Functional Servicing Report

Burwash Road Development

Township of Hamilton

Engage Project Number 23121

Revision Summary

Revision No.	Revision Title	Date	Revision Summary
1	Issued for Draft Plan Application	2026-08-04	Final

Accessibility for Ontarians With Disabilities (AODA)

This report was prepared and formatted to comply with AODA guidelines. Engage is committed to taking all reasonable steps to ensure that anyone who accesses our services will be able to do so in a way that respects their needs. As such, if you require any assistance with the access or interpretation of this proposal, please do not hesitate to reach out and we will be pleased to accommodate you.

Rights and Distribution

This report was prepared by Engage Engineering Ltd. (Engage) for Stalwood Homes and is intended for their sole use only. This report is considered our professional work product and remains the property of Engage. Any unauthorized reuse, redistribution of, or reliance upon the report, shall be at the users risk, without liability to Engage.

Table of Contents

1.0 Introduction 1

1.1 Purpose 1
1.2 Site Description 1

2.0 Sanitary Servicing 2

2.1 Existing Conditions 2
2.2 Proposed Conditions 2
2.3 Design Criteria 3
2.4 Design Flow Rates 3

3.0 Water Servicing 4

3.1 Existing Conditions 4
3.2 Design Criteria 5
3.3 Design Flow Rates 5
3.4 Well Operational Capacity 6
3.5 Water Treatment Plant Capacity 6
3.6 Treated Water Storage Capacity 7
3.7 Pressure and Flow Discussion 7
3.8 Proposed Design Solution 8

4.0 Summary 8

List of Figures

Figure 1: Location Plan 2
Figure 2: Draft Plan 10
Figure 3: Preliminary Servicing Plan 12
Figure 4: Groundwater Elevations Figure 14

List of Tables

Table 1 – Proposed Sewage Flow Rates 4
Table 2 – Proposed Water Demand 6
Table 3 – Water Treatment Plant Capacity 6

List of Appendices

Appendix A: Miranda Wastewater Treatment System 16
Appendix B: Sanitary Flow Calculations 17
Appendix C: Water Demand Calculations 18

1.0 Introduction

1.1 Purpose

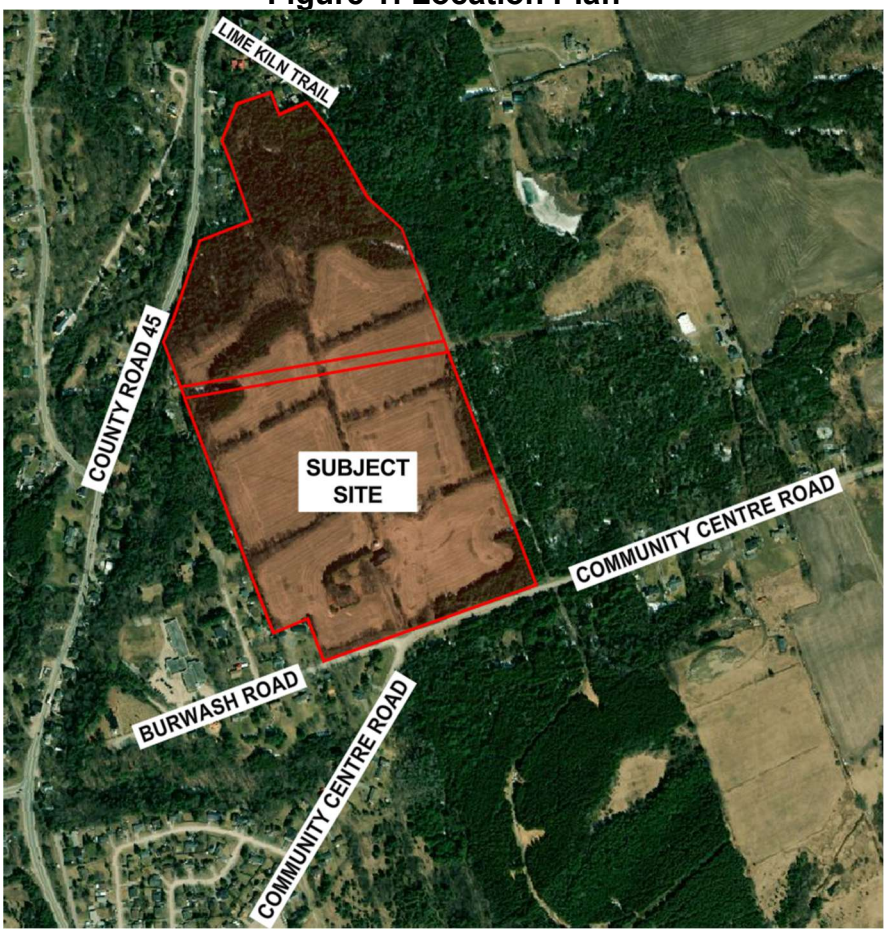
Engage Engineering Limited (Engage) has been retained by Stalwood Homes to prepare a Functional Servicing Report in support the Draft Plan Application for the proposed Burwash Road Subdivision located at 9380 Burwash Road in the Village of Baltimore, Township of Hamilton (Township). This report is being prepared to confirm the water and sanitary servicing strategy for the site.

1.2 Site Description

The subject site is the 29.9 ha parcel of land located between Burwash Road, County Road 45, and Lime Kiln Trail in the Township of Hamilton. The existing site primarily features farmland with a forested portion at the northern extent of the site. There is an existing house and barn located on the farmland with the existing house proposed to remain as part of the development. A watercourse runs through the forested portion of the site, and an existing gas line easement runs through the farmland portion of the site. The site is bound to the north by residential properties, Lime Kiln Trail, and County Road 45, to the east by forested areas, to the west by residential properties and County Road 45, and to the south by Burwash Road. The location of the proposed development is identified on the **Location Plan** included as **Figure 1**.

The proposed development will feature the existing house, a mix of single and semi-detached residential units, an internal road network, stormwater management blocks, park land, and a shared servicing block featuring a septic treatment area. The current draft plan shows a total unit count of 136 to 167 units. For this report, a unit count of 167 will be used for all calculations to verify servicing capacity. The layout of the proposed development can be seen on the **Draft Plan** included as **Figure 2**.

Figure 1: Location Plan



2.0 Sanitary Servicing

2.1 Existing Conditions

The Township of Hamilton does not have a municipal wastewater collection system. Most of the residential dwellings in the area surrounding the site are serviced with private on-site septic systems.

2.2 Proposed Conditions

The site is proposed to be serviced by a network of gravity sewer. The gravity sewer within Phase 1 and Phase 2A are proposed to drain directly to a shared wastewater treatment system on the servicing block east of the site entrance. The gravity sewer within Phase 2B of the development will drain to a sanitary pumping station located on the northern stormwater management block that will convey flows to the shared wastewater treatment system via a forcemain.

The shared wastewater treatment system will be a Miranda Water Technologies modular wastewater treatment system. Included in **Appendix A** are further details prepared by

Miranda Water Technologies regarding the wastewater treatment system, including design calculations, a process design report, and an MECP compliance addendum to the report. The wastewater treatment system will treat the sewage and discharge treated water to grade at the southeast corner of the property.

The proposed sanitary system including the gravity sewer, forcemain, sanitary pumping station, and servicing block for the modular wastewater treatment system can be seen on the **Preliminary Site Servicing Plan** included as **Figure 3**.

2.3 Design Criteria

Design criteria to analyze the wastewater flow for the development has been assembled from the Town of Cobourg Design Guidelines, MECP Design Guidelines for Sewage Works, and population statistics for the Township of Hamilton. The design criteria include:

- Residential sewage flow of 450 L/person/day (MECP)
- Single Family Unit population of 2.972 people/unit (from 2021 DC Study by Watson & Associates referenced in the Township of Hamilton Water Supply Master Plan Phase 1 Report prepared February 28, 2025, by J.L. Richards & Associates Limited)
- Harmon peaking formula to be applied to residential flows with a maximum value of 3.80 (Town of Cobourg)
- Infiltration rate of 0.26 L/s/ha for inflow and infiltration (Town of Cobourg)

2.4 Design Flow Rates

Sewage flow rates for the proposed development were calculated based on the design criteria presented in **Section 2.3**. The calculations are included in **Appendix B** and summarised in **Table 1**. The proposed sanitary flows for Phase 1 were calculated separately to be utilized in the phased deployment of the Miranda wastewater treatment system.

The residential sanitary flows were calculated using 167 single family units for the full development and 74 units for Phase 1. The area used for the inflow and infiltration was determined based on the groundwater elevations figure included in the Hydro Geotechnical Report prepared by GHD Engineering in November 2024, which shows the directions of groundwater flow within the site. The **Groundwater Elevations Figure** is included as **Figure 4**. The infiltration area includes the site area upstream of the internal roads where groundwater flow is directed towards the gravity sewer.

Table 1 – Proposed Sewage Flow Rates

Flow Type	Full Development Proposed Flow (m^3/day)	Full Development Proposed Flow (L/s)	Phase 1 Proposed Flow (m^3/day)	Phase 1 Proposed Flow (L/s)
Average Residential Flow	223.3	2.59	99.0	1.15
Peak Residential Flow	848.7	9.82	376.1	4.35
Infiltration Flow	430.6	4.98	229.1	2.65
Design Average Flow	**654.0**	**7.57**	**328.1**	**3.80**
Design Peak Flow	**1279.3**	**14.81**	**605.2**	**7.00**

3.0 Water Servicing

3.1 Existing Conditions

The Burwash Road Subdivision is proposed to be serviced by a connection to the Creighton Heights drinking water system. Information on the existing water system was obtained from the Township of Hamilton Water Supply Master Plan Phase 1 Report (Master Plan) prepared February 28, 2025, by J.L. Richards & Associates Limited. The Master Plan is heavily referenced throughout the water servicing portion of this report.

The Creighton Heights drinking water system is owned and operated by the Township of Hamilton and consists of the following infrastructure:

- Three groundwater wells (two primary production wells and one back-up well). The primary wells have a rated capacity of 1,303 m^3/day based on their Permit to Take Water (PTTW). However, the Master Plan states that the Township has defined an operational capacity of 700 m^3/day.
- A water treatment plant (WTP) with a rated capacity of 979 m^3/day. The WTP features chemical feed systems, two greensand filters, UV units, methane stripper, high lift pumps, and a backup power generator.
- A two-celled water storage reservoir with a capacity of 1,130 m^3 (874 m^3 effective storage)
- Four pressure zones
- Three PRV chambers
- Approximately 14 km of watermains
- 78 fire hydrants

The Master Plan contains historical flow data from the water treatment plan for four years of operation from 2020 to 2023. The historical flow data shows an average day demand of **327 m^3/day** and a maximum day demand of **668 m^3/day**.

There is an existing 200 mm Ø watermain in the Burwash Road Right-of-Way that connects to the Community Centre Road watermain, extending west from the intersection of the two streets and terminating at the Baltimore Public School with a dead end.

3.2 Design Criteria

The following design criteria used to analyze the water demand for the site were sourced from MECP guidelines and statistics from the Master Plan, and include:

- Residential water demand of 225 L/person/day (4-year average day demand per capita utilized in the Master Plan)
- Single Family Unit population of 2.972 people/unit (from 2021 DC Study by Watson & Associates referenced in the Master Plan)
- Maximum day factor of 2.50 (based on existing service population of 1,283)
- Peak hour factor of 3.75 (based on existing service population of 1,283)
- The minimum pressure of the system shall meet or exceed 40 psi (275 kPa) during normal operating conditions (from MECP Guidelines)
- The minimum pressure of the system shall meet or exceed 20 psi (138 kPa) during fire flow conditions (from MECP Guidelines)
- The optimal normal operating pressure of the system is between 50 and 70 psi (350 to 480 kPa) (from MECP Guidelines)
- The maximum pressure of the system shall not exceed 100 psi (700 kPa) during normal operating conditions (from MECP Guidelines)

3.3 Design Flow Rates

Water demand rates for the proposed development were calculated based on the design criteria present in **Section 3.2**. The calculations are included in **Appendix C** and summarised in **Table 2**.

A required fire flow of 4,000 L/min was used for the proposed demands, taken from Table 7 of the Fire Underwriters Survey (2020) which provides an estimate of fire flows for one- and two-family dwellings up to 450 m^2. It was assumed the dwellings would be no larger than 450 m^2 with exposure distances of 3 to 10 m.

Table 2 – Proposed Water Demand

Demand Type	Demand (L/min)	Demand (m^3/day)
Average Day Demand	77.6	112
Maximum Day Demand	193.9	279
Peak Hour Demand	**290.8**	**419**
Fire Flow	4,000.00	5,760
Maximum Day + Fire Flow	**4,193.9**	**6,039**

3.4 Well Operational Capacity

The Master Plan states that the raw water production is at 99% of the operational limit of the wells and 53% of the PTTW rated capacity. This indicates that the existing system is limited by the operational capacity of the wells and there is no residual capacity for the proposed development. The Master Plan recommends that the operational limits of the existing system be confirmed through on-site evaluations.

3.5 Water Treatment Plant Capacity

The capacity of the existing WTP was checked to confirm that it can accommodate the additional demand required to service the development. The existing maximum day demand was combined with the proposed maximum day demand for the development and compared to the rated capacity of the WTP. The results are presented in **Table 3** below.

Table 3 – Water Treatment Plant Capacity

Demand Type	Maximum Day Demand (m^3/day)	Rated Capacity of WTP (m^3/day)	Capacity (%)
Existing	668	979	68%
Existing + Proposed	947	979	97%

Under existing conditions, the WTP is operating at 68% of it's rated capacity and has limited residual capacity to accommodate new developments. With the addition of the proposed demands, the WTP capacity is over the 80% capacity threshold. Therefore, upgrades to the WTP should be considered prior to constructing the new development.

The WTP analysis included in the Master Plan has similar findings, projecting that the 80% capacity threshold with be reached in 2030 for the low growth scenario and in 2027 for the high growth scenario.

3.6 Treated Water Storage Capacity

The Master Plan states that the Creighton Heights system has sufficient available storage to accommodate existing conditions, with a required storage of 861 m^3 and an available storage of 874 m^3. This indicates that the storage is very near its maximum capacity and there is no residual capacity for the proposed development. Upgrades to the treated water storage are recommended prior to constructing the new development.

3.7 Pressure and Flow Discussion

The Master Plan includes modelling results for the existing conditions of the full Creighton Heights water system. At the intersection of Burwash Road and Community Centre Road, the modelling results show pressures of 49.6 psi under average day demands and 49.2 psi under peak hour demands. The modelling results also show an available fire flow of 942 L/min at this intersection. The node in the hydraulic model at the intersection of Burwash Road and Community Centre Road where these results were taken from is at an elevation of 180.8 m.

The modelled results indicate that under existing conditions, the water pressure near the proposed site entrance on Burwash Road is above the minimum operating pressure of 40 psi but falls below the optimal operating range of 50 to 70 psi. The available fire flow at this location of 942 L/min is well below the required fire flow of 4,000 L/min.

The elevation of the proposed site limits the available pressure for domestic water demands. Proposed elevations along the watermain within the site range from approximately 184 to 200 m. The maximum site elevation of 200 m is 19.2 m above the node where the modelling results were taken from, corresponding to a pressure drop of 27.3 psi due to elevation. Based on the static pressure of 49.2 psi at the modelled node, a maximum pressure of 21.9 psi can be expected at the high point of the site. Due to the topography of the site and the low pressures in the existing system near the site entrance, a pressure boosting station will be required to provide adequate pressures for domestic demands for the site.

Physical constraints within the distribution system also limit the available fire flow at the site. The modelling results from the Master Plan show a drastic decrease in available fire flows between Oriole Crescent and the site entrance. The hydrant near the site entrance fed by a single run of watermain approximately 930 m long, spanning from Oriole Crescent to Burwash Road. This stretch of watermain would experience significant pressures losses due to friction under fire flow conditions. The available fire flow could be improved by creating a looped system or upsizing the single stretch of watermain between Oriole Crescent and the site. In the meantime, a dry hydrant and storage tanks are recommended for the subject site to provide adequate volume for fire protection. Alternatively, the firefighting volume could be provided with a dry hydrant connection at the southern wet pond if adequate depth and volume are available to meet the firefighting requirements. The stage storage of the pond will be reviewed at the detailed design stage to determine if suitable depth and volumes are available within the pond’s permanent pool.

Hydraulic modelling is recommended to determine the impact of the site's water demands on the existing system and the extent of required upgrades to bring pressures and flows up to the recommended service levels.

3.8 Proposed Design Solution

Water servicing for the site is proposed to be provided by a connection to the Creighton Heights drinking water system. The site's internal watermain will connect to the existing water system at both of the proposed driveway entrances. The existing 200 mm Ø watermain along Burwash Road will need to be extended approximately 190 m east along the municipal right-of-way to reach the site's east entrance. The site will feature an internal network of watermain including the necessary valves and appurtenances. Hydrants will be provided throughout the site at a maximum spacing of 180 m.

Due to the lack of pressure for domestic demands, a pressure boosting station will be required. The pressure boosting station has been located within the shared servicing block near the eastern entrance to the site. A pressure reducing valve has been included near the western entrance to the site to regulate pressures and isolate the new pressure zone to the proposed development. The preliminary layout of the proposed water system including the watermain, pressure boosting station, and pressure reducing valve can be seen on the **Preliminary Servicing Plan** included as **Figure 3**.

To address the fire flow requirements, a dry hydrant system is recommended within the site to provide adequate water storage for fire fighting. The dry hydrant system is proposed to be located within the servicing block. Table 1 of the Fire Underwriters Survey specifies a required duration of 1.5 hours for a fire flow of 4,000 L/min. This corresponds to a recommended storage volume of 360,000 L.

The Master Plan shows that the pressures and available fire flows within the existing system do not meet the optimal service levels in the areas surrounding the site. To improve the overall function of the water system, the pressure boosting station should be constructed further upstream in the system, potentially along Community Centre Road. This would expand the proposed pressure zone, improving pressures in the surrounding area. Looping the system should also be considered to reduce the physical constraints within the distribution system. Coordination with the Township will be required to determine the ideal location and parameters of the pressure boosting station, pressure reducing valve(s), and watermain upgrades. The design of these upgrades should be considered in tandem with the necessary upgrades to the wells, water treatment plant, and treated water storage.

4.0 Summary

Engage was retained by Stalwood Homes to prepare a Functional Servicing Report in support of the Draft Plan Application for the proposed Burwash Road Subdivision. The development is proposed to include a mix of single family and semi-detached residential

units, as well as an internal road network, stormwater management blocks, and a shared servicing block.

Sanitary servicing for the development is proposed to be provided by a network of gravity sewer conveying flows to a modular wastewater treatment system designed by Miranda Water Technologies. A sanitary pumping station will also be required on the north end of the site to convey flows via forcemain to the modular wastewater treatment system.

Water servicing for the development is proposed to be provided by two connections to the existing Creighton Heights drinking water system. A network of internal watermain will be constructed with individual service connections to each unit. To provide adequate pressure for domestic demands, the preliminary design includes a pressure boosting station near the eastern site entrance and a pressure reducing valve at the west entrance. A dry hydrant and water storage tanks are proposed to be constructed on the shared servicing block to provide adequate volumes for fire fighting.

The Township of Hamilton Water Supply Master Plan shows that the Creighton Heights drinking water system is currently operating near its total capacity. The wells, water treatment plant, and treated water storage will require upgrades to adequately service the proposed development.

Physical constraints within the water distribution system are limiting the available fire flow at the site. While a dry hydrant and storage tanks are currently proposed, upgrades to the distribution system are recommended to improve the available flow and pressure at the site. Hydraulic modelling will be required to assess the impact of the site on the Township's water system and to determine the necessary upgrades to the system.

Prepared by:

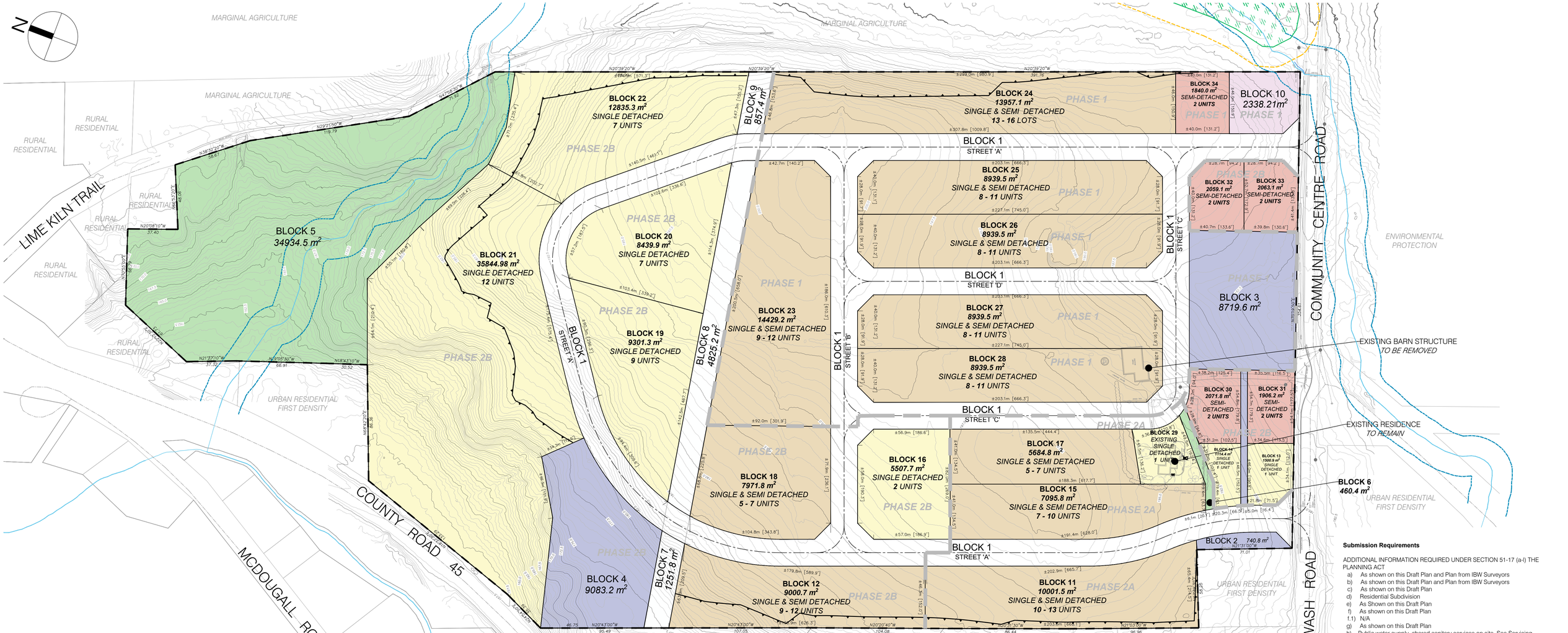


Alex Lautenbach, P. Eng.
Design Engineer

Reviewed by:

Jason Armstrong
Municipal Group Manager

Figure 2: Draft Plan



Submission Requirements

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51-17 (a-l) THE PLANNING ACT

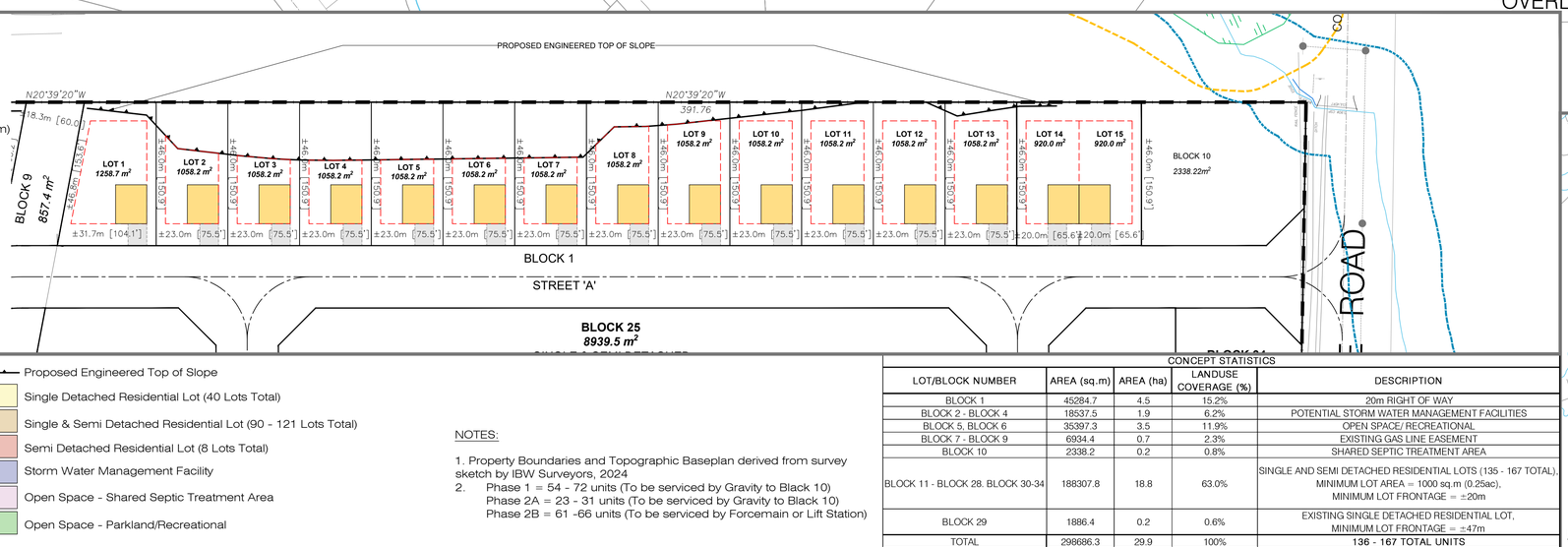
a) As shown on this Draft Plan and Plan from IBW Surveyors
b) As shown on this Draft Plan and Plan from IBW Surveyors
c) As shown on this Draft Plan
d) Residential Subdivision
e) As Shown on this Draft Plan
f) As shown on this Draft Plan
f.1) N/A
g) As shown on this Draft Plan
h) Public water supply, shared sanitary services on site. See Servicing report by Engage Engineering for full details.
i) N/A (Future Studies)
j) As shown on this Draft Plan. Proposed grading and elevations details shown on Grading Plans by Engage Engineering
k) Refer to Refer to Servicing Report from Engage Engineering for full details.
l) As shown on this Draft Plan

CONCEPT LOT LAYOUT (TYP.)

Building Envelope
Front Yard Setback: 7.0 m
Side Yard Setback: 3.0 m
Rear Yard Setback: 6.0 m

Potential 2 storey Residence (125 sq.m)
Gross Floor Area: 250 m²

Potential Driveway



LEGEND

- Subject Lands
- Existing Parcel Line
- Proposed Lot/Block Line
- Proposed ROW Centreline
- Existing Watercourse
- Watercourse Buffer (15m)
- Existing Unevaluated Wetland
- Wetland Buffer (15m)
- Proposed Engineered Top of Slope
- Single Detached Residential Lot (40 Lots Total)
- Single & Semi Detached Residential Lot (90 - 121 Lots Total)
- Semi Detached Residential Lot (8 Lots Total)
- Storm Water Management Facility
- Open Space - Shared Septic Treatment Area
- Open Space - Parkland/Recreational

NOTES:

1. Property Boundaries and Topographic Baseplan derived from survey sketch by IBW Surveyors, 2024
2. Phase 1 = 54 - 72 units (To be serviced by Gravity to Black 10)
   Phase 2A = 23 - 31 units (To be serviced by Gravity to Black 10)
   Phase 2B = 61 -66 units (To be serviced by Forcemain or Lift Station)

CONCEPT STATISTICS

LOT/BLOCK NUMBER	AREA (sq.m)	AREA (ha)	LANDUSE COVERAGE (%)	DESCRIPTION
BLOCK 1	45284.7	4.5	15.2%	20m RIGHT OF WAY
BLOCK 2 - BLOCK 4	18537.5	1.9	6.2%	POTENTIAL STORM WATER MANAGEMENT FACILITIES
BLOCK 5, BLOCK 6	35397.3	3.5	11.9%	OPEN SPACE/ RECREATIONAL
BLOCK 7 - BLOCK 9	6934.4	0.7	2.3%	EXISTING GAS LINE EASEMENT
BLOCK 10	2338.2	0.2	0.8%	SHARED SEPTIC TREATMENT AREA
BLOCK 11 - BLOCK 28. BLOCK 30-34	188307.8	18.8	63.0%	SINGLE AND SEMI DETACHED RESIDENTIAL LOTS (135 - 167 TOTAL), MINIMUM LOT AREA = 1000 sq.m (0.25ac), MINIMUM LOT FRONTAGE = ±20m
BLOCK 29	1886.4	0.2	0.6%	EXISTING SINGLE DETACHED RESIDENTIAL LOT, MINIMUM LOT FRONTAGE = ±47m
TOTAL	298686.3	29.9	100%	136 - 167 TOTAL UNITS



CONCEPT DRAFT PLAN
CDP1

BURWASH SUBDIVISION
Stalwood Homes - c/o Anthony Dew

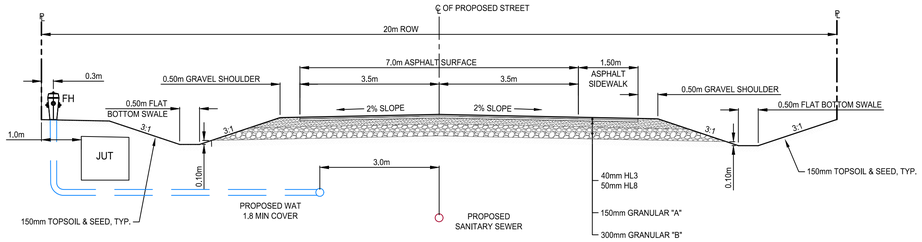
PART OF 9380 BURWASH ROAD

TOWNSHIP OF HAMILTON
COUNTY OF NORTHUMBERLAND

Project Number:	24-2700
Drawn By:	MC/PP
Horiz. Scale:	1:1300
Revision Date:	July 24, 2026

416 Chambers Street
Peterborough, ON K9H 3V1
T 705.876.8340 | F 705.742.8343

Figure 3: Preliminary Servicing Plan



TYPICAL ROAD SECTION

SCALE: 1:75

SURVEY

TOPOGRAPHIC SURVEY PROVIDED BY IBW SURVEYORS.
COORDINATE VALUES AND DIGITAL FILE ARE IN GRID SYSTEM, UTM ZONE 17, NAD83.
COMBINED SCALE FACTOR = 1.000223

ELEVATIONS

ELEVATIONS ARE GEODETIC AND REFERRED TO THE CANADIAN GEODETIC VERTICAL DATUM (COVD28) BY DIRECT MEASUREMENT TO A REAL TIME NETWORK SUPPLEMENTED WITH LIDAR DATA DERIVED FROM THE MNRF MAPPING UNIT FROM 2016-2018.

NOTES:

PR.	EX.	FEATURE
		EDGE OF PAVEMENT
		BACK OF CURB
		EDGE OF GRAVEL
		CENTRELINE OF ROAD
		DITCH
		SIDEWALK
		FENCE LINE
		RETAINING WALL
		LIGHT DUTY ASPHALT
		HEAVY DUTY ASPHALT
		GRASS/LANDSCAPED
		CONC. DRIVEWAY RAMP
		ASPH. DRIVEWAY RAMP
		INTERLOCK BRICK
		GAS MAIN
		OVERHEAD BELL
		UNDERGROUND BELL
		OVERHEAD HYDRO
		UNDERGROUND HYDRO
		WATERMAIN
		STORM SEWER
		SANITARY SEWER
		SANITARY FORCEMAIN
CS WS SS		WATER SERVICE
		SANITARY SERVICE
		PROPERTY LINE
		RIGHT-OF-WAY
		WATER VALVE
		WELL
		SANITARY MANHOLE
		CATCH BASIN
		DOUBLE CATCH BASIN
		CATCH BASIN MANHOLE
		DOUBLE CB MANHOLE
		STORM MANHOLE
		FIRE HYDRANT
		IRON BAR
		LIGHT STANDARD
		MAILBOX
		ROAD SIGN
		CONIFEROUS TREE
		DECIDUOUS TREE
		OVERHEAD DOOR
		MAN DOOR
		ACCESSIBLE MAN DOOR

No.	REVISION	BY	DATE
1.	ISSUED FOR REVIEW	SD	2026-06-22

ENGAGE ENGINEERING
www.engageeng.ca ★ 171 King Street, Suite 120, Peterborough, ON ★ Phone: (705) 749-0407

BURWASH DEVELOPMENT

STALWOOD HOMES

PRELIMINARY SITE SERVICING PLAN

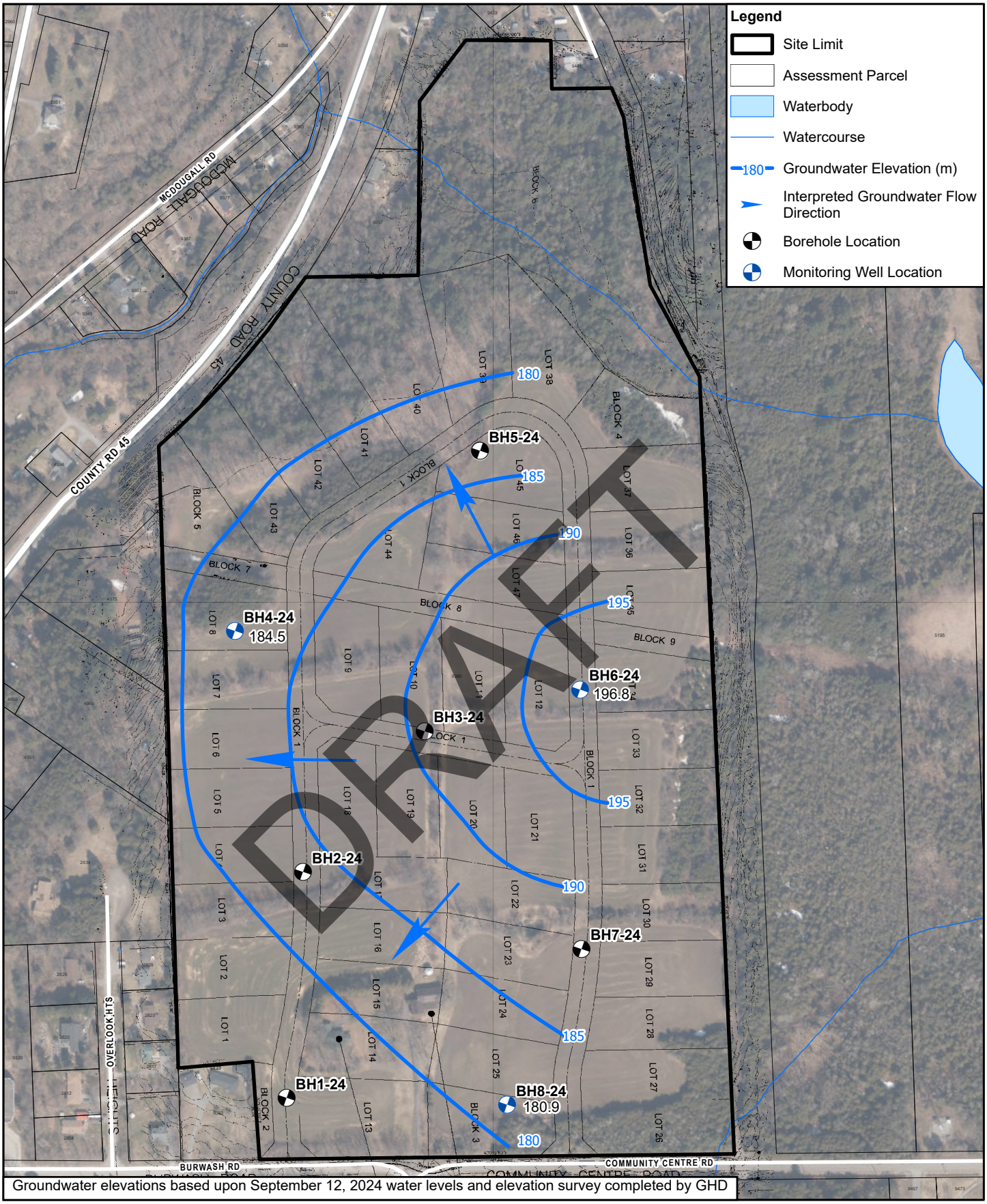
TOWNSHIP OF HAMILTON

DRAWN BY: S.DINGMAN	STAMP:
DESIGNED BY: S.DINGMAN	
APPROVED BY: J.ARMSTRONG	
DATE: 2024-06-10	
SCALE: 1:1250	

PROJECT NUMBER:	SHEET NAME:	SHEET:
23121	SS	2 of 2

23121 - SS
Date: Aug. 05, 2026 - 9:20 am Plotted by: sdingman File name: S:\01 - Projects\2300\23121 - 8380 Burwash Rd Ballinrod\02 Design\01 - Drawing\01 - Current Drawing\23121 - SS.dwg

Figure 4: Groundwater Elevations Figure



Groundwater elevations based upon September 12, 2024 water levels and elevation survey completed by GHD

1:4,000
0 40 80 120
Metres

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 17N

Stalwood Homes
9380 Burwash Road, Baltimore, ON
Municipality of Northumberland

Hydrogeological Assessment and Preliminary Geotechnical Services Report

Groundwater Elevations

Project No. 12639057
Revision No.
Date Nov 11, 2024

Figure 10

O:\GIS\PROJECTS\12639000s\12639057\GIS\Maps\Deliverables\202410_HydrogeologicalAssessmentAndPreliminaryGeotechnicalServicesReport\12639057_202410_GIS012_GWElevations.aprx
Print date: 11 Nov 2024 - 17:21

Attribution: Concept Plan: CP1, Prepared by Ecovue on June 14, 2024; Imagery accessed from the Ontario Imagery Web Map Service on Nov 11, 2024; Produced by GHD Limited under Licence with the Ontario Ministry of Natural Resources and Forestry© King's Printer

Appendix A: Miranda Wastewater Treatment System

MIRANDA water technologies	**MIRACELL MODULAR W.W.T.P.** **RBC SYSTEM DESIGN CALCULATION PROGRAM**	**Prepared By**	
		Emrah HANÇER	
		Date	7-31-2026
		Revision	

Project Name	:	FLC Group			
Design Parameters					
Population	:	484	Person		
Water Usage Per Capita	:	450	lt/PE.day		
BOD Load Per Capita	:	60	gr/PE.day		
COD Load Per Capita	:	120	gr/PE.day		
TSS Load Per Capita	:	60.0	gr/PE.day		
N Load Per Capita	:	13	gr/PE.day		
P Load Per Capita	:	1.8	gr/PE.day		
Flow	:	217.8	m³/day		
Flow Time (Effluent Time)	:	**18**	h		
Design Flow (Flow Per Hour)	:	12.10	m³/h		
Q14	:	15.56	m³/h		
Flow Peak Factor	:	**2**			
Peak Flow	:	24.20	m³/h		
Inlet Wastewater Parameters					
sBOD5/BOD5 Fraction (soluble fraction of BOD5)	:	0.5	–	*Literature: Typical domestic wastewater sBOD5/BOD5 = 40–65% (avg. 50%)* *0.40–0.50 is conservative if no field measurement is available.* *Used in Germain & denitrification carbon calculation.* *Field measurement: 0.45µm membrane filter BOD test* *Ref: Metcalf & Eddy 5th Ed. Table 3-17; WEF MOP-8 Ch.10*	
BOD5	:	133.33	mg/lt		29,040 gr/day
SBOD5	:	66.67	mg/lt	✓ sBOD/BOD = 50% (typical range 40–65%)	
COD	:	266.67	mg/lt		58,080 gr/day
TSS	:	133.33	mg/lt		29,040 gr/day
TKN	:	28.88888889	mg/lt		6,292 gr/day
T-P	:	4	mg/lt		871 gr/day
Oil & Grease	:	**50**	mg/lt		10,890 gr/day
Water Temperature	:	**14**	°C		
pH	:	**7**			
C:N:P Ratio	:	33.3:7.2:1			
BOD5/TKN Ratio	:	4.62			
COD/BOD5 Ratio	:	2.00			
cBOD5 (Carbonaceous BOD5 ≈ BOD5 − 4.57×NH4-N*)	:	40.92	mg/lt	ℹ cBOD5 (40.9 mg/L) < sBOD5 (66.7 mg/L) — use MIN; nBOD est. uses NH4-N≈70%×TKN	
BOD5 Load	:	29,040	gr/day	14520 g/day sBOD5 | 286 g/day cBOD5 | 286 g/day design BOD5	
TKN Load	:	6,292	gr/day		29 mg/lt
Design BOD5 for Germain [MIN(sBOD5, cBOD5*)]	:	41	mg/lt	⚠ cBOD5 controls (40.9 mg/L) — Germain As/Q may need review	
Effluent Water Parameters					
BOD5	:	15	mg/lt		89%
COD	:	50	mg/lt		81%
TSS	:	10	mg/lt		93%
N	:	20	mg/lt		31%
P	:	0.5	mg/lt		88%
Oil & Grease	:	50	mg/lt		0%
Water Temperature	:	**14**	°C		
pH	:	**7**			

Burwash Landing

Miracell Modular WWTP — MC 200 ULTRA

Design Capacity: 217.8 m3/day | 484 Persons | 2 Lines x 4 Modules (8 Modules Total)

Process Design Report

1. Introduction

This report presents the preliminary process design basis, calculations, and hydraulic flow configuration for the Miracell Modular Wastewater Treatment Plant (WWTP) at Burwash Landing. All calculations are derived from and consistent with the Miracell MC 200 ULTRA RBC System Design Calculation Program (Burwash_217_M3_D_MC_200_ULTRA_RBC_SYSTEM.xlsx, Rev. 0, 30.07.2026, prepared by Emrah Hancer).

The plant is designed for 217.8 m3/day (217,800 L/day), 484 persons, 2 Lines x 4 Modules (8 modules total) RBC configuration, at 14°C design temperature.

- Treat 217.8 m3/day from 484 persons (450 L/PE/day)
- Effluent targets: BOD5 ≤15, TN ≤20, TSS ≤10, TP ≤0.5 mg/L, E.Coli <200 CFU/100mL, pH 6.5-9.5
- Accommodate peak loading over 18-hour inflow period
- Operate reliably at 14°C design temperature
- Design basis: Miracell MC 200 ULTRA RBC system per design calculation program

2. Design Basis

2.1 Project and Flow Parameters

Parameter	Value	Unit
Project	Burwash Landing WWTP	–
Location	Burwash Landing, Ontario	–
Design Population	484	persons
Water Usage per Capita	450	L/PE/day
Average Daily Flow (ADF)	217.8	m3/day
Flow Time (Inflow Period)	18	h/day
Design Flow Q18h	12.100	m3/h (= ADF / 18 h)
Continuous Feed Rate Q24h	9.0750	m3/h (= ADF / 24 h)
Flow Peak Factor	2.0	–
Peak Flow Qpeak	24.200	m3/h
Design Temperature	14	°C
Design pH	7.0	–

2.2 Influent Characteristics (Per-Capita Load Basis)

Parameter	Per-Capita (g/PE/day)	Design Conc. (mg/L)	Daily Load (g/day)
BOD5	60	133.33	29,040
Soluble BOD5 (sBOD5)	30	66.67	14,520
cBOD5 (carbonaceous, design basis)	—	40.92	—
COD	120	266.67	58,080
TSS	60	133.33	29,040
TKN	13	28.89	6,292
Total Phosphorus (TP)	1.8	4.00	871.2
Oil & Grease	—	50	10,890
BOD5/TKN Ratio	—	4.62	–
COD/BOD5 Ratio	—	2.00	–

sBOD5/BOD5 fraction = 50% (literature value for domestic wastewater, Metcalf & Eddy 5th Ed.). cBOD5 = 40.92 mg/L controls Germain model (cBOD5 < sBOD5). Field measurement recommended. Ref: sBOD5/BOD5 typical

range 40-65%.

2.3 Effluent Quality Targets

Parameter	Target Value	Unit	Standard / Basis
BOD5	≤ 15	mg/L	Ontario secondary treatment
COD	≤ 50	mg/L	Site-specific requirement
TSS	≤ 10	mg/L	Ontario secondary treatment
Total Nitrogen (TN)	≤ 20	mg/L	Nutrient-sensitive receiving water — confirm permit
Total Phosphorus (TP)	≤ 0.5	mg/L	Enhanced TP removal — alum coagulation required
Oil & Grease	≤ 50	mg/L	Provincial standard
E.Coli	< 200	CFU/100mL	Ontario ECA standard — UF + UV dual barrier
pH	6.5 – 9.5	–	Discharge requirement (Miranda guarantee)
Water Temperature	14	°C	Design basis

3. Process Description

No	Unit Process	Tag	Function
1	Pre-Settling Tank	T-100	25% BOD removal, 10% TKN removal by settling
2	Denitrification Tank (2-stage)	T-101	Pre-anoxic denitrification — raw WW carbon; Cdenit=0.06; 75% recirculation
3	Equalization Tank	T-102	Hydraulic buffering for 18-hour peak inflow
—	Transfer Pumps (all)	P-101 to P-106	All transfer pumps with VFDs for flow control
—	Flow Meter	FT-101	Recirculation flow meter upstream of Denitrification Tank
4	RBC Miracell 2L × 4M	MRC-101 to 108	Biological BOD removal + nitrification at 14°C (MC 200 ULTRA)
5	Lamella Clarifier	T-103	Secondary clarification
6	Sand Filter	SF-101	Tertiary polishing — BOD, TSS, TP
7	Ultrafiltration	UF-101	Membrane polishing (required: RBC effluent TSS < 35 mg/L)
8	UV Disinfection	UV-101/102	Disinfection — E.Coli < 200 CFU/100mL
9	Sludge Storage Tank	SDT-101	Sludge thickening & storage

4. Pre-Settling Tank Design

4.1 Tank Dimensions

Parameter	Required (Design Program)	Provided (Civil)	Unit

Volume (2.5 h HRT basis)	22.69	**TBD**	m3
HRT at Q24h	2.5 h (target)	**TBD**	h
BOD5 Removal Efficiency	25	25	%
TKN Removal Efficiency (settling)	10	10	%

Note: BOD5 removal efficiency for this system is 25% (vs 33% for Curve Lake). This is appropriate for a dilute influent (BOD5 = 133 mg/L due to high per-capita water usage of 450 L/PE/day). Civil tank volume will be determined during detailed design. Monthly desludging recommended.

4.2 BOD and TKN Reduction Calculations

```
BOD5 Removal (25%) = 29,040 g/day x 0.25 = 7,260 g/day  (33.33 mg/L)
BOD5 Load to Denitrification = 29,040 - 7,260 = 21,780 g/day  => 100.0 mg/L

TKN Removal (settling, 10%) = 6,292 x 0.10 = 629.2 g/day  (2.89 mg/L)
TKN after Primary Settling = 6,292 - 629.2 = 5662.8 g/day  => 26.0 mg/L

TKN Assimilation by Biomass = 5% x TKN after Primary Settling
  = 0.05 x 5662.8 = 283.14 g/day  => 1.3 mg/L

TKN Load to RBC (Biodisk) = 5662.8 - 283.14 = 5379.66 g/day  => 24.7 mg/L
```

5. Denitrification Tank Design

5.1 Configuration

Parameter	Value	Unit
Configuration	2 independent stages per line	–
Stage 1	Mixing zone (passive)	–
Stage 2	Reaction zone (active mixer, MIX-101)	–
Total Effective Volume per Line	27.79	m3 (civil — per GA drawing)
Total Effective Volume (2 lines)	55.58	m3
Denitrification HRT	3.5	h (at Q + Q_recirc)
HRT (design-flow basis)	6.1	h (at ADF only)

5.2 Design Parameters

Parameter	Value	Unit
Recirculation Rate	75	%
Recirculation Flow	163.35	m3/day = 2 × 3.403 m3/h per line
Recirculation Lines	2	lines
Denitrification Efficiency	42.86	% (= R/(R+100) = 75/175)
Cdenit (Miracell Standard)	0.06	kg NO3-N / kg BOD5
BOD5 Consumed per NO3-N	5	g BOD5 / g NO3-N
N Denitrifiable from Raw WW	8.0	mg/L NO3-N (= 0.06 × 133.33)
Minimum Denitrification Potential (design prog.)	5379.66	g NO3-N/day = 24.7 mg/L

5.3 Denitrification Calculation

```
HRT (at Q + Q_recirc = 381.15 m3/day):
  = 55.58 / (381.15/24) = 3.5 h

HRT (design-flow basis only):
  = 55.58 / 217.8 x 24 = 6.1 h
```

```
NO3-N Recirculated & Denitrified = 2190.43 g NO3-N/day  (= 10.06 mg/L)
BOD5 Consumed in Denitrification = 2190.43 x 5 g BOD5/g NO3-N = 10952.14 g/day
BOD5 Removal Efficiency (Denitrif.) = 10952.14 / 21,780 = 50.3%
BOD5 Load to RBC (after denitrif.)  = 21,780 - 10952.14 = 10827.86 g/day  =>  49.71 mg/L
```

5.4 Nitrogen Balance Notes

- Cdenit = 0.06: 5 g BOD5 consumed per g NO3-N denitrified (Miracell standard)
- N denitrifiable from raw WW carbon: 0.06 × 133 mg/L = 8.0 mg/L NO3-N available
- N in biomass (5% of TKN after settling): 0.05 × 26.0 = 1.3 mg/L
- N not needing denitrification: NOrg(2) + NH4-N(0) + NO3-N target(10) + N-biomass(6.67) = 18.67 mg/L
- No external carbon (methanol) required. The system achieves effluent TN = 16 mg/L using raw wastewater carbon alone — well within the ≤20 mg/L target.
- Operator instruction: if TN monitoring during commissioning approaches 20 mg/L, contact Miranda before considering any chemical carbon addition.

6. Equalization (EQ) Tank Design

Parameter	Required (Design Program)	Provided (Civil)	Unit
EQ Volume	54.45	126 m3 (civil structure)	m3
HRT at Q24h	6.0	13.9 h	h
Aeration	Not required	Not required	–

```
Q18h (inflow rate over 18h period) = ADF / 18 = 217.8 / 18 = 12.100 m3/h
Q24h (continuous biodisk feed)     = ADF / 24 = 217.8 / 24 = 9.0750 m3/h
dQ (variation in hourly flow)       = 12.100 - 9.0750 = 3.0250 m3/h
V_EQ_required = dQ x 18h = 3.0250 x 18 = 54.45 m3

Provided EQ Tank (Civil Structure): 126 m3
HRT = 126 / 217.8 x 24 = 13.9 h
```

7. RBC (Miracell MC 200 ULTRA) System Design — 2 Lines x 4 Modules

7.1 System Configuration

Parameter	Value	Unit
Configuration	2 Lines x 4 Modules (4 modules per line)	–
Total Modules Selected	8	pcs
Discs per Module	66	pcs (1,800 mm diameter)
Total Discs	528	pcs
Disc Surface Area (at 14°C)	6.634	m2/disc
Disc Surface Area per Module	437.844	m2 (= 66 × 6.634)
Drum Surface Area per Module	28.162	m2
Total Surface Area per Module	466.006	m2 (Disc + Drum)
Total Provided Surface Area (8 modules)	3,728.048	m2
Module Volume (each)	3.19	m3
Total Module Volume (8 modules)	25.52	m3
Drum Rotation Speed	3.7	rpm

Drum Peripheral Speed	0.349	m/s

7.2 Required Surface Area (Design Program Methodology)

The Miracell Design Program applies OLR to BOD REMOVED (not inlet BOD), sizes TKN area to remove TKN to the effluent target, and verifies hydraulics by HRT and hydraulic loading rate.

```
BOD Area Required:
  BOD to remove = BOD to RBC - Effluent BOD target
  = 10827.86 - (15 mg/L x 217.8 m3/day) = 10827.86 - 3267 = 7560.86 g/day
  Selected OLR = 4.0 g BOD5 removed / m2 / day  (from design program)
  A_BOD = 7560.86 / 4.0 = 1890.2 m2

TKN Area Required (to reach TN target of 20 mg/L):
  TKN to nitrify = TKN to RBC - TN effluent target
  = 5379.66 - (20 mg/L x 217.8) = 5379.66 - 4356 = 1023.66 g/day
  Selected OLR = 3 g NH4-N / m2 / day
  A_TKN = 1023.66 / 3 = 341.22 m2

Total Required (process) = A_BOD + A_TKN = 1890.2 + 341.22 = 2686.39 m2
Required (hydraulic check) = Q_hyd / hydraulic loading = 3465 m2
Governing requirement = 3465 m2 (hydraulic controls)

Provided Surface Area (8 modules) = 3,728 m2  ->  Margin = 1.08x (hydraulic)
Process margin = 1.39x  ->  BOTH OK

Hydraulic Check (Miracell MC 200 ULTRA criterion: HRT >= 120 min):
  HRT = 25.52 m3 / 9.0750 m3/h x 60 = 168.7 min  >=  120 min  OK
```

Note on OLR = 6 g/m2/day: This is higher than the Curve Lake design (3.94 g/m2/day) because the hydraulic area requirement (3,465 m2) governs the module selection. The installed 3,728 m2 provides 1.08× the hydraulic area requirement with a process margin of 1.81×. RBC HRT of 169 min well exceeds the 120-minute MC 200 ULTRA criterion.

7.3 Germain Model BOD Removal Verification (As/Q = 2.445 d/m)

Performance verified by Germain model using design BOD5 = 40.92 mg/L (= cBOD5, controls because cBOD5 < sBOD5) as substrate parameter, As/Q = 2.445 d/m (calculated from installed area and flow):

Stage	BOD5 Residual (mg/L)	Notes
Inlet to RBC	40.92	Design BOD5 = cBOD5 = MIN(sBOD5=66.67, cBOD5=40.92) mg/L
After Stage 1	27.87	Stage 1 OLR: 15.58 g sBOD5/m2/day
After Stage 2	20.69	Cumulative removal: 49.5%
After Stage 3	16.25	Cumulative removal: 60.3%
After Stage 4	13.29	Cumulative removal: 67.5% => < 15 mg/L target OK

Note: Stage 4 residual of 13.29 mg/L is based on cBOD5 = 40.92 mg/L (carbonaceous BOD). Effluent BOD5 target of ≤15 mg/L is met. The sBOD5/BOD5 = 50% assumption (conservative for domestic wastewater) drives the lower design BOD5 and should be verified with field data.

8. Secondary Clarification (Lamella Clarifier)

Parameter	Value	Unit
Lines Provided	2	pcs
Flow per Line (ADF)	108.9	m3/day
Critical Settlement Velocity	8	m/day
Plate Angle	57	degrees
Plate Length (L)	54	cm
Space Between Plates (b)	3.17	cm
Reynolds Number (Re)	8.72	< 2,000 (Laminar OK)
Froude Number (Fr)	3.34 × 10-6	< 0.00001 (Stable OK)
Lamella Length	1.674	m (<= 2.0 m OK)
Plates per Line	37	pcs
Surface Overflow Rate (at ADF)	2.59	m/h (MECP limit 2.0-2.5 m/h — see note)

9. Filtration System (Sand Filter + UF)

9.1 Sand Filter

Parameter	Value	Unit
Filter Diameter	1,200	mm
Filter Area	1.131	m2
Design Flow Rate	9.0750	m3/h (at Q24h, EQ-protected)
Actual Filtration Velocity	8.02	m/h (design max: 12 m/h OK)
Filtration Cycle Time	7	min (range: 5-9 min OK)
Media	80% Quartz Sand + 20% Gravel	–
Backwater Percent	40	% (media fluidization freeboard for backwash)

9.2 Filtered Water and Treated Water Tanks

Tank	Required (Design Program)	Provided (Civil)	HRT (provided)
Filtered Water Tank (12h basis)	108.90 m3	**TBD**	12h target
Treated Water Tank (1.5h basis)	13.61 m3	**TBD**	1.5h target

Civil tank sizes for Filtered Water (108.9 m3) and Treated Water (13.6 m3) tanks to be determined during detailed design. These are minimum volumes; oversizing is recommended for operational flexibility, consistent with Curve Lake approach.

10. Sludge Management

```
BOD5 Reduction in Biodisk:
  = BOD5 to RBC - Effluent BOD5 target
  = 10827.86 - (15 x 217.8) = 10827.86 - 3267 = 7560.86 g/day

Total Sludge Dry Solids (all sources: BOD + Nitrification + Chemical P) = 8.466 kg/day
Sludge Dry Solid Percent = 3%
Total Sludge Volume (wet, all sources) = 0.2822 m3/day  (282.2 L/day)

Required Sludge Storage (30 days per design program) = 8.466 m3
```

Parameter	Required (30-day)	Provided (Civil)	Unit

Effective Volume	8.466	TBD	m3
30-day Accumulation	8.466	–	m3
Time to Fill (projected)	30 days	TBD	–

Note: Sludge storage design basis is 30 days (vs 60 days for Curve Lake). Total sludge includes BOD removal sludge, nitrification sludge, and chemical P sludge. Civil tank to be sized during detailed design. Sludge dewatering system recommended for on-site volume reduction.

11. Chemical Dosing

11.1 NaOCl — UF Membrane Chemical Enhanced Backwash (CEB)

NaOCl is provided for UF membrane backwash (CEB) purposes only — not primary disinfection.

Parameter	Value	Unit
NaOCl Dosage (CEB reference)	10	g/m3
Daily NaOCl Demand	2178	g/day (= 10 x 217.8)
NaOCl Flow Rate	0.5	L/h (from chlorine dosage design sheet)
Selected Dosing Pump	5	L/h
Storage Tank Volume	100	L (~1-day supply)

11.2 Additional Chemical Systems

- NaOH: Not required. Net effluent alkalinity = 59.5 mg/L as CaCO3 (sufficient for nitrification). NaOH dosing system retained as precautionary provision.
- HCl: Acid wash for UF membrane cleaning (stored separately from NaOCl)
- Alum (Al2(SO4)3): Required for TP ≤0.5 mg/L target (stricter than Ontario secondary standard). Biological treatment insufficient. Chemical coagulation upstream of lamella clarifier required. Estimated dose: Alum ~6.32 kg/day (Al:P = 1.5:1 molar) OR FeCl3 ~5.99 kg/day (Fe:P = 1.5:1 molar). Chemical selection to be confirmed during detailed design.
- Methanol (external carbon): Not required. System achieves TN = 16 mg/L without external carbon. No methanol storage or dosing provision required.

11.3 UV Disinfection and E.Coli Elimination

Dual-barrier approach: UF (physical, 0.04 µm pore) + UV-C (254 nm, Ecoline ECO 100 or equivalent). E.Coli target: <200 CFU/100mL. UV system to be sized for Q24h design flow with adequate dose at peak flow. Detailed UV sizing TBD during equipment specification.

12. HRT Summary — Required vs. Provided

All civil tank sizes are TBD in this preliminary design. Values below show design program minimum requirements. Final civil sizing to be confirmed during detailed design.

Unit Process	Required V	Provided V (Preliminary)	HRT / Performance	Notes
Pre-Settling Tank	22.69 m3	Civil — per GA drawing	2.5h min @ Q24h	MECP ≥0.75h @ PHF ✓

Denitrification Tank	55.58 m3	Civil — per GA drawing	3.5h (at Q+Q_recirc) / 6.1h (ADF)	MECP ≥2.0h @ PDF ✓
Equalization Tank	54.45 m3	126 m3 (civil)	13.3h @ Q24h	2.3× minimum required
RBC System (2L × 4M)	3,465 m2 (hyd.)	3,728 m2	168.7 min (≥120 min OK)	1.08× hyd. margin
Sludge Storage Tank	8.47 m3 (30d)	Civil — per GA drawing	30-day design basis	All sludge sources
Treated Water Tank	13.61 m3 (1.5h)	Civil — per GA drawing	1.5h target	Min. 1.5h × Q24h
Filtered Water Tank	108.90 m3 (12h)	Civil — per GA drawing	12h target	12h × Q24h

13. Key Design Considerations and Preliminary Flags

13.1 Design Assumptions Consistent with Curve Lake

- Cdenit = 0.06 kg NO3-N/kg BOD5 (Miracell standard)
- Pre-anoxic denitrification with raw WW carbon source
- UF membrane required (RBC effluent TSS may exceed 35 mg/L)
- UV + UF dual barrier for E.Coli < 200 CFU/100mL
- All transfer pumps with VFD control
- FT-101 electromagnetic flow meter at denitrification tank inlet
- ORP and DO monitoring in Denitrification Tank Stage 2
- pH and Temperature monitoring in EQ Tank

14. Conclusion (Preliminary)

This preliminary process design report presents a feasibility-level design for the Miracell MC 200 ULTRA Modular WWTP at Burwash Landing: 217.8 m3/day, 484 PE, 14°C, 2 Lines x 4 Modules (8 modules total). All calculations are based on Burwash_217_M3_D_MC_200_ULTRA_RBC_SYSTEM.xlsx Rev. 0.

Objective	Design Provision	Status
BOD5 ≤ 15 mg/L	Germain stage 4: 13.29 mg/L; As/Q=2.445 d/m	OK
TN ≤ 20 mg/L	Pre-anoxic denitrif. + nitrification; methanol ~5.98 kg/day if needed	OK / verify
TSS ≤ 10 mg/L	Lamella + sand filter + UF	OK
TP ≤ 0.5 mg/L	**Alum coagulation required; system TBD**	Confirm
E.Coli < 200 CFU/100mL	UF + UV dual barrier	OK
pH 6.5-9.5	Miranda confirms achievable	OK
Peak flow buffering	EQ required: 54.5 m3 — recommend ≥110 m3	**⚠ Size TBD**
RBC surface area	3,728 m2 provided vs 3465 m2 required (hyd.) — 1.08x	OK (tight)
RBC HRT	169 min >= 120 min MC 200 ULTRA criterion	OK
Denitrif. tank volume	55.58 m3 total (2 × 27.79 m3 per line)	MECP ✓ (2.30h @ PDF)

Appendix A — Design Parameter Summary

Parameter	Value	Parameter	Value
ADF	217.8 m3/day	Q24h	9.0750 m3/h
Population	484 PE	Q18h	12.100 m3/h
Qpeak	24.200 m3/h	Water Temperature	14°C
BOD5 / sBOD5 / cBOD5	133.3 / 66.7 / 40.92 mg/L	TKN (influent)	28.89 mg/L
BOD5 Load	29,040 g/day	TKN Load	6,292 g/day
EQ Tank Required	54.45 m3	Recommended civil EQ	≥ 110 m3
BOD to RBC	10827.86 g/day = 49.71 mg/L	TKN to RBC	5379.66 g/day = 24.7 mg/L
Cdenit	0.06 kg NO3-N/kg BOD5	NO3-N Denitrified	2190.43 g/day
BOD Consumed (Denitrif.)	10952.14 g/day	Denitrif. Efficiency	50.3%
RBC Area (process req.)	2686.39 m2	RBC Area (hyd. req.)	3465 m2
RBC Area Provided	3,728 m2	Margin (hyd.)	1.08x
RBC HRT	169 min (≥120 min OK)	OLR (BOD removed)	6.0 g/m2/day
Stage 1 OLR	15.58 g sBOD5/m2/day	As/Q	2.445 d/m
BOD Germain Stage 4	13.29 mg/L	Nitrif. OLR	3 g NH4-N/m2/day
Sludge Volume	0.2822 m3/day	Sludge Dry Solids	8.466 kg/day
Denitrif. Tank Volume	55.58 m3 (both lines)	Denitrif. HRT	3.5h (Q+Q_recirc)

End of Report — Miranda Water Technologies, July 31, 2026 | Burwash Landing | Revision R0 (Preliminary) | All calculations per Burwash_217_M3_D_MC_200_ULTRA_RBC_SYSTEM.xlsx Rev.0

MIRANDA
water technologies

Burwash Landing

Miracell Modular WWTP — MC 200 ULTRA

Process Design Report — Addendum A

MECP Design Guidelines for Sewage Works — Peak Flow Compliance Verification

⚠ **PRELIMINARY: This MECP compliance addendum is based on preliminary design parameters. Civil tank sizes are TBD. Final compliance verification to be updated following detailed civil design.**

1. Purpose of This Addendum

This addendum verifies compliance with the Ministry of the Environment, Conservation and Parks (MECP) Design Guidelines for Sewage Works (2008), Table 8-2, at applicable peak flow rates for the Burwash Landing Miracell Modular WWTP. All unit processes are evaluated at Peak Hourly Flow (PHF) or Peak Daily Flow (PDF) as applicable.

2. Design Flow Parameters

Flow Parameter	Value	Basis / Notes
Average Daily Flow (ADF)	217.8 m3/day	484 PE × 450 L/PE/day
Continuous Feed Rate (Q24h)	9.0750 m3/h	ADF ÷ 24 h
Design Inflow Rate (Q18h)	12.100 m3/h	ADF ÷ 18 h
Peak Hourly Flow (PHF)	24.200 m3/h	Q18h × Peak Factor 2.0
Peak Daily Flow (PDF)	580.80 m3/day	PHF × 24 h
Peak Flow Factor	2.67×	PHF ÷ Q24h

Key Design Philosophy: The EQ Tank is the primary hydraulic buffer. When adequately sized (recommended ≥110 m3, vs minimum required 54.5 m3), it attenuates peak inflows before reaching downstream processes. The RBC, clarifier, sand filter, UF, and UV operate at near-constant Q24h = 9.0750 m3/h under normal conditions. Civil EQ tank sizing is critical for this protection.

3. MECP Table 8-2 — Peak Flow Compliance Verification

3.1 Primary Settling Tank (T-100)

Parameter	At ADF	At PHF	MECP Min. (PHF)
Effective Volume	22.69 m3 (min)	Same	—
HRT	2.50 h	0.94 h	≥ 0.75 h
Compliance Margin	—	—	1.2× MECP minimum
Status	—	—	✓ COMPLIANT

At PHF (24.200 m3/h), the minimum pre-settling tank (22.69 m3) delivers 0.94h HRT — 1.2× the MECP 0.75h minimum. Civil oversizing will improve this margin further.

3.2 Denitrification Tank (T-101)

Parameter	At ADF	At PDF	MECP Min. (PDF)
Effective Volume	55.58 m3	55.58 m3	—
HRT	6.1 h	2.30 h	≥ 2.0 h
Compliance Margin	—	—	1.1× MECP minimum
Status	—	—	✓ COMPLIANT

Denitrification tank (56 m3) provides 2.30h HRT at PDF — 1.1× the MECP 2.0h minimum. The EQ tank upstream ensures near-constant flow in practice; PDF is a conservative upper bound.

3.3 Equalization Tank (T-102)

Parameter	Value	MECP Criterion	Compliance
Minimum Required Volume	54.45 m3	Flow equalization (18h period)	—
Recommended Civil Volume	≥ 110 m3	—	—
Volume Margin (at 110 m3)	2.0× minimum	—	—
Peak Absorption (at 110 m3)	4.5 h @ PHF	> inflow period (18h) — see note	⚠ Partial
Protection of Downstream	Depends on civil sizing	—	✓ At Q24h (EQ protected)

⚠ WARNING: The minimum EQ volume (54.5 m3) provides only 2.2h of peak absorption — insufficient to fully buffer an 18-hour peak event. A civil tank of ≥110 m3 is recommended; this provides 4.5h of peak absorption and significantly protects downstream units. For complete 18-hour protection, a tank of 436 m3 would be needed, which is impractical. Operational protocols (VFD-controlled discharge, DO/pH monitoring) will be required to manage extended peak events.

3.4 RBC System — Miracell MC 200 ULTRA (MRC-101 to MRC-108)

Parameter	At Q24h (Design)	At PHF (hypothetical)	MECP Criterion
Surface Area	3,728 m2	3,728 m2	—
Hydraulic Loading	0.0584 m3/m2/day	0.1558 m3/m2/day	≤ Manufacturer spec.
HRT	169 min	63 min (hypothetical)	≥ 120 min (MC 200 ULTRA)
Design Basis	Q24h — EQ protected	EQ Tank prevents PHF reaching RBC	✓ COMPLIANT

RBC system is protected from peak flows by the upstream EQ Tank. Under normal operation, it receives steady feed at Q24h = 9.0750 m3/h, delivering HRT = 169 min — well above the MC 200 ULTRA 120-minute criterion.

3.5 Lamella Clarifier (T-103)

Parameter	At ADF / Q24h	At PHF	MECP Limit (PHF)
Flow per Line	9.0750 m3/h	12.100 m3/h	—
Surface Overflow Rate (SOR)	2.59 m/h	3.45 m/h	≤ 2.0-2.5 m/h
Reynolds Number	8.72	< 2,000 laminar	< 2,000 ✓
Plates per Line	37	37	—
Status	⚠ 2.59 m/h (near MECP limit)	⚠ 3.45 m/h (exceeds if unprotected)	EQ protection required

⚠ NOTE: SOR at ADF (2.59 m/h) is at the upper boundary of the MECP 2.0-2.5 m/h guideline. At PHF without EQ protection, SOR would be 3.45 m/h (exceeds limit). Under EQ-protected conditions, the clarifier operates at Q24h where SOR = 2.59 m/h. The 37-plate configuration significantly enhances effective settling area. Detailed clarifier design review required for final compliance confirmation.

3.6 Sand Filter (SF-101)

Parameter	At ADF / Q24h	At PHF	MECP / Design Max.
Filter Area	1.131 m2	1.131 m2	—
Filtration Velocity	8.02 m/h	21.40 m/h	≤ 12 m/h
% of Maximum at PHF	—	178%	≤ 100%
Status	67% of max (Q24h protected)	EQ tank prevents PHF reaching SF	✓ COMPLIANT (EQ protected)

At Q24h (EQ-protected), filtration velocity = 8.02 m/h (67% of 12 m/h max). At PHF without EQ, velocity would be

21.40 m/h — exceeding the design maximum. EQ Tank protection is therefore critical for sand filter compliance.

3.7 UV Disinfection

UV system to be sized during equipment specification. Preliminary compliance: At Q24h design flow, UV dose expected to be well above 40 mJ/cm2 MECP minimum. Final verification with selected UV equipment datasheet.

4. Summary — MECP Table 8-2 Compliance Matrix

Unit Process	MECP Criterion	Design Value	MECP Minimum	Status
Primary Settling Tank	PHF	HRT = 0.94 h	≥ 0.75 h @ PHF	✓ 1.2× minimum
Denitrification Tank	PDF	HRT = 2.30 h	≥ 2.0 h @ PDF	✓ 1.1× minimum
Equalization Tank	Flow equal.	54.5 m3 req. / ≥110 m3 rec.	54.5 m3 min	**⚠ Civil sizing TBD**
RBC System	ADF (EQ prot.)	HRT = 169 min	≥ 120 min (MC 200)	✓ COMPLIANT
Lamella Clarifier	PHF (EQ prot.)	SOR = 2.59 m/h (ADF)	≤ 2.0-2.5 m/h	**⚠ Review required**
Sand Filter	PHF (EQ prot.)	8.02 m/h (ADF)	≤ 12 m/h	✓ COMPLIANT (EQ prot.)
UV Disinfection	PHF	**TBD (spec. required)**	≥ 40 mJ/cm2	**TBD**

5. Conclusion (Preliminary)

Preliminary MECP Table 8-2 compliance verification for the Burwash Landing Miracell Modular WWTP Plant. Key findings:

- PRIMARY SETTLING TANK: HRT of 0.94h at PHF exceeds 0.75h minimum by 1.2×.
- DENITRIFICATION TANK: HRT of 2.30h at PDF exceeds 2.0h minimum by 1.1×.
- EQUALIZATION TANK: Minimum required 54.5 m3. Civil oversizing to ≥110 m3 strongly recommended for adequate downstream protection.
- RBC SYSTEM: HRT = 169 min at Q24h (EQ protected) exceeds 120-minute MC 200 ULTRA criterion.
- LAMELLA CLARIFIER: SOR at ADF = 2.59 m/h, near MECP 2.0-2.5 m/h limit. Detailed review required.
- SAND FILTER: Compliant at Q24h (EQ protected). EQ Tank critical for peak flow protection.
- UV DISINFECTION: System to be specified. Compliance to be confirmed with equipment datasheet.

This preliminary compliance assessment should be updated following: (1) confirmation of civil tank sizes, especially EQ tank; (2) lamella clarifier detailed sizing; (3) UV equipment selection. Miranda confirms that MECP Table 8-2 compliance is achievable for this preliminary design with appropriate civil sizing and EQ tank oversizing.

Appendix B: Sanitary Flow Calculations

Sewage Flows (Phase 1)

Project Name:	Burwash Road Subdivision	**Designed By:**	AL
Project No:	23121	**Date:**	2026-07-27

Design Criteria			
Residential Sewage Flows:	450	L/p/day	A
No. of Units (Single Family):	74	units	B
No. of Persons/Unit (Single Family):	2.972	p/unit	C
Total Development Population	220	persons	D = B x C
Drainage Area:	10.20	ha	E
Inflow and Infiltration Rate	0.26	L/s/ha	F

Calculations

Harmon Peaking Factor for Residential Flows

$K_H = 1+(14 / 4+(D)^{1/2})$ (Max = 3.80)

$K_H = 3.80$

Residential Sewage Flows

$F_{RES} = A \times D$

$= 98968$ L/day

$= 1.15$ L/s

$= 98.97$ m³/day

Peak Residential Sewage Flows

$F_{RES_PEAK} = F_{RES} \times K_H$

$= 376077$ L/day

$= 4.35$ L/s

$= 376.08$ m³/day

Inflow and Infiltration Rate

$F_{I\&I} = E \times F$

$= 229133$ L/day

$= 2.65$ L/s

$= 229.13$ m³/day

Total Average Sewage Flows

$F_{TOT_AVG} = F_{RES} + F_{I\&I}$

$= 328100$ L/day

$= 3.80$ L/s

$=$ **328.10** **m³/day**

Total Peak Sewage Flows

$F_{TOT_PEAK} = F_{RES_PEAK} + F_{I\&I}$

$= 605210$ L/day

$= 7.00$ L/s

$=$ **605.21** **m³/day**

Sewage Flows (Full Development)

Project Name:	Burwash Road Subdivision	**Designed By:**	AL
Project No:	23121	**Date:**	2026-07-27

Design Criteria				
Residential Sewage Flows:		450	L/p/day	A
No. of Units (Single Family):		167	units	B
No. of Persons/Unit (Single Family):		2.972	p/unit	C
Total Development Population		496	persons	D = B x C
Drainage Area:		19.17	ha	E
Inflow and Infiltration Rate		0.26	L/s/ha	F
Calculations				
Harmon Peaking Factor for Residential Flows				
K_H	=	$1+(14 / 4+(D)^{1/2})$		(Max = 3.80)
K_H	=	3.80		
Residential Sewage Flows				
F_{RES}	=	A x D		
	=	223346	L/day	
	=	2.59	L/s	
	=	223.35	m³/day	
Peak Residential Sewage Flows				
F_{RES_PEAK}	=	F_{RES} x K_H		
	=	848714	L/day	
	=	9.82	L/s	
	=	848.71	m³/day	
Inflow and Infiltration Rate				
$F_{I\&I}$	=	E x F		
	=	430635	L/day	
	=	4.98	L/s	
	=	430.63	m³/day	
Total Average Sewage Flows				
F_{TOT_AVG}	=	F_{RES} + $F_{I\&I}$		
	=	653981	L/day	
	=	7.57	L/s	
	=	**653.98**	**m³/day**	
Total Peak Sewage Flows				
F_{TOT_PEAK}	=	F_{RES_PEAK} + $F_{I\&I}$		
	=	1279349	L/day	
	=	14.81	L/s	
	=	**1279.35**	**m³/day**	

Appendix C: Water Demand Calculations

Water Demand

Project Name:	Burwash Road Subdivision	**Designed By:**	AL
Project No:	23121	**Date:**	2026-07-27

Design Criteria			
Domestic Water Demand:	225	L/p/day	A
No. of Units (Single Family):	167		B
No. of Persons/Unit (Single Family):	2.972	p/unit	C
Total Population	496.3		D = B x C
Max. Day Peak Factor (MECP):	2.50		E
Peak Hour Peak Factor (MECP):	3.75		F
Fire Flow:	4,000	L/min	G

Average Day Demand				
	Q_{AVG}	=	A x D	
		=	111673	L/day
		=	77.6	L/min
		=	111.7	m³/day
Maximum Day Demand				
	Q_{MAX}	=	Q_{AVG} * E	
		=	279182	L/day
		=	193.9	L/min
		=	279.2	m³/day
Peak Hour Demand				
	Q_{PHD}	=	Q_{AVG} * F	
		=	418773	L/day
		=	**290.8**	**L/min**
		=	**418.8**	**m³/day**
Fire Fighting Demand (MDD + Fire Flow)				
	Q_{TD}	=	Q_{MDD} + G	
		=	6039182	L/day
		=	**4193.9**	**L/min**
		=	**6039.2**	**m³/day**