106	**0.106**	**0.240**	**1.915**

TATHAM ENGINEERING

Modified-Rational Method Calculation

Project Details

7582 County Road 9, Creemore	424350

Municipality

Township of Clearview

Prepared By

JRL	7/2/2025

IDF Parameters

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
a	20.700	27.400	31.900	37.500	41.600	45.800
b	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Rational Method Formula

$$Q = \frac{C \times I \times A}{360} \quad (m^3/s)$$

where; C = Runoff Coefficient
I = Rainfall Intensity (mm/hr)
A = Drainage Area (ha)

Rainfall Intensity Formula

$$I = A \cdot \left(\frac{T_C}{60}\right)^B$$

where; I = Rainfall Intensity for 2-100 year storms (mm/hr)
A = IDF Parameter
B = IDF Parameter
T_c = Time of Concentration (min)

Runoff Coefficient Factors

2-yr	C_2 =	1.00
5-yr	C_5 =	1.00
10-yr	C_{10} =	1.00
25-yr	C_{25} =	1.10
50-yr	C_{50} =	1.20
100-yr	C_{100} =	1.25

Runoff Volume

$V_{Runoff} = Q_{Runoff} \times t_d \quad (m^3)$

where; Q_{Runoff} = Runoff Peak Flow Rate (m^3/s)
t_d = Storm Duration (sec)

Released Volume

$V_{Released} = Q_{Released} \times (t_d + T_c)/2 \quad (m^3)$

where; $Q_{released}$ = Max. Release Rate (m^3/s)
t_d = Storm Duration (sec)
T_c = Time of Concentration (sec)

Max. Storage Required

$V_{Storage} = V_{Runoff} - V_{Released} \quad (m^3)$

where; V_{Runoff} = Runoff Volume (m^3)
$V_{Released}$ = Released Volume (m^3)

Catchment ID	A (ha)	C	Tc (min)	T_i (min)	$Q_{existing}$	$Q_{uncontrolled}$	$Q_{released}$ (m^3/s)	Q_{total}	$V_{Storage}$ (m^3)
POST-DEVELOPMENT									
2-yr	5.93	0.51	10.0	5.0	0.312	0.087	0.225	0.312	245
5-yr	5.93	0.51	10.0	5.0	0.413	0.115	0.298	0.413	325
10-yr	5.93	0.51	10.0	5.0	0.481	0.134	0.279	0.413	445
25-yr	5.93	0.56	10.0	5.0	0.622	0.173	0.240	0.413	744
50-yr	5.93	0.61	10.0	5.0	0.753	0.209	0.204	0.413	1089
100-yr	5.93	0.63	10.0	5.0	0.864	0.240	0.173	0.413	1450

Notes: Post-development release rates per design storm overcontrolled to overall site 5-yr pre develpoment release rate (0.413 m3/s) less the uncontrolled release rate. Volume storage accounts for external drainage area.

Design volume tank to store 1450 cu.m

PROJECT	7582 County Road 9, Creemore	FILE	424350
		DATE	04 29 2025
SUBJECT	Water Quality - Treatment Train Calculation	NAME	JRL
		PAGE	1 OF 1

Treatment Train

Drainage Area to Controls= 5.93 ha Imperviousness of Drainage Area to Controls= 80%

Device	Target Total Suspended Solids (TSS) Removal	
Primary Treatment	OGS	60%
Secondary Treatment	Underground Storage Tank Sediment Debris Row	84%

$$\boldsymbol{TSS\ Removal} = \boldsymbol{(1 - ((1 - R_P) * (1 - R_S))}$$

where:

R_P = % TSS Removal provided by Primary Treatment Strategy

R_S = % TSS Removal provided by Secondary Treatment Strategy

TSS Removal Provided By Controls : 94%

Note: 1. Refer to Appendix G for %TSS removal values and verification reports
2. Treatment train has been designed to overtreat received runoff to offset runoff TSS from catchments (Catchment 202, 203, 205) unable to drain to treatment system

DM-OS +HS Water Quality Flow Rate Design Report

DM-OS + HS290 Design Project Details
7529 County Road 9, Creemore ON

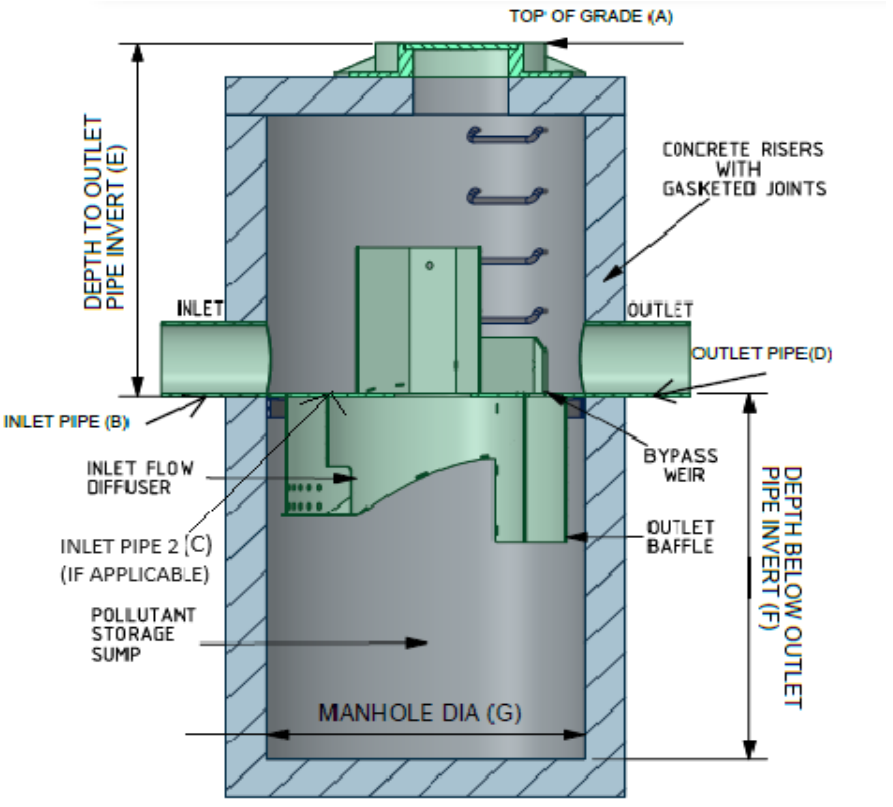
Project Name:	7529 County Road 9	**Engineer:**	**Tatham Engineering**
Location:	Creemore, ON	**Contact:**	James Lovett, B.Eng
OGS #:	OGS 50% Split Catchment	**Report Date:**	23/Apr/25
Area	3.72 ha	**OGS Designer**	**Rob Rainford**
C value	0.47	**Email**	Info@Echelonenvironmental.ca
		Phone	905-948-0000

Water Quality Design Parameters

Structure ID	OGS 50% Split Catchment	**DM-OS Unit Volumes**	
OGS Pretreatment TSSR	60%	**Sediment Capacity (L)**	6500
TSS Particle Size	ETV	**Oil Capacity (L)**	3600
Peak Storm Flow Rate (L/s)	Per Engineer of Record	**Total Holding Volume (L)**	12092
Peak Storm Event (Yr)	Per Engineer of Record	**Estimated Maintence Freq.**	Exceeds 2 Years

DM-OS Site Specific Characteristics*

A (masl)	**
B - Diameter (mm)	**
B - Material	**
B - Invert Elev. (masl)	**
C Diameter (mm)	**
C Material	**
C Invert Elev. (masl)	**
D Diameter	**
D Material	**
D Invert Elev. (masl)	**
E (mm)	Varies
F (mm)	2591
G (mm)	2400



*For special design considerations, contact Echelon Environmental for a custom design proposal.

**See formal drawing package for all design characteristics

ECHELON ENVIRONMENTAL

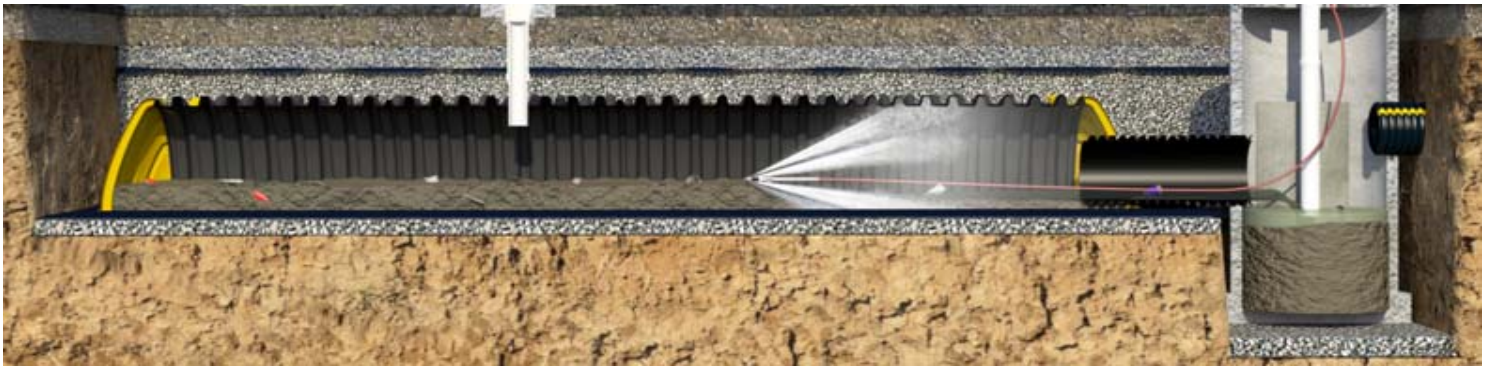
PRINSCO WATER MANAGEMENT SOLUTIONS

DM-OS Net Annual Load Removal Efficiency Results

Rainfall Intensity (mm/hr)	Percent Rainfall Volume	Rainfall Intensity (mm/hr)	Percent Rainfall Volume
1.0	20.0%	8.0	2.9%
1.5	10.0%	9.0	3.0%
2.0	8.5%	10.0	2.3%
2.5	7.3%	15.0	7.4%
3.0	6.8%	20.0	2.7%
3.5	4.8%	25.0	1.4%
4.0	4.9%	30.0	0.8%
4.5	3.9%	35.0	0.3%
5.0	2.9%	40.0	0.0%
6.0	5.3%	45.0	0.2%
7.0	3.7%	50.0	0.0%

HydroStor®

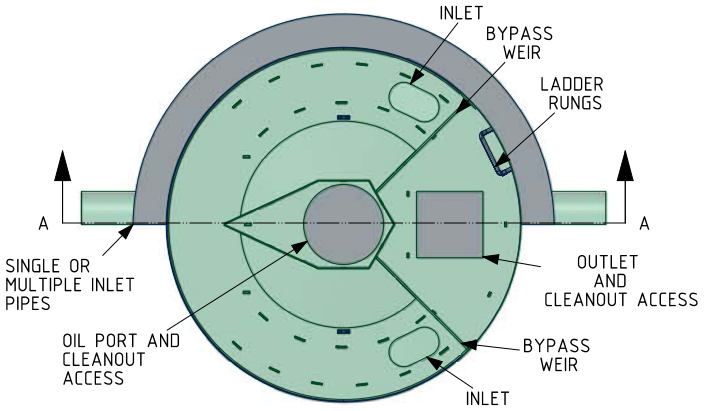
Design Storm Flow:	63.10	l/s (13mm)	Sediment Row Area:	64.6	m2
Design SLR:	0.0234	l/s/m2	Design Flow:	175.5	l/s

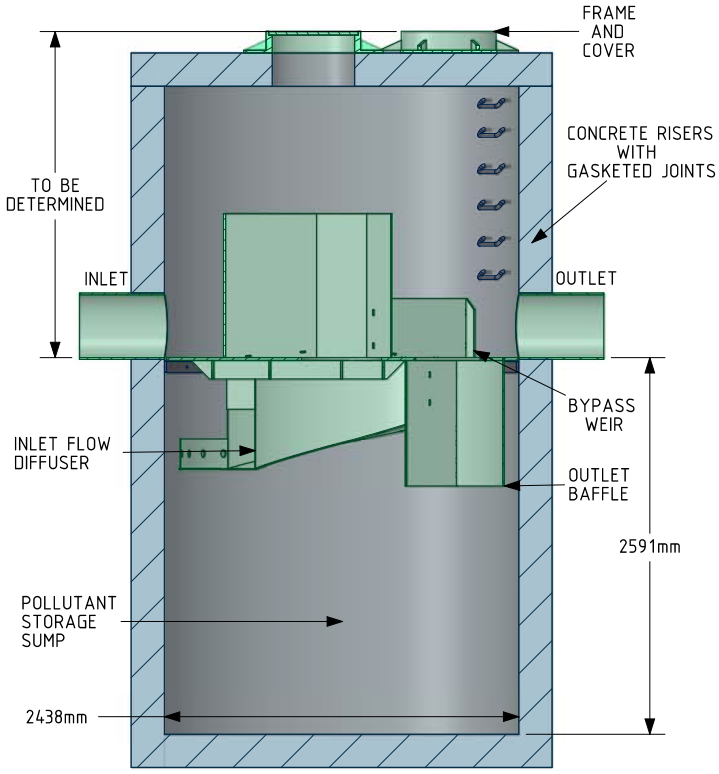


Combined Treatment Train Efficiency***	84%
Total Annual Rainfall Treated	>90%

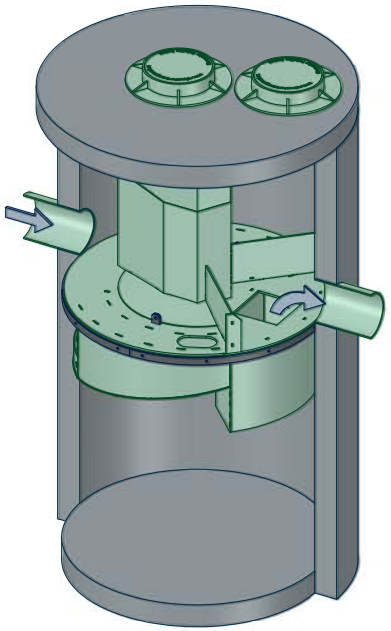
Design Notes:

1) Based on 27 years of hourly rainfall data from Canadian Station 6110557, Barrie ON
2) RWM DM-OS Efficiency based on ETV 2013 Test Protocol testing
3) HydroStor Sediment Row Efficiency based on NJCAT testing verification dated 2017
4) Combined Treatment Train Efficiency based on efficiency of highest removal system





SECTION A-A



SCALE 1:68

TYPICAL DRAWING NOT FOR CONSTRUCTION

DESIGN NOTES

1. INLET AND OUTLET PIPE CAN BE UP TO 90 DEGREES APART DEPENDING ON PIPE AND MANHOLE SIZE. IF IN DOUBT, PLEASE CONTACT RAINWATER MANAGEMENT.

2. THE RWM DM-OS UNIT CAN HANDLE MULTIPLE INLET PIPES AS WELL AS A TOP INLET.

GENERAL NOTES

1. RAINWATER MANAGEMENT TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.

2. ALL DIMENSIONS ARE SHOWN IN MILLIMETERS.

3. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHTS, CONTACT RAINWATER MANAGEMENT. www.rainwatermanagement.ca

4. RWM WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.

5. STRUCTURE AND CASTINGS SHALL MEET REQUIRED LOAD RATINGS, ASSUMING GROUNDWATER ELEVATION AT, OR BELOW THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION.

6. MANHOLE MANUFACTURED TO LOCAL SPECIFICATIONS.

INSTALLATION NOTES

1. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY THE ENGINEER OF RECORD.

2. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE MANHOLE STRUCTURE (LIFTING DEVICES NOT PROVIDED). HEAVIEST LIFT DEPENDS ON RISER HEIGHTS.

3. CONTRACTOR TO ADD GASKETS OR JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS, AND ASSEMBLE STRUCTURE AS REQUIRED.

4. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES, MATCH PIPE INVERTS WITH ELEVATIONS SHOWN.

5. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO ASSURE UNIT IS WATER TIGHT. IT IS SUGGESTED THAT ALL JOINTS BELOW PIPE INVERTS ARE GROUTED.

rainwater MANAGEMENT
www.rainwatermanagment.ca
604-944-9265

FILE		MODEL			
RWM-DM-2400-OS TYPICAL		RWM-DM-2400-OS			
JOB / IDENTIFIER		REV	CREATOR	APPROVED	SHEET
DM-OS			AG		1 of 1
MATERIAL	THICKNESS	SCALE	CREATED	LAST SAVED	
--		1:52	1/9/2023	1/9/2023 3:44:47 PM	

TATHAM ENGINEERING

Water Budget

Mitigation Measures

LID Design

Project Details

7582 County Road 9	424350

Prepared By

JRL	7/7/2025

LID System Design Details

LID Measure	Infiltraction Cell
LID Impervious Drainage Area (ha)	1.72
Number of LIDs	1
Void Ratio	0.40
Footprint of LID (m^2)	1154
Depth of LID (m)	0.33
Storage Volume Required (m^3)	46.2
Volume Required / LID (m^3)	46.2
Volume Provided / LID (m^3)	152.3
Volume Provided (m^3)	152.3
Design Precipitation Depth (mm)	8.9
Annual Volume Captured (mm)	589.0
Annual Volume Captured excluding Evapotransiration (m^3)	10,131
Annual Volume Captured after Evapotranspiration (m^3)	8,105

Additional Notes

TATHAM ENGINEERING

Water Budget
Summary

Project Details

7582 County Road 9	424350

Prepared By

JRL	7/7/2025

Summary

Existing Infiltration (m^3)	6,074
Proposed Infiltration (m^3) - No Mitigtation	2,220
Infiltration Deficit Prior to Mitigation (m^3)	3,854
Proposed Infiltration Measures	
[ ] Increase Topsoil Depth	
[x] Infiltration LID	
[ ] Impervious Area Routed Over Pervious Area	
Mitigation - Increase Topsoil Reduction in Pervious Runoff (m^3)	0
Mitigation Measure - Implementing LID (m^3)	8,105
Mitigation Measure - Impervious Area Routed over Pervious Area (m^3)	0
Proposed Infiltration (m^3)	10,325
Infiltration Deficit after Mitigation (m^3)	-4,250

Additional Notes

TATHAM ENGINEERING

Phosphorus Budget Assessment

Project Details

7582 County Road 9	424350

Prepared By

JRL	7/2/2025

Watershed

Mad River (NVCA)

Treatment Method

Treatment Train

LAND USE CATEGORY	Phosphorus Loading Rate (kg/ha/yr)	Pre-Development		Post-Development		Post-Development	
		Total Area		Treated Area		Untreated Area	
		Area (ha)	Loading (kg/yr)	Area (ha)	Loading (kg/year)	Area (ha)	Loading (kg/year)
Cropland	0.39	0.00	0.00	0.00	0.00	0.00	0.00
Hay-Pasture	0.08	1.71	0.14	0.00	0.00	0.00	0.00
Turf -Sod	0.11	0.00	0.00	0.00	0.00	0.00	0.00
High Intensity Development - C/I	0.41	0.00	0.00	0.00	0.00	0.00	0.00
High Intensity Development - R	0.41	0.00	0.00	2.33	0.96	0.07	0.03
Low Intensity Development	0.13	0.10	0.01	0.15	0.02	0.48	0.06
Quarry	0.15	0.00	0.00	0.00	0.00	0.00	0.00
Unpaved Road	0.83	0.00	0.00	0.00	0.00	0.00	0.00
Forest	0.06	1.21	0.07	0.00	0.00	0.00	0.00
Transition	0.16	0.00	0.00	0.00	0.00	0.00	0.00
Wetland	0.05	0.00	0.00	0.00	0.00	0.00	0.00
Open Water	0.26	0.00	0.00	0.00	0.00	0.00	0.00
Total		**3.02**	**0.22**	**2.48**	**0.97**	**0.54**	**0.09**

CONTROLS

Proposed Treatment Method	Removal Efficiency (%)	Area (ha)	Loading (kg/year)
OGS	20%	2.48	0.78
Underground Storage Tank Infiltration	60%	2.48	0.31
Effective Removal Efficiency	**68%**	**2.48**	0.31

SUMMARY

Existing Phosphorous Load	0.22 kg/year
Post Development Phosphorous Load (no controls)	1.06 kg/year
Post Development Phosphorous Load (with controls)	0.40 kg/year
Overall Increase in Phosphorus Load	0.18 kg/year

NJCAT TECHNOLOGY VERIFICATION

StormKeeper® Chamber Sediment Strip®

Lane Enterprises Inc.

May 2017

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1. Introduction

1.1 New Jersey Corporation for Advanced Technology (NJCAT) Program

NJCAT is a not-for-profit corporation to promote in New Jersey the retention and growth of technology-based businesses in emerging fields such as environmental and energy technologies. NJCAT provides innovators with the regulatory, commercial, technological and financial assistance required to bring their ideas to market successfully. Specifically, NJCAT functions to:

- Advance policy strategies and regulatory mechanisms to promote technology commercialization;
- Identify, evaluate, and recommend specific technologies for which the regulatory and commercialization process should be facilitated;
- Facilitate funding and developing commercial relationships/alliances to bring new technologies to market and new business to the state; and
- Assist in the identification of markets and applications for commercialized technologies.

The technology verification program specifically encourages collaboration between vendors and users of technology. Through this program, teams of academic and business professionals are formed to implement a comprehensive evaluation of vendor specific performance claims. Thus, suppliers have the competitive edge of an independent third party confirmation of claims.

Pursuant to N.J.S.A. 13:1D-134 et seq. (Energy and Environmental Technology Verification Program) the New Jersey Department of Environmental Protection (NJDEP) and NJCAT have established a Performance Partnership Agreement (PPA) whereby NJCAT performs the technology verification review and NJDEP certifies the net beneficial environmental effect of the technology. In addition, NJDEP/NJCAT work in conjunction to develop expedited or more efficient timeframes for review and decision-making of permits or approvals associated with the verified/certified technology.

The PPA also requires that:

- The NJDEP shall enter into reciprocal environmental technology agreements concerning the evaluation and verification protocols with the United States Environmental Protection Agency, other local required or national environmental agencies, entities or groups in other states and New Jersey for the purpose of encouraging and permitting the reciprocal acceptance of technology data and information concerning the evaluation and verification of energy and environmental technologies; and

- The NJDEP shall work closely with the State Treasurer to include in State bid specifications, as deemed appropriate by the State Treasurer, any technology verified under the Energy and Environment Technology Verification Program.

1.2 Description of Technology

The StormKeeper® Sediment Strip® is a row of StormKeeper® chambers utilizing a woven fabric wrapping to treat storm water and remove sediment. The sediment strip can act as a stand – alone device or as a pretreatment device in series with other water quality treatment devices and storm water retention and detention systems. The Sediment Strip is constructed from open bottom StormKeeper® chambers wrapped with a woven geotextile. The chambers are connected to each other with an overlapping joint allowing the sediment strip to be sized as needed by adding chambers to the row. The chambers are isolated with the placement of an endcap at each end of the row. The end caps are also wrapped with the woven geotextile fabric meeting AASHTO M288 Class 1.

Each sediment strip is designed with a diversion structure to allow the initial runoff at the beginning of a storm to enter the strip for filtration. The diversion structure is most commonly a manhole with a diversion weir but could be some other type of structure based on the requirements of the site. After the first flush volume or flow rate has been exceeded the remaining water will continue into the detention or retention system that is being utilized (**Figure 1)**.

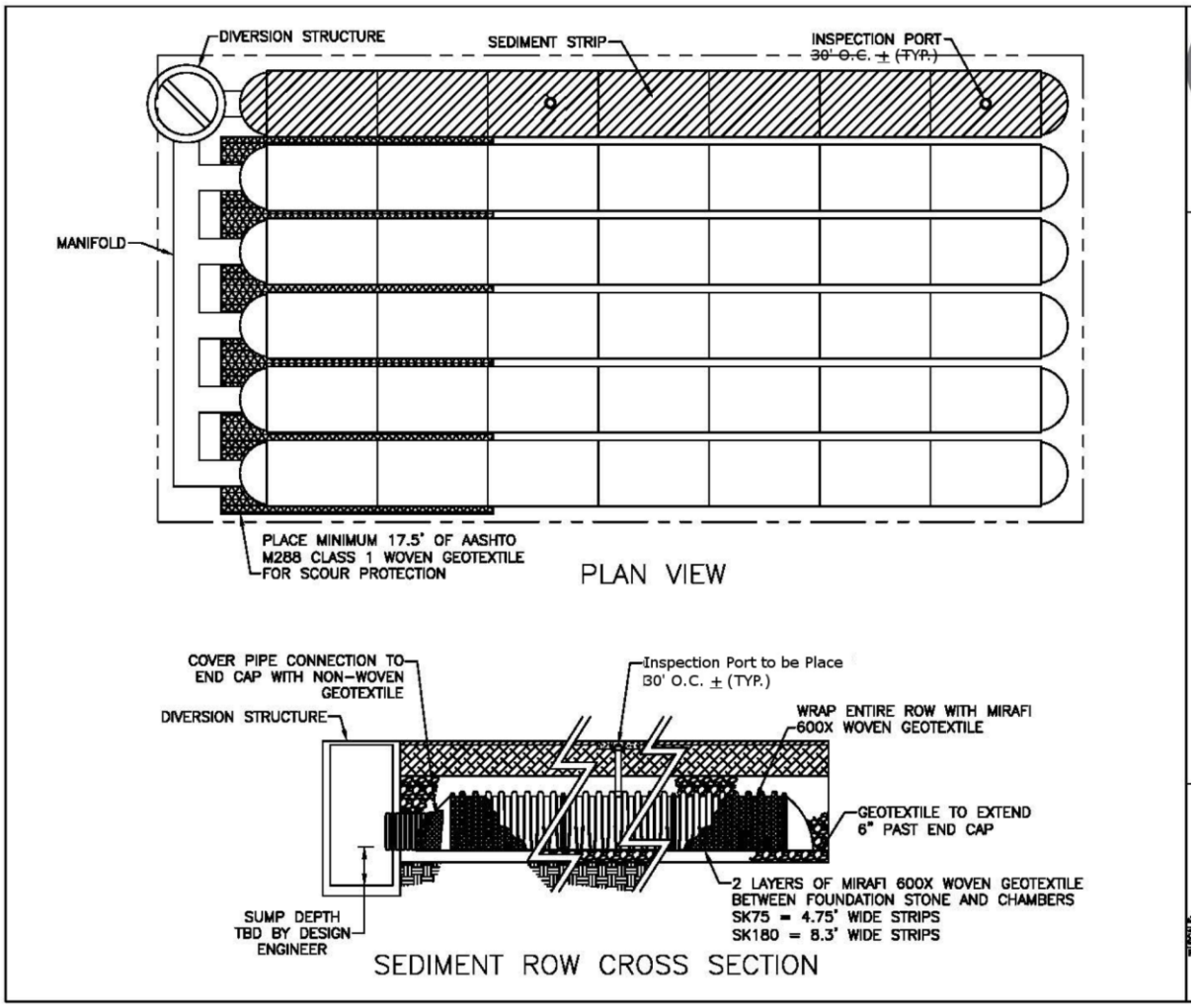


Figure 1 Conceptual Layout with Sediment Row

As the first flush water enters the chamber system it is forced to exit through the bottom of the chamber and through the woven geotextile surrounding the chamber. This results in a filtration of the storm water allowing the water to be treated and the sediment and pollutants to be left behind while the treated water is infiltrated into the ground if soil permeability studies allow. The diversion structure also provides access for jetting the system to flush the sediment and other stormwater pollutants to a manhole or other structure.

Prinsco, Inc. and **Lane Enterprises, Inc.** jointly developed the open bottom chamber. Lane uses the StormKeeper trade name and markets and sells the SK75 and SK180 StormKeeper Chamber Sediment Strip. Prinsco uses the HydroStor™ trade name and markets and sells the HS75 and HS180 Sediment Row. The Sediment Strip and Sediment Row chambers are identical products and produced during the same production runs.

2. Laboratory Testing

The purpose of the testing was to define the performance characteristics of the StormKeeper® Chamber Sediment Strip® under controlled laboratory conditions, utilizing established standard testing methodologies. The testing was conducted in accordance with *"New Jersey Department of Environmental Protection Laboratory Protocol to Assess Total Suspended Solids Removal by a Filtration Manufactured Treatment Device",* 2013, to establish the following parameters:

- Hydraulic Characteristic Curves
- Sediment Removal Efficiency at Maximum Treatment Flow Rate (MTFR)
- Filter Blinding (Occlusion)

Testing was conducted at Alden Research Laboratory, Inc. (Alden), Holden, Massachusetts. **It is not planned to submit this verification report to NJDEP for certification at this time.**

2.1 Test Unit

The SK75 StormKeeper® Chamber Sediment Strip® test unit was an arched stormwater detention/retention sediment collection and filtering device, measuring approximately 51" wide x 30" high x 7 ft long, constructed from polypropylene Open Bottom StormKeeper® chambers and Mirafi® 600X geotextile. The geotextile is composed of high-tenacity polypropylene yarns, which are woven into a stable network such that the yarns retain their relative position. 600X is inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids. Both ends of the chamber were sealed with the use of end caps. A water-tight tank was used to house the test chamber system. The chamber was installed on top of a 1-ft base of ¾"-2" double-washed stone containing a 6" underdrain pipe, which penetrated the downstream tank wall. (The purpose of the 12" stone was to allow for the underdrain to be put in for testing in the laboratory boxed in condition as opposed to buried in the ground. The stone does not contribute to the treatment of the sediment strip and is only necessary for foundation support. It is not part of the field installation.)

Two layers of Mirafi® 600X geotextile were placed between the stone base and chamber to collect particulate contaminants, as well as protect the stone base from scouring. The geotextile had an open-area of 1% and opening size of 425 microns. The top geotextile layer was used to fully wrap

the chamber and end caps. The chamber floor filtration area was approximately 30 ft^2. Drawings and specifications of the SK75 test unit are shown on **Figure 2** and **Figure 3.**

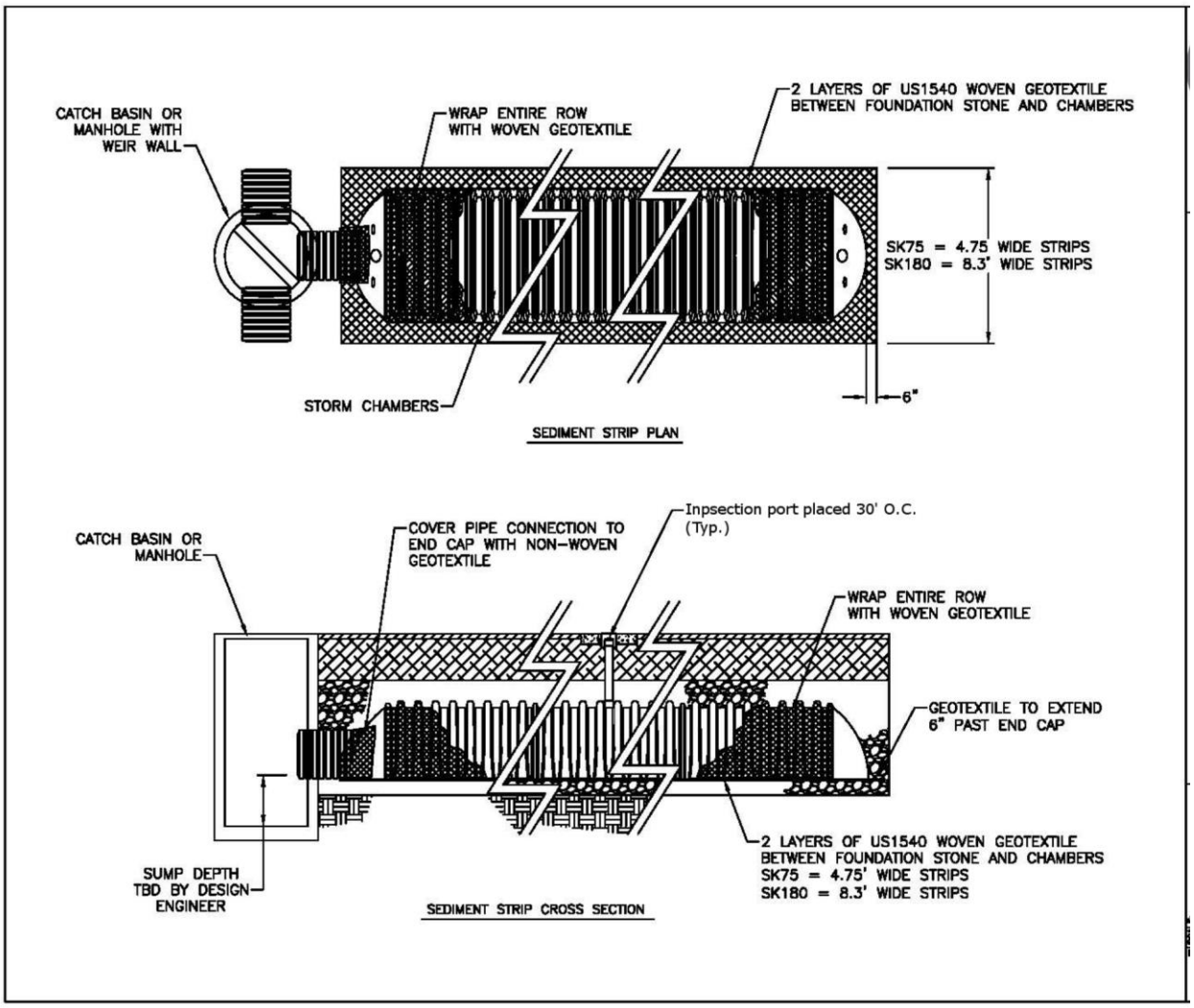


Figure 2 Stormkeeper Chambers Sediment Strip Detail

Water was conveyed into the chamber by means of a 12" diameter inlet pipe, which penetrated the upstream end cap. The junction was wrapped in non-woven fabric. The invert of the pipe was approximately flush with the chamber floor. Additional stone was installed around the outside of the chamber until fully buried. A 4" diameter x 2-ft tall PVC standpipe was installed into the crown of the chamber. In a typical field installation, water passing through the base fabric seeps into the stone base and is either re-infiltrated into the surrounding soil, enters the underdrain and is conveyed into an outlet control structure, or is distributed into other chambers in the stormwater management system. Although the primary function of the sediment strip chamber is to capture and retain sediment particles, the geotextile membrane possesses filtering characteristics and therefore, was tested as such. On-line scour testing was not conducted, as the system is designed

for an off-line application with the inclusion of an upstream bypass weir. The bypass weir was not included in the laboratory set-up.

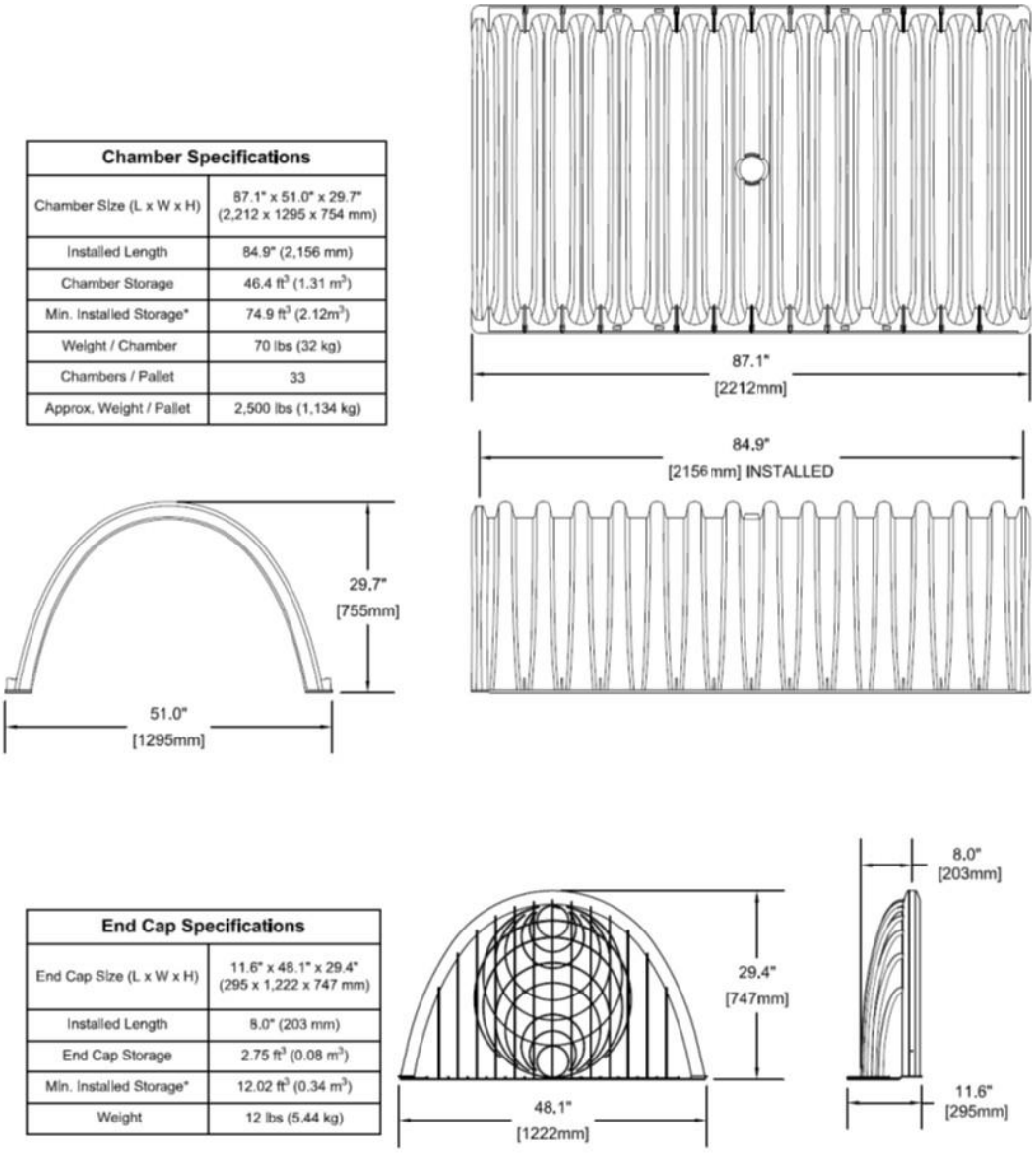


Figure 3 Drawing and Specifications of the SK 75 Treatment Unit

Photographs showing the wrapped SK75 installed in the test tank prior to backfilling and after final installation are shown on **Figure 4** and **Figure 5.**



Figure 4 SK75 Test Unit Installed in the Test Tank Prior to Backfilling



Figure 5 SK75 Test Unit Fully Installed in the Test Tank

A photograph of the test set up is shown on **Figure 6.**



Figure 6 Photograph of the Test Set Up

2.2 Test Setup

The SK75 test unit was installed in the Alden test loop, shown in **Figure 7,** which is set up as a recirculation system.

The loop is designed to provide metered flow up to approximately 17 cfs. Flow was supplied to the unit with one or two selected laboratory pumps (20HP, 50HP), drawing water from a 50,000-gallon supply sump. The test flow was set and measured using one of six differential-pressure meters and corresponding control valves (2", 4", 6", 8", 10", or 12"). A Differential Pressure (DP) cell and computer Data Acquisition (DA) program was used to record the test flow. Twenty-five (25) feet of straight 12-inch PVC influent pipe conveyed the metered flow to the test unit. Two (2) feet of 6" PVC pipe free-discharged the effluent flows to a receiving tank, which contained a calibrated V-notch weir at the downstream end for measuring the drawdown flow. The influent and effluent pipes were set at 1% slopes. A 12-inch tee was located 4 pipe-diameters (4 ft) upstream of the test unit for injecting sediment into the crown of the influent pipe using a variable-speed auger feeder.

Filtration of the supply sump, to further reduce background concentration, was performed with an in-situ filter wall containing 1-micron filter bags.

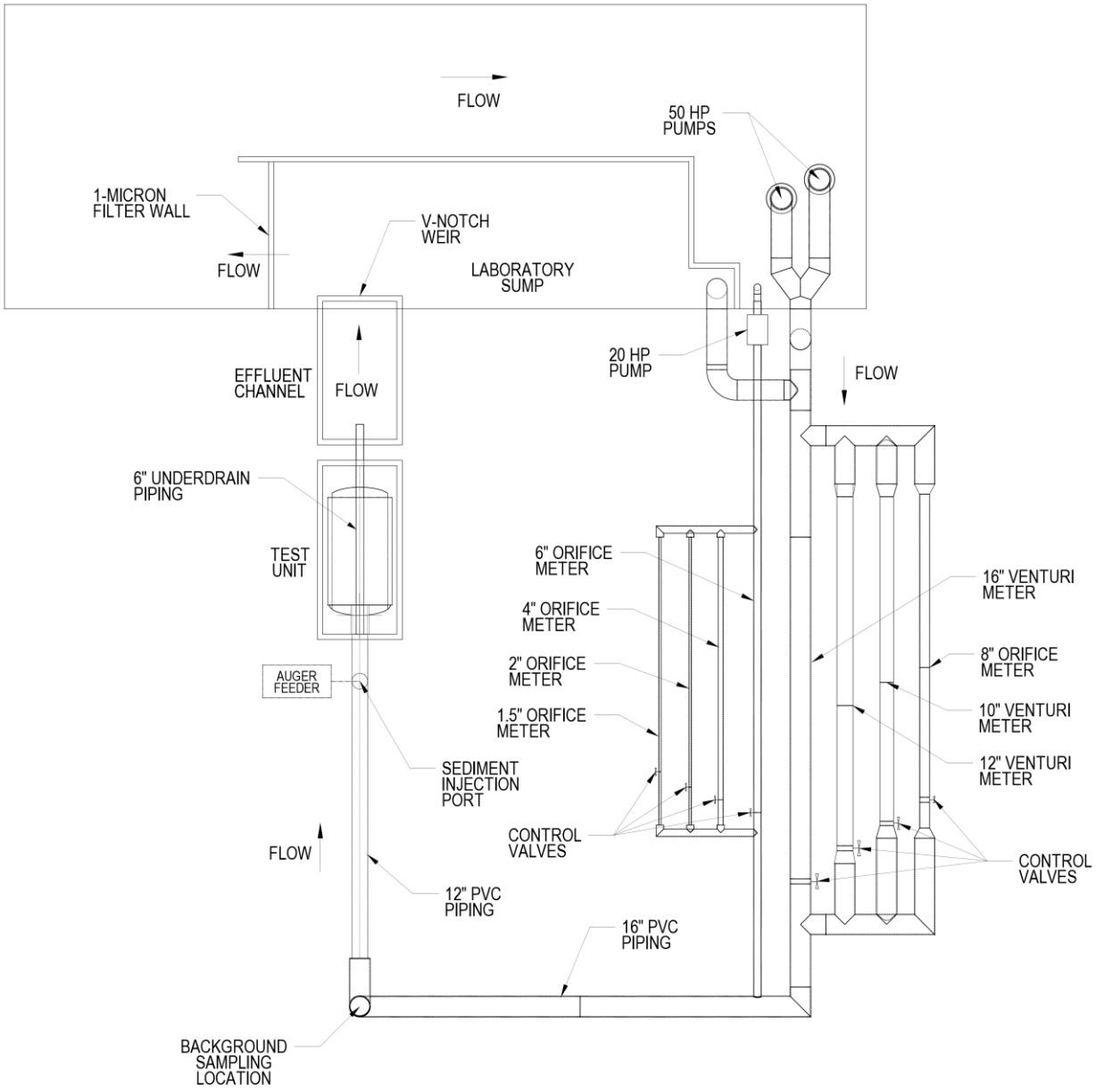


Figure 7 Plan View of Alden Flow Loop

2.3 Hydraulic Testing

The SK75 unit was tested with clean water to determine its hydraulic characteristic curves, including loss coefficients (Cd's) and/or K factors. Flow and water level measurements were recorded during steady-state flow conditions using a computerized Data-Acquisition (DA) system, which included a data collect program, 0-250" Rosemount Differential Pressure (DP) cell (flow), and Omegadyne PX419 0-2.5 psi Single-ended Pressure (SP) cell (water elevations). Flows were set and measured using the calibrated flow meters and control valves. Each test flow was set and operated at steady state for approximately 10 minutes, after which time a minimum of 30 seconds of flow and pressure data were averaged and recorded for each pressure tap location. Water

elevations were measured above and below the fabric layer outside of the chamber. Measurements within the influent and effluent pipes were taken one pipe-diameter upstream and downstream of the unit.

2.4 Removal Efficiency Testing

Sediment testing was conducted to determine the removal efficiency, as well as sediment mass loading capacity. The sediment testing was conducted on an initially clean system at the 100% MTFR of 120 gpm (as selected by Lane). A minimum of ten 30-minute test runs were required to be conducted. The captured sediment was not removed from the chamber between tests.

The total mass injected into the system was quantified at the conclusion of the 10 runs. This data was used for determination of the required maintenance frequency.

The test sediment was prepared by Alden to meet the PSD gradation of 1-1000 microns in accordance with the distribution shown in column 2 of **Table 1**. The sediment is silica based, with a specific gravity of 2.65. Three random PSD samples of the test sediment were analyzed by an independent certified analytical laboratory using ASTM D 422-63 (Reapproved 2007) "Standard Test Method for Particle Size Analysis of Soils". The average of the three samples was used for compliance with the protocol.

Table 1 Test Sediment Particle Size Distribution

Particle Size[1] (Microns)	Target Minimum % Less Than[2]
1,000	100
500	95
250	90
150	75
100	60
75	50
50	45
20	35
8	20
5	10
2	5

1. The material shall be hard, firm, and inorganic with a specific gravity of 2.65. The various particle sizes shall be uniformly distributed throughout the material prior to use.
2. A measured value may be lower than a target minimum % less than value by up to two percentage points, provided the measured d_{50} value does not exceed 75 microns.

The target influent sediment concentration was 200 mg/L (+/-20 mg/L) for all tests. The concentration was verified by collecting a minimum of six timed dry samples at the injector and correlating the data with the measured average flow to produce the resulting influent concentration values for each test. The allowed Coefficient of Variance (COV) for the measured samples is 0.10.

The moisture content of the test sediment was determined using ASTM D4959-07 for each test conducted and was utilized in the final removal calculation.

The protocol requires the temperature of the supply water to be below 80 degrees F.

Five (5) time-stamped effluent samples were collected from the end of the outlet pipe during each run. A minimum of three detention times were allowed to pass before collecting a sample after the start of sediment feed and when the feed was interrupted for measurements. Three (3) background samples of the supply water were collected during each run. The samples were collected with each odd-numbered effluent sample (1, 3 and 5). Collected samples were analyzed for Suspended Solids Concentration (SSC) using the ASTM D3977-97 (2013).

At the conclusion of a run, the injection feed was stopped and time-stamped. The flow was stopped after one (1) detention time had passed. The drawdown flow was measured at the V-notch weir every five (5) seconds until the effluent was reduced to 1% of the test flow. Two (2) evenly-spaced effluent samples were collected from the pipe during drawdown.

2.5 Instrumentation and Measuring Techniques

Flow

The inflow to the test unit was measured using one of six (6) calibrated differential-pressure flow meters (2", 4", 6", 8", 10", or 12"). Each meter is fabricated per ASME guidelines and calibrated in Alden's Calibration Department prior to the start of testing. The high and low pressure lines from each meter were connected to manifolds containing isolation valves. Flows were set with a butterfly valve, and the differential head from the selected meter was measured using a Rosemount® 0 to 250-inch Differential Pressure (DP) cell, also calibrated at Alden prior to testing. All pressure lines and cells were purged of air (bled) prior to the start of each test. The test flow was averaged and recorded every 5 seconds throughout the duration of the test using an in-house computerized data acquisition (DA) program. The accuracy of the flow measurement is ±2%. A photograph of the flow meters is shown on **Figure 8** and the pumps on **Figure 9.**

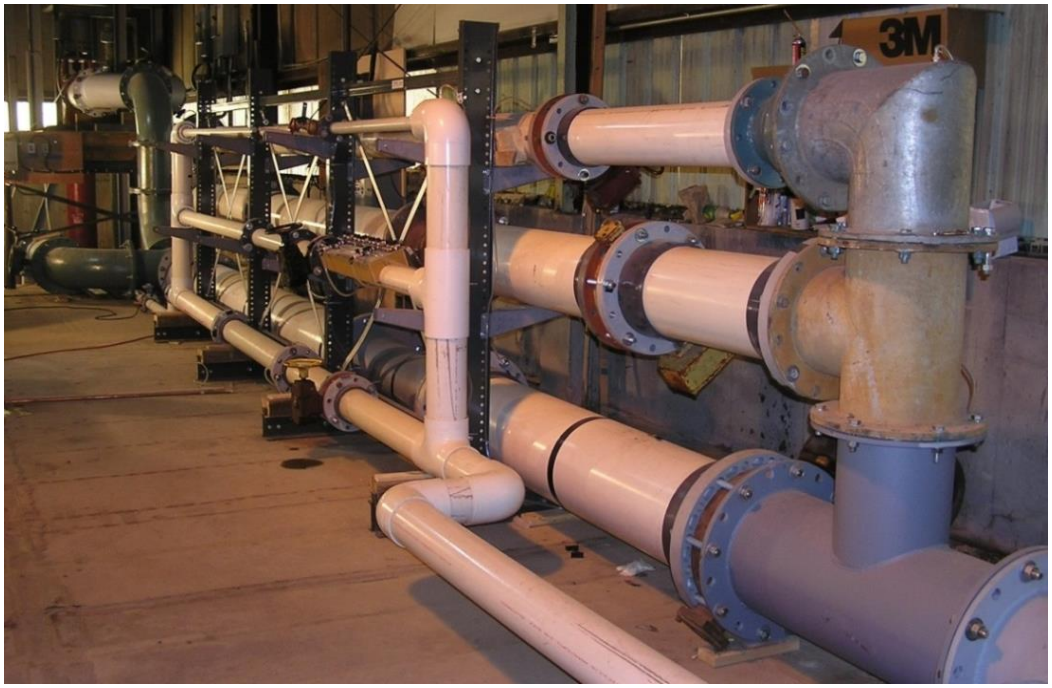


Figure 8 Photograph Showing Laboratory Flow Meters



Figure 9 Photograph Showing Laboratory Pumps

Drawdown Flow

The drawdown flow was measured with the use of a V-notch weir installed at the end of the effluent tank. The weir was fabricated in accordance with the Bureau of Reclamation Water Measurement Manual guidelines. The calculated (theoretical) curve was obtained using equation 7-3 found in Chapter 7, Section 7, of the "Water Measurement Manual", U.S. Department of the Interior Bureau

of Reclamation, Third Edition, 2001. The measured curve is the V-notch weir calibration performed at Alden. The points were obtained by running steady-state flows through a selected calibrated flow meters and measuring the water elevation above the invert of the V-notch using a calibrated absolute pressure cell and computerized data acquisition system.

The calculated and measured weir curves are shown on **Figure 10.** The measured (calibrated) curve equation was used to calculate the instantaneous flow passing over the weir, based on the corresponding recorded pressure readings, which were taken every 1-second during drawdown.

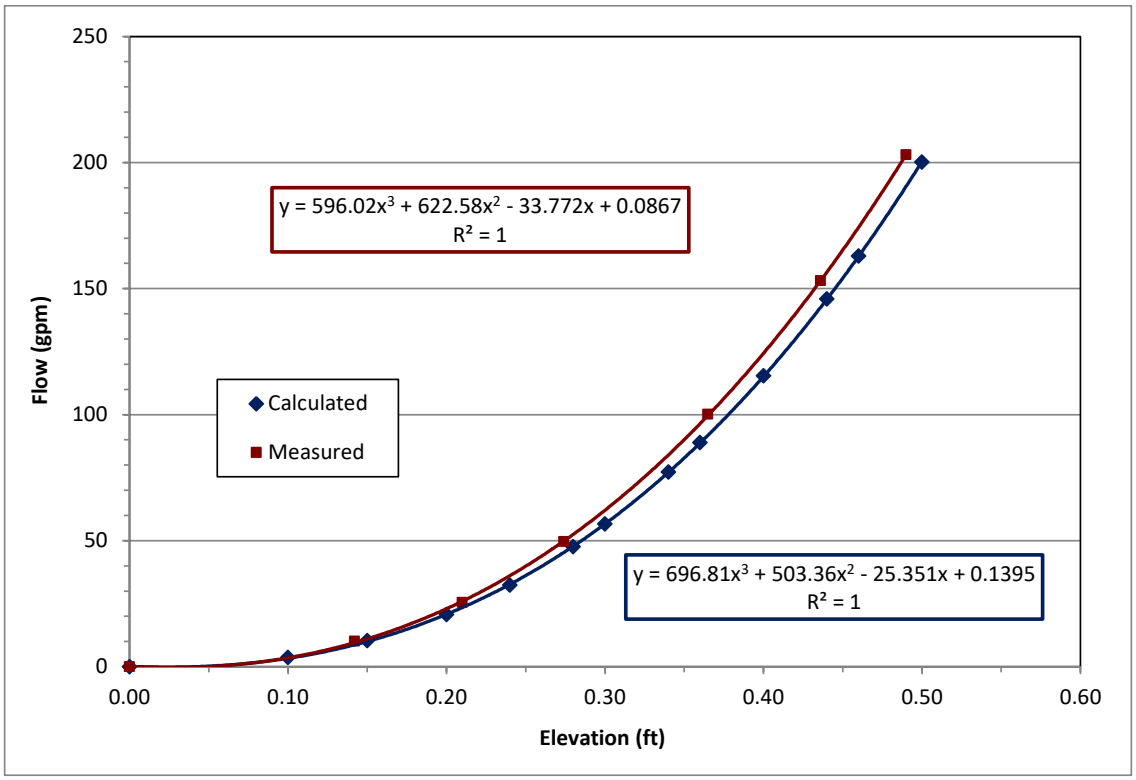


Figure 10 Drawdown V-notch Weir Flow vs Head Curves

Temperature

Water temperature measurements within the supply sump were obtained using a calibrated Omega® DP25 temperature probe and readout device. The calibration was performed at the laboratory prior to testing. The temperature reading was documented at the start and end of each test, to assure an acceptable testing temperature of less than 80 degrees F.

Pressure Head

Pressure head measurements were recorded at multiple locations using piezometer taps and a Omegadyne PX419, 0 - 2.5 psi cell. The pressure cell was calibrated at Alden prior to testing. Accuracy of the readings is ± 0.001 ft. The cell was installed at a known datum in relation to the tank floor, allowing for elevation readings through the full range of flows. A minimum of 30 seconds of pressure data was averaged and recorded for each pressure tap during hydraulic testing, under steady-state flow conditions, using the computerized DA program. Driving head and

effluent weir measurements were averaged and recorded every 5 seconds during removal efficiency testing. A photograph of the pressure instrumentation is shown on **Figure 11**.

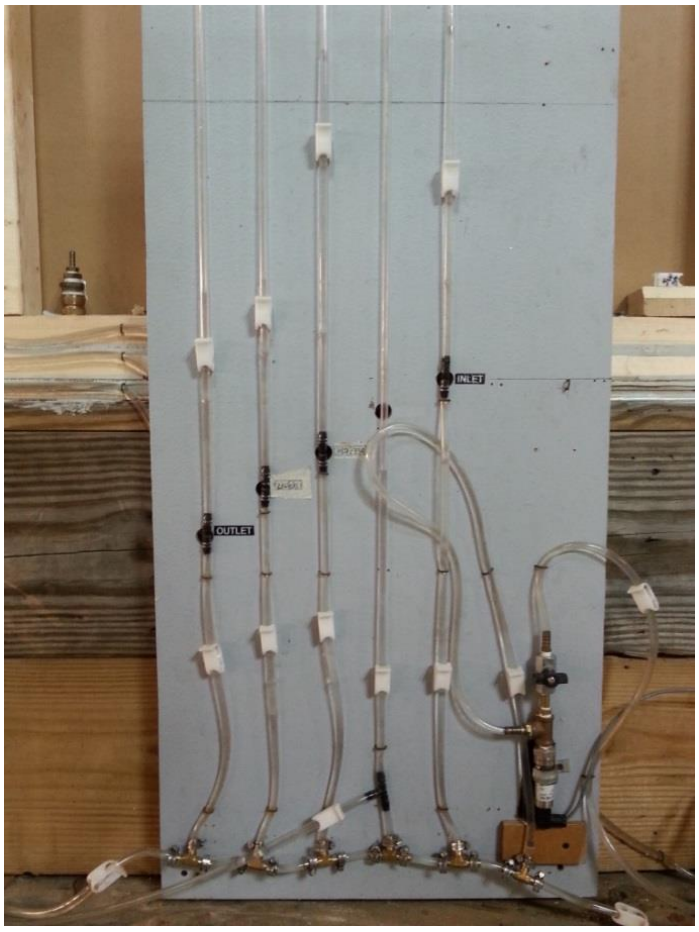


Figure 11 Pressure Measurement Instrumentation

Sediment Injection

The test sediment was injected into the crown of the influent pipe using an Auger® volumetric screw feeder, model VF-1, shown on **Figure 12**. The auger feed screw, driven with a variable-speed drive, was calibrated with the test sediment prior to testing, to establish a relationship between the auger speed (0-100%) and feed rate in grams/minute. The calibration, as well as test verification of the sediment feed was accomplished by collecting timed dry samples of 0.1-liter, up to a maximum of 1-minute, and weighing them on an Ohaus® 4000g x 0.1g, model SCD-010 digital scale. The feeder has a hopper at the upper end of the auger to provide a constant supply of dry test sand. The allowable Coefficient of Variance (COV) for the injection is 0.10.

Sample Collection

Effluent samples were collected in 2-liter containers from the end of the 6-inch effluent pipe. Background concentration samples were collected from the center of the vertical pipe upstream of the test unit with the use of a calibrated isokinetic sampler, shown on **Figure 13**.



Figure 12 Photograph of Variable-Speed Auger Feeder

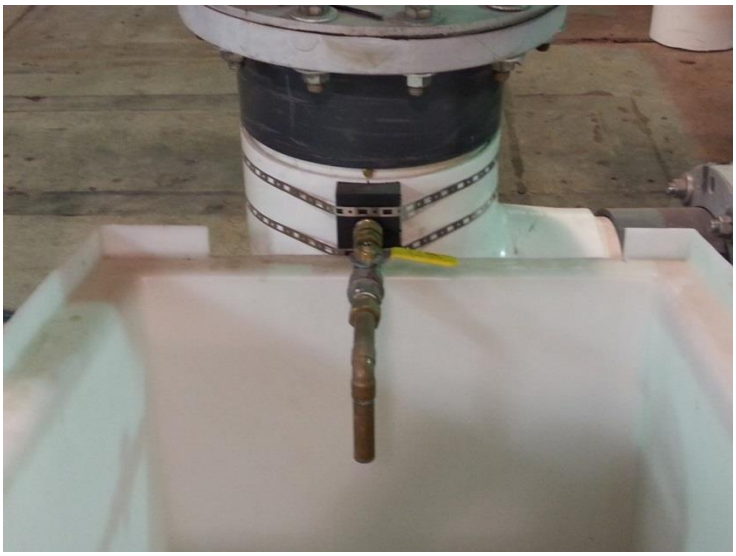


Figure 13 Photograph of Background Isokinetic Sampler

Sample Concentration Analysis

Effluent and background concentration samples were analyzed by Alden in accordance with Method B, as described in ASTM Designation: D 3977-97 (Reapproved 2013), "Standard Test Methods for Determining Sediment Concentration in Water Samples". The required silica sand used in the sediment testing did not result in any dissolved solids in the samples and therefore,

simplified the ASTM testing methods for determining sediment concentration. Associated instrumentation included:

- 2-Liter collection beakers
- Ohaus® 4000g x 0.1g digital scale, model SCD-010
- Oakton® StableTemp gravity convection oven, model 05015-59
- Sanplatec Dry Keeper® desiccator, model H42056-0001
- AND® 0.0001-gram analytical balance, model ER-182A
- Advantec 3-way filtration manifold
- Whatman® 934-AH, 47-mm, 1.5-micron, glass microfiber filter paper

Samples were collected in graduated 2-Liter beakers which were cleaned, dried and weighed to the nearest 0.1-gram, using an Ohaus® 4000g x 0.1g digital scale, model SCD-010, prior to sampling. Collected samples were also weighed to the nearest 0.1-gram using the Ohaus® digital scale. Each collected sample was filtered through a pre-rinsed Whatman® 934-AH, 47-mm, 1.5-micron, glass microfiber filter paper, using a laboratory vacuum-filtering system. Prior to processing, each filter was rinsed with distilled water and placed in a designated dish and dried in an Oakton® StableTemp gravity convection oven, model 05015-59, at 225 degrees F for a minimum of 2.5 hours. Each dried filter was placed in a Sanplatec Dry Keeper® desiccator, model H42056-0001, to cool and then weighed to the nearest 0.0001-gram to determine the tare weight, using an AND® analytical balance, model ER-182A. Once filtered, each sample and dish was dried at a temperature between 175 and 210 degrees F (below boiling) for 20 to 30 minutes until visually dry. The oven temperature was increased to 225 degrees F and the samples were dried for an additional 2.5 hours. The dry samples and dishes were then cooled in the desiccator and weighed to the nearest 0.0001-gram, using the AND® balance. Net sediment weight (mg) was determined by subtracting the dried filter weight (tare) from the dried sample weight and multiplying the result by 1,000. The net sample volume, in liters, was determined by subtracting the beaker and net sediment weight from the overall sample weight and dividing by 1,000. Each sample sediment concentration, in mg/L, was determined by dividing the net sediment weight by the net sample volume.

2.6 Data Management and Acquisition

A designated Laboratory Records Book was used to document the conditions and pertinent data entries for each test conducted. All entries are initialed and dated.

A personal computer running an Alden in-house Labview® Data Acquisition program was used to record all data related to instrument calibration and testing. A 16-bit National Instruments® NI6212 Analog to Digital board was used to convert the signal from the pressure cells. Alden's in-house data collection software, by default, collects one second averages of data collected at a raw rate of 250 Hz. The system allows very long contiguous data collection by continuously writing the collected 1-second averages and their RMS values to disk. The data output from the program is in tab delimited text format with a user-defined number of significant figures.

Test flow and pressure data was continuously collected at a frequency of 250 Hz. The flow data was averaged and recorded to file every 5 seconds. Steady-state pressure data was averaged and

recorded over a duration of 30 seconds for each point. The recorded data files were imported into a spreadsheet for further analysis and plotting.

Excel based data sheets were used to record all sediment related data used for quantifying injection rate, effluent and background sample concentrations. The data was input to the designated spreadsheet for final processing.

2.7 Laboratory Analysis

The following Test Methods were used to analyze the dry and aqueous sediment samples:

- Sediment Concentration
  ASTM Designation: D 3977-97 (Reapproved 2013), "Standard Test Methods for Determining Sediment Concentration in Water Samples"

- Sediment Moisture Content
  ASTM Designation: D4959-07, "Standard Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating"

- Dry Sediment Particle Size Distribution
  ASTM D422-63 (2007), "Standard Test Method for Particle Size Analysis of Soils"

2.8 Quality Assurance and Control

All instruments were calibrated prior to testing and periodically checked throughout the test program. Instrumentation calibrations were provided.

Flow

The flow meters and pressure cells were calibrated in Alden's Calibration Laboratory, which is ISO/IEC 17025 accredited. All pressure lines were purged of air prior to initiating each test. A standard water manometer board and Engineers Rule were used to measure the differential pressure from the meter and verify the computer measurement of each flow meter.

Sediment Injection

The sediment feed (g/min) was verified with the use of a digital stop watch and 4000g calibrated digital scale. The tare weight of the sample container was recorded prior to collection of each sample. The samples were a minimum of 0.1 liters in size, with a maximum collection time of 1-minute.

Sediment Concentration Analysis

All sediment concentration samples were processed in accordance with the ASTM D3977-97 (2013) analytical method. Gross sample weights were measured using a 4000g x 0.1g calibrated digital scale. The dried sample weights were measured with a calibrated 0.0001g analytical balance. Any change in filter weight due to processing was accounted for by including three control filters with each test set. The average of the three values, which was +/- 0.1-0.5 mg, was used in the final concentration calculations.

Analytical accuracy was verified by preparing two blind control samples and processing using the ASTM method. The final calculated values were within 0.26% and 0.87% of the theoretical sample concentrations, with an average of 0.57% accuracy. This value was not corrected for particles smaller than the filter designation of 1.5 microns and therefore is considered conservative.

3. Performance Test Results

The test sediment PSD analysis, removal efficiency test results and mass loading capacity results are summarized in this section.

3.1 Test Sediment PSD Analysis

The sediment particle size distribution (PSD) used for removal efficiency testing and sediment mass loading capacity was comprised of 1-1000 micron silica particles, as shown in **Table 1**. The Specific Gravity (SG) of the sediment mixes was 2.65. A commercially-available blend of each mix was provided by AGSCO Corp., a QAS International ISO-9001 certified company, and adjusted by Alden as required. Samples were collected from random bags and analyzed in accordance with ASTM D422-63 (2007), by GeoTesting Express, an ISO/IEC 17025 accredited independent laboratory. The average %-finer values of the stock material were found to be outside of the NJDEP acceptance criteria of 2% for particle sizes ≤ 20 microns. The Alden test mix was adjusted to within the NJDEP acceptance criteria with the addition of commercially-available US-Silica Min-U-Sil 10, with a PSD of approximately 1-25 microns. Test batches of approximately 30 lbs each, were prepared in individual 5-gallon buckets, which were arbitrarily selected for the removal testing. A well-mixed random sample was collected from three random test batches and analyzed for PSD by GeoTesting Express. The average of the samples was used for compliance to the protocol specifications listed in Column 2 of **Table 1**. The D_{50} of the samples ranged from 61 to 63 microns, with an average of 62 microns. The PSD data of the samples are shown in **Table 2** and the corresponding curves are shown on **Figure 12**.

Table 2 PSD Analyses of Alden NJDEP 1-1000 µm Mix

Particle size (µm)	NJDEP	Sample 1	Sample 2	Sample 3	Average
1000	100%	100%	100%	100%	100%
500	95%	96%	95%	96%	96%
250	90%	92%	91%	92%	92%
150	75%	79%	79%	79%	79%
110	60%	65%	65%	65%	65%
75	50%	53%	53%	53%	53%
53	45%	48%	47%	47%	47%
20	35%	34%	32%	36%	34%
8	20%	20%	19%	19%	19%
5	10%	14%	13%	13%	13%
2	5%	4%	4%	4%	4%
75	D_{50}	61	63	63	62

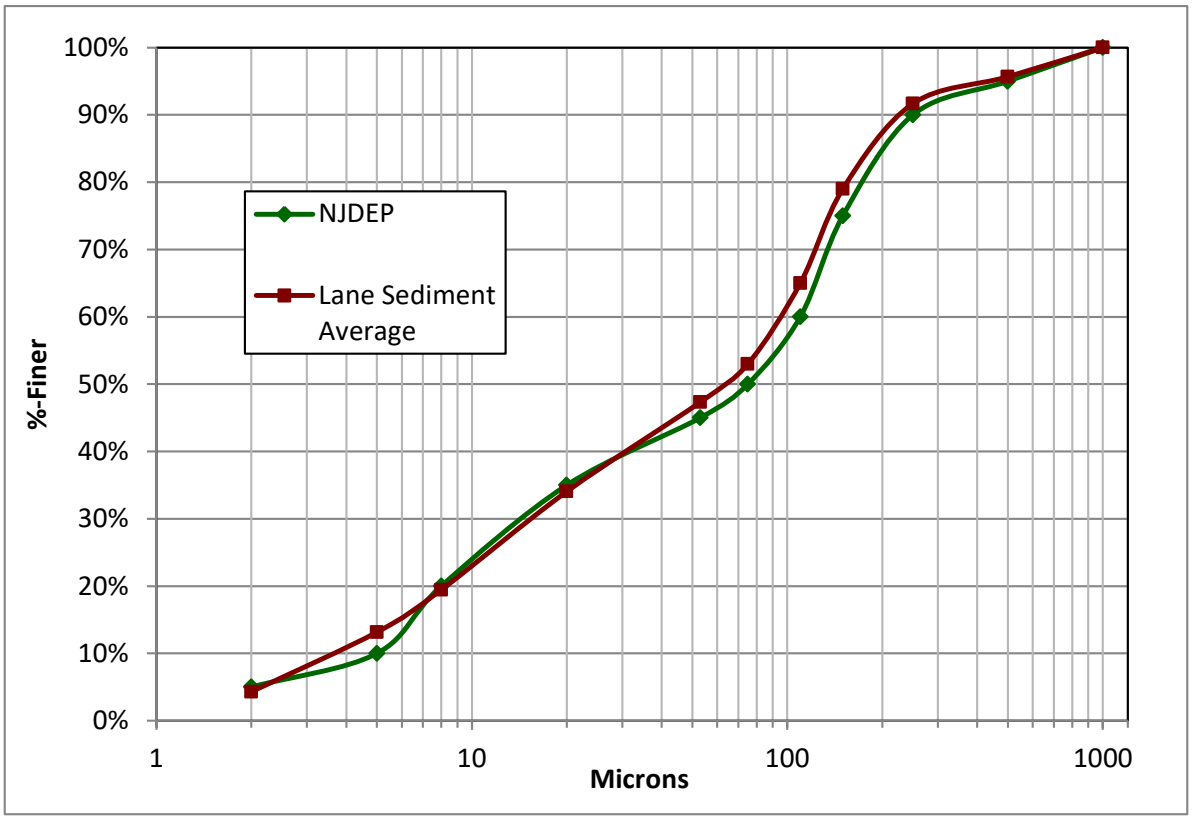


Figure 14 PSD Curves of Alden 1-1000 µm Test Sediment vs NJDEP Specifications

3.2 Removal Efficiency Testing

Ten (10) removal efficiency test runs were conducted at a target flow of 120 gpm (100% MTFR), corresponding to a normalized flow of 4 gpm/ft^2 of geotextile fabric. The minimum duration of the runs was 38 minutes, with a target influent sediment concentration of 200 mg/l. All test runs met or exceeded the protocol testing criteria. An additional run (Run11) was conducted at a flow of 108 gpm (90% MTFR) and influent concentration of 200 mg/L, to meet the mass loading capacity testing protocol. The duration of the run was 69 minutes, during which nine (9) effluent samples (11 including drawdown) and five (5) background samples were collected.

The measured flow for the 10 runs ranged from 119.5 gpm to 120.8 gpm, with an average flow of 119.9 gpm. The calculated COVs ranged from 0.001 to 0.004. The average measured flow for the mass loading run was 107.7 gpm, with a COV of 0.001. The maximum recorded temperature for all the runs ranged from 67.2 to 78.4 degrees F. The calculated mass/volume influent concentrations ranged from 189 to 207 mg/L, with an average concentration of 201 mg/L. The measured injected influent concentrations ranged from 198 to 203 mg/L, with an average concentration of 200 mg/L. The injection COVs ranged from 0.001 to 0.005. (**Table 3**)

The average adjusted effluent concentrations ranged from 29.2 to 41.5 mg/L, the maximum background concentration never exceeded 6.6 mg/L and the average drawdown concentrations ranged from 5.0 to 25.7 mg/L. The drawdown duration for the eleven (11) runs increased sequentially from 29 minutes to approximately 80 minutes. (**Table 4)**

Table 3 Measured Influent Parameters

Run #	Measured Flow		Max Temp	Influent Concentration (mg/L)		
	gpm	COV	Deg. F	mass/volume	Injector Concentration	Injector COV
1	120.8	0.004	78.4	198	198	0.002
2	119.6	0.001	78.1	207	201	0.002
3	119.7	0.002	74.8	189	200	0.003
4	119.7	0.001	74.6	203	200	0.001
5	119.5	0.001	74.8	204	201	0.001
6	120.2	0.001	70.5	202	200	0.001
7	119.7	0.001	70.7	204	200	0.002
8	120.1	0.001	69.7	196	199	0.002
9	119.8	0.001	68.6	204	200	0.002
10	119.9	0.001	67.2	203	200	0.001
11 (90% MTFR)	107.7	0.001	71.7	199	203	0.005
Average #1-10	**119.9**		**Average**	**201**	**200**	

Table 4 Measured Sample Concentrations

Run #	Maximum Background	Adjusted Effluent Concentrations (mg/L)						Drawdown Concentrations (mg/L)		
	mg/L	#1	#2	#3	#4	#5	Average	#1	#2	Average
1	1.7	21.39	19.71	19.94	43.56	41.30	**29.18**	12.33	39.14	**25.73**
2	3.0	19.75	23.25	25.22	63.47	44.35	**35.21**	9.19	5.51	**7.35**
3	4.3	38.33	35.94	39.79	40.91	39.69	**38.93**	9.39	4.76	**7.07**
4	0.9	37.31	40.21	41.01	41.24	41.60	**40.28**	7.32	3.43	**5.37**
5	1.0	36.04	37.24	41.02	40.58	41.34	**39.24**	7.86	3.11	**5.48**
6	0.2	37.81	37.48	41.27	40.86	40.02	**39.49**	7.60	3.28	**5.44**
7	0.6	37.44	39.24	40.07	42.90	43.45	**40.62**	7.69	3.77	**5.73**
8	1.0	39.21	39.05	41.13	42.64	40.48	**40.50**	6.50	3.39	**4.95**
9	0.9	38.24	41.77	42.78	41.84	42.91	**41.51**	9.69	4.58	**7.14**
10	6.6	32.24	35.76	46.27	39.88	36.87	**38.20**	12.14	5.56	**8.85**
Mass Loading Test		#1 / #2	#3 / #4	#5 / #6	#7 / #8	#9				
11	1.1	36.06 / 38.25	41.89 / 39.93	38.14 / 37.42	40.14 / 36.55	39.92	**38.70**	9.82	6.09	**7.95**

The removal efficiency on a mass removal basis (mass captured/mass loading) for the 10 removal test runs varied from 83.2% to 85.5%, with an average removal efficiency of 83.9% (**Table 5).**

Table 5 Removal Efficiency Test Results

Run #	Average Influent TSS	Average Adjusted Effluent TSS	Average Adjusted Drawdown TSS	Influent Volume	Effluent Volume	Drawdown Volume	Mass Loading	Mass Captured	Removal Efficiency by Mass
	(mg/L)	(mg/L)	(mg/L)	(L)	(L)	(L)	(g)	(g)	(%)
1	198.3	29.2	25.7	18,962	16,448	2,514	3760.1	3215.5	85.5
2	200.9	35.2	7.3	15,384	12,636	2,748	3090.1	2625.1	85.0
3	199.8	38.9	7.1	15,392	12,950	2,442	3075.1	2570.9	83.6
4	200.0	40.3	5.4	15,390	12,614	2,776	3078.3	2570.3	83.5
5	200.6	39.2	5.5	15,364	12,631	2,734	3081.5	2585.8	83.9
6	199.5	39.5	5.4	15,454	12,590	2,864	3083.4	2586.2	83.9
7	199.8	40.6	5.7	15,397	12,667	2,730	3076.6	2562.1	83.3
8	199.2	40.5	4.9	15,447	12,752	2,695	3077.0	2560.5	83.2
9	199.8	41.5	7.1	15,406	12,728	2,678	3078.7	2550.3	82.8
10	199.8	38.2	8.8	15,419	12,740	2,678	3080.7	2594.0	84.2
						Total Mass (kg)	**31.48**	**26.42**	
							Average Removal Efficiency by Mass		**83.9**

The maximum driving head, 3.26 ft, was recorded at the end of Run 10, which is 0.76 ft above the crown of the chamber. This value was set as the driving head criteria for sediment mass loading capacity testing. Lane SK75 recorded driving head elevations during the 10 sediment removal efficiency tests are shown in **Table 6** and on **Figure 15**.

Table 6 SK75 Recorded Driving Head Elevations

Run #	Driving Head (ft)
1	1.820
2	2.276
3	2.323
4	2.512
5	2.611
6	3.008

7	3.187
8	3.210
9	3.209
10	3.257
11	3.261

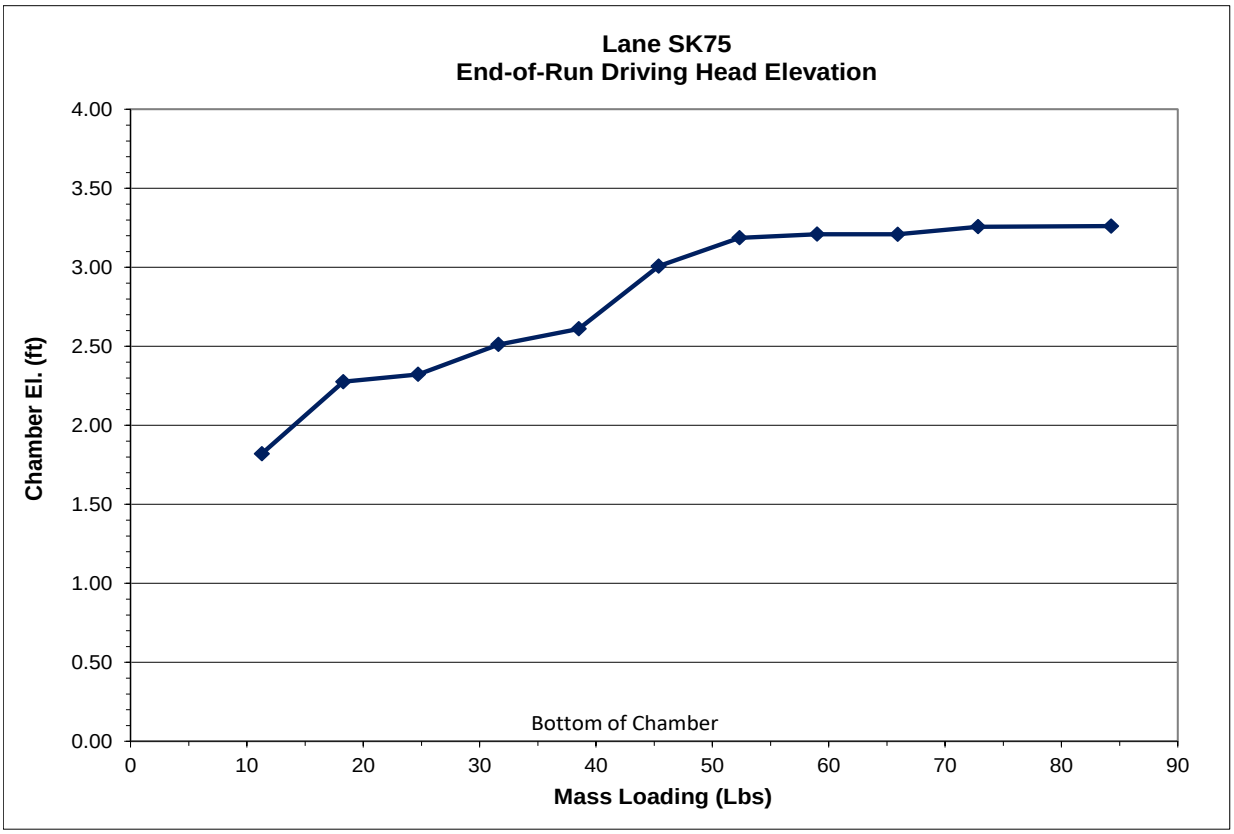


Figure 15 SK75 Recorded Driving Head Elevations

3.3 Sediment Mass Loading Capacity Testing

As mention previously, an additional run (Run 11) was conducted at 90% of the MTFR since the maximum driving head was reached during Run 10. It was anticipated that the lower MTFR would result in a lower driving head. This did not happen. The driving head remained essentially the same as at the end of Run 10. Hence it was concluded that the sediment mass loading capacity had been reached and testing was terminated. An additional 4.38 kg was captured by the SK75 Stormkeeper Chamber Sediment Strip during Run 11 increasing the total sediment captured for the 11 test runs to 30.8 kg (67.8 lb). This equates to 2.26 lb/geotextile fabric surface area. The maximum impervious inflow drainage area **per** SK75 Stormkeeper Chamber Sediment Strip calculated by the equation in the Appendix in the *New Jersey Department of Environmental Protection Laboratory Protocol to Assess Total Suspended Solids Removal by a Filtration Manufactured Treatment Device" (January 2013)* is 0.11 acre.

3.4 Hydraulic Characteristics

Steady-state pressure measurements were recorded on the clean chamber to establish the hydraulic characteristic curves. Recorded flows ranged from 10 to 353 gpm (0.33 to 11.75 gpm/ft^2). The recorded data is shown in **Table 7** and corresponding curves on **Figure 16**.

Table 7 Measured Hydraulic Data

Flow			Inlet El. (A')	Outlet El. (D')	System Energy Loss
gpm	cfs	gpm/sq-ft	Corrected for Energy ft	Corrected for Energy ft	A'-D'
0	0				
10.1	0.02	0.34	**1.281**	**0.389**	**0.892**
25.4	0.06	0.85	**1.331**	**0.446**	**0.886**
49.6	0.11	1.65	**1.385**	**0.513**	**0.873**
100.1	0.22	3.34	**1.494**	**0.619**	**0.875**
153.1	0.34	5.10	**1.578**	**0.708**	**0.870**
203.1	0.45	6.77	**1.832**	**0.787**	**1.046**
246.1	0.55	8.20	**2.112**	**0.855**	**1.257**
305.5	0.68	10.18	**2.583**	**0.939**	**1.644**
352.5	0.79	11.75	**2.984**	**1.010**	**1.975**

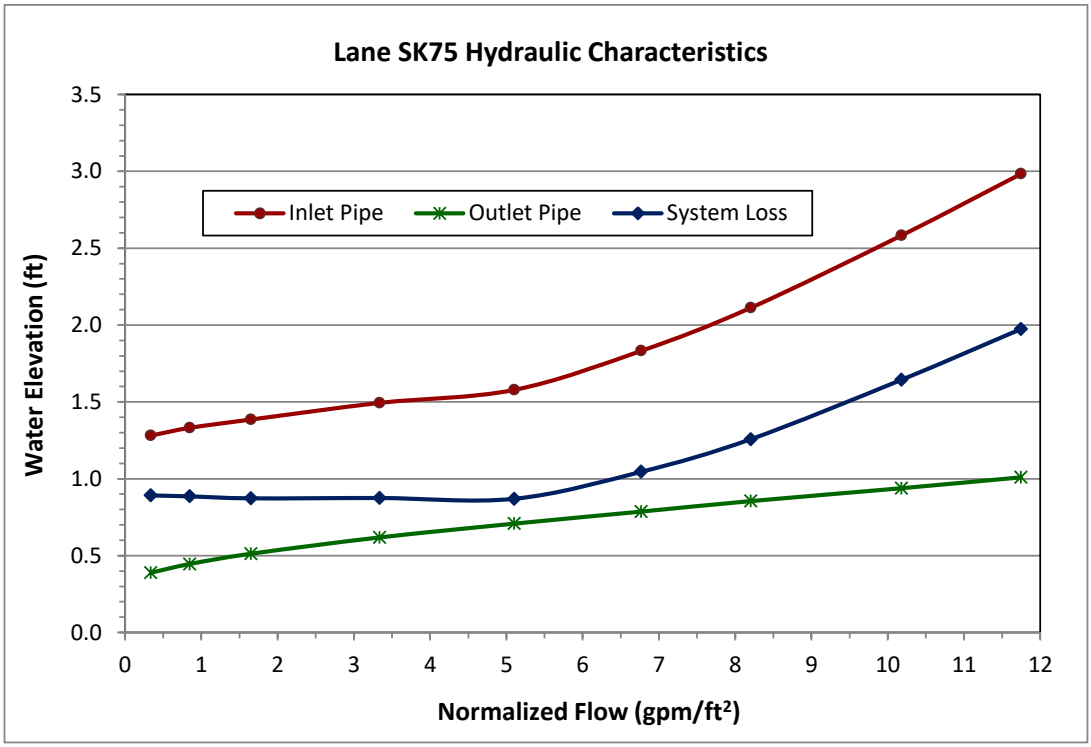


Figure 16 SK75 Hydraulic Characteristic Curves

4. Maintenance

Maintenance is accomplished by jetting the system to flush the sediment and other storm water pollutants to a manhole or other structure. Once the material has been flushed to the structure, the contaminants are moved from the system via a vacuum truck or similar operation. Inspection ports are installed along the length of the sediment strip to determine when maintenance is required. The Stormkeeper Chambers should be inspected every 6 months during the first year of operation. The inspection interval should be adjusted based on previous observations of sediment accumulation and standing water levels. If sediment is at or above 3-inches, or standing water remains 24 hours after a storm event, the chambers should be cleaned out using a Jet-Vac (jetting) process. In general, the system should be maintained yearly. A greater interval can be specified once the site has stabilized and monitoring has occurred.

During the life of the StormKeeper Sediment Strip, the woven polypropylene fabric will be subject to repeated loading of sediment and cleaning by forces that may cause abrasion and degradation. Although unlikely to be needed, due to the extremely high abrasion resistance of polypropylene, a method has been established to replace the fabric without creating the need to excavate the entire system. The following protocol describes methods for inspecting and replacing the fabric if necessary.

Inspection

The sediment strip is designed to allow inspection ports to be installed in any chamber along the length of the strip. Inspection ports are installed on approximately 30 foot intervals to facilitate regular inspections and maintenance activities. The inspection ports are typically 6 inches in diameter and are used to determine the amount of sediment in the structure as well as the quality of the fabric under the system. In most cases the fabric under the system will never have to be replaced due to the inherent high resistance to abrasion of polypropylene. Removal of sediment is covered under a separate document for the Lane StormKeeper System. After the system has been cleaned during a maintenance cycle, the fabric should be inspected to determine its condition. Should the fabric show excessive wear the top layer shall be subject to a replacement procedure. Wear of the fabric is indicated by frayed edges, discoloration, and broken filaments in the fabric. Should the amount of wear cover an area of over 25% of the fabric surface area, then replacement or patching in accordance with the manufacturer's recommendations will be required.

Fabric Replacement

Should the fabric require replacement the following steps should be followed:

A. Excavate to the top of the Sediment Strip row on one end of the system. It is not necessary to excavate the entire row. If there is a header system, the access port can be placed at either end. If there is only one header, the access port should be at the end opposite of the header.
B. Once the top of the Sediment Strip has been located an 18-inch access port should be cut centered on the top of the chamber and at least 12 inches from the end of the chamber row. Care should be taken to prevent material from entering the system through the access port.

C. An 18-inch riser section can be connected to the chamber to prevent material from entering the system.
D. Once the access port has been installed, entrance into the system is possible. Adequate confined space entry protocols should be followed.
E. The existing top layer of fabric should be removed. Removal is completed by cutting the fabric where it meets the foot of the chamber and disposing of it properly.
F. A new fabric will be the same fabric as specified in the original specifications or meeting the same material properties.
G. New woven geotextile will be folded lengthwise and pulled through the 18-inch access port which was installed. The fabric can be pulled through sediment strip row either by hand or using mechanical means. The header can be utilized to pull fabric through the length of the sediment strip if required.
H. Once the fabric has been pulled through the length of the sediment strip it is unfolded to cover the bottom of the chamber floor and the existing fabric which has remained in place.
I. The new fabric is attached to the floor of the chamber by welding the edges of the fabric to the feet of the chambers. Both materials are polypropylene allowing them to be attached to each other via welding. A qualified professional plastics welder should perform the work utilizing appropriate tools and welding rod.
J. A continuous weld should be run along all edges of the fabric and chamber to connect the fabric permanently to the chamber bottom. At the ends, the fabric shall be secured to the endcaps via welding.
K. Once installation of the new fabric is complete the chamber access can either be removed or left in place. If left in place, a flowable low strength concrete fill should be placed around the structure up to 12 inches above the chamber and 12 inches around the structure. If the structure is removed, a polypropylene patch should be placed over the chamber and welded in place. #57 stone should be placed around and above the patch up to 12 inches. Remaining backfill should be placed as required for the surface finish above the sediment strip.

Surface Treatment

Upon completion of the fabric replacement, the backfill and ground finish above the patch will be matched to the surrounding ground finish or as directed by the owner.

5. Performance Verification

The SK75 StormKeeper® Chamber Sediment Strip® test unit wrapped with Mirafi® 600X geotextile demonstrated an 83.9% average TSS removal efficiency and a sediment mass loading capacity of 2.26 lb/geotextile fabric filtration area when operated with a driving head <3.26 ft at a hydraulic loading rate of 4 gpm/ft^2 of geotextile fabric filtration area. The MTFRs and maximum allowable drainage area for other Lane and Prinsco models are shown in **Table 8**.

Table 8 Lane and Prinsco Model MTFRs and Maximum Allowable Drainage Area

Model	Driving Head (ft)	Hydraulic Rate (gpm/ft^2)	MTFR (gpm)	Mass Loading Capacity (lb/ft^2)	Mass Storage Capacity (lb)	Maximum Allowable Drainage Area (acres)[1]
Lane						
SK75	3.26	4.0	120	2.26	67.8	0.113
SK180	3.26	4.0	240	2.26	135.6	0.226
Prinsco						
HS75	3.26	4.0	120	2.26	67.8	0.113
HS180	3.26	4.0	240	2.26	135.6	0.226

1. Based upon the equation in the NJDEP Filter Protocol: Maximum Inflow Drainage Area (acres) = Weight of TSS Captured before Efficiency Drops below 80% / 600 lbs per Acre of Drainage Area Annually.

6. References

ASTM (2013*), "Standard Test Methods for Determining Sediment Concentration in Water Samples"*, Annual Book of ASTM Standards, D3977-97, Vol. 11.02.

ASTM (2007), *"Standard Test Method for Particle Size Analysis of Soils",* Annual Book of ASTM Standards, D422-63, Vol. 04.08.

ASTM (2007), *"Standard Test Methods for Determination of Water (Moisture) Content of Soil by Direct Heating",* Annual Book of ASTM Standards, D4959-07, Vol. 04.08.

ASME (1971), *"Fluid Meters Their Theory and Application- Sixth Edition".*

NJDEP (2013), *"New Jersey Department of Environmental Protection Laboratory Protocol to Assess Total Suspended Solids Removal by a Filtration Manufactured Treatment Device".* Trenton, NJ. January 25, 2013.

◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

RWM-DM-1200 & DM-1200-OS Stormwater Systems

Developed by Rainwater Management Ltd.
Port Coquitlam, BC, Canada

Registration: GPS-ETV_V2022-09-15

In accordance with

ISO 14034:2016

Environmental Management — Environmental Technology Verification (ETV)

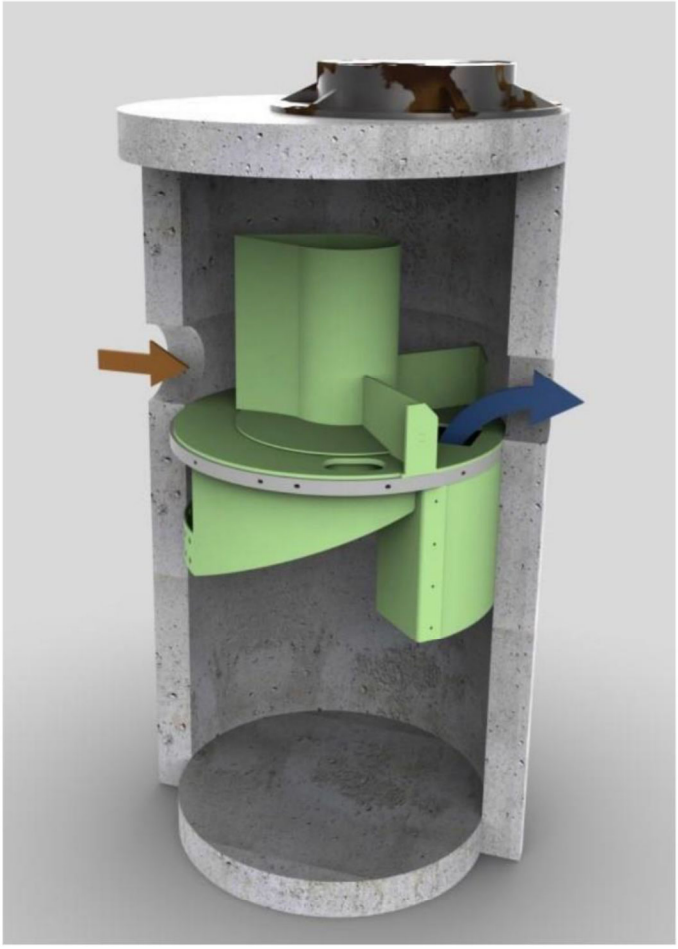
John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

September 15, 2022
Vancouver, BC, Canada

Verification Body
GLOBE Performance Solutions
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Technology description and application

The RWM-DM-1200 and RWM-DM-1200-OS test unit is a full scale 1.22 m (4 foot) diameter by approximately 2.44 m (8 foot) high stormwater treatment device that facilitates the capture of oil (OS only) and sediment. The test tank was fabricated from plastic and included 305 mm (12 inch) diameter inlet and outlet pipes, oriented along the centerline of the tank. The pipe inverts were located 1.52 m (60 in.) above the sump floor with 1.5% slopes. The effective sedimentation area was 1.17 m^2 (12.57 ft^2)



The test units consist of a treatment disk containing a 406 mm (16 inch) diameter flow splitter and cleanout tube, two oval inlet openings, followed by two angled bypass weirs. The weir height of the RWM-DM-1200 was 229 mm (9 inches) and the RWM-DM-1200-OS was 152 mm (6 inches). A 254mm x 254mm (10" x 10") outlet opening was located downstream of the bypass weirs. Flow entering the unit was split to either side of center and directed to the oval inlets. The flow is conveyed into the collection sump volume by means of two sloped channels that curved along the tank wall and converged at the upstream side. An enclosed outlet baffle conveys the flow upward to the square outlet hole and into the outlet pipe.

Figure 1: Graphic of a typical inline RWM-DM-1200 unit and core components

Performance conditions

The data and results published in this Verification Statement were obtained from the testing program conducted on the Rainwater Management Ltd.'s DM-1200 and DM-1200-OS OGS devices, in accordance with the Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014) and associated bulletins, including *CETV 2021-04-0001* which requires more stringent test conditions that limit the potential for sediment to accumulate in the inlet pipe and *CETV 2018-09-0001* which changes how the light liquid simulation test is undertaken. The Procedure was prepared by the Toronto and Region Conservation Authority (TRCA) for Environment Canada's Environmental Technology Verification (ETV) Program. A copy of the Procedure may be accessed on the Canadian ETV website at www.etvcanada.ca.

Performance claim(s)

Capture test[1]:
During the capture test, the RWM-DM-1200, with a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and a constant influent test sediment concentration of 200 mg/L, removes 67.7, 63.3, 54.9, 45.4, 42.1, 35.5, and 33.6 percent of influent sediment by mass at surface loading rates (SLR) of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m^2, respectively.

During the capture test, the RWM-DM-1200-OS, with a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and a constant influent test sediment concentration of 200 mg/L, removes 67.7, 63.3, 54.9, 45.4, 42.1, 35.0, and 28.7 percent of influent sediment by mass at surface loading rates of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m^2, respectively.

Scour test[1]:
During the scour test, the RWM-DM-1200, with 15.2 cm (6 inches) of test sediment pre-loaded onto a false floor reaching 50% of the manufacturer's recommended maximum sediment sump storage depth, generate corrected effluent concentrations of 0.4, 3.4, 2.0, 0.0, and 0.4 mg/L at 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2, respectively.

Scour testing was not performed on the DM-1200-OS unit because the unit is identical to the DM-1200 model with the exception of the bypass weir, which is set at a lower elevation (i.e. the unit bypasses at lower flow rates). Tests at SLRs up to the lower weir height would experience the same scour as the DM-1200 unit. Test SLRs that overtop the bypass weir would have lower scour rates than the DM-1200 because there would be lower velocity and turbulence within the collection sump.

Light liquid retention test[1]:

DM-1200-OS, with the lower chamber oil collection zone pre-loaded with floatable low density polyethylene beads as an oil surrogate, and representing oil volume equal to a depth of 5 cm over the sedimentation area, retains 99.9% of the bead mass and volume loaded into the unit both initially during the preloading process and at 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2.

Performance results

The test sediment consisted of ground silica (1 – 1000 micron) with a specific gravity of 2.65, uniformly mixed to meet the particle size distribution specified in the testing procedure. The *Procedure for Laboratory Testing of Oil Grit Separators* requires that the average of the test sediment particle size distribution (PSD) meet the specified PSD percent less than values within a boundary threshold of 6%. The comparison of the average test sediment PSD (n=7) to the CETV specified

[1] The claim can be applied to other units smaller or larger than the tested unit as long as the untested units meet the scaling rule specified in the Procedure for Laboratory of Testing of Oil Grit Separators (Version 3.0, June 2014)

PSD in Figure 2 indicates that the test sediment used for the capture and scour tests met this condition.

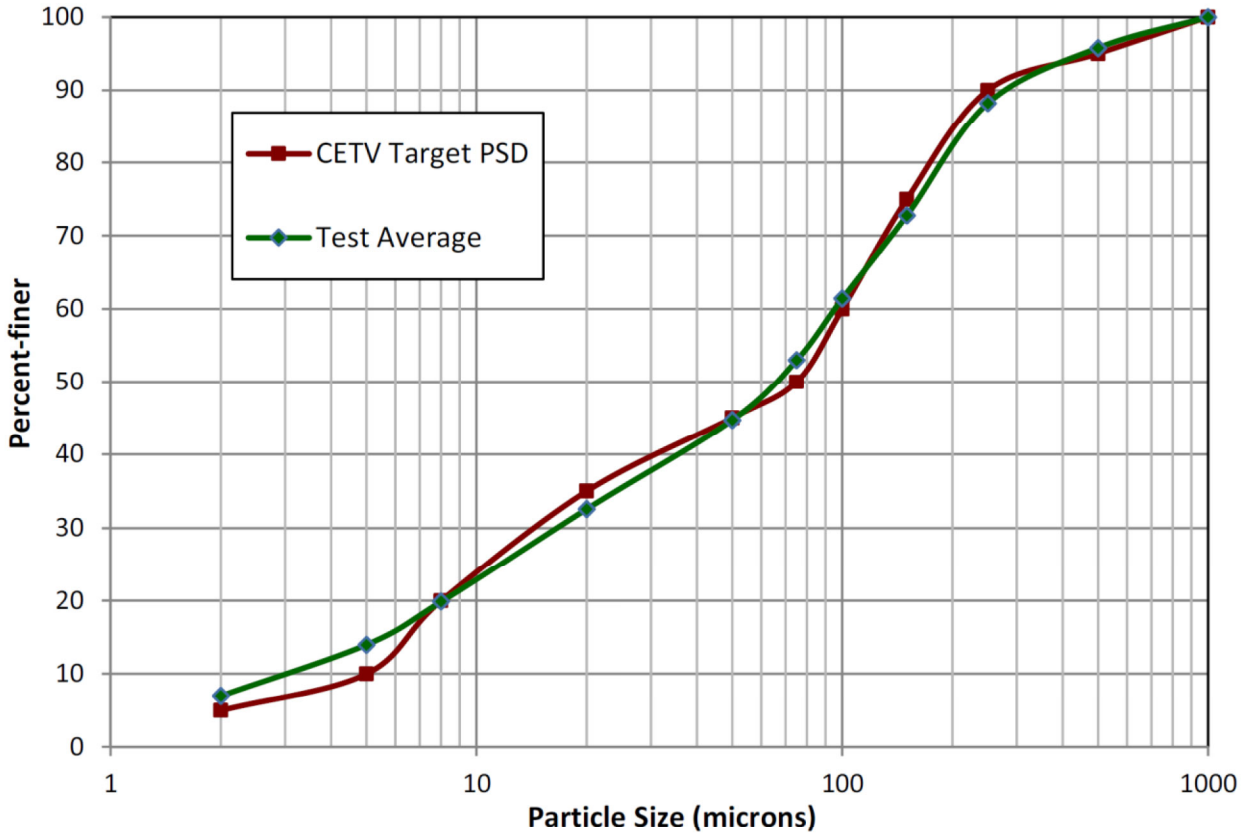


Figure 2. Particle size distribution of test sediment compared with the CETV Target PSD.

The capacity of the device to retain sediment was determined at seven surface loading rates using the modified mass balance method. This method involves measuring the mass and particle size distribution of the injected and retained sediment for each test run as the basis for calculating removal efficiencies. Performance was evaluated with a false floor simulating the technology filled to 50% of the manufacturer's recommended maximum sediment storage depth. The test was carried out with clean water that maintained a sediment concentration below 20 mg/L. Based on these conditions, removal efficiencies for individual particle size classes and for the test sediment were determined for each of the tested surface loading rates (Table 1). Since the DM-1200 and DM-1200-OS models are identical except for the weir height, which was shortened from 9" to 6" for the DM-1200-OS unit, sediment capture tests at surface loading rates from 40 to 1400 L/min/m^2 were only performed on the DM-1200 unit. Two additional tests were conducted on the DM-1200-OS unit at the bypass SLRs of 1000 and 1400 L/min/m^2.

Table 1. Removal efficiencies (%) of the DM-1200 and DM-1200-OS units at specified surface loading rates

Particle size fraction (µm)	Surface loading rate (L/min/m^2)								
	40	80	200	400	600	1000	1400	OS-1000	OS-1400
>500	100*	100*	93	89	100*	57	93	100*	86
250 - 500	97	88	86	92	95	92	89	100*	72
150 - 250	97	100*	91	100*	100*	70	58	66	65
105 - 150	100*	100	100*	81	58	44	44	40	47
75 - 105	100*	100*	85	64	46	29	22	31	23
53 - 75	87	78	67	37	30	18	19	17	11
20 - 53	68	38	26	13	6	3	5	6	4
8 - 20	24	17	0	0	0	0	1	3	2
5 – 8	5	0	0	0	0	0	1	0	0
<5	14	14	8	7	6	7	6	0	0
All particle sizes by mass balance including SS in the inlet pipe	**67.7**	**63.3**	**54.9**	**45.4**	**42.1**	**35.5**	**33.6**	**35.0**	**28.7**
All particle sizes by mass balance not including SS in the inlet pipe	67.7	63.3	54.9	45.4	42.0	30.2	29.7	35.0	28.7

* Removal efficiencies were calculated to be above 100%. Calculated values ranged between 101 and 133% (average 110%). See text and Bulletin # CETV 2016-11-0001 for more information.

In some instances, the removal efficiencies were above 100% for certain particle size fractions (Table 1). These discrepancies are not unique to any one test laboratory and may be attributed to errors relating to the blending of sediment, collection of representative samples for laboratory submission, and laboratory analysis of PSD. Due to these errors, caution should be exercised in applying the removal efficiencies by particle size fraction for the purposes of sizing the tested device (see Bulletin # *CETV 2016-11-0001*). The results for "all particle sizes by mass balance" shown in Table 1 are based on measurements of the total injected and retained sediment mass, and are therefore not subject to blending, sampling or PSD analysis errors.

Mass balance sediment removal efficiencies are provided for sediment capture in the unit and inlet pipe as well as for the sediment settling only in the unit. These values show that sediment accumulated in the inlet pipe primarily during the 1000 and 1400 SLR tests on the DM-1200 model (5.3% and 4.0% of the injected mass, respectively). Sediment deposition in the inlet pipe consisted of coarse sediment > 100 µm. The *Procedure* allows for the mass of sediment accumulated in the inlet pipe to be claimed as captured mass. Measures were implemented in 2021 through *Bulletin CETV 2021-04-0001* to limit the potential for fine sediment accumulation in the inlet during lab tests.

Figure 3 compares the PSD average of the of the test sediment to the PSD of the sediment retained by the DM-1200 at each of the tested surface loading rates. As expected, the capture efficiency for fine particles in both units was generally found to decrease as surface loading rates increased.

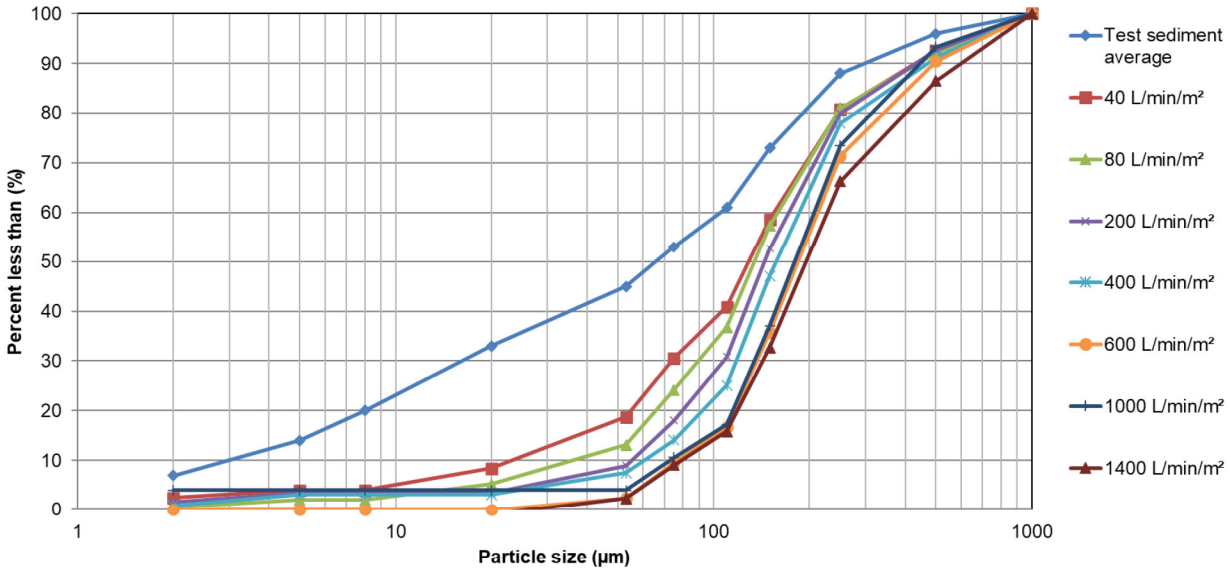


Figure 3. PSD average of the of the test sediment to the PSD of the sediment retained by the DM-1200 at each of the tested surface loading rates

Table 2 shows the results of the sediment scour test for the DM-1200 unit. The scour test involved preloading 10.2 cm of fresh test sediment into the sedimentation sump of the device. The sediment was placed on a false floor to mimic a device filled to 50% of the maximum recommended sediment storage depth. Clean water was run through the device at five surface loading rates over a 30-minute period. Each flow rate was maintained for 5 minutes with a one-minute transition time between flow rates. The test was stopped and started after the 800 L/min/m^2 test to change flow meters. Effluent samples were collected at one minute sampling intervals and analyzed for Suspended Sediment Concentration (SSC) and PSD by approved methods. The effluent samples were adjusted based on the background concentration of the influent water. Typically, the smallest 5% of particles captured during the 40 L/min/m^2 sediment capture test is also used to adjust the concentration, as per the method described in *Bulletin #* *CETV 2016-09-0001*. However, the low flow rate particle size adjustment was not made because the concentrations were very low even without adjustment. Results showed average effluent sediment concentrations below 4 mg/L at all tested surface loading rates.

Table 2. Measured scour effluent concentration (mg/L)

Measured concentration at each surface loading rate					
Effluent Sample No.	200 L/min/m^2	800 L/min/m^2	1400 L/min/m^2	2000 L/min/m^2	2600 L/min/m^2
1	1.5	0.0	1.7	0.0	0.0
2	0.1	0.0	4.5	0.0	0.7
3	0.0	2.0	2.8	0.0	1.2
4	0.0	8.2	0.7	0.0	0.2
5	0.5	6.6	0.0	0.0	0.0
Average	**0.4**	**3.4**	**2.0**	**0.0**	**0.4**

The results of the light liquid retention test used to evaluate the unit's capacity to capture and prevent re-entrainment of light liquids are reported in Table 3. The test involved preloading

33.4 kg (corresponding to a 5 cm depth over the collection sump area of 1.17m^2) of surrogate low-density polyethylene beads (Dowlex™ 2517) within the oil collection skirt and running clean water through the device continuously at five surface loading rates (200, 800, 1400, 2000, and 2600 L/min/m^2). Each flow rate was maintained for 5 minutes with approximately 1 minute transition time between flow rates. The effluent flow was screened to capture all the pellets that were lost during the preloading process (i.e. that did not enter the collection skirt) and all re-entrained pellets throughout the test. Results showed that 99.9% of the bead mass injected into the unit were retained.

Table 3. Light-liquid retention test results

Surface loading rate	Collected mass (grams)	Remaining Mass (grams)	Retained Mass (%)
Not captured during preloading	20.5	33,380	99.9
200 L/min/m^2	0.0	33,380	99.9
800 L/min/m^2	4.6	33,375	99.9
1400 L/min/m^2	2.1	33,373	99.9
2000 L/min/m^2	0.9	33,372	99.9
2600 L/min/m^2	0.5	33,371	99.9

The DM-1200 and DM-1200-OS were tested with clean water to establish head loss through the units. Flow and water level measurements were recorded at steady-state flow conditions. Flows were set and measured using calibrated differential-pressure flow meters and control valves. Each test flow was set and operated at steady state for approximately 5 minutes, after which time a minimum of 60 seconds of flow and pressure data were averaged and recorded for each pressure tap location. Water elevations were measured one pipe-diameter upstream and downstream of the unit, as well as upstream and downstream of the bypass weir. The measured bypass rate for the DM-1200 and DM-1200-OS was 21 and 11.7 L/s (1,077 and 600 L/min/m^2), respectively. Results of the measurements, corrected for velocity head, are provided in Table 4.

Table 4. Head loss measurements by flow rate for the DM-1200 and DM-1200-OS

Water Elevations (adjusted to outlet invert)			Losses (DM-1200)		
Measured Flow	Inlet Pipe	Outlet Pipe	Inlet El. (A')	Outlet El. (D')	System Energy Loss
	A	D	Corrected for Energy	Corrected for Energy	A'-D'
L/sec	m	m	m	m	m
1.6	0.045	0.025	0.050	0.041	0.009
3.2	0.062	0.036	0.069	0.058	0.011
6.3	0.093	0.054	0.100	0.081	0.020
11.7	0.140	0.074	0.147	0.111	0.036
21.2	0.220	0.102	0.228	0.152	0.076
25.3	0.237	0.114	0.247	0.166	0.080
30.0	0.252	0.125	0.264	0.183	0.081
38.0	0.287	0.159	0.302	0.209	0.093
44.6	0.302	0.169	0.322	0.228	0.094
50.9	0.318	0.177	0.343	0.245	0.097
60.2	0.341	0.188	0.375	0.271	0.105

Water Elevations (adjusted to outlet invert)			Losses (DM-1200-OS		
Measured Flow	Inlet Pipe	Outlet Pipe	Inlet El. (A')	Outlet El. (D')	System Energy Loss
	A	D	Corrected for	Corrected for Energy	A'-D'
L/sec	m	m	m	m	m
1.6	0.045	0.025	0.050	0.041	0.009
3.2	0.062	0.036	0.069	0.058	0.011
6.3	0.093	0.054	0.100	0.081	0.020
11.7	0.141	0.075	0.148	0.111	0.037
15.8	0.161	0.085	0.171	0.131	0.040
25.3	0.197	0.109	0.211	0.169	0.043
30.0	0.212	0.117	0.229	0.186	0.043
37.8	0.250	0.146	0.269	0.207	0.062
47.3	0.280	0.163	0.304	0.235	0.069
57.2	0.310	0.180	0.341	0.263	0.078
63.6	0.328	0.190	0.367	0.280	0.087
72.7	0.360	0.196	0.411	0.306	0.105

Variations from the Procedure

Minor variances from the *Procedure for Laboratory Testing of Oil-Grit Separators* used as the basis of testing for this verification were as follows:

1. The *Procedure* states that the tested device "must be a full scale commercially available device with the same configuration and components as would be typical for an actual installation." The unit tested for this verification had the same internal components as would be typical for a commercial installation, but the internal components were placed inside a structure constructed of plastic, rather than a manhole made of concrete, the latter of which is typical for most installations. The dimensions of the structure were the same as would have been the case had the manhole been concrete. The use of alternate materials for the structure was not believed to significantly affect system performance and the variance was approved by the verifier prior to testing

2. As part of the capture test, evaluation of the 40 and 80 L/min/m^2 surface loading rate was split into 3 and 2 parts, respectively. The test was conducted in parts because of the long duration (i.e. over 10 hours) needed to feed the required minimum of 11.3 kg of test sediment into the unit. At the end of the first and second parts of the test, the flow rates were gradually shutdown to prevent capture of particles that would have been washed out under normal circumstances. The requirement to split the test into parts was not anticipated during the writing of the *Procedure* but has been a common feature of testing at the lower surface loading rate. The breaks were not likely to have significantly impacted results and the variance was approved prior to testing.

3. It was necessary to change flow meters during the sediment scour and light liquid re-entrainment test, as the required flows exceeded the minimum and/or maximum range of any single meter. When the flow capacity of the selected meter was reached, the flow was shut down over a period of approximately 10 seconds and all flow data saved. The next data acquisition file was executed, and flow increased at a rate that corresponded to reaching each previous target flow after a period of 1-minute. This procedure was approved by the verifier prior to testing, in recognition that most particles susceptible to scour at low flows would not be in the sump at higher flows. Similarly, re-entrainment of the oil beads was not expected to be significantly affected by the flow meter change.

Verification

This verification was completed by the Verification Experts, at the Toronto and Region Conservation Authority (TRCA), contracted by GLOBE Performance Solutions, using the International Standard ***ISO 14034:2016 Environmental management -- Environmental technology verification (ETV)***. Data and information provided by Rainwater Management Ltd. to support the performance claim included the Technical Evaluation Report No. 1292RWM-R0 prepared by Alden Research Laboratory of Holden, Massachusetts, dated May 2022. This report is based on testing completed in accordance with the *Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014)* and associated Bulletins.

What is ISO14034:2016 Environmental management – Environmental technology verification (ETV)?

ISO 14034:2016 specifies principles, procedures and requirements for environmental technology verification (ETV) and was developed and published by the *International Organization for Standardization (ISO)*. The objective of ETV is to provide credible, reliable and independent verification of the performance of environmental technologies. An environmental technology is a technology that either results in an environmental added value or measures parameters that indicate an environmental impact. Such technologies have an increasingly important role in addressing environmental challenges and achieving sustainable development.

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