FUNCTIONAL SERVICING REPORT
BLACK BEAR RIDGE GP INC. – BLACK BEAR RIDGE SUBDIVISION

January 31, 2025

Belleville	Kingston	Oakville
1 – 71 Millennium Pkwy	208 – 4 Cataraqui St	214 – 231 Oak Park Blvd
Belleville, ON	Kingston, ON	Oakville, ON
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1 Background

Jewell Engineering (Jewell) has been engaged by Black Bear Ridge GP Inc., known as The Developer, to assist with the servicing design for the proposed development of the Blackbear Ridge lands to include residential and commercial uses. The area is within the hamlet of Corbyville, north of the City of Belleville. The development site plan can be found in Appendix A.

This servicing report has been prepared to support the Draft Plan application for Black Bear Ridge Subdivision. The location of the subdivision is shown below in Figure 1.

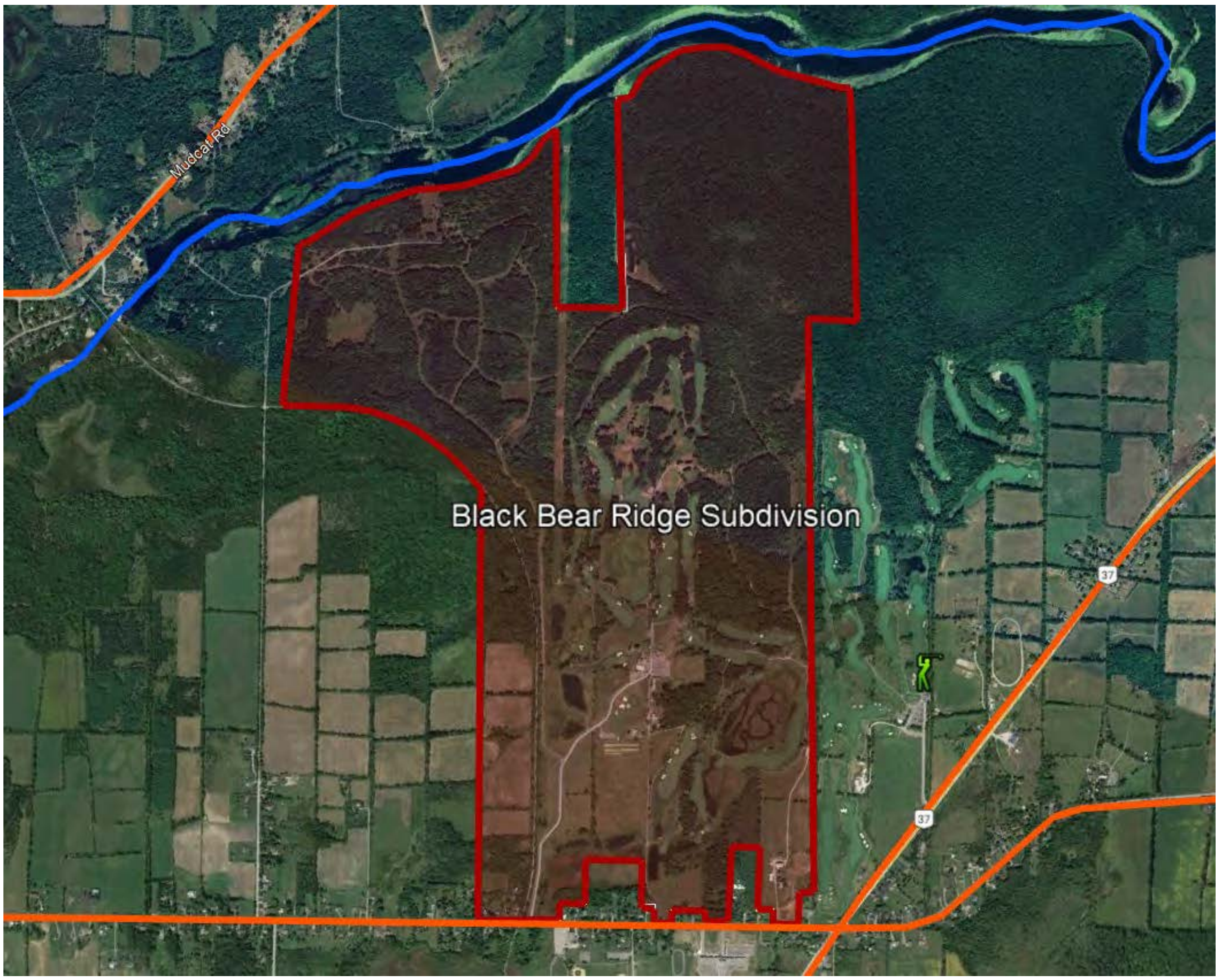


Figure 1: Subdivision Location (Adapted from Google Earth, 2025)

Water and sanitary sewer servicing have been considered in this report while stormwater management has been evaluated under a separate cover.

1.1 Site Description

The proposed subdivision area is approximately 370 hectares (ha) that fronts onto Harmony Road, however only 200 hectares are being considered for development. The surrounding land uses are low density residential to the south, agricultural to the west, and recreational to the east. A watercourse runs across the north border of the property.

The topography is smooth and gently sloping. The northern half gently slopes towards the watercourse and Homan Road to the west, while the southern portion slopes towards Harmony Road.

1.2 Proposed Development

The development lands will weave around the existing golf course and environmentally sensitive areas. Housing clusters are proposed with a mix of low and medium-high density units connected to each other with new municipal roads. Access to the subdivision will be gained from the existing Golf Course entrance on Harmony Road with the potential for a second entrance onto Homan Road. A third possible entrance would be gained with a connection to the 16-Lot condominium development situated at the east side along Harmony Road.

The municipal zoning order provides zoning approval for up to 3,049 dwelling units. The current concept plans illustrate less than 2,000 dwelling units and it is anticipated that the concept will change in future submissions as further environmental studies are performed and additional opportunities for intensification are found.

The southern portion of the concept plan is considered to be more stable and the servicing study applied the proposed densities for those areas. By contrast, the layout of the northwestern quadrant is less certain and the remaining dwelling units are assumed to be placed there. The full 3,049 units are considered.

Municipal water and sanitary sewer infrastructure will be provided for the subdivision servicing. Water and sanitary sewer are present on River Road over 3km south of the development limits. Both services must be extended from the intersection of Short Street and River Road (south of the hydro corridor crossing) to the development limits and throughout the development. Jewell investigated the feasibility of extending water and sewer (Jewell 2022) and found that an elevated water storage structure and booster would be required for water servicing. The elevated tank volume is 3,000,000 gallons which would be filled from 10PM to 7AM daily. The local (BBR) distribution would be supplied via a distribution system from the tank. The tank operating range provides the service pressures.

Jewell also found that a gravity solution for sanitary was not possible and instead a sewage lift station would be required.

Concurrent with the BBR development design, the City of Belleville has initiated a servicing study that will consider the water and sewer needs for the for the area north of the 401.

Municipal infrastructure has been designed to specifications set out by the Ministry of Environment, Conservation, and Parks (MECP) and City of Belleville Standards. The Development Site Plan is provided in Appendix A.

2 Water Distribution System

The municipal water system is terminated on River Road with a 300mm PVC DR18 main. According to the Jewell 2022 Feasibility study, a 400mm main must be extended into the subdivision and to the elevated water storage tank that would be located near the golf course clubhouse. The City had also requested that a provision be made for a future loop through Scott Street to Farnham Road.

Since this time, with the assistance of Dillon Consulting, a revised water concept was investigated that would see municipal water supplied to a larger area from Foxboro in the west to Hwy 37 in the east. The newly serviced land would be fed entirely from an elevated water storage tank with an operating range from 165.7 to 176.5 metres HGL. The tank would be situated on BBR lands approximately on topographical heights anywhere close to 130m elevation. The MDD was determined to be 139.6L/s and the transmission main from the point of interconnection to the tank would be increased to 450mm diameter.

Dillon found the Adam Street pumping station had insufficient head to fill the elevated water storage tank to the desired elevation and that an additional 30.2m of head would be required. The water booster station must be situated between the point of interconnection and the tower.

Jewell reviewed the internal pipe network and updated EPAnet model results from Dillon and proposes the water servicing scheme as discussed in the following sections.

2.1 Design Criteria

The watermain design criteria used are based on City of Belleville and MECP guidelines and are summarized below.

- Minimum Watermain Diameter Size: 200 mm
- Average Residential Daily Domestic Demand: 350 L/d*cap
- Population Factors:
  - Residential: 3.0 per unit
  - Future Development: 35 people per hectare
- Resort Hotel Demand: 500L/day per person
- Commercial Demand: 5L/m^2/day
- Maximum Day Plus Fire Flow Demand Minimum Pressure: 140 kPa (20 psi)
- Normal Operating Conditions Minimum Pressure: 280 kPa (40 psi)
- Normal Operating Conditions Maximum Pressure: 700 kPa (100 psi)
- Normal Operating Conditions Targeted Range: 350 kPa (50 psi) – 480 kPa (70 psi)

2.2 Watermain Design

Design calculations were completed by Dillon in the attached memo in Appendix D.

Table 2-1: Population Served

Unit Type	Number of Units	Assumed Population Per Unit [Cap]	Population Equivalent [Cap]
RESIDENTIAL	3049	3	9147
RESORT A/B	250	2	500
SUBTOTAL (BBR)			**9,647**
HARMONY ROAD	162	3	486
RESORT	1	250	250
SCHOOL	1	900	900
FOXBORO		450	450
JAMES PROPERTY	1,520	3	4,560
TOTAL			**16,293**

Table 2-2: Calculation of ADD (from Dillon)

Water User	Servicing Report [L/c/d]	MOECC [L/c/d]	Design [L/c/d]	Population [cap] or Area [m_2]	ADD [L/s]
RESIDENTIAL	350	270 to 450	350	9147 cap	37.1
RESORT A	500	225	500	250 cap	1.5
RESORT B	500	225	500	250 cap	1.5
HARMONY RD.	350	270 to 450	350	486 cap	1.97
FOXBORO	350	270 to 450	350	450 cap	1.82
JAMES PROPERTY	NA	270 to 450	350	4560 cap	18.48
SCHOOL (HARMONY RD.)	NA	70 to 140 L/Student/8 hrs	140 L/Student/8 hrs	900 cap	4.38
COMMUNITY CENTER (HARMONY RD.)	NA	NA	140 L/person/8 hrs	173 cap	0.84
COMMERCIAL	5 L/m_2/day	2.5 to 5 L/m_2/day	5 L/m_2/day	38,277 m2	2.22
Total					**69.81**

Peaking factors applied to the Average Day Demand (ADD) to find the Maximum Day Demand (MDD) and Peak Hour Demand (PHD) were:

Table 2-3: Peaking Factors (from Dillon)

Demand	Peaking Factor	Demand (L/s)
Average Day	1	69.81
Maximum Day	2	139.6
Peak Hour	3	209.41

The local distribution was reviewed to ensure pressures in the system remain within the desired operational band of 50psi – 70psi. Watermain sizes required are 200mm to 400mm and are shown in drawing WTR-1 in Appendix B.

The operating range of the elevated storage tank will be 165.7m to 176.5m in a tank with diameter 20.65m. This provides 3,680m^3 of storage that exceeds the requirement of 3,016m^3.

2.3 Fire Flow Requirements

Given that the BBR development will be fed entirely from the BBR elevated storage tank, fire flows and storage were calculated following Table 8-1 in the MOE guidelines. A conservative fire flow rate of 250 L/s for a 4-hour period for a population of 17,000 people was considered.

The reservoir must contain the sum of the 4-hr fire storage, 25% of the MDD and an emergency storage volume of 25% of the sum of the fire storage and 25% of the MDD.

Fire storage of 3,600m^3 is calculated. Twenty-five percent of the MDD is 3,016m^3 and the emergency storage volume is 1,654m^3. The total reservoir storage volume required is 8,270m^3 (see Dillon, Appendix D, p8). The operational demands for the tank is provided in the range of 146.9m to 176.5m through which 9,900m^3 of storage is made available.

The fire protection for the community is demonstrated to meet the ministry design guidelines.

2.4 Conclusions and Recommendations

The proposed water distribution scheme relies on an elevated water storage tank that will be filled with supply from the City of Belleville water system using a booster pump and a dedicated 450mm main. The tank will be positioned on the lands near the clubhouse on BBR lands.

Local supply will be provided through watermains ranging from 200mm to 400mm that will be fed directly from the tank.

The storage is sufficient to provide both domestic and fire suppression supply for the planned service area.

3 Sanitary Sewer System

The topography of the Black Bear Ridge Subdivision is challenging for a purely gravity system; thus, pump stations will be required at strategic positions to overcome the adverse conditions. There is a 300mm municipal sanitary sewer on River Road that has operated for decades and is under consideration for the point of interconnection at MH40. In previous work, Jewell determined the sanitary main would need to be increased in capacity to convey the expected flows from BBR.

The City of Belleville has commissioned a servicing study that is currently underway that will investigate the City's current and future infrastructure needs, in particular for the growth areas north of the 401 highway as well as the receiving trunkline and WWTP capacity. In this circumstance, the scope of the BBR Functional Servicing Report has been adjusted to consider the on-site sewage demands and collection and transmission. Since the eastern portion of the BBR site will drain to Harmony Road and then flow westerly to the central pumping station, the Harmony Public School, community centre and local residential area are also evaluated and included.

3.1 Design Criteria

The sanitary design criteria used are based on MECP guidelines as well as the City of Belleville Engineering Design Standards and are summarized below.

- Minimum Sanitary Sewer Diameter Size: 200 mm
- Average Daily Residential Domestic Design Flow: 350 L/d*cap
- Extraneous Flow Allowance: 0.28 L/s*ha
- Residential Population Factors:
    - Single Family: 3.0 persons/unit
    - Semi-detached: 3.0 persons/unit
    - Townhouse/Apartment: 2.5 persons/unit
    - Batchelor: 1.6 persons/unit
- Residential Peaking Factor Calculation: Harmon's Formula
    - Minimum: 2.00
- Commercial Peak Flow: 5.00 L/d*sq.m
- Friction Calculation: Manning's Formula
    - Manning's coefficient: 0.013
- Full Flow Velocity:
    - Minimum: 0.6 m/s
    - Maximum: 3.0 m/s

3.2 Sanitary Sewer Design

The proposed subdivision sanitary sewer network is shown in the general servicing plans in Appendix B.

All sanitary sewers in the Black Bear Ridge Subdivision have been designed to accommodate the entire development lands as shown in the Catchment Area Drawings in Appendix C. Sanitary sewer mains ranging in diameter from 200 mm to 450 mm with a minimum slope of 0.3% will service the development. The sanitary sewer design sheet is provided in Appendix C.

Given the large topographic variation across the BBR service area, gravity sewers serving the disparate development clusters will require local lift stations to convey sewage to the central sewage pumping station. Gravity sewers are planned to generally maintain sewer depths between 3m and 6m from centreline of road.

Jewell previously reviewed gravity sewer along River Road in Cannifton (Jewell 2022, p14) and found the existing pipes had insufficient capacity to receive the additional flows and must be upsized to 600mm. The 675mm pipe crossing under the 401 was found to be capable of conveying the expected flows, but that commercial and industrial development contemplated by the Official Plan and the Cannifton Servicing Study (GGG 2014) would increase usage to 108%. Since the serviced area now contemplated by the City will be larger, it is expected that the new study may find larger sewers are needed along River Road. This is currently under review by others and outside of the scope of the current analysis.

Five lift stations are required. The locations are shown on SAN-1 in Appendix B. The gravity mains leading to each lift station are sized using slopes ranging from 0.3% to 0.4%. In Table 3-1 the calculations for peak flow and the largest gravity mains to each pump station are summarised. Commercial flows are included within the peak flows and details can be found in Appendix C. The percent use of each gravity sewer is also listed.

Table 3-1: Pump Stations with Population Served, Peak Flow and Gravity Main Sizes

Pump Station	Population Served	Peak Flow (L/s)	Gravity Main Sizes (L/s)	% Full
A	10,130	153.7	375, 450	56.6, 65.1
B	177	4.6	200	22
C	1,755	31.1	300	58.8
D	6,496	94.5	450	60.5
E	225	4.5	200	21.5

Table 3-2: Sanitary Pump Station Data

Pump Station	Peak Flow (L/s)	Ground Elevation (m)	Lowest Pump Setting (m)	Discharge Elevation (m)	Lift (m)
A	153.7	109	102	101	7*
B	4.6	107	103	108	5
C	31.1	108	103	110	7
D	94.5	104	97	110	13
E	4.5	112	107	111	4

** Discharge is 3km in length*

Since the concept plans are fluid and were still changing at the time of completion of the FSR, generalized servicing areas were used to understand the depth, sizes, and slopes of the largest downstream sewers leading to each pump station. As the development progresses to detailed design stage it is expected that some fluctuation in pipe size will occur.

Pumping stations are subject to change as the concept plans are further refined and as detailed design stages commence. The first station to be installed will be Station A. This is the largest station and will receive all site flows as well as flows along Harmony Road to Hwy 37. At this stage, it is not known if gravity will be extended west to Foxboro. Other stations will be designed during detailed design stages.

Pump Station A will be fitted with three 75hp pumps and twin 200mm HDPE forcemains that will discharge to MH40. The route of the discharge was conceived to be through the snowmobile trail down to River Road. This concept is subject to change depending on the findings of the City of Belleville Servicing Study

Lands have been set aside for each of the five pump stations and are shown on the sanitary servicing plan.

3.3 Conclusions and Recommendations

Sanitary servicing needs have been reviewed and peak flows identified for the current BBR development concept while maintaining a full buildout of 3,049 dwelling units. External serviced areas were not part of the scope of the study, with the exception of the existing land uses along Harmony Road from the intersection of the BBR entrance to Hwy 37.

Gravity sanitary sewers have been conceptually designed throughout the Black Bear Ridge Subdivision. The system will drain to various pump stations and ultimately to Pump Station A located near Harmony Road and the existing entrance. This station will discharge pumped sewage to MH40 on River Road approximately 3km south of Harmony Road. All sanitary sewer within the Black Bear Ridge Subdivision have been designed with sufficient capacity for current development (as shown on the Catchment Area Drawings in Appendix C).

The sanitary concept is subject to the recommendations that are expected from the City's on-going servicing study.

4 Storm Sewers

Underground storm sewers are proposed for all roads. The storm sewer design follows the MECP 2008 guidelines. Most notably the standards followed include:

• Design Method	Rational Method
• Design Event	5-Year
• IDF Source	Environment Canada Station 6150689 – Belleville
• Manning's n	0.013
• Tc minimum	15 min
• Maximum Pipe Velocities	<6m/s
• Minimum Sewer Size	300mm

Pipes were sized using the standard Storm Sewer Design Sheet as required in the 2008 guidelines. Pipe sizes range from 300mm to 825mm in size. Pipes are conceptually designed at this stage and will be advanced to full design during detailed design stages.

Storm sewer design sheets are included in Appendix E as are the storm sewer catchment drawings.

5 Conclusions

The BBR lands will be developed with as many as 3,049 dwelling units. The development is conceptually conceived to proceed in stages that would start in the south and move northeast and finalize in the northwest.

The development will be municipally serviced with water and sanitary sewers. Streets will be fully urbanized and will be publicly owned. Stormwater management has been reviewed under a separate cover.

Water servicing requires a 450mm feeder main to be extended from the limits of the current City system 3km to an elevated water storage tank near the BBR clubhouse. The city system is believed to be able to provide adequate supply for the development, but will require a booster station to deliver a 30.2m head lift to fill the storage. Distribution piping ranges from 200mm to 400mm.

Sanitary servicing will include a combination of gravity sewers and lift stations. Five lift stations are currently conceived with all discharging to a central lift station situated at the northwest intersection of the BBR entrance and Harmony Road. Gravity sewers range in size from 200mm to 450mm with slopes generally planned at 0.3% to 0.4%.

Storm sewers are proposed with pipes generally ranging from 300mm to 825mm.

Upon the conclusion of the City servicing study that is underway at the time of writing of this report, the findings may highlight servicing limitations or challenges that require some further consideration of the proposed BBR servicing strategy. The detailed design stage for the lands subject to the current draft plan of subdivision application will be more fully informed with the findings and recommendations of the city study.

Prepared by:

Julie Humphries

Julie Humphries, C.E.T.
Jewell Engineering Inc.

Submitted by:

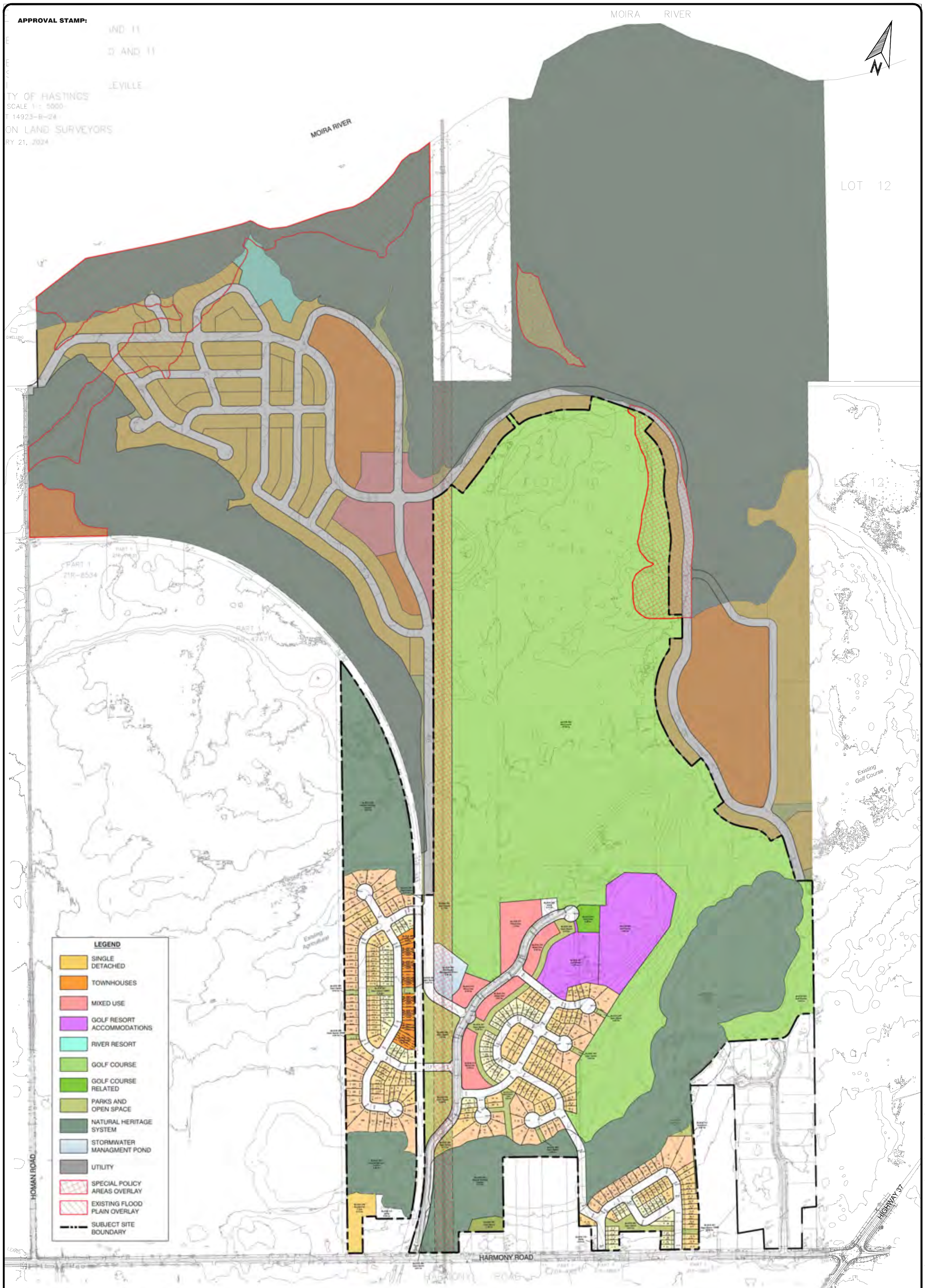
Bryon Keene, P.Eng.
Jewell Engineering Inc.

6 References

The information used to prepare this report is based on the following documents and information provided as noted below:

- Ontario Ministry of Environment. Conservation, and Parks
    - Design Guidelines for Sewage Works, 2007
    - Design Guidelines for Drinking-Water Systems, 2007
- Cannifton Servicing Study, Greer Galloway Group, 2014
- Black Bear Ridge Feasibility Study, Jewell Engineering, 2022
- City of Belleville Engineering Guidelines and Manual, draft 2023
- As-constructed drawings Cannifton Area, 1990

APPENDIX A:
DEVELOPMENT SITE PLAN



C:\Users\rpeltgnes\The Biglieri Group\TBG_Project - 2020-2024\2022\22826\2 - TBG Design\1 - TBG Working Drawings\1 - AutoCAD\2024-12\22826_Draft Plan_2024.12.19.dwg

TITLE:

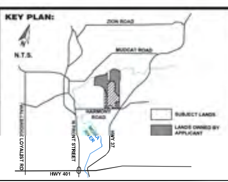
CONCEPTUAL MASTER PLAN

LEGAL DESCRIPTION:

Part of Lots 8, 9, 10 and 11, Concession 5
Part of Lots 7, 8, 9, 10 and 11, Concession 6
Township of Thurlow
Now in the City of Belleville
County of Hastings

BLACK BEAR RIDGE GP INC.

KEY PLAN:



BLACK BEAR RIDGE VILLAGE

SURVEYOR'S CERTIFICATE:

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATE AND CORRECTLY SHOWN IN ACCORDANCE WITH A PLAN OF SURVEY PREPARED BY

SURVEYOR
WATSON LAND SURVEYORS, OLS

DATE

OWNER'S CERTIFICATE:

I HEREBY AUTHORIZE THE **BIGLIERI GROUP LTD.** TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE CITY OF BELLEVILLE

NAME
COMPANY

JULY 30, 2024
DATE

REQUIRED INFORMATION:

AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT R.S.O. 1990

(a) SEE PLAN
(b) SEE PLAN
(c) SEE KEY MAP
(d) SEE SCHEDULE OF LAND USE
(e) SEE PLAN
(f) SEE PLAN
(f 1) N/A
(g) SEE PLAN
(h) MUNICIPAL SERVICES AVAILABLE
(i) SANDY SILT, SILTY SAND, AND CLAYEY SILT
(j) SEE PLAN
(k) MUNICIPAL SERVICES AVAILABLE
(l) SEE PLAN

NOTE:
DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

REVISIONS

No.	Description	Date	Int.
3			
2			
1	REVISED PER 1ST SUBMISSION COMMENTS	12/19/2024	MP

PROJECT No.: 22826
DATE: July 30, 2024
SCALE: 1:3000
DRAFTED BY: EC
CHECKED BY: MP
DRAWING No.: **CMP-01**

BIGLIERI GROUP

2472 Kingston Road, Toronto
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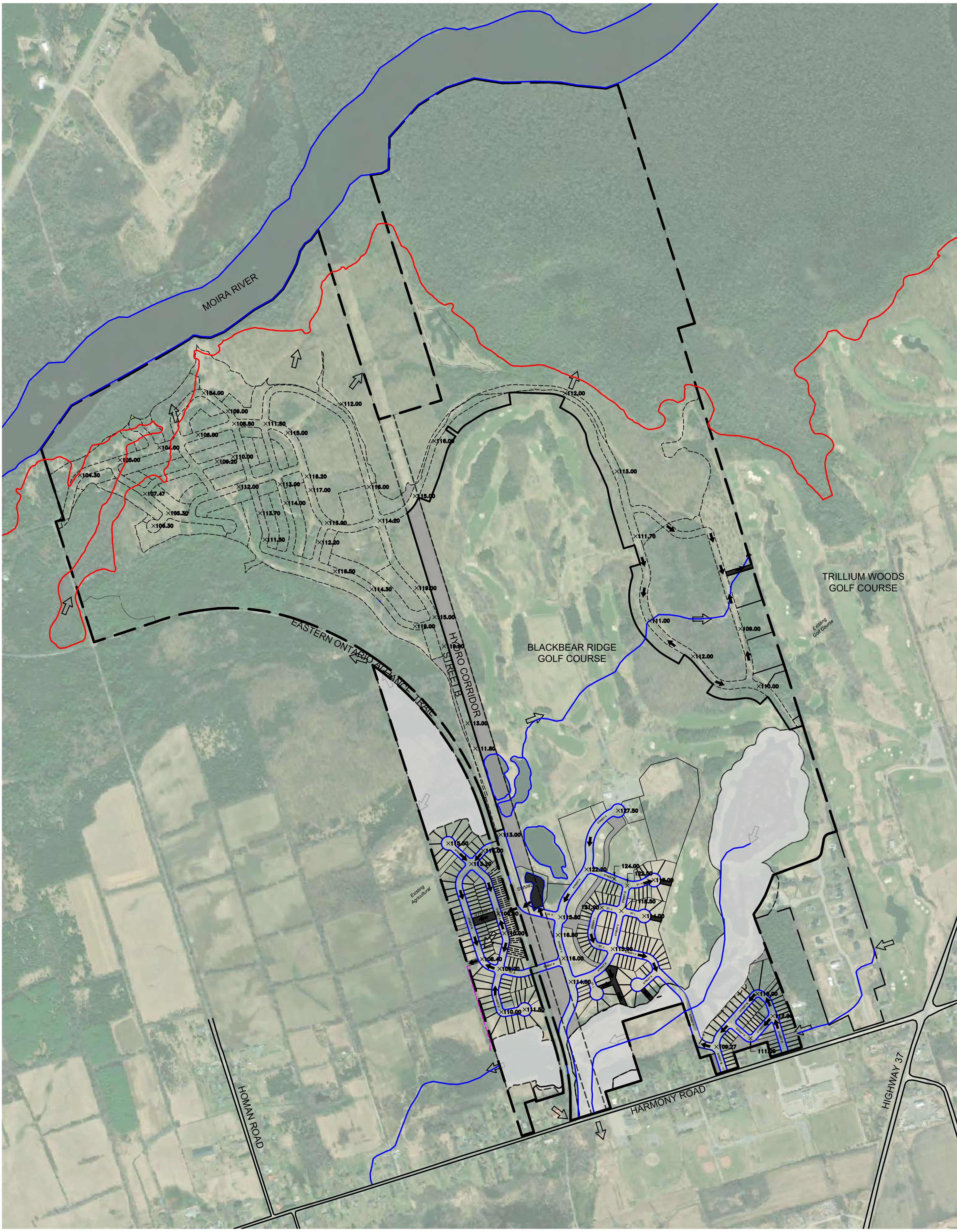
APPENDIX B:
PROPOSED SERVICING PLANS



JEWELL ENGINEERING

GENERAL NOTES:

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

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GEOMETRIC NOTE:

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.
- LIDAR CGVD 2013

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

- SWM FEATURE
- PROPOSED OVERLAND FLOW ROUTE
- EXISTING OVERLAND FLOW ROUTE
- ENVIRONMENTAL CONSTRAINTS
- DITCHING
- 100 YEAR FLOOD LIMIT
- STORM OUTLET ID

N
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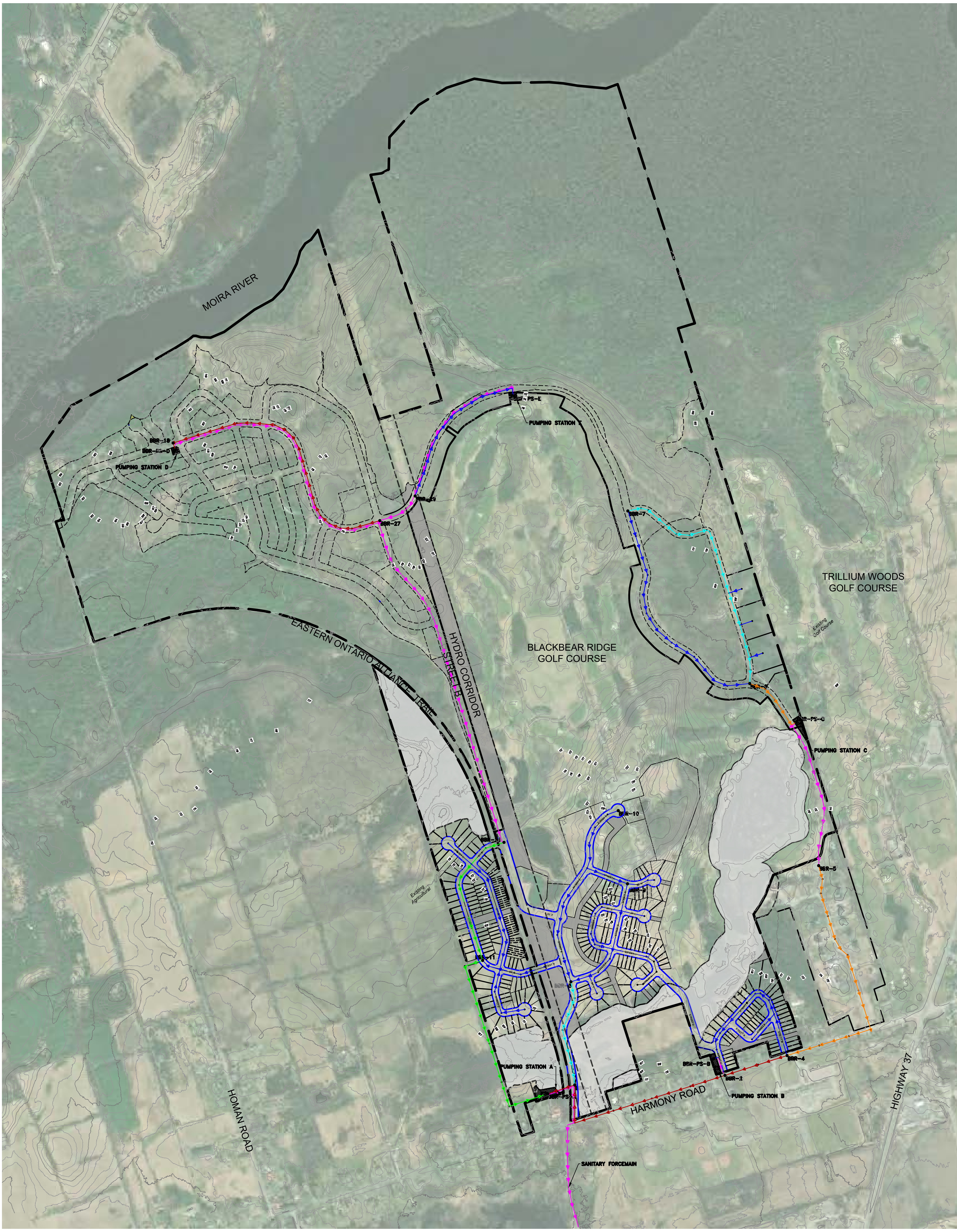
S

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PRELIMINARY GRADING PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
HORIZONTAL - 1:5000
VERTICAL -
DRAWING NO: PG-2



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- LIDAR CGVD 2013

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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

- 200mmø SANITARY MAIN
- 250mmø SANITARY MAIN
- 300mmø SANITARY MAIN
- 375mmø SANITARY MAIN
- 450mmø SANITARY TRUNK
- SANITARY FORCE MAIN
- PUMPING STATION
- ENVIRONMENTAL CONSTRAINTS
- 100 YEAR FLOOD LIMIT

N
W E
S

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PRELIMINARY SANITARY PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
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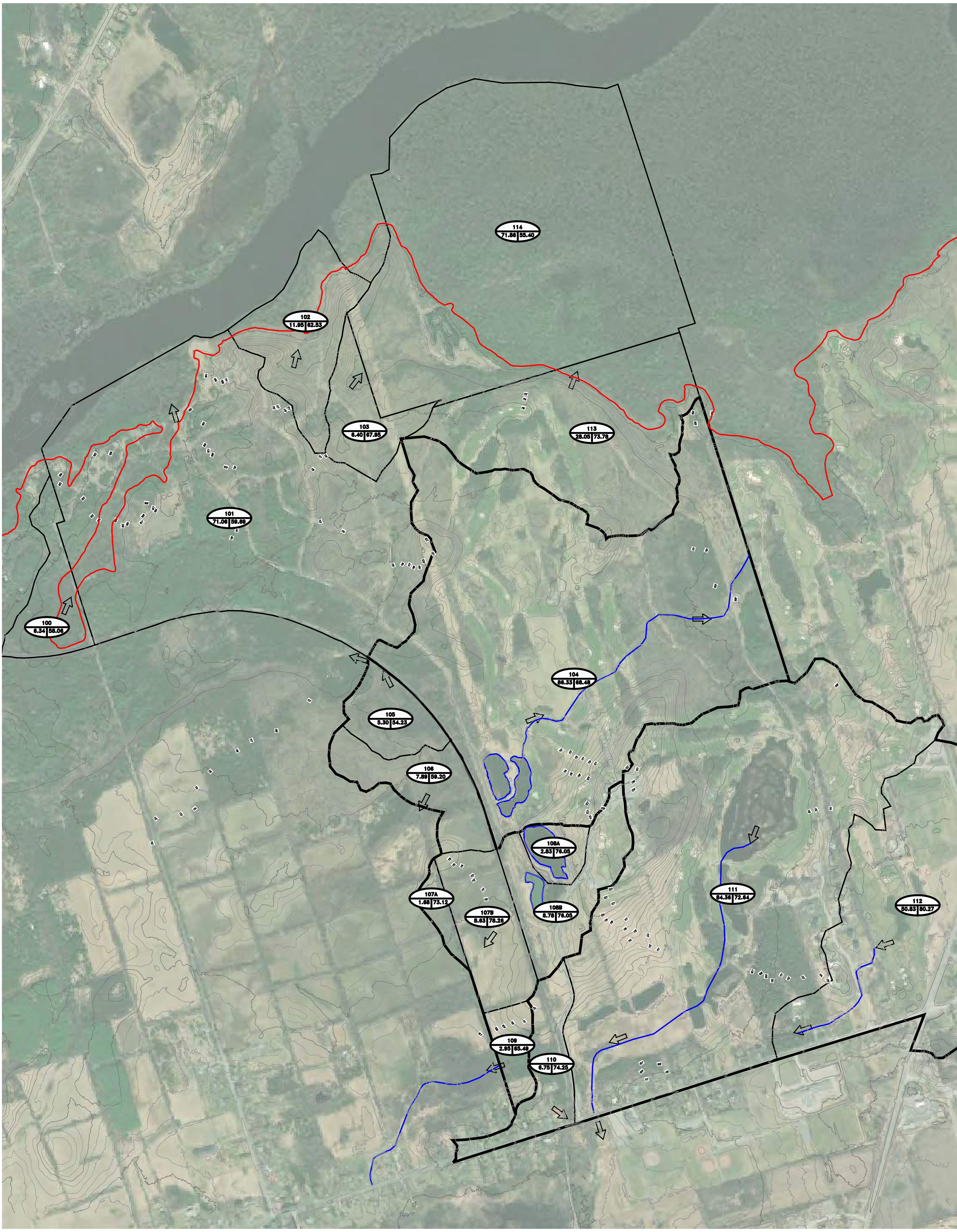

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- LIDAR CGVD 2013

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

EXISTING OVERLAND FLOW ROUTE

110
6.75 | 74.25

CATCHMENT ID NUMBER
AREA(ha) AND CURVE NUMBER

100 YEAR FLOOD LIMIT

N
W E
S

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

EXISTING STORM CATCHMENT AREA PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
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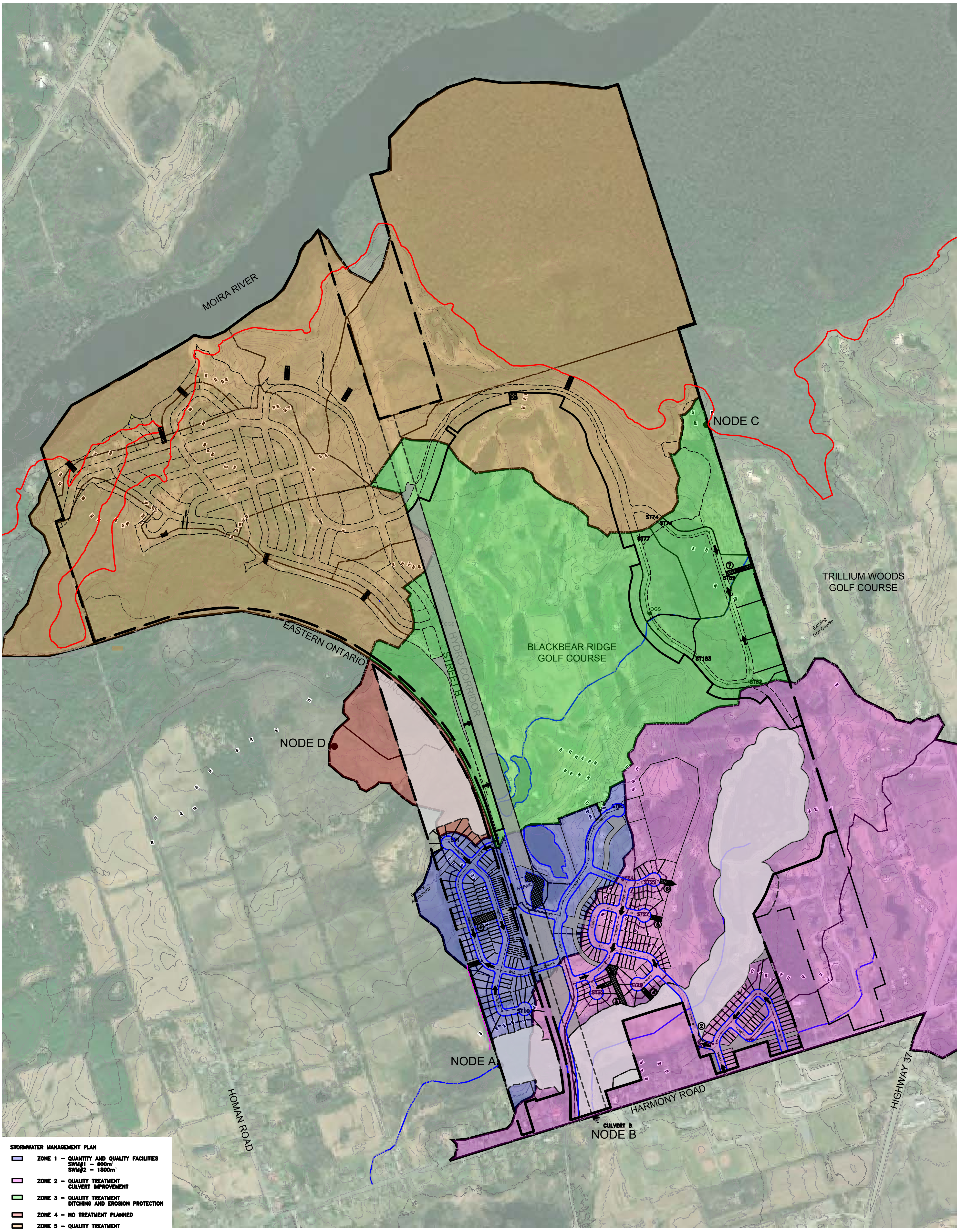
VERTICAL -
DRAWING NO: STM-1



STORMWATER MANAGEMENT PLAN

- ZONE 1 – QUANTITY AND QUALITY FACILITIES
  SWM#1 – 600m³
  SWM#2 – 1800m³
- ZONE 2 – QUALITY TREATMENT
  CULVERT IMPROVEMENT
- ZONE 3 – QUALITY TREATMENT
  DITCHING AND EROSION PROTECTION
- ZONE 4 – NO TREATMENT PLANNED
- ZONE 5 – QUALITY TREATMENT

JEWELL ENGINEERING

GENERAL NOTES:

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- LIDAR CGVD 2013

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

- SWM FEATURE
- PROPOSED OVERLAND FLOW ROUTE
- EXISTING OVERLAND FLOW ROUTE
- ENVIRONMENTAL CONSTRAINTS
- DITCHING
- 100 YEAR FLOOD LIMIT
- ① STORM OUTLET ID

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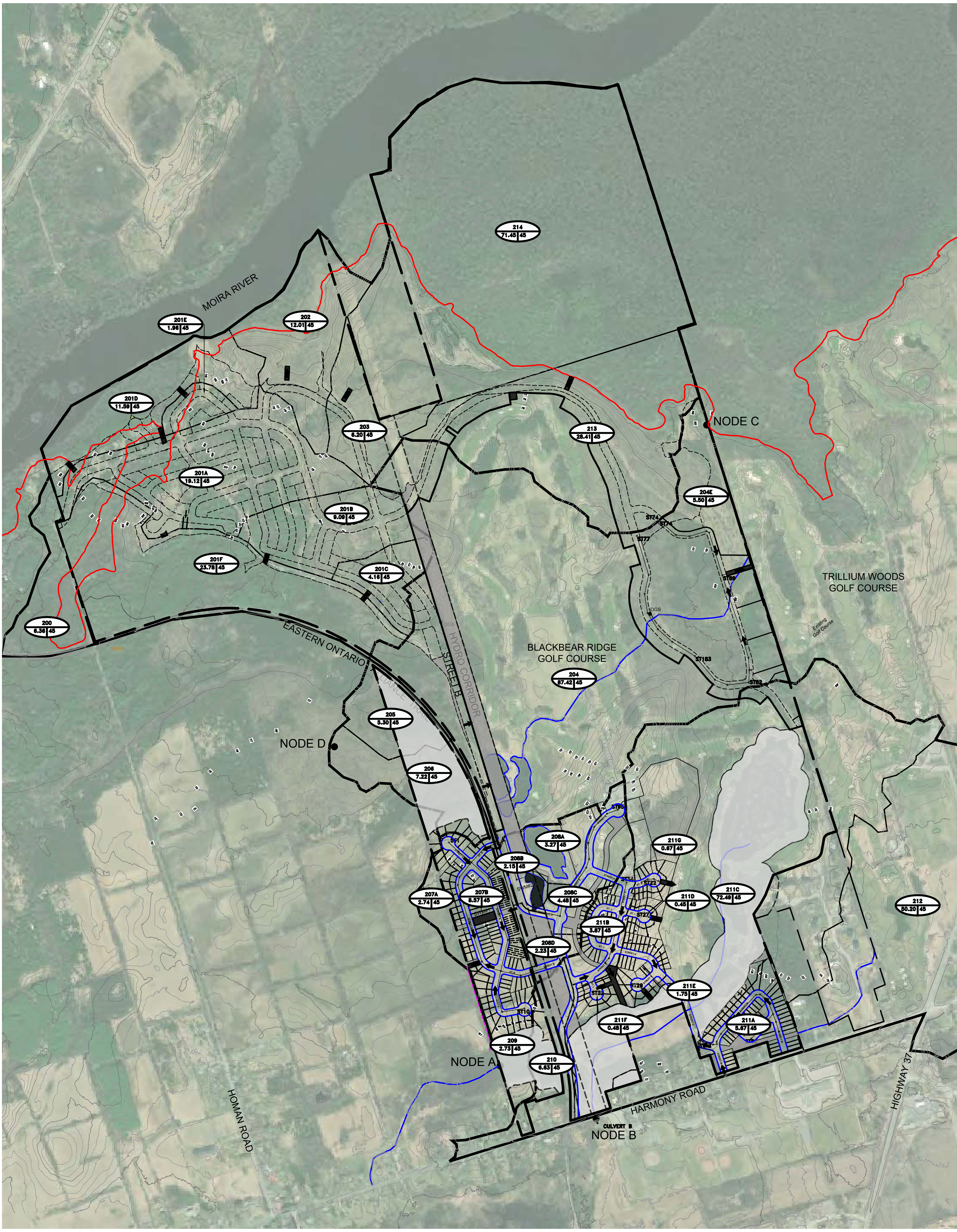
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PROPOSED STORMWATER
MASTER PLAN

DRAWN BY: DFM/AG	PROJECT NO: 2205260
DESIGNED BY: DFM/AG	DATE: February 2025
CHECKED BY: BK	CONTRACT NO:
APPROVED BY: BK	
SCALE: HORIZONTAL - 1:5000 VERTICAL -	
	DRAWING NO: STM-2



JEWELL ENGINEERING

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- LIDAR CGVD 2013

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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

- SWM FEATURE
- PROPOSED OVERLAND FLOW ROUTE
- EXISTING OVERLAND FLOW ROUTE
- ENVIRONMENTAL CONSTRAINTS
- DITCHING
- 100 YEAR FLOOD LIMIT
- 211F / 0.48 | 45 — CATCHMENT AREA ID / AREA(ha) & IMPERVIOUSNESS

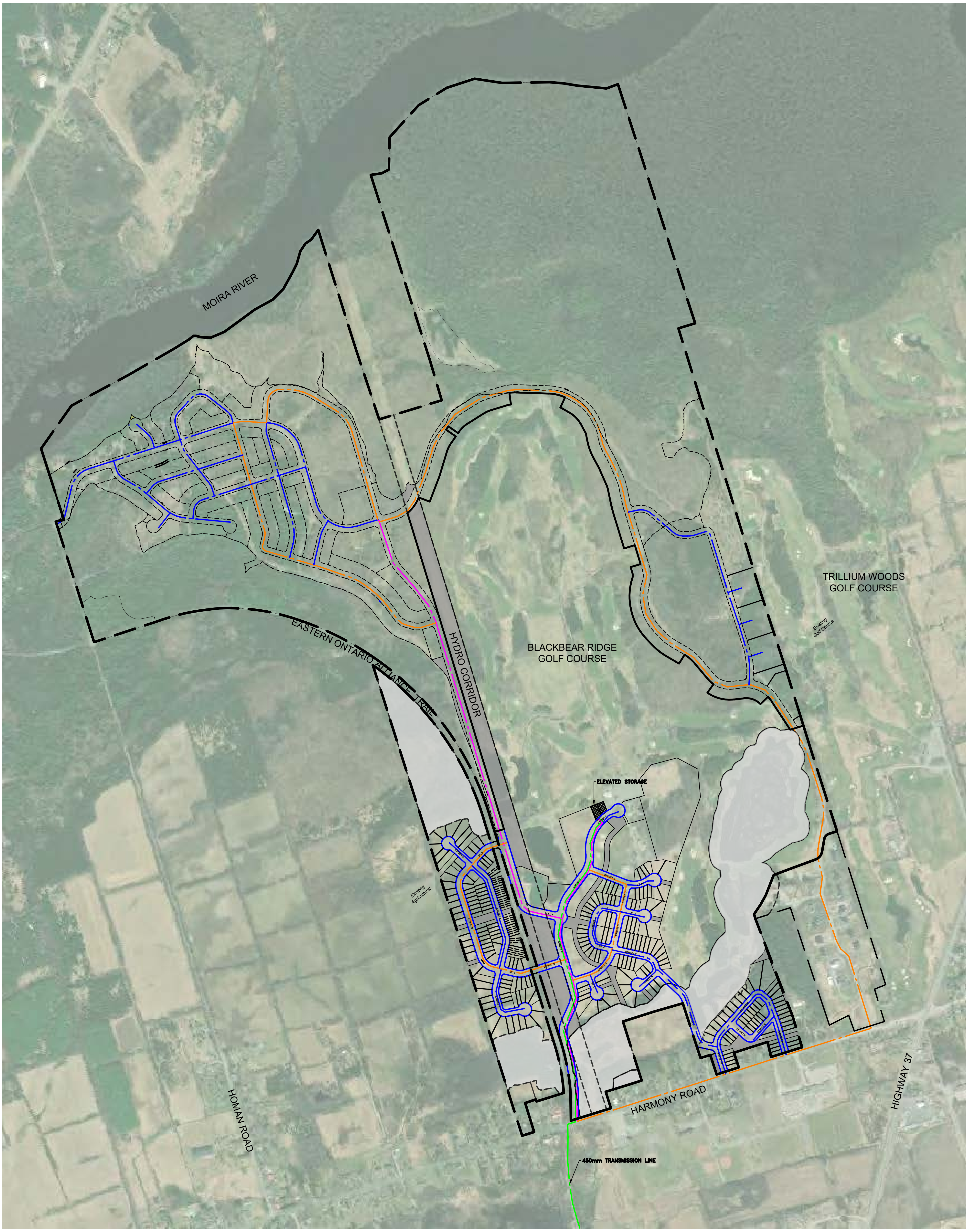
N W E S

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PROPOSED STORMWATER
CATCHMENT AREA PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
HORIZONTAL - 1:5000
VERTICAL -
DRAWING NO: STM-3



JEWELL ENGINEERING

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- LIDAR CGVD 2013

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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

- 200mmø WATERMAIN
- 300mmø WATERMAIN
- 400mmø WATERMAIN
- 450mmø WATERMAIN
- ELEVATED STORAGE
- ENVIRONMENTAL CONSTRAINTS

N W E S

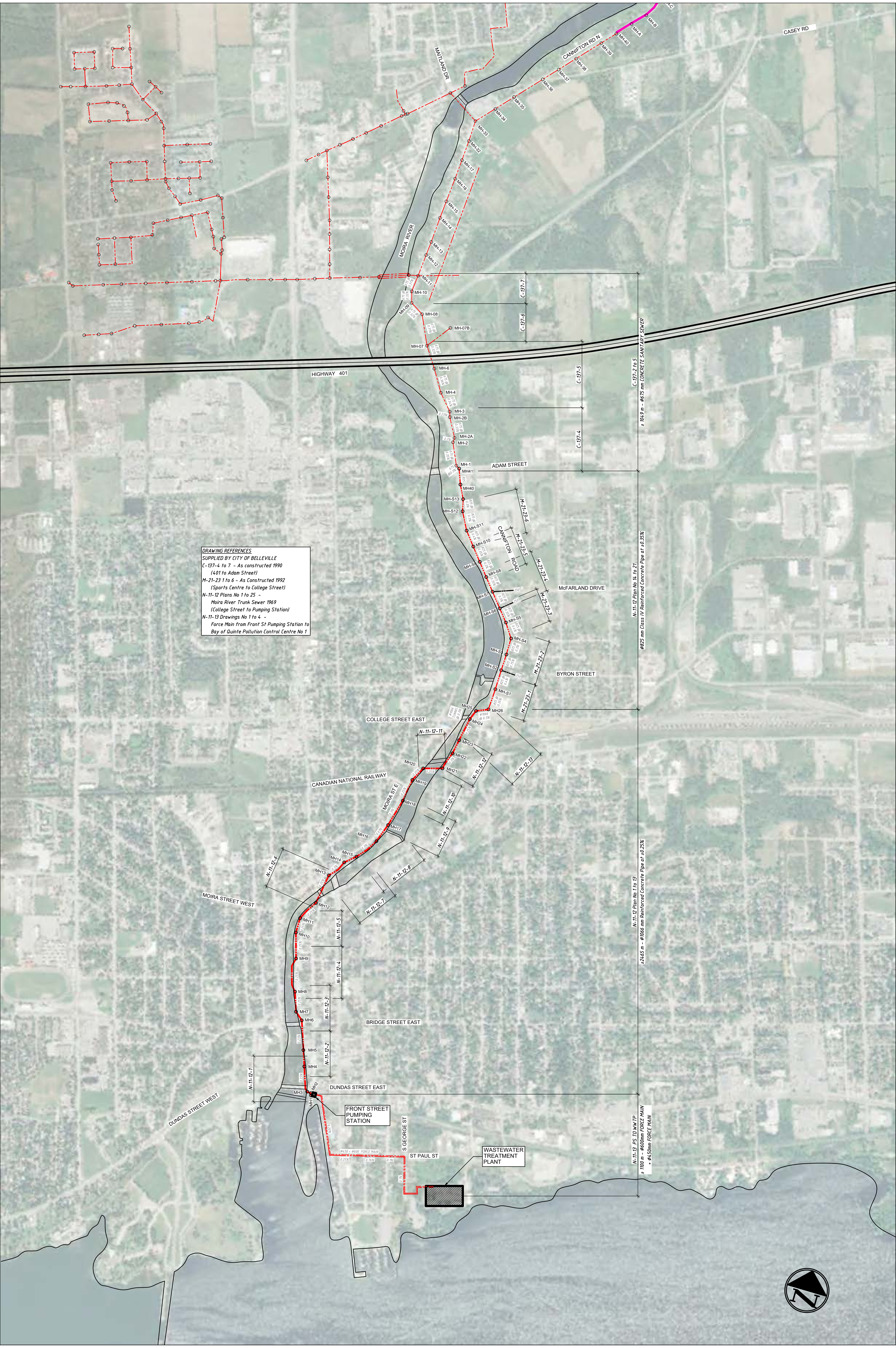
LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Jan. 31/25
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PRELIMINARY WATER SERVICING PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
HORIZONTAL - 1:5000
VERTICAL -
DRAWING NO: WTR-1

APPENDIX C:
SANITARY CATCHMENT AREAS AND DESIGN SHEET



CASEY RD
CANNIFTON RD N
MAITLAND DR
MOIRA RIVER
HIGHWAY 401
ADAM STREET
CANNIFTON ROAD
McFARLAND DRIVE
BYRON STREET
COLLEGE STREET EAST
CANADIAN NATIONAL RAILWAY
MOIRA ST E
MOIRA STREET WEST
BRIDGE STREET EAST
DUNDAS STREET EAST
DUNDAS STREET WEST
S GEORGE ST
ST PAUL ST
FRONT STREET PUMPING STATION
WASTEWATER TREATMENT PLANT
DRAWING REFERENCES
SUPPLIED BY CITY OF BELLEVILLE
C-137-4 to 7 - As constructed 1990
(401 to Adam Street)
M-21-23 1 to 6 - As Constructed 1992
(Sports Centre to College Street)
N-11-12 Plans No 1 to 25 -
Moira River Trunk Sewer 1969
(College Street to Pumping Station)
N-11-13 Drawings No 1 to 4 -
Force Main from Front St Pumping Station to
Bay of Quinte Pollution Control Centre No 1
C-137-2 to 5
± 1049 m - Ø675 mm CONCRETE SANITARY SEWER
N-11-12 Plan No 14 to 21
Ø825 mm Class IV Reinforced Concrete Pipe at ±0.35%
N-11-12 Plan No 1 to 13
±2465 m - Ø1066 mm Reinforced Concrete Pipe at ±0.25%
N-11-13 PS TO WWTP
± 1100 m - Ø600mm FORCE MAIN
+ Ø450mm FORCE MAIN
C-137-7
C-137-6
C-137-5
C-137-4
M-21-23-6
M-21-23-5
M-21-23-4
M-21-23-3
M-21-23-2
M-21-23-1
N-11-12-13
N-11-12-12
N-11-12-11
N-11-12-10
N-11-12-9
N-11-12-8
N-11-12-7
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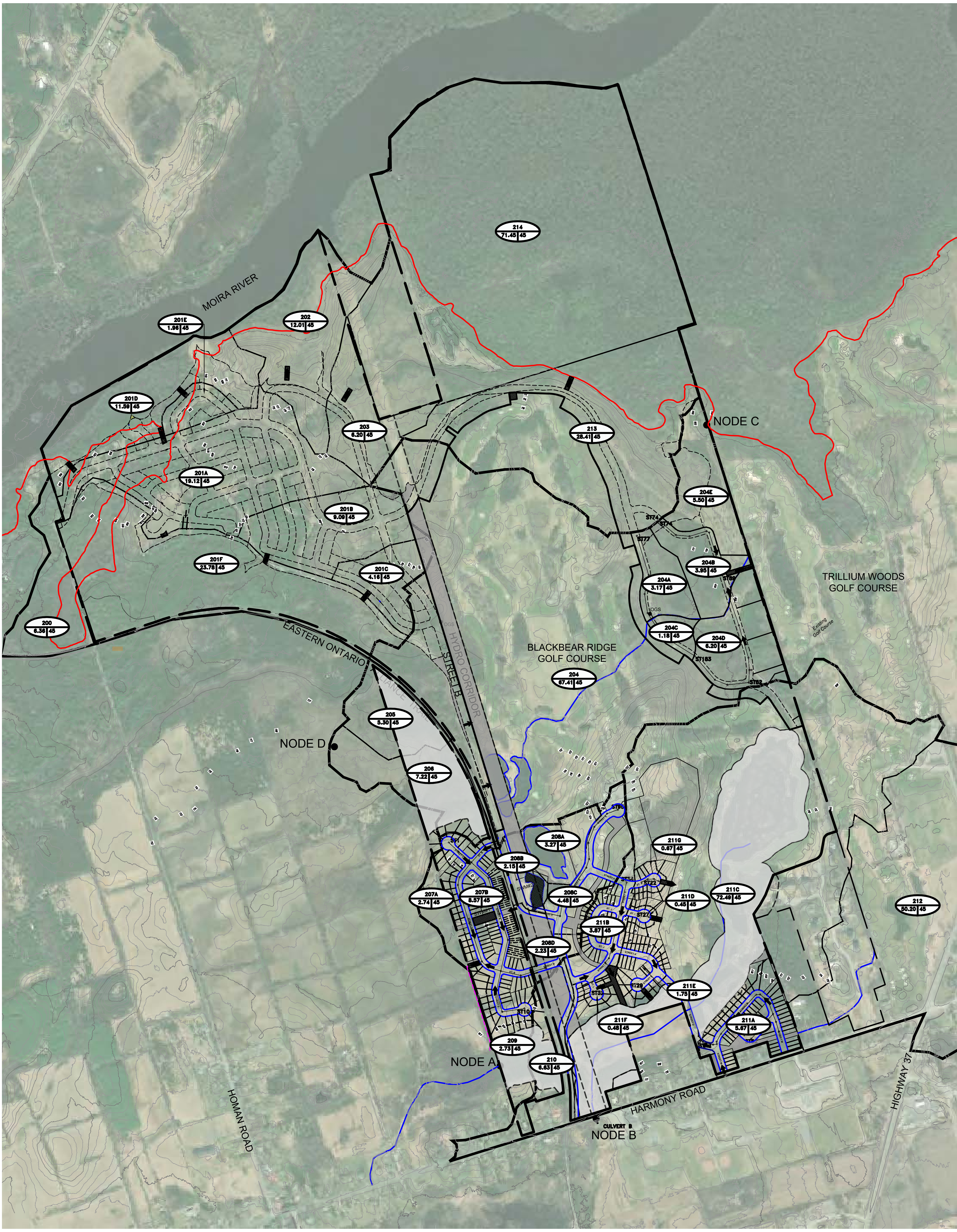
N-11-12-1
N



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REVISIONS

NO.	DATE	DESCRIPTION	BY
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- 100 YEAR FLOOD LIMIT
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LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

JEWELL ENGINEERING

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PROPOSED STORMWATER
CATCHMENT AREA PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
HORIZONTAL - 1:5000
VERTICAL -
DRAWING NO: STM-4

SANITARY SEWER DESIGN SHEET - Black Bear Ridge Development 3,049 Homes + Commercial

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

- q = Average daily per capita flow **350** L/d*cap
- I = Unit of peak extraneous flow **0.28** L/s*ha
- M = Harmon peaking factor (min = 2)
- P = Population in 1000's
- A = Area in hectares

Population Flows (Persons/Unit)

Single Family	3.0
Semi (3bdrm)	3.0
Townhouse/Apartment	2.5
Batchelor	1.6

Commercial Flows

Peaking Factor	1.00	(Note: GGG used no peaking factor)
Average Commercial Flows	5.00	L/d*sq.m
Commercial Peak Flow	5.00	L/d*sq.m

Floor Area = 30% Gross Area

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

- A = area of pipe in m^2
- R = Hydraulic radius = A / P
- P = Wetted perimeter
- S = Slope (m/m)
- n = Manning's friction coef.

Check

$$Q_d \leq 0.8 \cdot (Pipe\ Capacity)$$

$$0.6 \leq V \leq 3.0$$

use Actual V if d:D < **0.3**

LOCATION				PEAK FLOW CALCULATION																				
				Residential					Commercial															
				Individual		Cumulative			Individual		Cumulative													
DESCRIPTION	AREA	FROM	TO	Pop.	Area (A) (ha)	Pop.	Area (A) (ha)	Peaking Factor M	Floor Area (sq.m)	Area (ha)	Floor Area (sq.m)	Area (ha)	Pop. Flow $Q_{(p)}$ (L/s)	Commer. Flow $Q_{(c)}$ (L/s)	Peak Ex. Flow $Q_{(i)}$ (L/s)	Design Flow $Q_{(d)}$ (L/s)	Pipe Size (mm)	Grade (use m/m) (%)	Capacity n = 0.013 (L/s)	Full Flow Velocity (m/s)	Ratio d:D	Actual Velocity at Q_d (m/s)	Check	% Full q/Q
BBR - North-Central	600	BBR-29	BBR-PS-E	225	2.51	225	2.51	4.13			0.0	0.00	3.8	0.0	0.7	4.5	200	0.40%	20.74	0.66	0.32	0.66	OK	21.5%
Pump Station E		BBR-PS-E	**BBR-27**			225	2.51	4.13			0.0	0.00	3.8	0.0	0.7	4.5								
BBR - North-West	501	BBR-27	**BBR-18**	5,141	32.83	5,366	35.34	3.22			0.0	0.00	69.9	0.0	9.9	79.8	375	0.40%	110.89	1.00	0.63	1.00	OK	72.0%
	500	**BBR-18**	BBR-PS-D	1,130	7.22	6,496	42.56	3.14			0.0	0.00	82.6	0.0	11.9	94.5	450	0.30%	156.16	0.98	0.56	0.98	OK	60.5%
Street B		BBR-PS-D	BBR-13			6,496	42.56	3.14			0.0	0.00	82.6	0.0	11.9	94.5								
Street C	200, 204-205	BBR-13	**BBR-11**	195	4.56	6,691	47.12	3.13			0.0	0.00	84.7	0.0	13.2	97.9	450	0.30%	156.16	0.98	0.58	0.98	OK	62.7%
Street C	201-203	BBR-12	**BBR-11**	210	4.93	210	4.93	4.14			0.0	0.00	3.5	0.0	1.4	4.9	200	0.40%	20.74	0.66	0.33	0.66	OK	23.6%
		BBR-11	**BBR-PS-A**			6,901	52.05	3.11			0.0	0.00	87.0	0.0	14.6	101.6	450	0.30%	156.16	0.98	0.59	0.98	OK	65.1%
Street A	108-111	BBR-10	**BBR-8**	451	5.01	451	5.01	4.00	16,650.0	5.55	16,650.0	5.55	7.3	1.0	3.0	11.2	200	0.40%	20.74	0.66	0.52	0.66	OK	54.1%
Street E	100-107	BBR-9	**BBR-8**	246	10.29	246	10.29	4.11	5,730.0	1.91	5,730.0	1.91	4.1	0.3	3.4	7.8	200	0.40%	20.74	0.66	0.43	0.66	OK	37.8%
Street A		**BBR-8**	**BBR-1**			697	15.30	3.90			22,380.0	7.46	11.0	1.3	6.4	18.7	250	0.30%	32.57	0.66	0.54	0.66	OK	57.3%
BBR - East	401	BBR-7-S	**BBR-6**	579	6.41	579	6.41	3.94			0.0	0.00	9.2	0.0	1.8	11.0	200	0.40%	20.74	0.66	0.52	0.66	OK	53.2%
	402	BBR-7-E	**BBR-6**	1,017	11.42	1,017	11.42	3.80			0.0	0.00	15.6	0.0	3.2	18.8	250	0.30%	32.57	0.66	0.55	0.66	OK	57.8%
	400	**BBR-6**	BBR-PS-C	159	1.17	1,755	19.00	3.63			0.0	0.00	25.8	0.0	5.3	31.1	300	0.30%	52.97	0.75	0.55	0.75	OK	58.8%
Pump Station C		BBR-PS-C	BBR-5			1,755	19.00	3.63			0.0	0.00	25.8	0.0	5.3	31.1								
		BBR-5	BBR-4			1,755	19.00	3.63			0.0	0.00	25.8	0.0	5.3	31.1	300	0.25%	48.35	0.68	0.59	0.68	OK	64.4%
Harmony Public School	800	BBR-4	**BBR-2**	600	5.27	2,355	24.27	3.53	5,105.0	5.27	5,105.0	5.27	33.7	0.3	8.3	42.2	375	0.25%	87.67	0.79	0.49	0.79	OK	48.2%
BBR - South-Central	300-303	BBR-3	BBR-PS-B	177	5.65	177	5.65	4.17			0.0	0.00	3.0	0.0	1.6	4.6	200	0.40%	20.74	0.66	0.32	0.66	OK	22.0%
Harmony Rd		BBR-PS-B	**BBR-2**			177	5.65	4.17			0.0	0.00	3.0	0.0	1.6	4.6								
Gerry Masterson Hall	700	**BBR-2**	**BBR-1**			2,532	29.92	3.50	1,011.0	0.78	6,116.0	6.05	35.9	0.4	10.1	46.4	375	0.40%	110.89	1.00	0.45	1.00	OK	41.8%
		BBR-1	**BBR-PS-A**			3,229	45.22	3.42			28,496.0	13.51	44.7	1.6	16.4	62.8	375	0.40%	110.89	1.00	0.54	1.00	OK	56.6%
Pump Station A		**BBR-PS-A**	CAN-40			10,130	97.27	2.95			28,496.0	13.51	121.0	1.6	31.0	153.7								

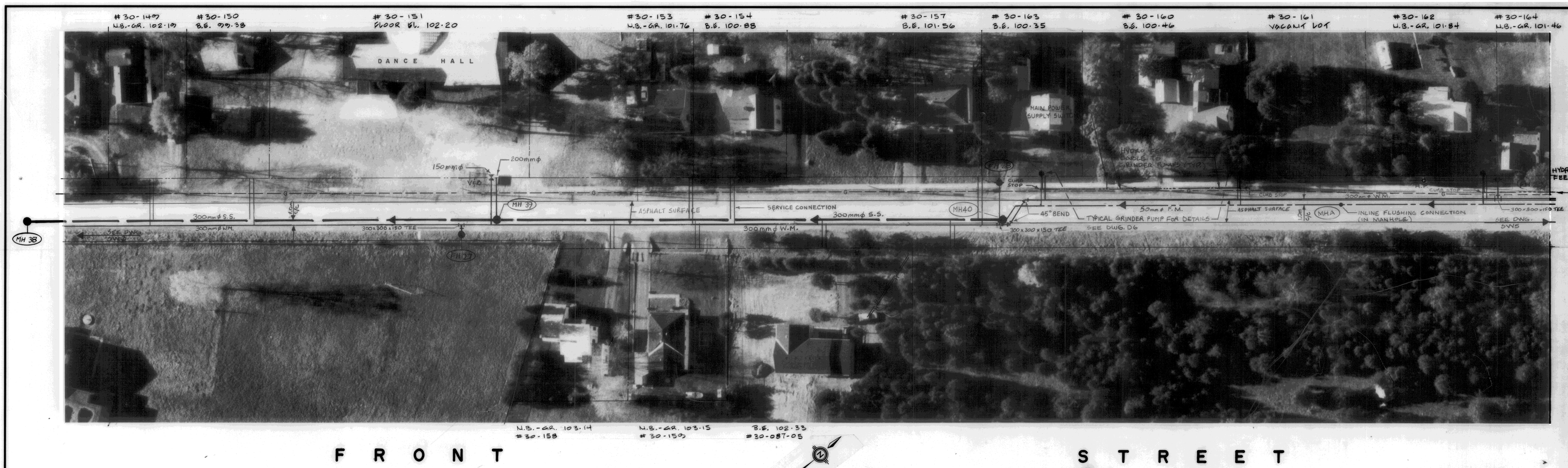
Jewell Engineering Inc
1-71 Millennium Parkway
Belleville, ON, K8N 4Z5
Ph. 613-969-1111
Fx. 613-989-8988
www.jewelleng.ca

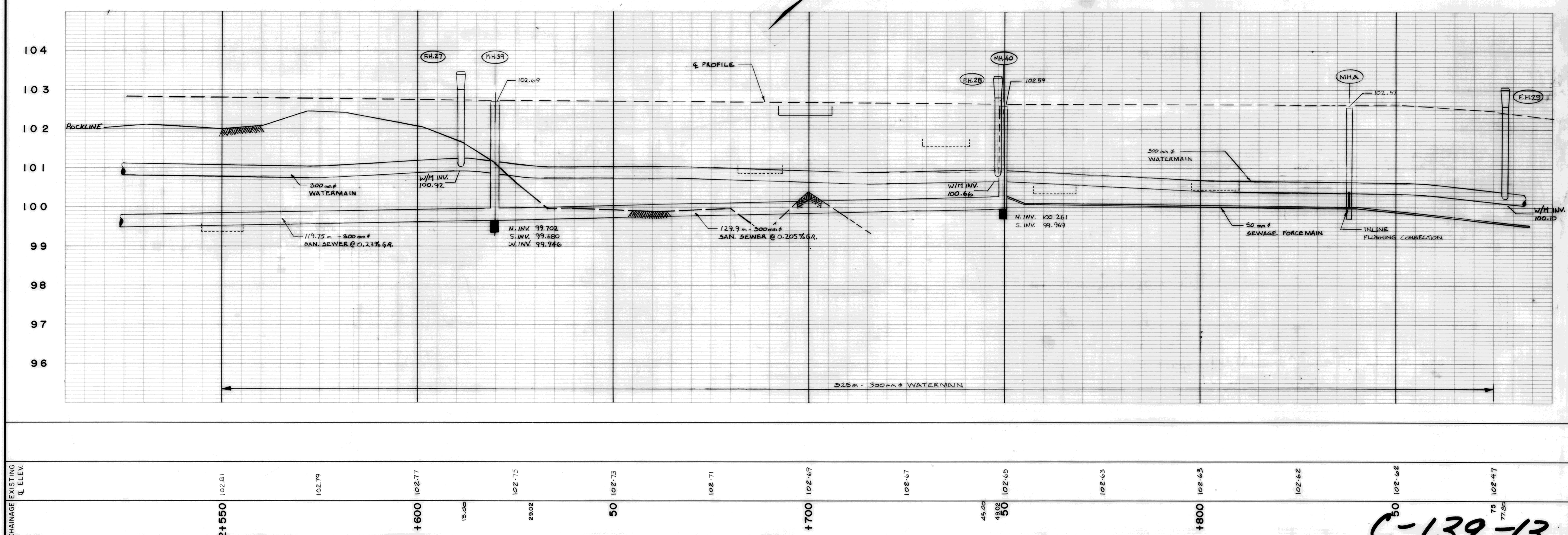
Legend

- Contributes to siphon
- Black Bear Ridge Development

Designed:	Julie Humphries, C.E.T.
Checked:	Bryon Keene P.Eng.
Date:	July 11, 2024

Project:
Black Bear Ridge Development
3,049 Homes plus Commercial





C-139-13

Notes:

1) REFER TO SOILS REPORT FOR MORE INFORMATION ABOUT BOREHOLES AND AUGER HOLES.
2) EXACT LOCATION OF SERVICE CONNECTIONS TO BE DETERMINED IN THE FIELD.
3) LOCATION OF ALL UTILITIES ARE APPROXIMATE. CONTRACTOR TO OBTAIN UTILITY LOCATES AS REQUIRED.
4) ALL MANHOLES ARE 1200 mm Ø UNLESS OTHERWISE NOTED.
5) ALL WATER SERVICE CONNECTIONS TO BE 19 mm Ø UNLESS OTHERWISE NOTED.
6) ALL SANITARY SERVICE CONNECTIONS TO BE 125 mm Ø UNLESS OTHERWISE NOTED.

LEGEND

—— B ——	BURIED BELL CABLE
—— H ——	BURIED HYDRO CABLE
— — G — —	GASMAIN
———	WATERMAIN
━━ ━━	SANITARY SEWER
=========	EXISTING STORM SEWER
	BASEMENT TO RIGHT OF ℄
	BASEMENT TO LEFT OF ℄
	BOREHOLE
	AUGER HOLE
GR.96.06	GROUND ELEVATION
B.E.94.39	BASEMENT ELEVATION
●	GRINDER PUMP

NO.	REVISIONS	DATE	INITIAL
4	AS-CONSTRUCTED	JUN./90	C.G.
3	GENERAL REVISIONS	OCT./88	C.K.K.
2	REVISION NOTE	OCT/88	R.M.
1	HYDRO ADDED FOR PUMP UNITS FORCEMAIN SIZE REVISED	JULY/88	R.M.

Approved

REGISTERED PROFESSIONAL ENGINEER C. K. KENT PROVINCE OF ONTARIO

TOWNSHIP OF THURLOW
(CANNIFTON AREA)

ONTARIO MINISTRY OF THE ENVIRONMENT
DIRECT GRANT PROJECTS № 3-0176 & 7-0245

PLAN AND PROFILE
FRONT STREET – STA. 2+550 TO STA. 2+875

Ainley and Associates Ltd.
Consulting Engineers and Planners
Collingwood — Barrie — Belleville

SCALE: HORZ. 1:500 VERT. 1:50	METRIC	CONTRACT 4	
DESIGN R.M.	CHECKED C.K.K.	DWG. № 187574-SW4	
DRAWN D.E.	DATE MAY, 1988		

APPENDIX D:
DILLON CONSULTING – WATER SERVICING MEMO

Memo

To:	Bryon Keene, P.Eng., Jewell Engineering Inc. (Jewell)
From:	Abdallah Alhalbouni, Dillon Consulting Limited (Dillon)
cc:	Justin Doiron, (Dillon) Matthew Murdock, (Dillon) Saher Ghanem (Dillon)
Date:	June 4, 2024
Subject:	Design Basis and Water Supply Feasibility for the Black Bear Ridge Subdivision (Bellville, ON)
Our file:	23-6045

Introduction

The Black Bear Ridge development is a proposed residential and commercial area north of Harmony Road in Belleville, Ontario and generally surrounding the existing golf course of the same name. Dillon Consulting Limited (Dillon) was retained by Jewell Engineering Inc (Jewell) to review the design criteria for the proposed development water system, including preliminary assessment of water service supply from the existing Belleville system to the proposed development, including functional size estimation of key infrastructure for water supply.

The following technical memorandum is presented in two parts, with the first part outlining the design expectations for the proposed Black Bear Ridge development's ultimate capacity, and the second part demonstrating a preliminary configuration for a first phase of development.

Table of Contents

List of Tables

List of Figures

Attachments

Part 1: Ultimate Development Water Distribution Design Basis

The proposed development design basis for the water distribution system follows the ultimate development density according to the planning documents provided to Dillon by Jewell. The demand is calculated from the available number of units for each planning block, by the unit demand rate for each unit type, and according to peaking factors representing steady state average day, maximum day, and peak hour demands. The proposed development area will be assessed for suitable water service according to acceptable pressure range during each of the steady state demand scenarios.

Development Population

The demands are allocated for the proposed development on the basis of the number of units (residential) or by gross area (commercial/industrial) at a unit rate for each development block or as otherwise shown on the concept plan provided by Jewell (**Attachment A**).

The total number of proposed residential units in the ultimate build-out is understood to be **3049 units.** The proposed development is also understood to include non-residential retail and commercial allowances which are provided in concept plans. Demand units are identified in the Black Bear Ridge (BBR) conceptual master plan and the Servicing Feasibility Report documents provided to Dillon (**Attachment A**). These plans are used for the purpose of estimating non-residential demand requirements.

Jewell's feasibility study included an allowance for three people per unit (3 ppu) in residential occupancy (**Attachment B**)**.** The following unit density was applied to the development concept:

- Low Density Residential (LDR): 25 u/ha @ 3 ppu
- Medium Density Residential (MDR): 60 u/ha @ 3 ppu
- Resort Hotel: 500 L/person.

Jewel has requested that areas adjacent to the BBR development be considered in the water demand of the proposed water tower. These areas are Harmony Road, James Property and Foxboro. The school located on Harmony Road has an existing water demand of 7000 L/d and is expected to supply up to 900 students in the future. The Gerry Masterson Community Centre at 516 Harmony Road has a dining capacity of 173 people. Using the latest version of google maps, the approximate number of houses on Harmony Road, Homan Road, and River Road is 85, 17, and 60 single homes, respectively. Harmony Road shall represent the units on Homan and River Road in this report. Statistics Canada shows that the 2016 population of Foxboro was 422 up from 408 in 2011. Dillon has allowed for a population of 450 people for Foxboro for the purpose of estimating water demand flowing through the proposed BBR development.

The James property is understood to have significant environmental constraints and may not be fully developable. As such, Jewell has recommended that half of the property be considered with low to medium density residential development while the rest will be environmental property. The area for the James property is roughly 715,315 m^2 or 71.53 ha (**Attachment A**). Therefore, 17.88 ha will be allocated to LDR and 17.88 ha will also be allocated to MDR.

The resort areas A & B as seen in **Figure A-12**, **Figure A-13**, and **Figure A-14** are expected to serve as areas for commercial development. As such, an approximate area from google earth is taken to estimate the water demand required for commercial services. The estimated land **area of the commercial space is approximately 38,277 m^2**.

Table 1: Population Distribution in Unit Types

Unit Type	Number of Units	Assumed Population Per Unit [Cap]	Population Equivalent [Cap]
RESIDENTIAL	3049	3	9147
RESORT A/B	2	250	500
		SUBTOTAL (BBR)	**9,647**
HARMONY ROAD	162	3	486
RESORT	1	250	250
SCHOOL	1	900	900
FOXBORO		450	450
JAMES PROPERTY	1,520	3	4,560
		TOTAL	**16,293**

The ultimate service plan includes a proposed development population of approximately 9,647 capita, which is 59.2% of the total population service via flow-through connections of 16,293 capita. These estimates exclude the proposed commercial areas.

Development Demand

The previous servicing analysis applied a consumption rate of 350 L/cap/day for all residential demands. The assumed flows and demands were compared to the relevant design guidelines available for review. The City of Bellville deign standards are not well developed and as such were not used for the Servicing Report. The Ontario Design Guidelines for Drinking Water Systems (Ontario Ministry of the Environment, 2023) includes ranges of water demands and peaking for planning. The normal design range for domestic (residential) consumption is between 270 to 450 L/(cap.d). The best estimate of existing Belleville water demand is extracted from the available EPANET model with a gross blended (residential/commercial) maximum day demand of 351.41 L/s for a census population of 50,720 people (2016 census) or 591.61 L/c/d under MDD conditions. The actual maximum day peaking factor was not available to Dillon; however, the Ontario Design Guidelines for Drinking Water Systems suggests a population-based MDD peaking factor of 1.75 times average day (after Table 3-1) for a blended ADD per-capita demand equivalent of 342.06 L/c/d which is less than the feasibility allowance of 350 L/c/d.

The non-residential demands of the development are understood to consist of commercial areas within the areas allocated for resort activities. The commercial demand rate is taken from the upper limit of the Design Guidelines for shopping centre area. The reasonable and conservative factor of 5 L/m^2/day is used[1]. This demand is conservatively applied to the floor space of commercial and commercial reserve properties. The average day demand unit rates are summarized in **Table 2.**

Table 2: Average Day Water Demand Rates (Ultimate Development and Flow-Through Service)

Water User	Servicing Report [L/c/d]	MOECC [L/c/d]	Design [L/c/d]	Population [cap] or Area [m^2]	ADD [L/s]
RESIDENTIAL	350	270 to 450	350	9147 cap	37.1
RESORT A	500	225	500	250 cap	1.5
RESORT B	500	225	500	250 cap	1.5
HARMONY RD.	350	270 to 450	350	486 cap	1.97
FOXBORO	350	270 to 450	350	450 cap	1.82
JAMES PROPERTY	NA	270 to 450	350	4560 cap	18.48
SCHOOL (HARMONY RD.)	NA	70 to 140 L/Student/8 hrs	140 L/Student/8 hrs	900 cap	4.38
COMMUNITY CENTER (HARMONY RD.)	NA	NA	140 L/person/8 hrs	173 cap	0.84
COMMERCIAL	5 L/m^2/day	2.5 to 5 L/m^2/day	5 L/m^2/day	38,277 m^2	2.22
Total					**69.81**

The water demand rate is conservatively selected to avoid arbitrarily restricting the demand flow under the analysis of the proposed development. In particular, the residential demand basis of 350 L/c/d is within the MOECC design guideline range and conservative with respect to the blended average historic demand of approximately 342 L/c/d.

As a result of the estimated ultimate development capability, the proposed development is expected to consume approximately 40.1 L/s under average day, which is *57.4% of the overall water service demand* including flow-through connections of 69.81 L/s under average day. The inclusion of flow-through services from the proposed development is expected to provide significant benefit to adjacent lands and existing developments.

[1] The Design Guidelines for Drinking Water Systems contains an error, which overstates the demand by one thousand times. The actual factor is intended to be equal to 5 L/m^2/day (as reported in the drinking water systems design guideline).

Water Demand Peaking Factors

The steady state peaking factors are applied to the analysis in order to represent typical demands that are valuable for engineering design of infrastructure. In particular these include the maximum day demand (MDD) peaking factor, and the peak hour demand factor. The servicing report and the Ontario Design Guidelines for Drinking Water Systems are summarized with the design basis in the development demand section above. The peaking factors are conservatively applied to both domestic and non-domestic demands in the analysis. Typical non-residential peaking factors historically tend to show less exaggeration except for heavy industry. The application of peaking factors for all development demands is considered conservative. Actual historic peaking factors were not provided to Dillon for the purpose of this analysis. The design basis peaking factors are summarized in **Table 3**.

Table 3: Design Basis Demand Peaking Factors

Demand Scenario	Servicing Study [X ADD]	MOE [1] [X ADD]	Design [X ADD]
AVERAGE DAY DEMAND (ADD) – BLENDED	1	1	1
MAXIMUM DAY DEMAND (MDD) - BLENDED	2	1.80	2
PEAK HOUR DEMAND (PHD) - BLENDED	3	2.85	3

Notes:

1. Peaking factors are adapted from Table 3-1 of the Design Guidelines for Drinking Water Systems, 2023 appropriate for serviced populations between 10001 up to 25000.

Water Distribution Pressure

The pressure requirements for the proposed development areas are proposed in Jewell's feasibility report in accordance with the design guidelines for drinking water systems in Ontario (Sections 8.3 and 10.2.2.1) and are summarized in **Table 4**.

Table 4: Design Pressure Guidelines (adapted from MOECC, Section 10.2.2.1)

Demand Scenario	Minimum Pressure	Maximum Pressure
MDD + FIRE FLOW	20 psi (140 kPa)	-
PHD	40 psi (275 kPa)	-
NORMAL DEMAND	50 psi (350 kPa)	70 psi (480 kPa)
MINIMUM & MAXIMUM ALLOWABLE PRESSURE IN THE SYSTEM	20 psi (140 kPa)	100 psi (700 kPa)

Water Distribution Elevated Storage

Jewell's Feasibility report has adopted the fire flow and emergency storage requirements in the MOE guidelines. The elevated tower is expected to supply water to a population of approximately 16,293 people (per **Table 1**). A conservative fire flow rate of 250 L/s for a 4-hour period was used for a population of 17,000 people from Table 8-1 in the MOE guidelines. The emergency storage was calculated to be 25% of the sum of the fire storage and the equalization storage (25% of the MDD). The calculations can be seen summarized in **Table 5**. The water system feasibility review will consider the water transfer (pumping) requirements to meet the industry standard worst case pumping conditions in later sections.

Table 5: Fire Flow and Storage Requirements and Calculations

MOECC Volume Fraction	Calculation	Volume (m^3)
Fire Volume (A)	$250\frac{L}{s} * \frac{3600s * 1m3}{1\,hr * 1000L} * 4\,hrs$	**3,600**
ADD	$69.81\frac{L}{s} * \frac{3600s * 24\,hr * 1m3}{1\,hr * 1\,day * 1000L}$	**6,032**
MDD	$MDD = ADD\ x\ MDD\ p.f.$ $MDD = (6032\,m^3)\ x\ (2)$ $MDD = 8985.6\,m^3$	**12,063**
Equalization Storage (B)	$Volume\ B = 25\%\ x\ MDD$ $= (0.25) * (12063)$	**3,016**
Emergency Storage (C)	$Volume\ C = 25\% \times (Volume\ A + Volume\ B)$	**1,654**
Total Storage	$Total\ Volume = Volume\ A + Volume\ B + Volume\ C$	**8,270**

The proposed service area ground elevation ranges between 105 and 130 metres of elevation. Applying the design pressure criteria from **Table 5** for normal demand conditions results in a desired water tower hydraulic grade line of nominally 165.7 to 176.5 metres HGL for the Equalization Storage (B) working volume. The bottom of the elevated storage liquid would be 142.7 metres elevation. (Hydraulic grade lines are conservatively set to meet design pressure guidelines at grade, with model results presented in Part 2).

The elevated water storage tower would be **nominally 20.64 metres diameter** to provide the volume required in the equalization storage (B) within the nominal working elevations. Assuming a cylindrical design, the resulting elevated storage tower would have a top water elevation and bottom water elevation of 176.5 metres and 146.9 metres, respectively, to achieve the estimated total storage capacity. It is recommended to employ a two-pipe elevated storage tower design to promote mixing within the storage tank to address potential water quality and freezing challenges.

The important hydraulic grade parameters are summarized as follows:

- Tank Inside working diameter: 20.64 metres

- Bottom of Elevated Storage: 152.5 metres elevation
- Top of Emergency Storage (C): 157.5 metres elevation
- Top of Fire Volume Storage (A): 168.3 metres elevation
- Top of Equalization Storage (B): 176.5 metres elevation
- Ground Elevation: 115.08 metres elevation (approximately)
- Pedestal Elevation: 37.42 metres
- Configuration: Two-pipe with top inlet and bottom withdrawal.

The existing Belleville distribution system consists of five existing elevated water storage tanks. These tanks are represented in the background hydraulic model as tanks with high water level (HWL) between 75.48 metres up to 138.02 metres, with the John Elevated Tower operating with the highest HGL. These elevations are below the minimum required HGL for the Black Bear Ridge development as identified above. This means that additional transfer pumping will be required to meet the proposed operating pressures of the proposed development. The nearest existing pumping station is on Cannifton Road south of Highway 401 operating at a discharge HGL of nominally 146.32 metres (510 kPa discharge pressure). A review of the requirements for water transfer into the proposed Black Bear Ridge elevated storage is described in the following section.

Water Transmission to Black Bear Ridge Development

The industry standard for pumping systems feeding elevated storage is to achieve maximum day demands while filling the elevated storage to top water level. The transfer of water into the proposed Black Bear Ridge development requires the capacity in the existing Belleville system to deliver both flow and pressure to achieve filling of the proposed elevated storage to top water level under maximum day background demand. The proposed elevated water storage top water level elevation has been demonstrated to be sufficient for the design basis service pressures in the distribution system. The proposed development would be filled from the existing distribution system via a dedicated 400 mm water transmission main. The nearest point of interconnection for a transmission main into existing 400 mm DI near Tank Farm Road and Short Road. This area is presently supplied via pumping station located on Cannifton Road between Highway 401 and Adam St. The required maximum day flow and pressure requirements at the proposed elevated tower in the Black Bear Ridge development is:

- MDD Demand: 139.6 L/s (**Table 2** and **Table 3**)
- BBR TWL HGL: 176.5 m HGL
- Supply Min. HGL: 146.32 m HGL (Cannifton area pressure zone).

A comparison of the nominal hydraulic grade line minimum and maximum within the existing pressure zone supplied by the Cannifton Road pumping facility is summarized against the proposed development area pressure zone in **Figure 1**.

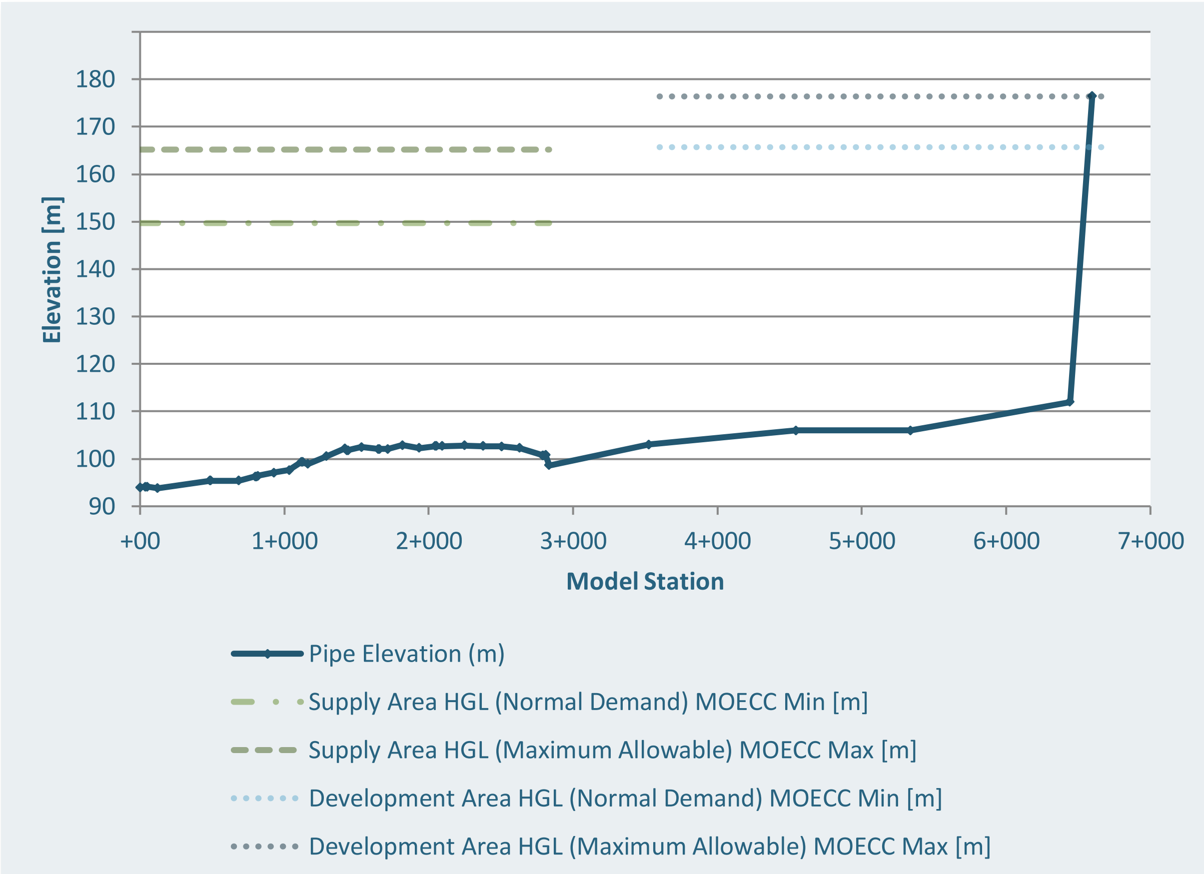


Figure 1: Hydraulic Model Supply Area HGL versus Proposed Development HGL

The supply area requires a pressure increase of approximately 30.2 metres of pressure head (296 kPa) in order to providing filling flow to the proposed elevated storage high water level under maximum day demand. The present supply area pressure zone operates at nominal pressures of between 315 kPa up to 514 kPa under normal maximum day demand. Increasing the existing pressure zone by 296 kPa to provide sufficient pressure for transfer to the Black Bear Ridge development pressure zone would exceed 700 kPa design guideline maximum within the supply area. This increase in pressure can not be achieved by increasing the existing discharge pressure at the Cannifton Road pumping facility without exceeding the pressure in the existing supply area pressure zone. An *additional booster pumping station* would be required on the transmission main alignment between River Road at Cannifton Road North and the proposed development area.

The existing Cannifton Road pumping facility would be required to achieve maximum day demand flow rates for both the existing pressure zone, as well as the additional flow to supply to proposed booster pumping station. The booster pumping station in turn would increase the supply pressure to fill the proposed elevated water storage tank in the Black Bear Ridge development area.

The existing water distribution system at the point of interconnection for supply to Black Bear Ridge consists of a duty/standby/jockey pump configuration with the duty pumps summarized according to **Figure 2**, as adapted from the EPANET water model provided to Dillon from Jewell (**Attachment B**).

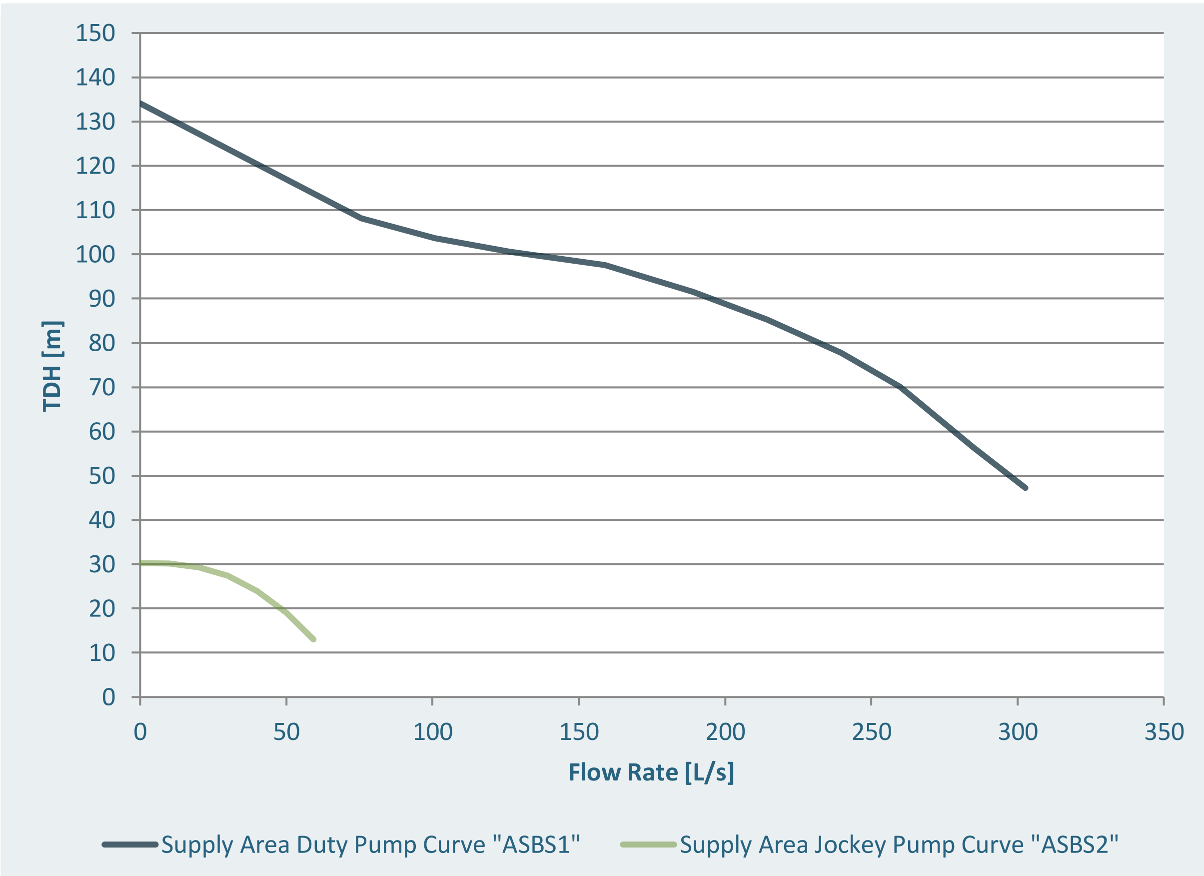


Figure 2: EPANET Hydraulic Model – Cannifton Road Jockey and Lead Pump Curves

The existing Cannifton Road pumping facility delivers approximately 16.05 L/s under maximum day demand within the existing pressure zone, according to the EPANET hydraulic model provided to Dillon. The MDD demands are met with the jockey pump operating at a duty point of 16.09 L/s at 14.08 m TDH into the existing discharge pressure reducing valve set for 525 kPa. The larger duty pump is capable of achieving greater flow and head required to deliver to the proposed transfer booster pumping station.

Combining the ultimate maximum day water demand for the proposed Black Bear Ridge development, including flow-through demands, results in a combined MDD demand at the Cannifton pumping station of approximately 155.65 L/s from the Cannifton Lead Pump, which appears to be within the available capacity of the pump curve.

The interconnection requirements for the ultimate development are summarized as follows:

- Maximum Day Demand (Transfer from existing pressure zone): 139.6 L/s
- Supply Pressure (Booster PS Suction Pressure): > 395 kPa
- Interconnection Point: Ex. 400 mm diameter (Short Road North of Tank Farm Road)
- Transmission Main: 3800 m of 450 mm diameter
- Cannifton Road Pumping Facility Requirements:
  - Large Duty Pump active to meet MDD water transfer to proposed BPS
  - Maintain existing PRV and setpoint (with potential for optimization of setpoint)
- Proposed Booster Pumping Station:
  - Booster configuration with Duty/Standby pumping ON/OFF control
  - Elevated Water Storage Tank filling only – interconnect with SCADA level control
  - Design Duty: 140 L/s @ 60.1 m TDH
  - Pump Power: 96.6 kW (130 hp) ea.
  - Consideration for reinforcement back-feeding existing pressure zone via PRSV chamber
- Proposed Elevated Water Storage Tank Configuration:
  - Two-pipe configuration with top fill, bottom withdrawal
  - Top Water Elevation: 176.50 m HGL
  - Nominal Diameter: 20.64 m
  - Nominal Total Volume: 8270 m^3.

The key observations related to the water supply to the proposed development include:

- The infrastructure sizing includes service to areas flowing through the proposed development where the development demands represent approximately 57.4% of the total design basis.
- The required water supply infrastructure for initial build-out will include:
  - Installation of proposed elevated water storage tank with filling control signal to booster pumping (start filling at HGL = 168.3 m, stop filling at HGL = 176.5 m)
  - Installation of proposed booster pumping station with coordinated control to existing Cannifton service area pumping station
  - Installation of proposed transmission water main
- The impact and confirmation of supply capacity within the existing Cannifton service area will require additional review by others.

Part 2: Water System Review for the Black Bear Ridge Concept Plan (August 2023)

The proposed development has been phased according to two plans provided to Dillon. These are the old conceptual master plan, and the new concept master plan. Both plans were provided to Dillon by Jewell. These plans form the basis for the following review of water supply infrastructure to a subset of the proposed ultimate development.

The old conceptual master plan (recreated in **Attachment B**) divided the BBR development into low and medium density residential zones. The Low density residential (LDR) development zone accounted for 64.48 ha with a proposed unit density of 25 units/ha for a total of 1612 units. The medium density residential (MDR) development zone accounts for 23.95 ha and a unit density of 60 units/ha for a total of 1437 units.

The new concept master plan (**Figure 3, Figure 4, and Figure 5**) divides the BBR development area into West, South, East, North and North-West Sites. The total number of residential units in this plan is 1416 units (refer to **Figure 5** and **Table 6**). The sites are divided into single family lots, townhouse typology, quad typology, courtyard typology, multi units, cabins in the woods and a resort. A population of 3 ppu is assumed consistent with the allowances of the ultimate development plan, with the exception of the resort that has a population of 250 ppu. A detailed distribution of the concept plan is provided in **Table 6**, **Table 7**, and **Figure 3**.

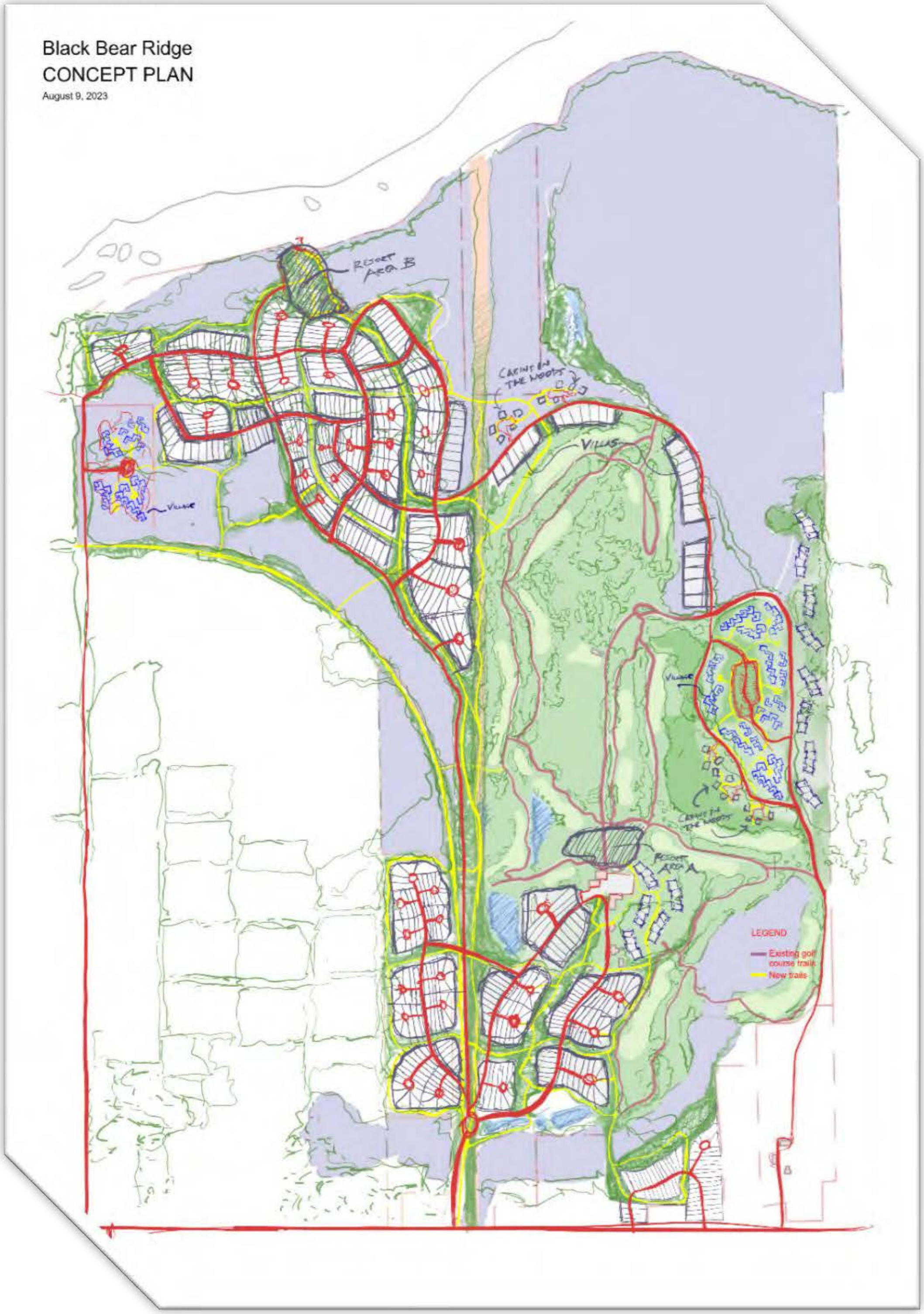


Figure 3: New Concept Plans for the BBR Development

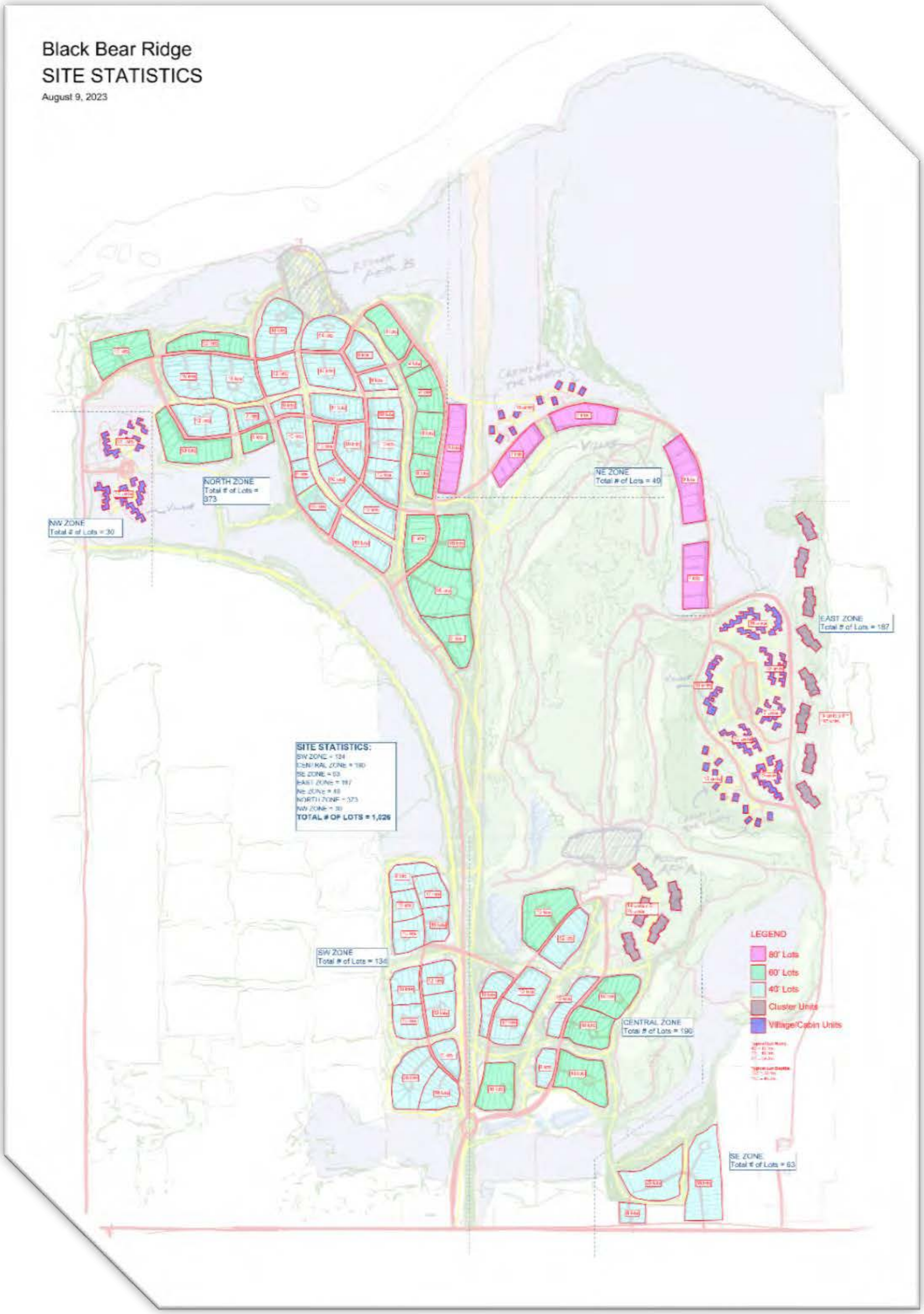


Figure 4: New Concept Plans for the BBR Development

Figure 5: New Updated Concept Plan for the BBR Development

The distribution of development sizes for the proposed development plan for Black Bear Ridge, as well as secondary servicing for adjacent areas are summarized in **Table 6**.

Table 6: Detailed Distribution of Unit Types in the BBR Development Area

Development Site	Low Density	Medium Density	40'x100' Single Family Lots	60'x100' Single Family Lots	80'x100' Single Family Lots	Townhouse Typology	Quad Typology	Courtyard Typology	Multi Unit	Cabins in the Woods	Resort	Total
WEST SITE	NA	NA	81	NA	64	62	84	NA	126	NA	1	418
SOUTH SITE	NA	NA	61	NA	NA	NA	NA	NA	NA	NA	NA	61
EAST SITE	NA	NA	40	NA	NA	NA	NA	51	70	18	NA	179
NORTH SITE	NA	NA	NA	NA	13	NA	NA	NA	NA	11	NA	24
NORTH-WEST SITE	NA	NA	200	NA	NA	240	408	NA	NA	37	NA	885
BBR AREA	SUBTOTAL BBR DEVELOPMENT DOMESTIC UNITS											1,567
HARMONEY RD.	NA	NA	NA	NA	162	NA	NA	NA	NA	NA	NA	162
JAMES PROPERTY	447	1,073	NA	NA	NA	NA	NA	NA	NA	NA	NA	1,520
FOXBORO	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	150
ALL SITES	TOTAL DOMESTIC UNITS											3,399

Table 7: Population Distribution in Unit Types

Unit Type	Number of Units	Assumed Population Per Unit [Cap]	Population Equivalent [Cap]
40'X100' SINGLE FAMILY LOTS	446	3	1,338
60'X100' SINGLE FAMILY LOTS	62	3	186
80'X100' SINGLE FAMILY LOTS	13	3	39
TOWNHOUSE TYPOLOGY	240	3	720
QUAD TYPOLOGY	492	3	1,476
COURTYARD TYPOLOGY	51	3	153
MULTI UNIT	196	3	588
CABINS IN THE WOODS	66	3	198
RESORT	1	250	250
SCHOOL	1	900	900
HARMONY RD.	162	3	486
FOXBORO	150	3	450
JAMES PROPERTY	1,520	3	4,560
TOTAL			**11,344**

Development Demand

The unit demand rates and peaking factors applied to the concept plan layout are the same as those summarized in **Table 2** on page 6. These factors are used together with the population totals for the concept to estimate the concept plan average day water demand rates as summarized in **Table 8**.

Table 8: Average Day Water Demand Rates (Concept Plan and Flow-Through Service)

Water User	Servicing Report [L/c/d]	MOECC [L/c/d]	Design [L/c/d]	POP. [cap] or Area [m^2]	Add [L/s]
WEST SITE	350	270 to 450	350	1254 cap	5.08
SOUTH SITE	350	270 to 450	350	183 cap	0.74
EAST SITE	350	270 to 450	350	483 cap	1.96

Water User	Servicing Report [L/c/d]	MOECC [L/c/d]	Design [L/c/d]	POP. [cap] or Area [m^2]	Add [L/s]
NORTH SITE	350	270 to 450	350	39 cap	0.16
NORTH-WEST SITE	350	270 to 450	350	2544 cap	10.31
HARMONY ROAD	350	270 to 450	350	486 cap	1.97
FOXBORO	350	270 to 450	350	450 cap	1.82
JAMES PROPERTY	NA	270 to 450	350	4560 cap	18.48
CABIN IN THE WOODS	NA	225 to 570 L/campsite/d	397.5 L/campsite/d	198 cap	0.91
RESORT A	500	225	500	250 cap	1.5
RESORT B	500	225	500	250 cap	1.5
SCHOOL (HARMONY ROAD)	NA	70 to 140 L/Student/8 hrs	140 L/Student/8 hrs	900 cap	4.37
COMMUNITY CENTER (HARMONY ROAD)	NA	NA	140 L/person/8 hrs	173 cap	0.84
COMMERCIAL	5 L/m^2/day	2.5 to 5 L/m^2/day	5 L/m^2/day	38,277 m^2	2.22
TOTAL					**51.86**

The water demand rate is conservatively selected to avoid arbitrarily restricting the demand flow under the analysis of the proposed development. In particular, the residential demand basis of 350 L/c/d is within the MOECC design guideline range and conservative with respect to the blended average historic demand of approximately 342 L/c/d.

Hydraulic Model Simulation

The proposed Black Bear Ridge development was reviewed with a computer numeric simulation using Bentley WaterGEMS Connect software environment. The City of Belleville water model was provided to Dillon by Jewell. The background model was adapted from EPANet format into WaterGEMS. This model is understood to represent the City of Belleville under maximum day demand (MDD) conditions. The model was extended with additional hydraulic elements representing the proposed development, as well as secondary connections including Harmony Road, the James' Property, and an allowance for extension to Foxboro. The hydraulic conditions are modelled at ground elevation, conservatively estimating available system pressure. Elevation data were provided to Dillon by Jewell in the form of a lidar-sourced ground surface AutoCAD file.

A preliminary watermain sizing includes transmission to the elevated storage in 400 mm diameter transmission watermain, intra-development distribution mains in 200 mm diameter, and other water mains in 150 mm diameter. All new development piping is simulated equivalent to thermoplastic (PVC or HDPE) with a conservative Hazen-Williams coefficient of friction equal to 130. A representation of the development hydraulic model is shown in **Figure 6.**

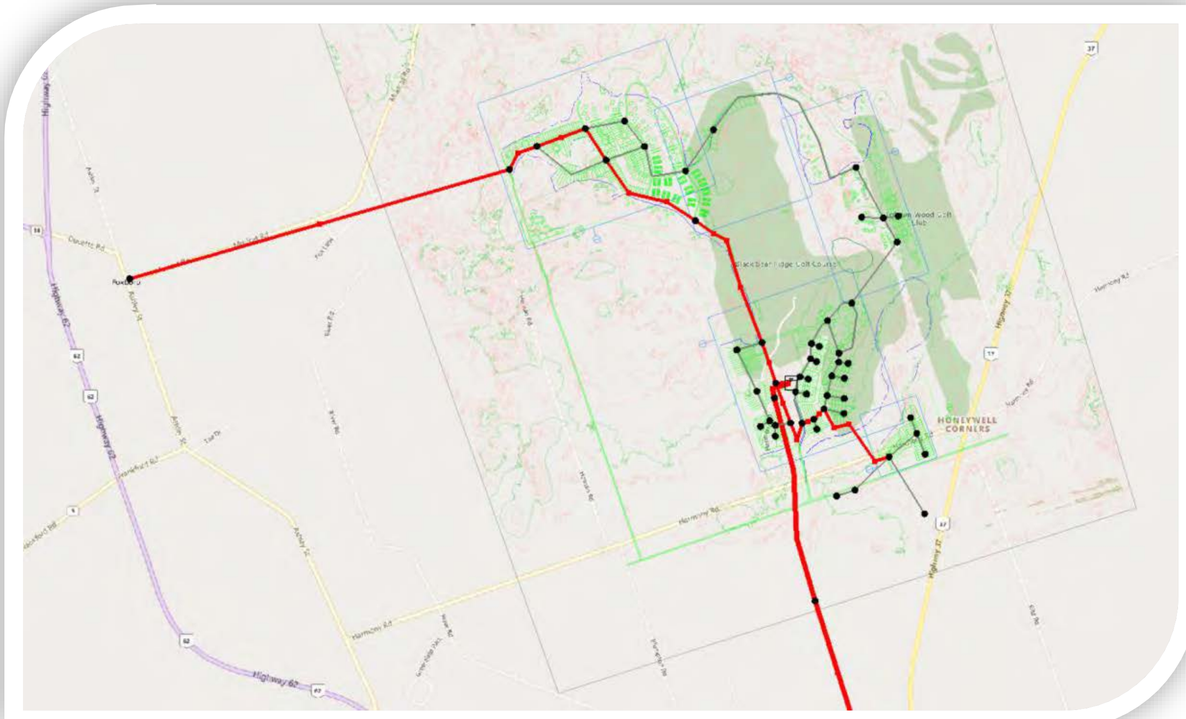


Figure 6: Model Representation of Proposed Black Bear Ridge Development (Transmission and Distribution Watermains in Red)

The distribution system was assessed on the basis of a functional water main layout shown in **Figure 6** consisting of the following water main diameters and lengths:

- 150 mm service mains: 7,519 metres
- 200 mm water mains: 2,995 metres
- 250 mm water mains: 2,375 metres
- 300 mm water mains: 492 metres

The proposed development area was modelled under average day demand (ADD), maximum day demand (MDD), and peak hour demand (PHD) according to **Table 9**.

Water elevation in the proposed elevated tower were assessed at 176.5 metres HGL (high water level) down to 168.3 metres HGL (low water level) to confirm the suitability of the proposed water storage elevations against hydraulic losses in the distribution system as modelled. The modelled pressures are summarized in **Table 9** and generally indicate that the proposed elevated water storage size and elevation parameters are suitable to meet service pressure design basis criteria.

Table 9: Proposed Development – Model Observed Service Pressures with Elevated Storage

Demand Scenario	Model Observed Pressure		Design Basis Pressure		Meets Requirements
	Min. [kPa]	Max. [kPa]	Min. [kPa]	Max. [kPa]	
PHD	309	523	275	-	**Yes**
MDD	335	656	350	700	**Yes** [2]
ADD	336	686	350	700	**Yes** [2]

Notes:

1. Minimum and Maximum head simulation results are determined with elevated storage at low water level (LWL) and high water level (HWL), respectively. Results are evaluated over all of the hydraulic model nodes within the proposed development area model.
2. Model results for MDD and ADD service pressures meet the design criteria for maximum pressure and are slightly (4.2%) below normal design pressures (but superior to minimum peak hour design criteria). It is expected that a detailed design would include distribution system optimization and additional reinforcement through the network to meet the design basis pressures.

The model results demonstrate that the proposed elevated water tower operating levels provide a reasonable water service performance under normal operating demands. The results are dependent upon the distribution network providing the minimum looping and reinforcement summarized above, including the use of 300 mm diameter water mains from the tank outlet through the south development as summarized in **Figure 7**. The model conservatively omits some opportunities for additional looping through the road network rights of way.

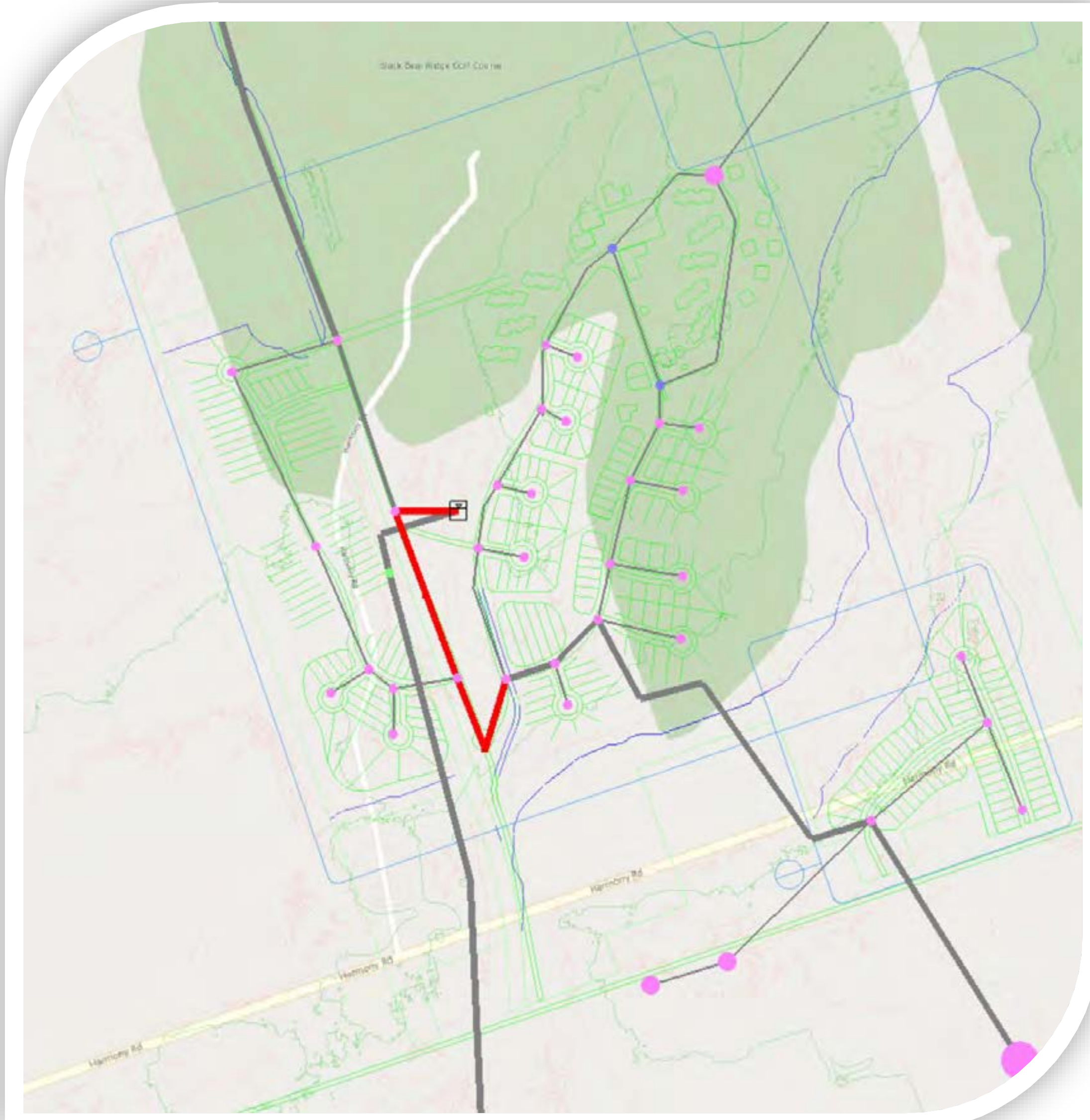


Figure 7: Hydraulic Model Representation of 300 mm Elevated Storage Outlet Through South Development Area (Red Highlight)

Water Transmission to Black Bear Ridge Development

The industry standard for pumping systems feeding elevated storage is to achieve maximum day demands while filling the elevated storage to top water level. The transfer of water into the proposed Black Bear Ridge development requires the capacity in the existing Belleville system to deliver both flow and pressure to achieve filling of the proposed elevated storage to top water level under maximum day background demand.

The proposed elevated water storage top water level elevation has been demonstrated to be sufficient for the design basis service pressures in the distribution system. The proposed development would be filled from the existing distribution system via a dedicated 450 mm water transmission main.

The nearest point of interconnection for a transmission main into existing 400 mm DI near Tank Farm Road and Short Road. This area is presently supplied via pumping station located on Cannifton Road between Highway 401 and Adam St. The required maximum day flow and pressure requirements at the proposed elevated tower in the Black Bear Ridge development is:

- MDD Demand: 103.481 L/s (**Table 8** and **Table 9**)
- HGL: 176.5 m
- Supply Pressure: 602.3 kPa

As noted in Part 1 of this memorandum, the requirement to deliver water at maximum day capacity into the proposed Black Bear Ridge development from the existing pressure zone serviced by the Cannifton Road pumping station will require an additional booster pumping station and transmission main to avoid over-pressure in the existing supply area.

The existing Cannifton Road pumping facility delivers approximately 16.05 L/s under maximum day demand within the existing pressure zone, according to the hydraulic model provided to Dillon. The MDD demands are met with the jockey pump operating at a duty point of 16.09 L/s at 14.08 m TDH into the existing discharge pressure reducing valve set for 525 kPa. The larger duty pump is capable of achieving greater flow and head required to deliver to the proposed transfer booster pumping station.

The proposed maximum day background demand would require the Cannifton Road pumping facility to deliver the present 16.05 L/s plus the Black Bear Ridge maximum day demand of 119.53 L/s to satisfy both the existing pressure zone and the filling requirements of the proposed development elevated water storage tank simultaneously. A model scenario was developed to review the impact of these demands on the existing Cannifton Road pumping facility, as well as to identify a nominal booster pumping duty point for providing adequate supply and transmission. The hydraulic model results under maximum day background demand for the transmission water main to the Black Bear Ridge elevated tower—including the large duty pump operating at the Cannifton Road and the Booster Pumping facilities—demonstrate steady state pressures below 850 kPa within the transmission system. The proposed transmission main operation under booster pumping is summarized in **Figure 8**.

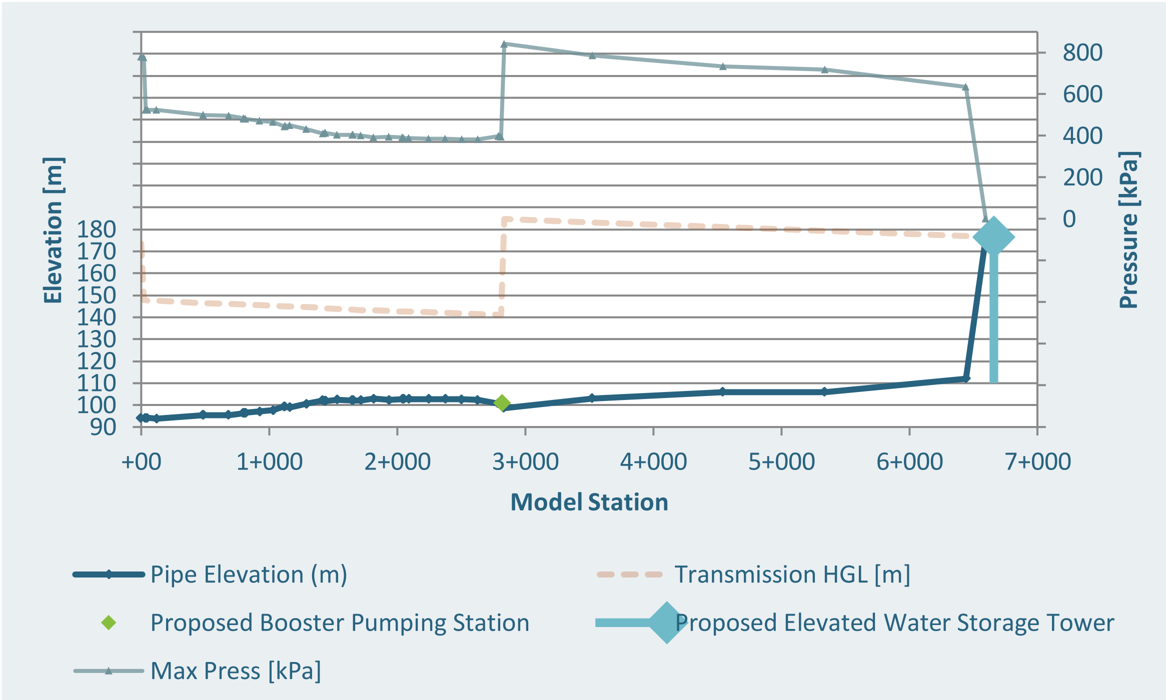


Figure 8: Black Bear Ridge Elevated Tower Booster Pumping and Transmission Water Main

The existing Cannifton Road pumping facility was simulated using the existing pump curves provided in the base model. The pumps discharge through the existing pressure reducing valve (PRV) with the setpoint unchanged at 525 kPa maximum discharge pressure. It is understood that the PRV provides the service pressure regulation within the existing pressure zone.

The operating conditions identified through modelling are summarized as follows:

- Existing Cannifton Road Pumping Facility Model Operating Conditions:
  - Duty Pump: TH1 (ASBS1 model pump definition)
  - Flow Rate: 125.82 L/s
  - TDH: 46.98 m
  - Discharge PRV Setting: 525 kPa (unchanged)
- Black Bear Ridge Booster Pump Model Conditions:
  - Duty Pump: Selection TBD during detailed design
  - Flow Rate: 103.48 L/s (109.733 L/s as modelled)
  - TDH: 43.62 m (as modelled)
  - Pump Power: 58.7 kW @ 80% Efficiency (approx. 80 hp).

The existing Cannifton Road pumping facility duty points under present MDD and proposed future MDD with water transfer to the Black Bear Ridge are summarized in **Figure 9.**

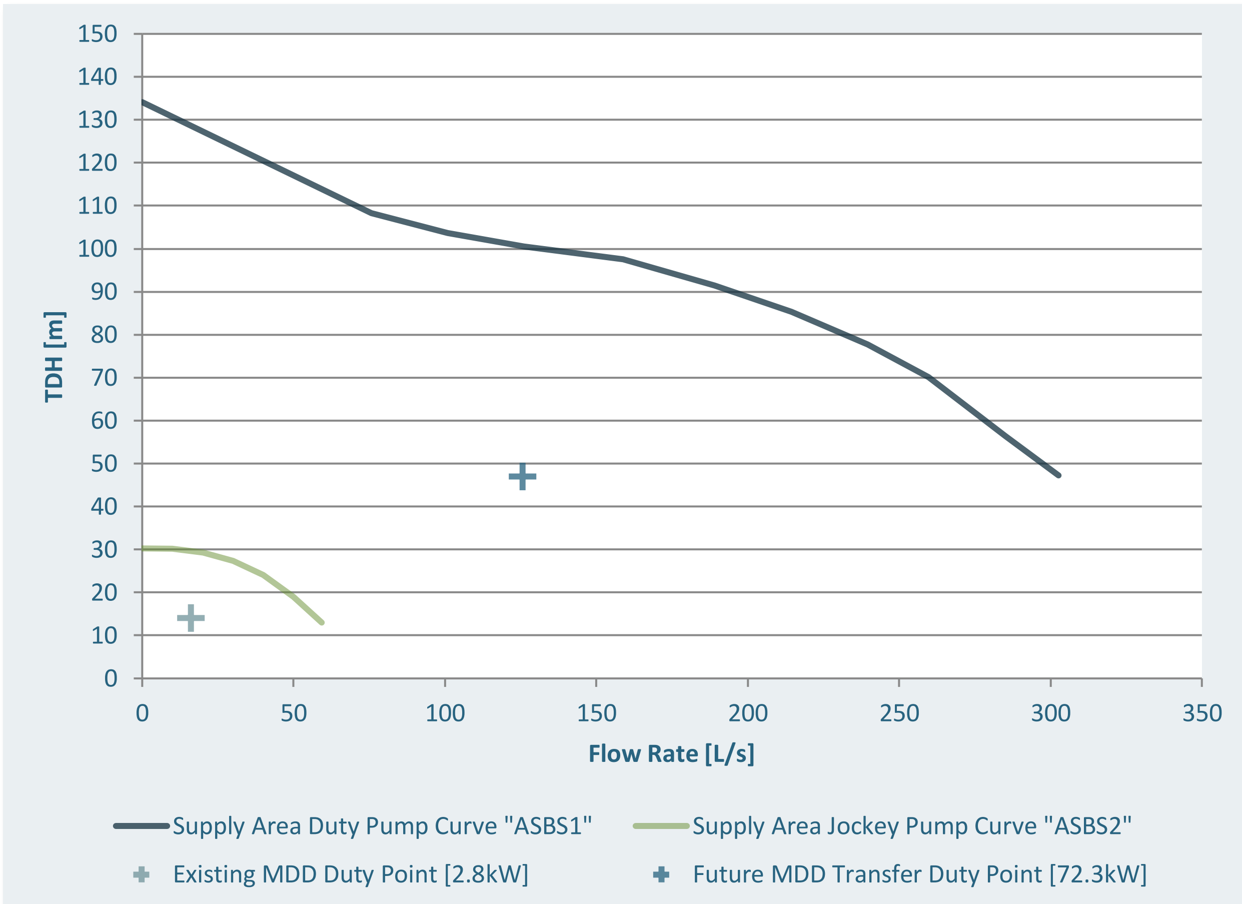


Figure 9: Model Representation of Existing Cannifton Road Pumping Facility Operation (Present and Proposed MDD)

Impact on Existing Distribution System Supply Pressure Zone

The estimated MDD service pressures in the existing pressure zone range between 316 kPa up to 515 kPa with a median of 390.5 kPa prior to development of the proposed Black Bear Ridge project. The impact of future MDD water transmission from the existing pressure zone to the Black Bear Ridge elevated water storage tower via booster pumping was evaluated in the hydraulic model. The impact of proposed water transfers, including operating the Cannifton Road pumping facility at 125.82 L/s but maintaining the existing PRV setting, suggests that the transfer can be achieved with minimal changes to the MDD service pressures in the existing pressure zone. The future service pressures range between 329 kPa up to 528 kPa with a median of 401 kPa during water transfers. These results exclude any future infill or planned development demands within the existing pressure zone serviced by the Cannifton Road pumping facility. The water model service pressures within the affected supply pressure zone are summarized in **Figure 10** and **Figure 11** for the existing and proposed future conditions, respectively.

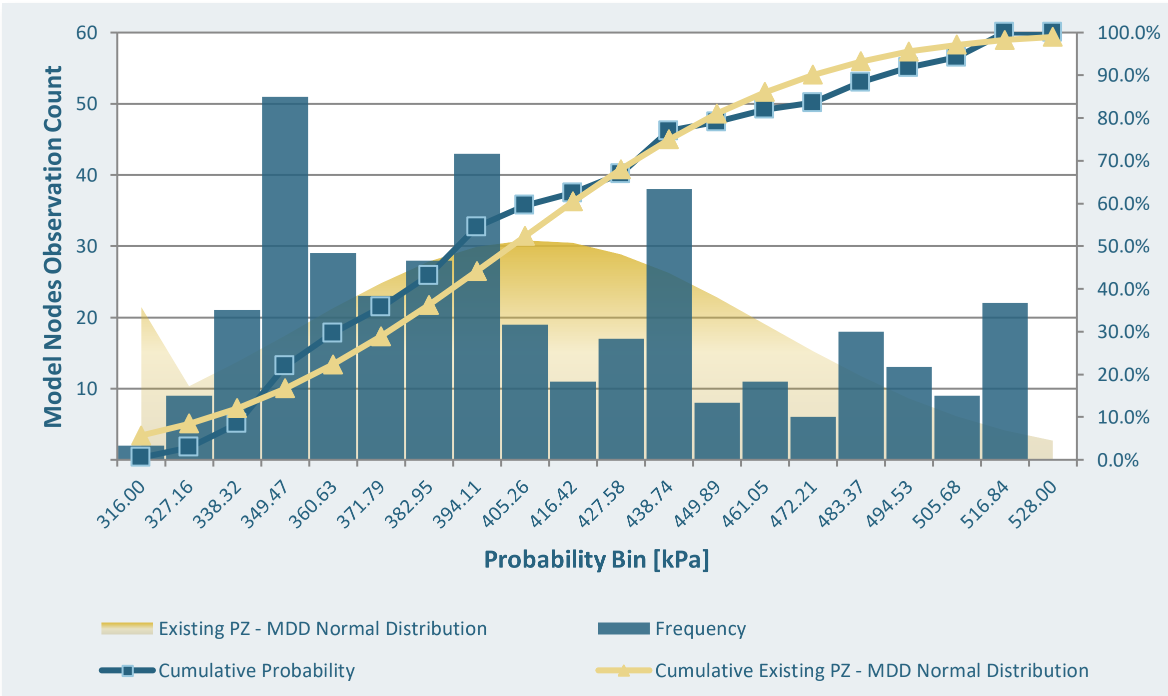


Figure 10: Existing Supply Pressure Zone Service Pressure (MDD Without Development)

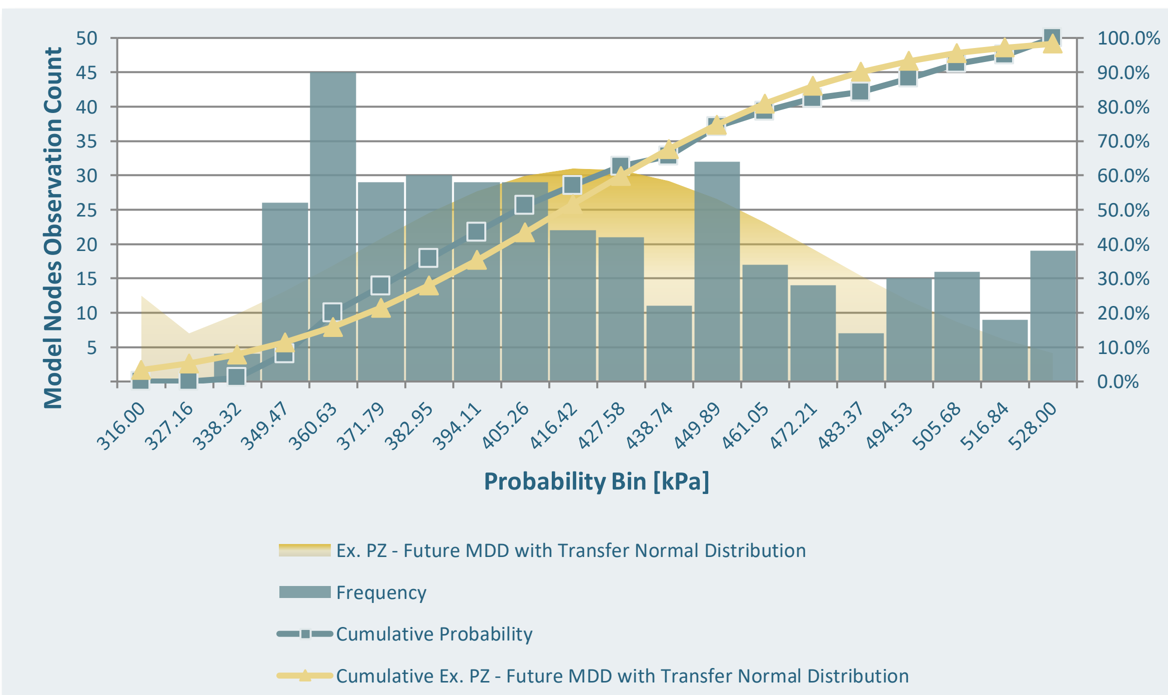


Figure 11: Existing Cannifton Road Supply Pressure Zone Service Pressure [kPa] (MDD With Transfer to Black Bear Ridge)

The existing distribution system infrastructure appears to be capable of providing the projected water transfer into the proposed Black Bear Ridge development. The service pressure within the existing pressure zone achieves similar pressure ranges under future development with the inclusion of a booster pumping station and increased use of the larger duty pumps at the existing Cannifton Road pumping facility.

Proposed Water System Supply Requirements (Concept Plan Buildout)

The proposed Black Bear Ridge development (including secondary water demand connections) will require water distribution system infrastructure expansions, including elevated water storage tower located in the development, a 450 mm diameter transmission water main connection from the existing 400 mm distribution main for a distance of approximately 3766 metres to a new elevated water storage tower. The transfer of water between the existing pressure zone serviced by the Cannifton Road pumping facility appears to be feasible with the existing duty pumps provided an additional booster pumping station is constructed along the water transmission main. Land rights of way and purchases along the transmission main may incur costs not otherwise considered in the proposed development.

- The interconnection requirements for the concept plan development phase are summarized as follows:
    - Maximum Day Demand (Transfer from existing pressure zone): 103.481 L/s
    - Supply Pressure (Booster PS Suction Pressure): > 395 kPa
    - Interconnection Point: Ex. 400 mm diameter (Short Road North of Tank Farm Road).
- Cannifton Road Pumping Facility Requirements:
    - Large Duty Pump active to meet MDD water transfer to proposed BPS
    - Maintain existing PRV and setpoint (with potential for optimization of setpoint).
- Proposed Booster Pumping Station:
    - Booster configuration with Duty/Standby pumping ON/OFF control
    - Elevated Water Storage Tank filling only – interconnect with SCADA level control
    - Pump Power: 58.7 kW (80 hp) ea.
    - Consideration for back-feeding existing pressure zone via PRSV chamber.
- Proposed Elevated Water Storage Tank Configuration:
    - Two-pipe configuration with top fill, bottom withdrawal
    - Top Water Elevation: 176.50 m HGL
    - Nominal Diameter: 20.64 m
    - Nominal Total Volume: 8270 m^3.

Attachment A

Development Area Figures

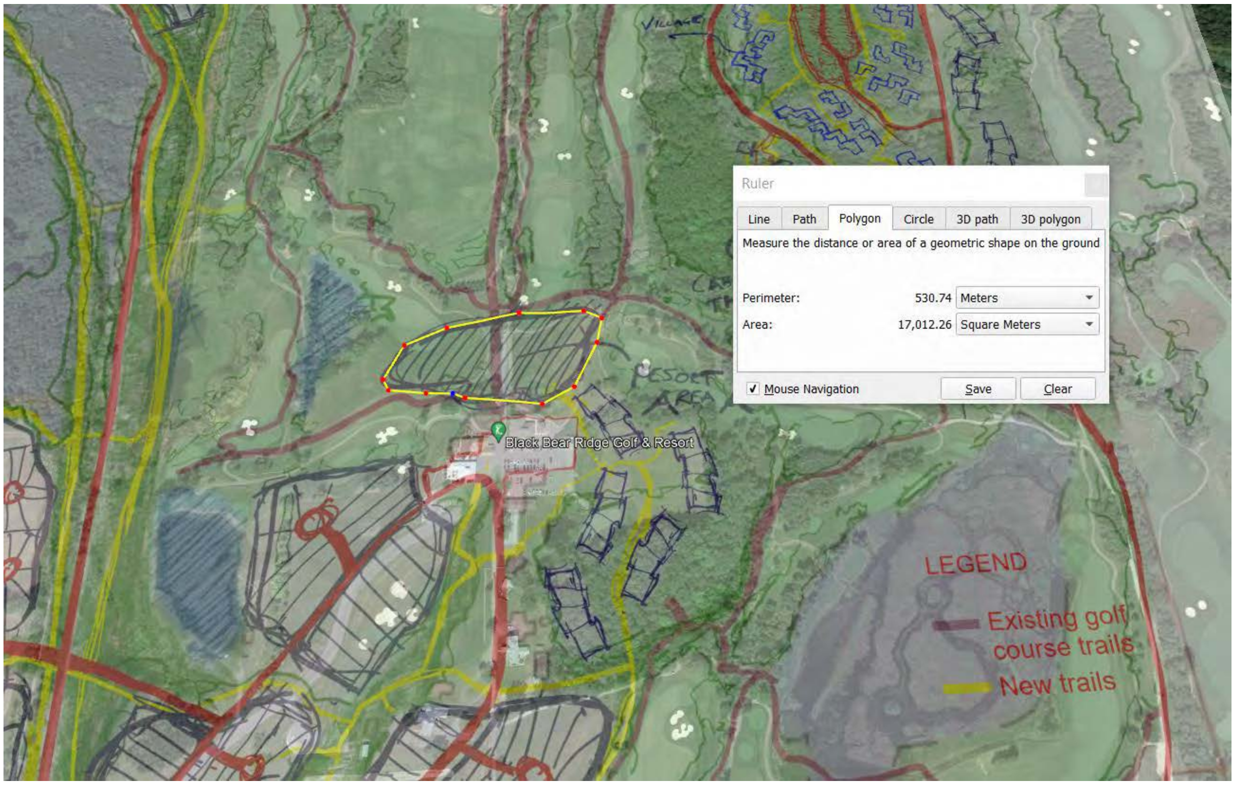


Figure A-12: Proposed Resort and Commercial Area A area: 17,013 m^2

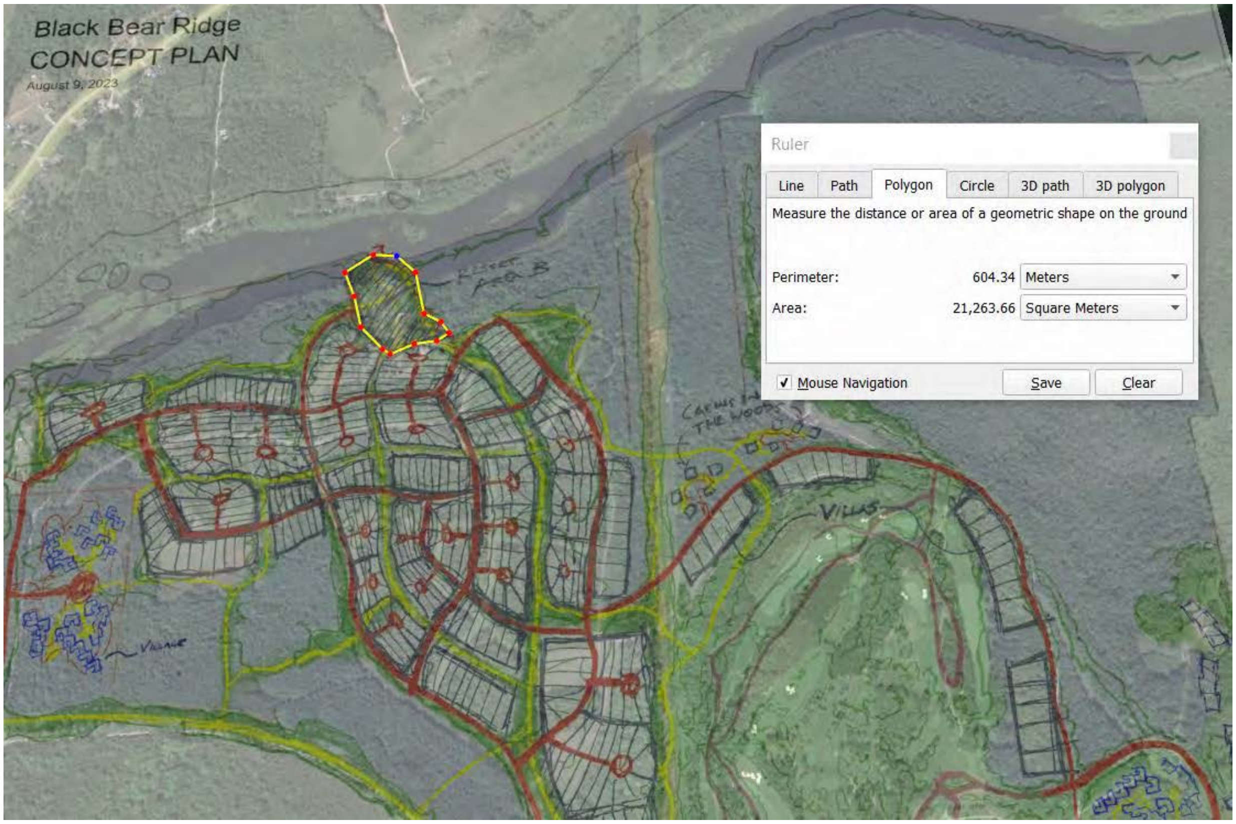


Figure A-13: Proposed Resort and Commercial Area B area: 21,264 m^2

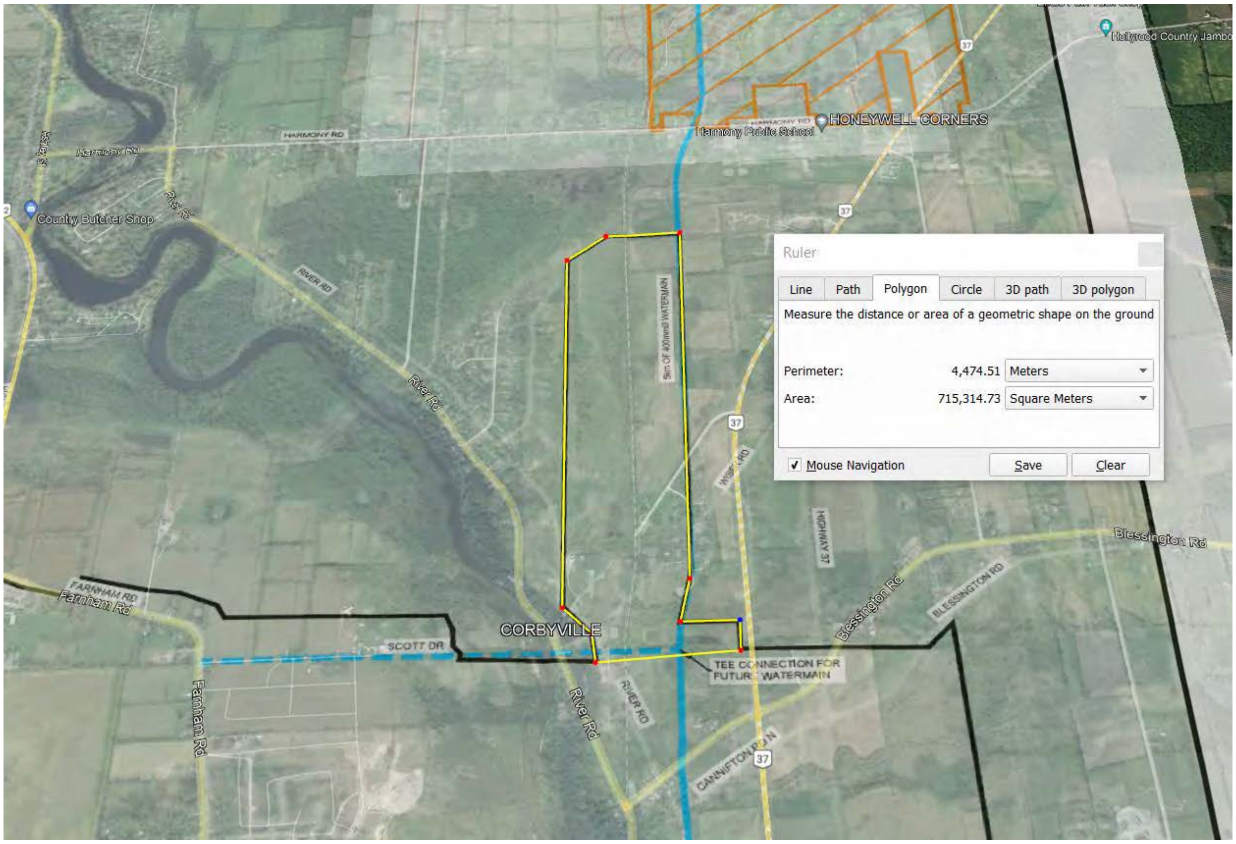


Figure A-14: James property area: roughly 715,315 m^2

Attachment B

Black Bear Ridge Servicing Feasibility Review (Jewell Engineering, February 28, 2022)

SERVICING FEASIBILITY REVIEW

Black Bear Ridge Development

City of Belleville

Final Report

February 28, 2022

Prepared by Bryon Keene, P.Eng.

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1 Background

Black Bear Ridge GP Inc. intends to build a large residential development on the Black Bear Ridge lands north of Harmony Road and west of the Trillium Woods Golf Course. The Black Bear Ridge lands cover an area of approximately 369ha and have frontage on Harmony Road as well as extensive water frontage along the Moira River (see Figure 1). The developers are contemplating an expansion of the golf course into a regional recreational resort featuring summer and winter activities, together with onsite accommodations and related services.

An initial residential component of 1,500 dwelling units of mixed density, proposed to be built out in phases based on market demand, is envisioned adjacent to the new regional resort. At full buildout the development could have as many as 3,049 units based on the most recent concept by Biglieri Group (Feb 18, 2022).

A large portion of the Black Bear Ridge lands is currently used for a 27 hole golf course. Other portions of the site are within environmental protection areas including both wetlands and floodplains. These are regulated by Quinte Conservation Authority and will be protected from any development. The limits of the environmental protection lands should be ground-truthed with topographic survey for floodplain areas and with the assistance of a biologist for the wetland areas.

The official plan identifies the lands are mostly recreational commercial with some rural land use and environmental protection (Figure 2). Proposed development within the rural land use designation would require an official plan amendment. Zoning (Figure 3) shows a mix of uses including community facility, rural, hazard, agricultural and environmentally sensitive lands.

The current work has been conducted at the request of the owners in advance of such a comprehensive study in order to determine the feasibility of expanding water and sewer servicing for the proposed development. The servicing study is intended to identify the City of Belleville infrastructure changes and the scale of potential costs that would be required in order to extend municipal services to meet the new demand. The local water distribution and sanitary collection systems have not been costed as part of this analysis.

Although the proposed BBR development is the focus of the analysis, Jewell also considered the potential for future demand on the system within the current urban area.

2 Proposed Development

The site owners are contemplating a development of approximately 3,049 dwelling units that would be primarily located within the northwest portion of the lands. Other potential areas include the east side adjacent Trillium Woods Golf Course and the south side below the BBR course.

A conceptual layout for the development was recently prepared by Biglieri Group. This shows a more intensive residential component that is distributed in smaller communities. The development also includes resort accommodations on 7.1ha of land and passive recreational land. There is no indication of the number of guests that could be accommodated in the resort and Jewell has estimated the number of guests to be 250. Additionally, an allowance of 38,900m^2 of gross floor area of commercial use for the development has been assumed.

It is anticipated that development will proceed in phases and take many years to be fully completed.

Stages of development are considered in this study to look at opportunities for servicing infrastructure to be constructed in phases as it becomes needed. The development phasing strategy is not yet in place and thus this recommendation for phasing is based on certain system limitations that would trigger additional infrastructure.

Due to the limitations in the existing gravity sanitary sewer, three stages are suggested.

Stage 1

The 300mm sanitary sewer from MH33 to MH40 must be replaced with a larger sewer in order to accommodate the first phase of construction. Approximately 850m sanitary sewer along River Road must be upsized. If development did not proceed beyond Stage 1, a replacement size of 450mm sewer would limit the development to about 1,200 homes and infiltration from about 90ha. Since the development is planned to be larger than 1,200 homes this section of sewer would need to be increased to 600mm diameter.

Stage 2

As development proceeds beyond 1,500 homes the next 840m section of existing 450mm sewer from MH11 to MH33 should be replaced. If development did not proceed beyond Stage 2 a 525mm sewer would be sufficient. This accommodates about 1,500 homes and infiltration from about 120ha of contributing area. Since the development is planned to be larger than this in Stage 3, a 600mm sanitary sewer pipe size is required.

Stage 3.

Any development beyond Stage 2 would require larger sanitary sewers for the two sections. The upperr reach must instead be replaced with 600mm sewers and the lower reach with 600mm sewers. Since Stage 3 is foreseen, the larger sewers should be installed. Stage 3 would include 3,049 dwelling units, the 250 person resort and 38,900m^2 of commercial use.

3 Servicing Options

Earlier investigations in the suitability of providing private water and septic found some limitations on water supply as it related to potential interference with existing wells. This was the case for the 17 unit condominium along the east side of BBR that was developed in 2020 where trickle systems were recommended to reduce the potential for interference. In addition, the density of development on private services would be in the order of 2 units per hectare and the maximum lot yield would be under 300 units.

To achieve the desired density of development, full water and sewer services must be provided. There are two possible means of servicing:

1) Communal

2) Municipal

Communal servicing requires a source of water and a means of disposal of septage. Such communal services are possible with on-site treatment facilities for both water and sewer. The Moira River would be a source of water for the drinking water supply. Treated sanitary flows could be disposed in-ground or to surface waters. Communal systems would require approval under the Ontario Water Resources Act and necessitate municipal responsibility agreements in the event the site operators fail to fulfill their obligations.

In early discussions with the City, it was determined that Belleville would have no interest in such a responsibility agreement. Further, concerns were expressed that any onsite disposal option for the treated sanitary flows would not be favourable.

Extension of municipal services remains as the only option available to realize the full potential of both the resort and the residential development lands.

Municipal water and sewer are present along Cannifton Road North approximately 700m north of the intersection of Tank Farm Road. A 300mm watermain and a 300mm sanitary sewer terminate just south of the pipeline crossing. Both the pipeline and Corbyville Creek must be crossed by the water and sewer extensions.

The extension of services would follow Cannifton Road North to the intersection of River Road at which point the services would divert to follow the former rail line that is now a snowmobile trail managed by the Eastern Ontario Trails Alliance. The trail continues north across Harmony Road and flanks the Black Bear Ridge property along the west side to the former rail bridge over the Moira River. This trail presents an excellent opportunity as a corridor for municipal services. The trail also marks the east extent of the long ‘finger’ of lands within Corbyville that form part of the City of Belleville’s urban centre.

Jewell completed a preliminary servicing strategy for water and sewer following the Ministry of the Environment’s guidelines:

- Design Guidelines for Drinking-Water Systems, 2008

- Design Guidelines for Sewage Works, 2008

Additional guidelines from the City of Belleville were also considered.

Information on the extent of the existing services was provided by City of Belleville. This included as-constructed engineering drawings and a study of the sanitary system completed by GGG in 2013. Jewell also received the City's EPANET water model.

4 Water Servicing

The City operates three remote storage facilities. They are referenced in this report as reservoirs, but are termed 'tanks' in the EPANET model. The three reservoirs are:

1. Pine Street 10,475m^3 max capacity
2. John Street 4,300m^3 max capacity
3. North Park Street 8,930m^3 max capacity

The Pine Street and North Park Street reservoirs are filled through the night using pumps on a schedule starting with North Park Street from 11:00PM until 2:00AM and Pine Street from 2:00AM until 4:30AM. The John Street reservoir is not filled on a schedule, but instead provides supplementary flow to the system during high demand periods and fills during low demand periods.

At the water treatment plant treated water is retained in a wet well and local storage is provided at the water treatment plant reservoir. This reservoir has a storage capacity of 4,500m^3.

The exchange of water between the wet well and the WTP reservoir is controlled by pumps and valves to maintain a wet well level of between 1.5m to 2.0m.

4.1 Proposed System

Jewell reviewed the EPANET water model and determined the water servicing strategy for Black Bear Ridge lands would require an elevated storage facility. Chapter 8.4.2 for water storage for systems providing fire protection gives guidance on the required storage volumes for an elevated tank. The required storage is calculated as a sum of:

A Fire Storage

B Equalization Storage (25% of maximum day demand)

C Emergency Storage (25% of A + B)

Because the existing City of Belleville system north of the 401 is operating with pressures in the lower end of the acceptable range, it was felt the best strategy would be for the new storage facility to be filled at night and supply all daily water demand for the proposed development. Thus, for this preliminary level of investigation the elevated storage tank volume should supply 100% of the max day demand plus fire flow and have an emergency storage reserve.

The City of Belleville required the extension of the watermain to be a minimum of 400mm diameter and to provide for a T connection for future looping at the extension of Scott's Road in Canniff Mills.

4.2 Demand

Assumptions:

- 3 Persons per household
- 350L/d per person

The number of single family dwelling units assumed is 3,049 for a total population of 9,147 persons. This represents an average day demand of 37.05L/s. The resort portion of the development is assumed to have 250 persons at 500L/p/day is per the OBC. This is equivalent to 1.45L/s average demand. The commercial use is estimated at 2.25L/s average demand plus an allowance for some reserve capacity. The total demand is rounded up to 42L/s. See Table 4-1 for the summary of demand calculations.

Table 4-1: Water Demand Summary

Water User	Assumption	Use	Demand (L/s)
Residential	9,147 People	350 L/p/day	37.05
Resort	250 Guests	500 L/p/day	1.45
Commercial	38,900m^2	5 L/m^2/day	2.25
Commercial Reserve	10,000m^2	5 L/m^2/day	0.58
Total Demand			**41.33 Rounded to 42 L/s**

** Storage Reservoir sizing is based on 41.33 L/s*

BBR demands were simulated following the demand Pattern 2 used for medium density residential developments. This is shown in Figure 4-1.

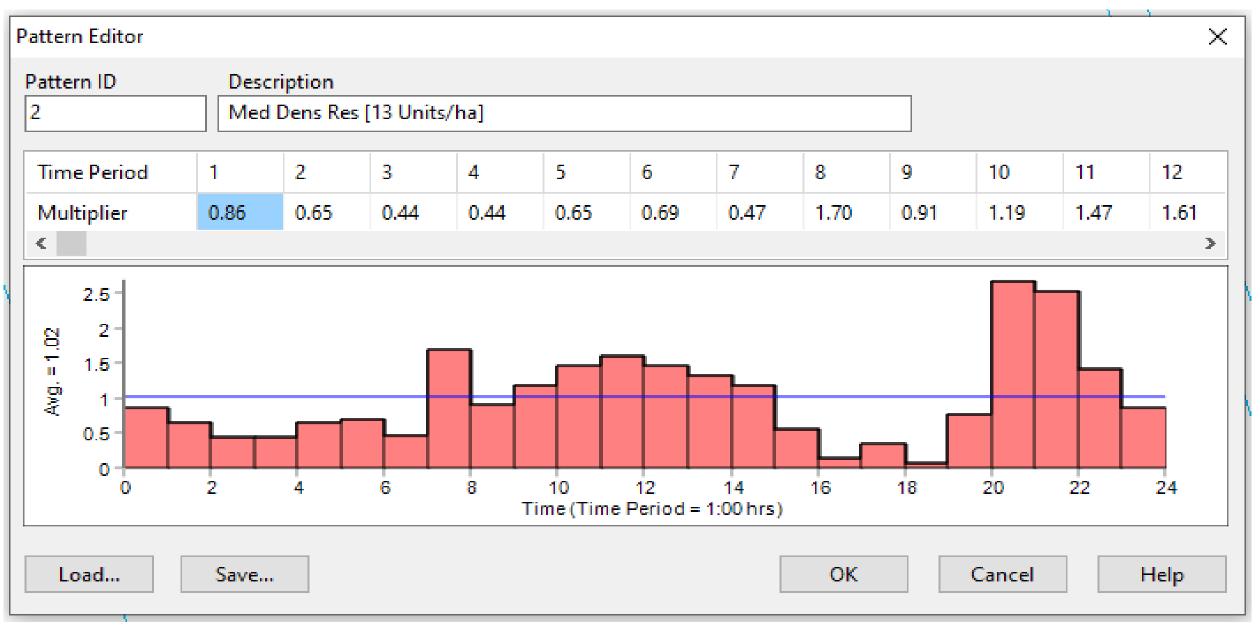


Figure 4-1: Local System Demand for BBR – following Demand Pattern 2

4.2.1 Fire flow

Fire flow requirements are guided by Table 8-1 (MOE, 2008) that lists a suggested flow rate of 189L/s for a population of 10,000 persons. The lesser population of 9,500 persons (residents + resort) would have a fire flow rate of 159L/s. Table 8-1 suggests a duration of 3 hours for the fire suppression storage. For conservatism, a fire flow rate of 189L/s is assumed for a duration of 3 hours.

The fire flow storage volume is 189L/s X 3 hours = 2,041m^3.

A = 2,041m^3

4.2.2 Average Day Demand (ADD)

Average demand is 41.33L/s. This is a total daily demand of 3,570,900L or 3,571m^3.

4.2.3 Max Day Demand (MDD)

The peaking factors from Table 3-1 (MOE, 2008) indicate a maximum day factor of 2.00 for a serviced population ranging from 3,001 to 10,000. Given the low system pressures north of the 401, it is assumed that all supply must come from the reservoir. Therefore, a maximum day demand is 2 X 3,571m^3 = 7,142m^3.

B = 7,142m^3

4.2.4 Peak Hour

The peak hour factor from Table 3-1 (MOE, 2008) is 3.00. The study concept is focussed on the ability of the City of Belleville system to supply a new reservoir from which a future distribution system would be tested for the capacity of delivering the required peak flow rates. The peak hour demand is not pertinent to the sizing of the storage facility and will be reviewed during the design of the distribution system for Black Bear Ridge.

4.2.5 Emergency Storage

Emergency storage is 25% of the sum of A and B = 25% of (2,041 + 7,142) = 2,296m^3.

C = 2,296m^3

4.3 Storage

The elevated tank should store, at a minimum, a volume of treated water equivalent to the sum of A + B + C = (2,041 + 7,142 + 2,296) = 11,479m^3. This is 3,036,800 US gal. Since there is planned to be a supplemental fill in the lower use period in the afternoon and some conservative assumptions were used in the demand calculations a 3,000,000 US gallon storage tank is sufficient.

This initial exercise investigates the potential storage criteria for an elevated tank to provide local system pressures assuming a night-time fill between 10:00PM and 7:00AM and usage following the demand pattern 2 in the City of Belleville model. The hourly demand is shown graphically in Figure 4-1.

In all cases, the City of Belleville system and the proposed new system at BBR should meet the criteria shown below (MOE, 2008 Section 10.2.2).

- Maximum Day plus Fire Flow Demand Pressure Minimum: 20 psi (140kPa)
- Peak Hour Demand Pressure Minimum: 40 psi (275kPa)
- Normal Operating Pressure: 50 psi (350kPa) – 70 psi (480kPa)
- Maximum Pressure: 100 psi (700kPa)

Water must be pumped from the distribution system up to the elevated tank. Three high lift pumps are proposed in a parallel arrangement modelled after the HL-5510 pumps in use at the Belleville plant. Pumping would begin at 10:00PM and continue until the tank is full.

A 3,000,000 US gallon tank was simulated. The elevation of the ground at the proposed location is 118m. The low water level in the tank should be 145m to meet minimum system pressures at BBR. The full storage elevation is 158.716m.

The model scenario was tested to review the ability of the City system to meet system demands with BBR at full development. The rate of fill will be limited to 105L/s by a flow control valve. An additional afternoon fill was added between 4:00PM and 7:00PM at a rate 25L/s. The filling summary is included in Table 4-2.

Table 4-2: Reservoir Filling Periods and Volumes

Reservoir Filling Period	Fill Rate (L/s)	Volume Supplied (ML)
10:00PM to 7:00AM	105	9,072
4:00PM to 7:00PM	25	270
Total Daily Supply		**9,342**

The model results indicate the net inflows to the reservoir vary from 45.4L/s to 86.5L/s. The system is capable of refilling the BBR tank daily. The demand of the reservoir assumes a water use of 42L/s (for some conservatism). As can be seen in Figure 4-2, the new BBR reservoir, the other remote reservoirs and the WTP storage (Wet Well and WTP Reservoir) can be sustained during the 4-day simulation. Jewell reviewed the system pressures and no new low pressure issues are induced.

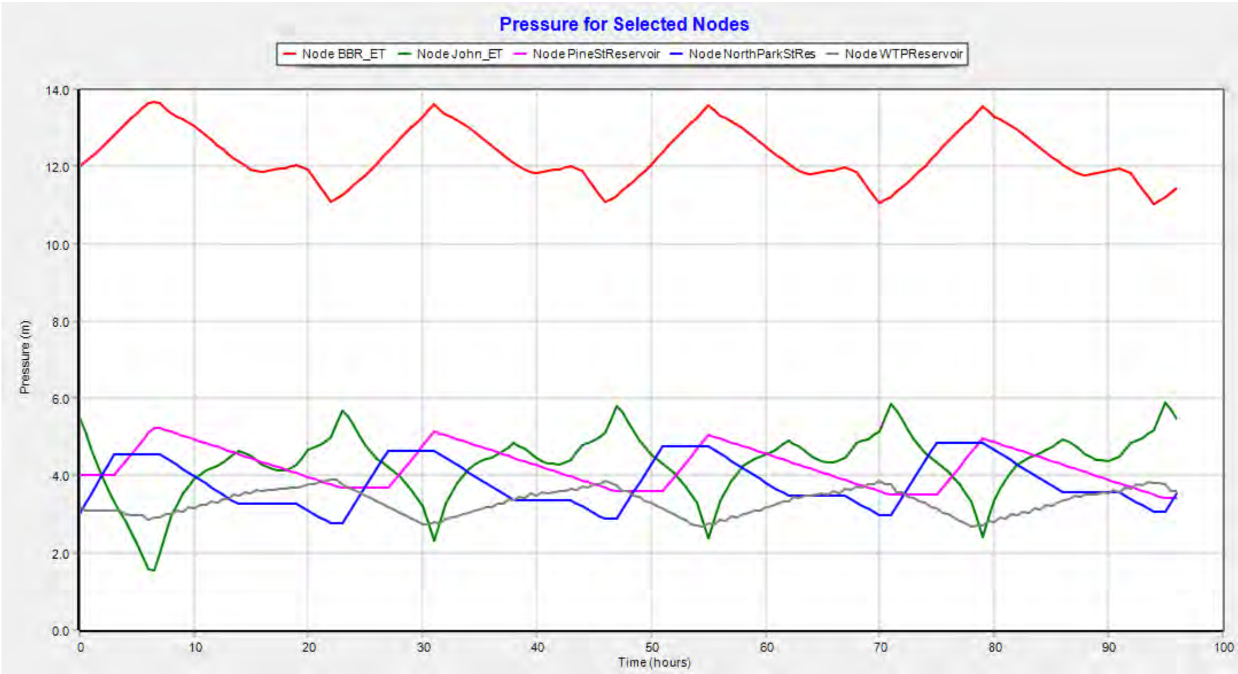


Figure 4-2: Time Series of Water Level in Remote Reservoirs with Proposed BBR Reservoir

Therefore, the BBR development can be serviced by municipal water assuming a 3,000,000 US Gal elevated tank with pumping station and 400mm watermain extension.

5 Sanitary Servicing

The potential to extend municipal sewer to the proposed BBR development was reviewed in light of capacity constraints in the existing Belleville system and demand from potential future growth.

5.1 Existing System

The City of Belleville sanitary sewers east of the Moira River and north of the 401 highway converge and cross the 401 highway as a 675mm pipe on Cannifton Road. The 675mm sewer extends north to the extension of Black Diamond Road where it is reduced at MH11 to a 450mm gravity pipe that continues along Cannifton Road to Maitland Road. The pipe size is again reduced to 300mm and continues north on Cannifton Road for 850m terminating at MH40.

Further north, for 300m a low pressure 50mm forcemain services several more residences.

At MH11 a 600mm pipe connects the sewers from the west side of Moira River through a siphon to another 600mm sewer.

The system was reviewed in 2013 by GGG on behalf of the City to determine potential constraints assuming full buildout of the urban area. GGG found several constraining sewers including:

- the 600mm pipes to and from the siphon and
- the 675mm pipes from MH11 to the highway 401 crossing.

The GGG investigators determined the siphon had the capacity for the projected growth, but the 600mm pipes did not. They also found the 675mm pipe would be at 172% capacity.

Jewell completed a full sanitary sewer spreadsheet model of the network to review the existing conditions with theoretical flow assumptions used by GGG and found:

- the 600mm pipes to and from the siphon are at 62% capacity on the west side and 72% capacity on the east side
- the 675mm pipes from MH11 to 401 crossing are at 62% capacity.

5.2 Constraints

Sanitary sewers would follow the same route as the proposed watermain extension from the existing termination points on Cannifton Road North to the snowmobile trail and along the trail into the BBR development. The BBR sewer would connect to the existing City sewers at MH40. This will require crossings of the pipeline corridor and Corbyville Creek.

5.2.1 Pipeline

A crossing of the pipeline is required. A gravity solution may conflict with the pipelines and their exact locations should be reviewed during a detailed feasibility study. A forcemain crossing is less of a concern because the elevation of the forcemain can be adjusted.

5.2.2 Corbyville Creek

The invert of the 300mm pipe at MH40 is 96.969m (per As-Constructed Dwgs). The bottom of the creek is approximately 98.6m (per the Floodrisk Mapping). This means that a gravity pipe could not be extended through the creek unless a siphon is used. A forcemain or siphon could be used. A siphon requires some grade differential from the upstream side to the downstream side. There is approximately 1m of grade difference and a siphon could be considered in conjunction with a gravity system. There would be no constraint on a forcemain crossing.

A permit will be required from the conservation authority for any crossing. Quinte Conservation encourages crossings to be completed with trenchless technologies such as a jack and bore solution. The presence of rock would add to the cost of crossing the creek.

5.2.3 Rock

The location of rock is known to be shallow in the vicinity of the current sewer based on the rock shots recorded on the As-Constructed drawings. No information is available to suggest the rock elevation along the proposed route, but it is suspected that deep excavations will encounter rock and increase costs. A geotechnical investigation of the proposed route is recommended.

5.2.4 Topography

A further constraint for a gravity system is a 2km long section of the route that has little grade difference. This constrains the slope and if a gravity pipe were used some segments would need to be buried about 6m to 7m deep.

5.2.5 Infiltration and Inflow

A fully serviced gravity solution must consider the infiltration and inflows along the route and serviced lands. Given that the City system has capacity constraints, a gravity solution will contribute more infiltration and inflows consuming capacity that could be allotted to population.

A proposed solution must limit the infiltration and inflows.

5.3 Servicing Concepts – Options

Jewell reviewed three different servicing strategies.

Option 1: Single pumping station on BBR lands with 5.7km of forcemain

Option 2: Pumping station on BBR with approx. 5km gravity and siphon

Option 3: Pumping station on BBR with approx. 5km gravity and second pumping station

Given the constraints discussed above, Jewell developed a solution following **Option 1**. A significant benefit of the forcemain is that there is no infiltration or inflow along the length of the forcemain and therefore more capacity is available for domestic flows. The forcemain can be buried shallower than a gravity pipe and can follow the land contour, whereas a gravity sewer must be deeper in some sections and will very likely encounter rock.

A single pumping station can be placed on BBR lands as shown in Figure 6.

5.4 Proposed System (Option 1)

The projected development was reviewed for three stages of development related to the capacity constraints in the existing gravity system for the 850m length of 300mm pipe from MH33 to MH40 and the 840m length of 450mm pipe from MH11 to MH33. The capacities are:

- 300mm section (MH33 to MH40) is 42.2L/s and
- 450mm section (MH11 to MH33) is 120.6L/s.

Sewage flows from the proposed development will exceed the capacity of each of the two pipe sections and each must be upsized to support the BBR development.

Given that the proposed full development will exceed the capacities of the trunk sewers north of the 401, Jewell considered a staged development to determine how many dwellings could be constructed before each of the two constraining sections of pipes must be increased.

To account for the growth of external areas, Jewell referenced the GGG study and assumed staged growth of the Cannifton region as shown on Figure 7 and discussed in Section 5.4.1.

5.4.1 External Lands

External developments that were considered to contribute flows for both Stages of BBR are 8B, 9A and the lands contributing to the Moira Lea pumping station.

Contributing to the gravity sewer MH33 – MH40:

- 8B with 553 persons and 10.53ha residential with 3,000m^2 of commercial floor area on 1ha of lands.
- 9A with 158 persons and 3ha of residential with 4,830m^2 of commercial floor area on 1.61ha of land.

Contributing to the gravity sewer MH11 – MH33:

- The Moira Lea pumping station contributes to the gravity sewer at MH33 and the pipes from MH11 – MH33 include pumped flows of 34.5L/s. Includes a population of 1411 persons and a residential area of 40.9ha, 12,900m^2 commercial floor area on 4.3ha of land.

5.4.2 Stage 1 – BBR with 1,200 Homes

The first stage assumes a population of 1,200 homes with 3 persons per dwelling = 3,600 persons. Extraneous flows were estimated on the basis of infiltration and inflow from 90ha of contributing lands.

- Population flows would be 49.2L/s assuming a Harmon peaking factor of 3.37.
- Infiltration and inflows are calculated at 0.28L/s/ha to be 25.2L/s
- **Total BBR flows are 74.4L/s**
- **Total flows at MH40 are 84.4L/s**

The 850m of existing 300m sanitary pipe from MH33 to MH40 must be upsized to permit any development of BBR. Jewell found that replacing the sewer with a 450mm pipe with slopes ranging from 0.19% to 0.23% would be adequate receive the expected flows. The 450mm pipe would have a capacity of **133.7L/s** if it replaced using an average slope of 0.22%.

However, ultimate development flows will be higher than this and the replacement size must be 600mm (see Stage 3).

5.4.3 Stage 2 – BBR with 1,500 Homes

This assumes 1,500 homes with a total population of 4,500 persons at full buildout covering an area of 120ha. Peak sewage flow, including infiltration and inflow, is 93.5L/s. With the addition of 8B, the peak flow will be 103.3L/s.

At this stage the existing 450mm pipes from MH11 to MH33 will have insufficient capacity and would need to be upsized to 525mm. The 525mm pipe with an average slope of 0.29% would have a capacity of **232.4L/s**.

Given that Stage 3 is the ultimate concept, the 525mm pipe would not be adequate and would also have to be increased to a 600mm pipe.

5.4.4 Stage 3 – BBR with 3,049 Homes and Resort Hotel with 38,900m² Commercial

The peak sewage flows at full buildout would exceed the capacity of both sections of pipe that were considered in Stages 1 and 2 and therefore larger sanitary sewers are required. The sewers north of the 401 highway from MH11 to MH33 and MH33 to MH40 must be increased to 600mm diameter. Peak flows were estimated using the standard sanitary sewer design sheet and include a peaking factor based on the Harmon formula. See Table 5-1.

Table 5-1: Proposed Sanitary Sewer Replacement – north of 401 Highway

Location	Manholes	Existing Size (mm)	New Size (mm)	Ave Slope	Peak Flow (L/s)	Capacity (L/s)	% Full
Maitland Drive to Pipeline	MH33 – MH40	300	600	0.22%	162.3	288	57.8
Siphon to Maitland Drive	MH11 – MH33	450	600	0.29%	193.2	331.8	58.2

5.5 Proposed Sanitary Servicing Summary

A Triplex T6 pump station with enclosure and backup generator is proposed. Descriptions of the sewer servicing is provided below for each of the two stages.

5.5.1 Stage 1 – 1,200 Homes

Pumping Station

The initial station will have:

- Ultimate duty capacity of 75 l/s @ 21m TDH (using two Forcemains – running two Gorman Rupp T6 6" pumps at 1300 rpm in parallel operation)
- Half capacity duty : 37.5 l/s @ 21m TDH (using one forcemain – running a single Gorman Rupp T6 6" pump at 1300 rpm)
- Quarter Capacity Duty : 18.75 l/s @ 14m TDH (using one forcemain – running a Single Gorman Rupp T6 6" pump at 1000 rpm) (easily done if station is equipped with VFD)

To run the Gorman Rupp T6A3S-B 6" pump at 1300 rpm will require a 25 HP motor. The electricity to operate (qty 2) T6A3S-B 6" pumps running in parallel at 1300 rpm will be 50 HP.

Forcemain

The forcemain configuration uses twin 200mm HDPE pipes discharging to MH40 for a total distance of 5.7km.

Gravity Sewer Improvements

The 300mm gravity sewer from MH33 to MH40 must be replaced to 600mm @ 0.22% slope.

5.5.2 Stage 2 – 1,500 Homes

Pumping Station

The next stage of the pumping station will increase the ultimate duty capacity to 95 l/s @ 26m TDH. The belts, shivs, starters and motors must be changed on the pumps.

Forcemain

No changes are required

Gravity Sewer

The 840m length of 450mm gravity sewer from MH11 to MH33 must be changed to 600mm at 0.29%, which increases the capacity to 331.8L/s.

5.5.3 Stage 3 – Full Development

The duty capacity must be increased to 153.2L/s. The pumps would need to be replaced.

A summary of the flows and capacities is shown in Table 5-2. The gravity sewer pipe changes are summarized in Table 5-3.

Table 5-2: Proposed Development Summary – Sanitary Servicing Strategy

Development Stage	Pop	Area (ha)	Peak Flow (L/s)	MH33 – MH40	MH11 – MH33
Pipe Size (mm)				600	600
Capacity (L/s)				**288.0**	**331.8**
BBR	9,500	147	157.5		
BBR + 8B			166.6		
9A + MLC Pump Stn			197.4		
Flow in Pipe (L/s)				166.6	193.2
Percent Full				57.8%	58.2%
Avail for 8A*				63.8L/s	L/s

** Assumes 80%*

Table 5-3: Gravity Pipe Changes per Stage

Stage	Manholes		Length	Existing Size	Proposed Size
	From	To	m	mm	mm
1	MH33	MH40	850	300	600
2	MH11	MH33	840	450	600

5.6 Development of 8A and 9A in Cannifton

Catchment 8A is the most northerly 'finger' of the urban lands that extends almost to Harmony Road. The development scenario of 8A includes:

Population of 189 persons on 5.48ha and

Commercial development of 68,040m^2 floor area on 22.68ha of land.

The total design flow is 20.9L/s including infiltration and inflow.

Catchment 9A is a large portion of Cannifton between Hwy 37 and Cannifton Road. At full development, 9A would have:

Population of 1,118 persons on 21.3ha and

Commercial / Industrial development of 93,600m^2 floor area on 31.2ha of land.

The total design flow is 37.2L/s including infiltration and inflow.

If 8A and 9A develop to full capacity, the proposed sanitary pipes from MH33 to MH40 recommended in Stages 1 and 2 will be at 68% capacity.

Other commercial and industrial development contemplated by the Official Plan and the GGG study in 9C, 9D, 9E, 10A and 11A would add to the demand in the 675mm pipe crossing Highway 401 putting that pipe at 108% capacity in the most constraining section. The Highway 401 crossing and downstream system will need to be reviewed as 9C, 9D, 9E, 10A and 11A are considered for development.

A gravity sewer cannot be extended to 8A and therefore a second pumping station will be required for 8A. The forcemain from the 8A pumping station can be directly connected to the forcemain from BBR.

6 Stormwater Management

The community that is conceived in the BBR Conceptual Master Plan encompasses a gross area of 369ha. Much of this includes the existing golf course and environmental areas as well as new parks and open space areas. The portions undergoing development includes up to 126.5ha at full development.

6.1 Quality Control

Quality Control facilities will treat runoff from the developing portions to an *Enhanced* treatment level. With reference to MOE 2003 Table 3-2 and assuming a 55% imperviousness for the developing area, Jewell calculates it would require a wet pond stormwater management facility with just over 24,000m^3 of storage to provide remedial water quality treatment to the *Enhanced* targets.

The total storage would be provided in up to three facilities distributed in several locations adjacent the topographical low areas.

6.2 Quantity Control

Lands that drain to the Moira River will require no quantity control. Lands that drain southerly contribute to a large wetland system that provides significant natural retention of runoff. The regulatory storm event is the 100-yr event. Typical storage volumes for control of the regulatory event are in the same scale of storage as the water quality. Quantity controls can often be accommodated within the lands set aside for quality treatment in combined facilities. The addition of quantity controls results in up to 1m depth of ponding over the permanent pool elevation and therefore is not restrictive on the pond size.

6.3 Land Requirement

A parcel set aside for stormwater management in the plan measures 2.64ha in area. This is a little larger than an area that would ordinarily be selected using 1ha per 10,000m^3 of storage. Storage volumes may be reduced using some Low Impact Design technologies that encourage infiltration.

6.4 Regulatory Requirements

Stormwater management facilities are approved under the Ontario Water Resources Act with an Environmental Compliance Approval through application to MECP. The plan is also reviewed by Quinte Conservation Authority and due to the proximity of the development to Hwy 37, MTO will also have an interest. The site does not drain to the MTO corridor and their interest will be reduced.

The City of Belleville will also review the stormwater management plan and provides approval through the plan of subdivision process.

6.5 Summary

There is sufficient land set aside for stormwater management. During detailed design smaller treatment areas may be identified that will require treatment through underground units or small filter strips or infiltration devices. Such areas are not land intensive and can be fit within the proposed land designations.

7 Conclusions

Municipal water and sewer services can be extended to the proposed Black Bear Ridge development to provide municipal servicing for up to 3,049 new homes on BBR lands. The water and sewer demands include:

- 3,049 Residential Dwelling Units (9,147 people) over 120ha of lands
- 250 Resort Hotel Guests
- 38,900m^2 GFA of Commercial Space
- 10,000m^2 of Reserve Commercial GFA

The existing limit of municipal servicing ends on Cannifton Road south of the intersection of River Road and the pipeline crossing. Both the sanitary and water mains are 300mm diameter and would not be sufficient to fully service the water and sewer needs of the development.

Water

Water servicing may be provided by connecting a 400mm watermain to the existing stub and extended 5.7km north to BBR. A water storage facility with a pumping station is required at BBR lands. This may be an elevated storage tank or an above ground tank with pumping station. The storage facility may be filled at night and will supply domestic and fire demands for the BBR development. Jewell found a 3,000,000 US Gal elevated tank filled from 10:00PM to 7:00AM at a flow rate of 105L/s with a supplemental filling period from 4:00PM to 7:00PM would meet the water needs.

Sanitary

Sanitary service requires replacement of 1,690m of existing undersized sewers with 600mm gravity pipes from MH11 to MH40. Gravity sewers cannot be extended to BBR and instead a pumped system is required. The pumping station will include a triplex pumping setup with 5.7km of twin 200mm forcemains to MH40.

Sufficient capacity can also be provided to accommodate the potential growth within the current urban boundary. The growth forecast was interpreted from the 2013 GGG study commissioned by the City.

Stormwater

Stormwater management needs were reviewed for provision of water quality and quantity control. The water quality target for the region is Enhanced. Jewell found that a wet pond with 24,000m^3 of storage including 18,975m^3 of retention and 5,060m^3 of extended detention would achieve the quality control targets. Water quantity controls are only required where the discharge is not to the Moira River. Sufficient land area would be available in the quality treatment ponds that would serve as combined facilities for water quality and quantity control. A depth of 1m of active storage would be sufficient. Opportunities for infiltration may be explored during detailed design stages that would reduce the quantity control storage volumes needed.

Costs for Extending Sanitary and Water

The Construction costs for the extension of municipal water and sanitary sewer to the BBR development have been estimated at $7.7M for the sanitary servicing and $10.8M for the water servicing. The costs include 15% contingency. Costs do not include design and environmental assessment costs. These may be estimated at an additional 15% of construction cost. The local servicing costs within the BBR development are also not included.

Table 7-1: Sanitary Servicing Cost Estimate

PRELIMINARY COST ESTIMATE City of Belleville Black Bear Ridge Development Watermain and Sanitary Sewer Servicing Study				
Item (Sanitary Sewers)	**Unit**	**Estimated Quantity**	**Estimated Cost per Unit**	**Total**
Mobilization and Demobilization	Lump Sum	1	$100,000.00	$100,000.00
Twin 200mm HDPE Sewer Forcemain (Open Cut Trench)	m	5800	$450.00	$2,610,000.00
Twin 200mm HDPE Sewer Forcemain (Directional Drilling @ Corbyville Creek)	m	50	$7,500.00	$375,000.00
Twin 200mm HDPE Sewer Forcemain (Directional Drilling @ Gas Pipe LIne)	m	50	$10,000.00	$500,000.00
600mm PVC Sanitary Sewers	m	838	$750.00	$628,500.00
1200mm Precast Concrete Maintenance Holes	each	16	$6,000.00	$96,000.00
Connect to Existing Sewer Systems	each	1	$2,500.00	$2,500.00
Above Grade Sanitary Sewer Pumping Station and Controls	Lump Sum	1	$1,500,000.00	$1,500,000.00
Reconnect Existing Sewer Services	each	110	$500.00	$55,000.00
Urban Roadway Restoration (3.5m Lane Width)	m	2390	$325.00	$776,750.00
Trail Restorations (150mm Granular 'A')	m	5200	$10.00	$52,000.00
			Sub-Total	**$6,695,750.00**
			15% Contingency	**$1,004,362.50**
TOTAL				***$7,700,112.50***

Table 7-2: Water Servicing Cost Estimate

PRELIMINARY COST ESTIMATE
City of Belleville
Black Bear Ridge Development
Watermain and Sanitary Sewer Servicing Study

Item (Watermains)	Unit	Estimated Quantity	Estimated Cost per Unit	Total
Mobilization and Demobilization	Lump Sum	1	$100,000.00	$100,000.00
400mm PVC Watermain (Open Cut Trench)	m	4900	$400.00	$1,960,000.00
400mm PVC Watermain (Directional Drilling @ Corbyville Creek)	m	50	$8,000.00	$400,000.00
400mm PVC Watermain (Directional Drilling @ Gas Pipeline)	m	50	$12,000.00	$600,000.00
400mm Gate Valve in Chamber	m	5	$12,500.00	$62,500.00
Fire Hydrant Assemblies	m	5	$7,500.00	$37,500.00
Air Release Valve in Chamber	each	2	$10,000.00	$20,000.00
Drain Valve in Chamber	each	2	$15,000.00	$30,000.00
Connect to Existing Watermain	Lump Sum	1	$5,000.00	$5,000.00
Water Tank and Controls	each	1	$6,000,000.00	$6,000,000.00
Urban Roadway Restoration (3.5m Lane Width)	m	400	$325.00	$130,000.00
Trail Restorations (150mm Granular 'A')	m	4600	$10.00	$46,000.00
			Sub-Total	**$9,391,000.00**
			15% Contingency	**$1,408,650.00**
TOTAL				***$10,799,650.00***

The water tank cost is estimated based on the on-ground concept

Prepared and Submitted by:

Bryon Keene

Bryon Keene, P.Eng.

Jewell Engineering Ltd.

LICENSED PROFESSIONAL ENGINEER
B. G. KEENE
Feb.28/22
PROVINCE OF ONTARIO

APPENDIX A

Figures

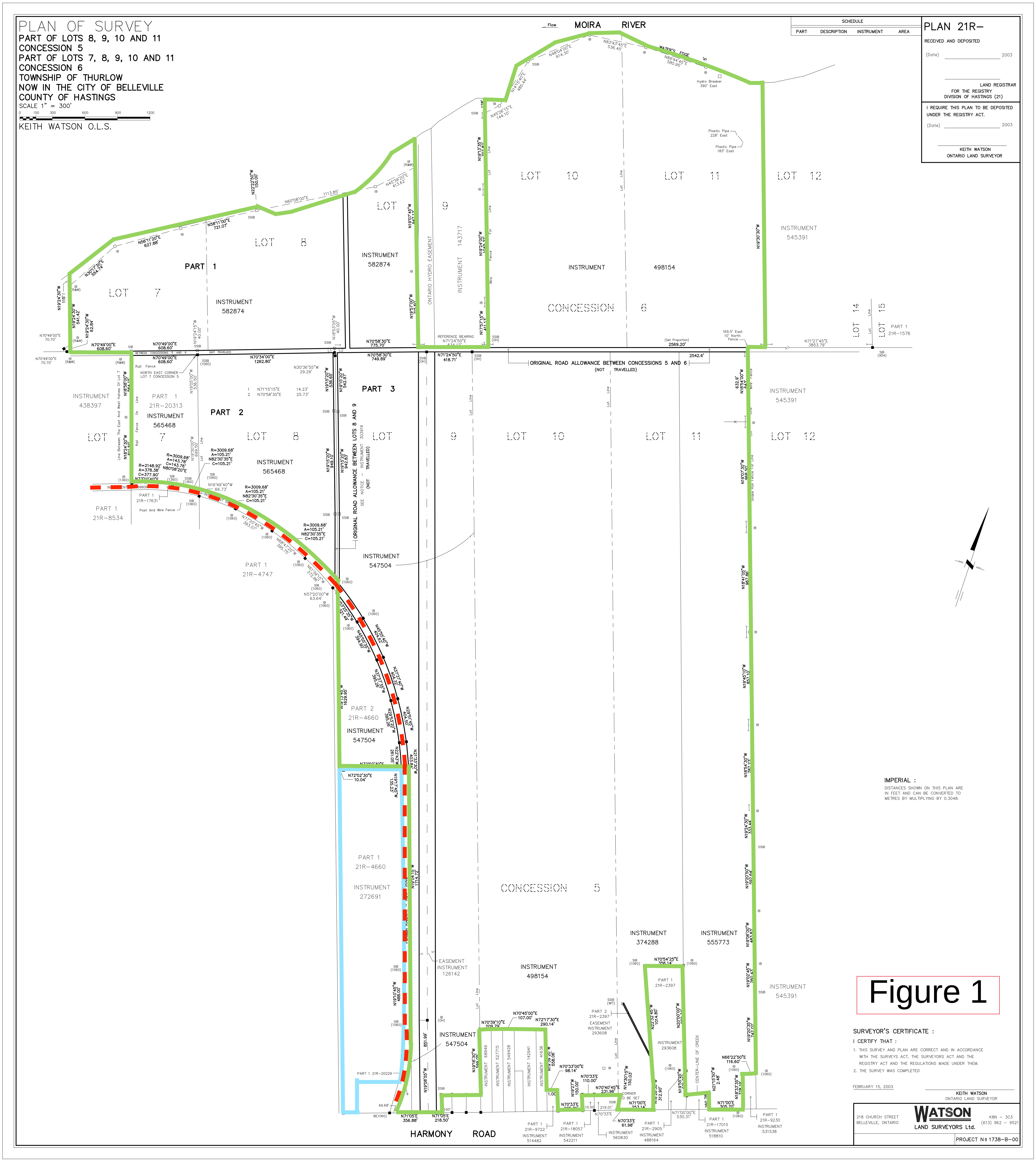

PLAN OF SURVEY
PART OF LOTS 8, 9, 10 AND 11
CONCESSION 5
PART OF LOTS 7, 8, 9, 10 AND 11
CONCESSION 6
TOWNSHIP OF THURLOW
NOW IN THE CITY OF BELLEVILLE
COUNTY OF HASTINGS
SCALE 1" = 300'
KEITH WATSON O.L.S.
MOIRA RIVER
PLAN 21R–
RECEIVED AND DEPOSITED
LAND REGISTRAR
FOR THE REGISTRY
DIVISION OF HASTINGS (21)
I REQUIRE THIS PLAN TO BE DEPOSITED UNDER THE REGISTRY ACT.
KEITH WATSON
ONTARIO LAND SURVEYOR
SCHEDULE
PART
DESCRIPTION
INSTRUMENT
AREA
PART 1
PART 2
PART 3
LOT 7
LOT 8
LOT 9
LOT 10
LOT 11
LOT 12
CONCESSION 6
CONCESSION 5
INSTRUMENT 582874
INSTRUMENT 498154
INSTRUMENT 545391
INSTRUMENT 565468
INSTRUMENT 547504
INSTRUMENT 272691
INSTRUMENT 438397
INSTRUMENT 374288
INSTRUMENT 555773
ONTARIO HYDRO EASEMENT
ORIGINAL ROAD ALLOWANCE BETWEEN CONCESSIONS 5 AND 6 (NOT TRAVELLED)
ORIGINAL ROAD ALLOWANCE BETWEEN LOTS 8 AND 9
PART 1 21R-20313
PART 1 21R-8534
PART 1 21R-4747
PART 2 21R-4660
PART 1 21R-4660
EASEMENT INSTRUMENT 126142
HARMONY ROAD
IMPERIAL :
DISTANCES SHOWN ON THIS PLAN ARE IN FEET AND CAN BE CONVERTED TO METRES BY MULTIPLYING BY 0.3048.
SURVEYOR'S CERTIFICATE :
I CERTIFY THAT :
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGISTRY ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED
FEBRUARY 15, 2003
KEITH WATSON
ONTARIO LAND SURVEYOR
218 CHURCH STREET
BELLEVILLE, ONTARIO
WATSON
LAND SURVEYORS Ltd.
K8N – 3C3
(613) 962 – 9521
PROJECT № 1738-B-00

Figure 1

F:\FTP\PROJECT FILES\1 CIVIL 3D PROJECT FILES\2104954 - SPIRE CAPITAL INVESTMENTS - BLACK BEAR RIDGE\2 - MODELS\2104954 BLACK BEAR RIDGE INC.- NORTH LOTS - SEPTEMBER 2021 10/01/2021

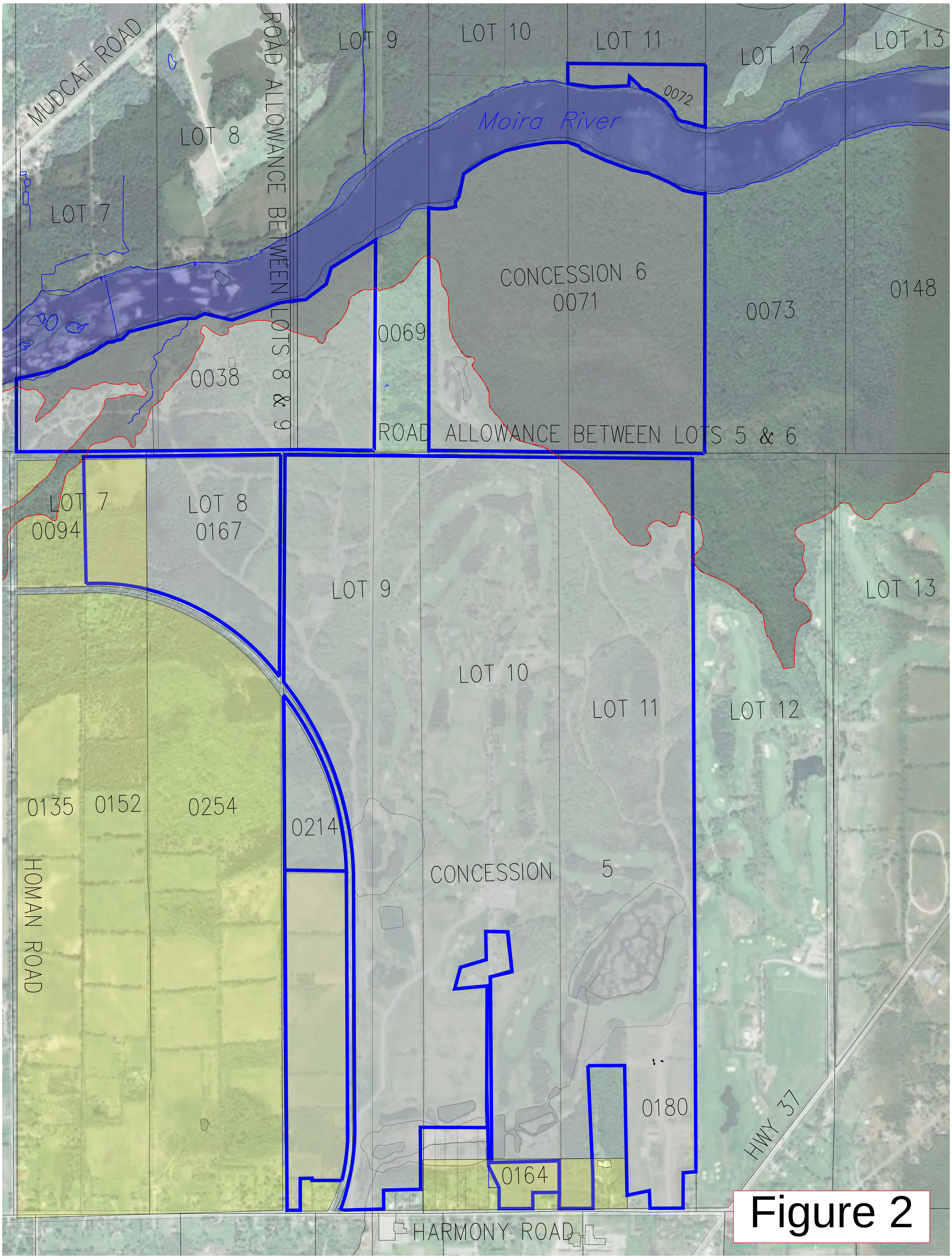


Figure 2

JEWELL ENGINEERING

GENERAL NOTES:

ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.

ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.

EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

ALL DIMENSIONS SHOWN ARE IN METRES OR MILLMETRES, UNLESS OTHERWISE NOTED.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

LEGEND:

- LANDS OWNED BY BRIAN MAGEE
- ENVIRONMENTAL PROTECTION AREA
- RECREATIONAL COMMERCIAL LAND USE
- RURAL LAND USE
- FLOODLINE - QUINTE CONSERVATION

RESIDENCES AT BLACK BEAR RIDGE

OFFICIAL PLAN CONSTRAINT MAP

DRAWN BY: BNW
PROJECT NO: 210-4954
DESIGNED BY: BNW
DATE: September 2021
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE: HORIZONTAL - VERTICAL - NOT TO SCALE
DRAWING NO: 01

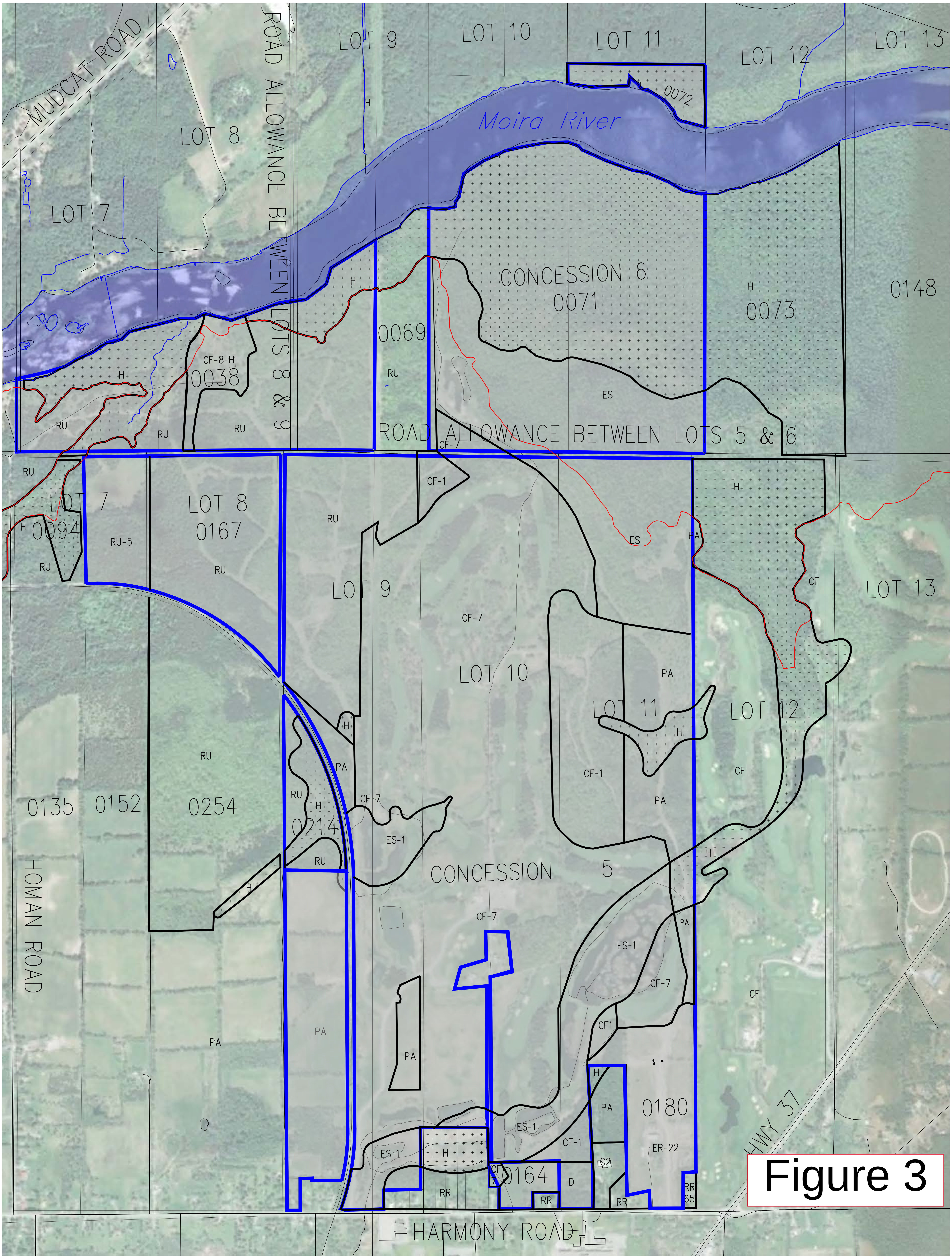


Figure 3

GENERAL NOTES:

ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.

ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.

EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

ALL DIMENSIONS SHOWN ARE IN METRES OR MILLMETRES, UNLESS OTHERWISE NOTED.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

LEGEND:

LANDS OWNED BY BRIAN MAGEE

ZONED HAZARD

FLOODPLAIN - QUINTE CONSERVATION

RESIDENCES AT BLACK BEAR RIDGE

ZONING CONSTRAINT MAP

DRAWN BY: BNW

PROJECT NO: 210-4954

DESIGNED BY: BNW

DATE: September 2021

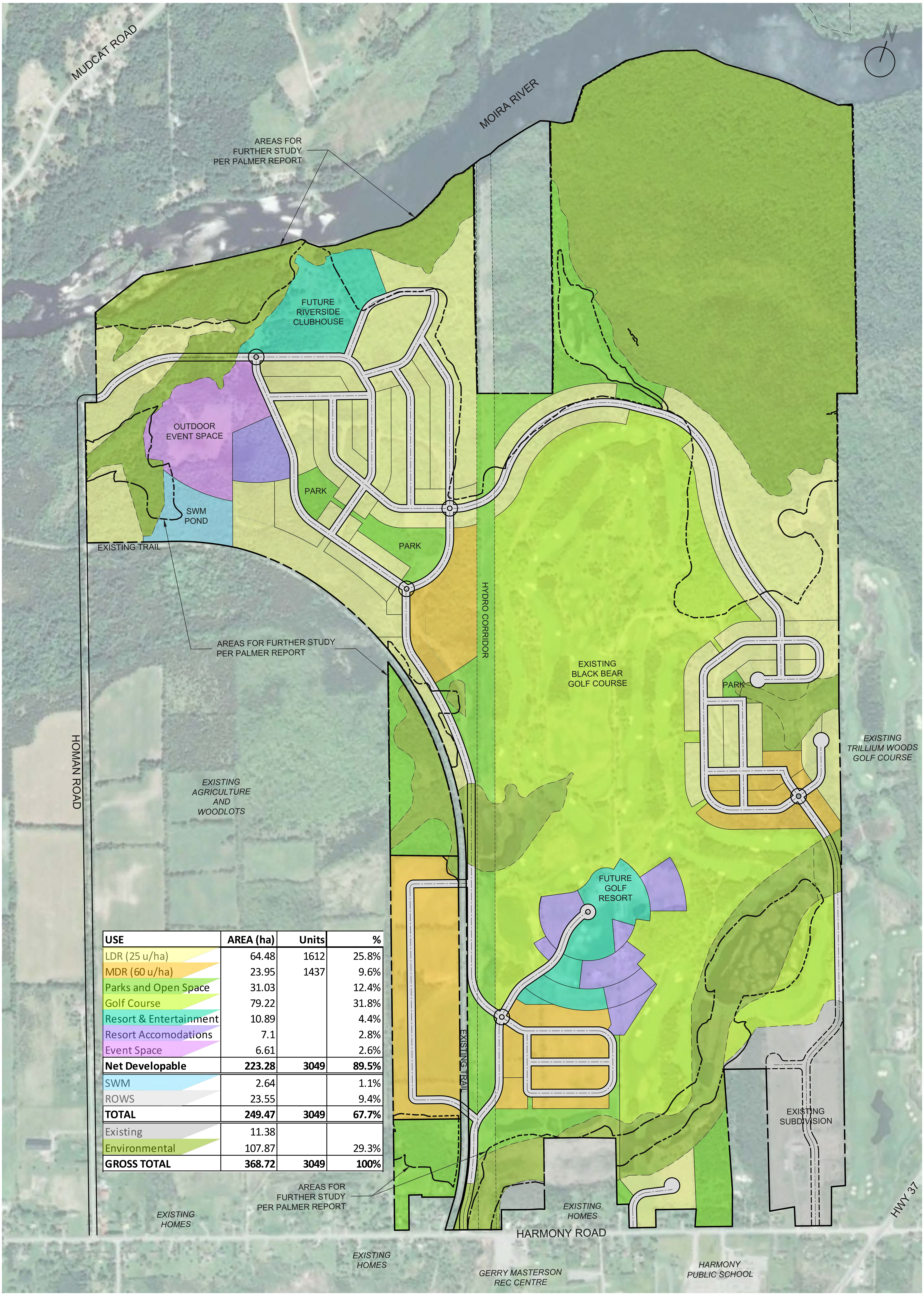
CHECKED BY: BK

CONTRACT NO:

APPROVED BY: BK

SCALE: HORIZONTAL - VERTICAL - NOT TO SCALE

DRAWING NO: 02



USE	AREA (ha)	Units	%
LDR (25 u/ha)	64.48	1612	25.8%
MDR (60 u/ha)	23.95	1437	9.6%
Parks and Open Space	31.03		12.4%
Golf Course	79.22		31.8%
Resort & Entertainment	10.89		4.4%
Resort Accomodations	7.1		2.8%
Event Space	6.61		2.6%
Net Developable	**223.28**	**3049**	**89.5%**
SWM	2.64		1.1%
ROWS	23.55		9.4%
TOTAL	**249.47**	**3049**	**67.7%**
Existing	11.38		
Environmental	107.87		29.3%
GROSS TOTAL	**368.72**	**3049**	**100%**

BLACK BEAR RIDGE CONCEPTUAL MASTER PLAN

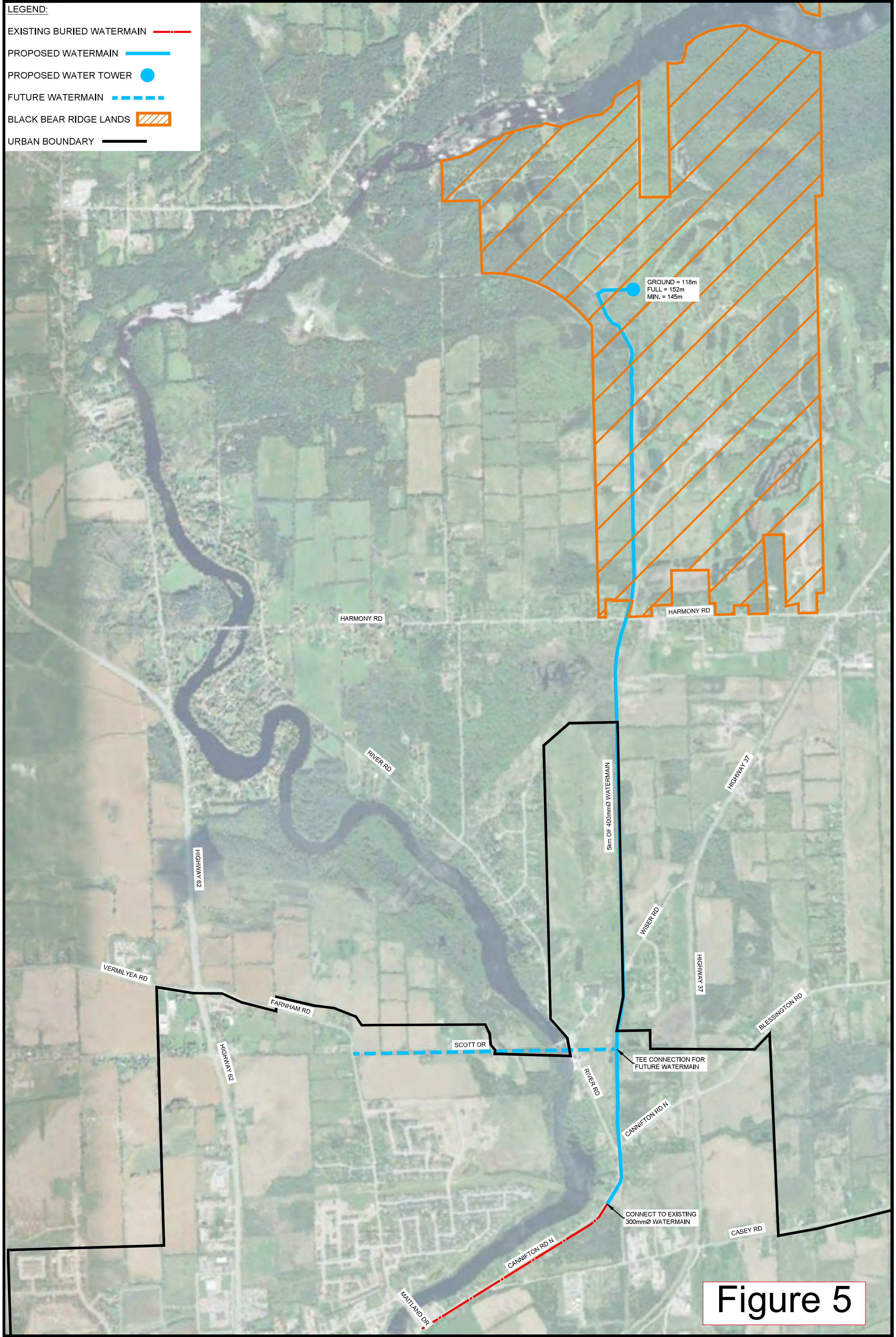
ADDRESS: 501 HARMONY ROAD, BELLEVILLE

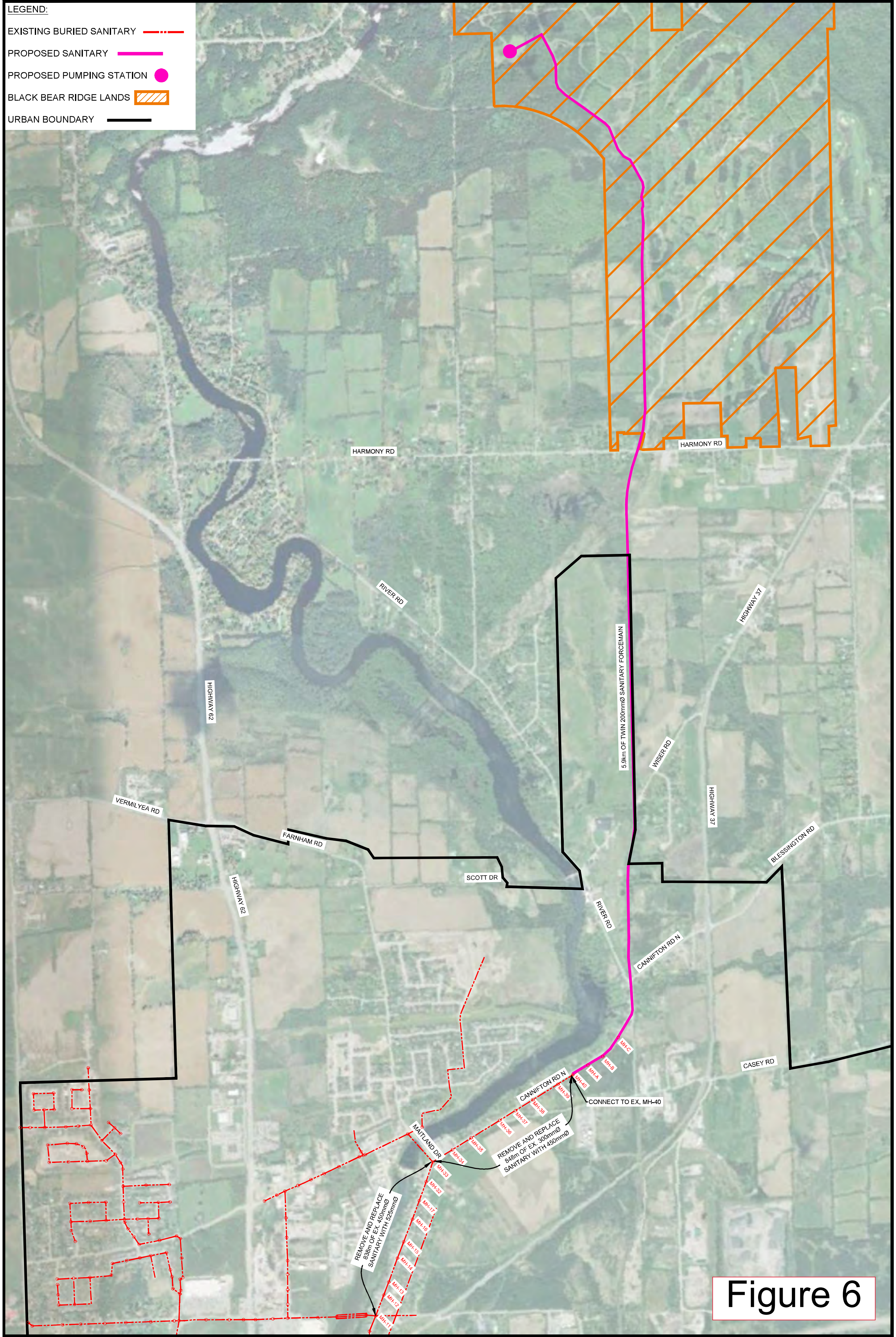
Project No: 22826

February 25, 2022
Scale: 1 : 4000

Designed By: MA

BIGLIERI GROUP


LEGEND:
EXISTING BURIED WATERMAIN
PROPOSED WATERMAIN
PROPOSED WATER TOWER
FUTURE WATERMAIN
BLACK BEAR RIDGE LANDS
URBAN BOUNDARY
GROUND = 118m
FULL = 152m
MIN. = 145m
HARMONY RD
HARMONY RD
RIVER RD
5km OF 400mmØ WATERMAIN
HIGHWAY 37
HIGHWAY 62
WISER RD
HIGHWAY 37
VERMILYEA RD
FARNHAM RD
BLESSINGTON RD
SCOTT DR
TEE CONNECTION FOR FUTURE WATERMAIN
HIGHWAY 62
RIVER RD
CANNIFTON RD N
CONNECT TO EXISTING 300mmØ WATERMAIN
CASEY RD
CANNIFTON RD N
MAITLAND DR
Figure 5



LEGEND:
EXISTING BURIED SANITARY
PROPOSED SANITARY
PROPOSED PUMPING STATION
BLACK BEAR RIDGE LANDS
URBAN BOUNDARY
HARMONY RD
HARMONY RD
RIVER RD
HIGHWAY 37
5.9km OF TWIN 200mmØ SANITARY FORCEMAIN
WISER RD
HIGHWAY 62
HIGHWAY 37
VERMILYEA RD
FARNHAM RD
SCOTT DR
BLESSINGTON RD
HIGHWAY 62
RIVER RD
CANNIFTON RD N
CASEY RD
CANNIFTON RD N
CONNECT TO EX. MH-40
MAITLAND DR
REMOVE AND REPLACE 848m OF EX. 300mmØ SANITARY WITH 450mmØ
REMOVE AND REPLACE 838m OF EX. 450mmØ SANITARY WITH 525mmØ

Figure 6

F:\FTP\PROJECT FILES\1 CIVIL 3D PROJECT FILES\1904502 - SETTLERS RIDGE\SETTLERS RIDGE DRAINAGE STUDY-CURRENT_1_6130_7916462B 04/10/2019

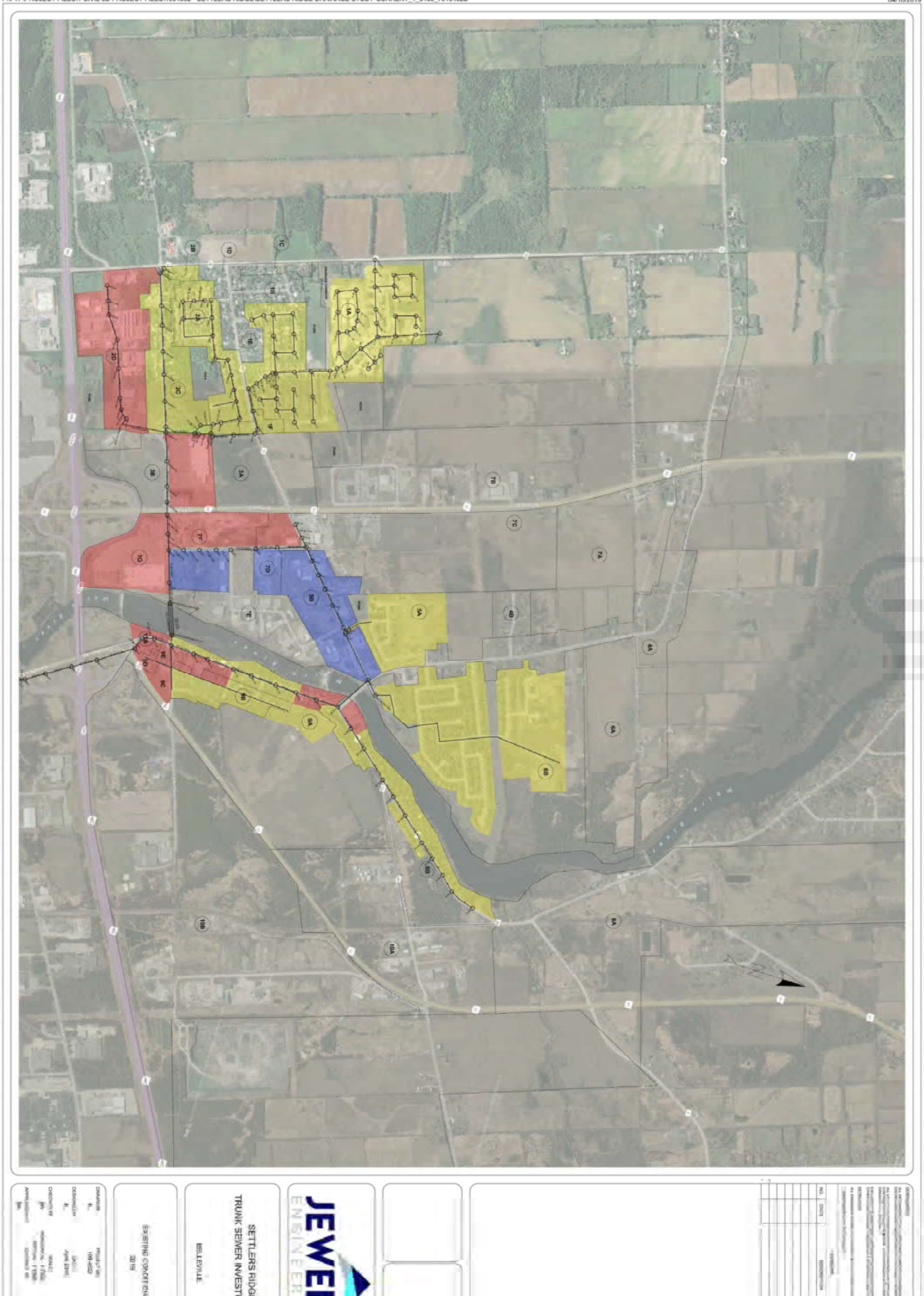


Figure 7

APPENDIX B

EPANET Rules

Existing and Proposed

EPAnet Rule Changes

Original Rules	Changed
RULE 1 ;Plant Reservoir Return Valve Control IF TANK HL_Wet_Well LEVEL < 1.5 THEN LINK FCV5554 STATUS IS ACTIVE AND LINK FCV5554 SETTING IS 100	RULE 1 ;Plant Reservoir Return Valve Control IF TANK HL_Wet_Well LEVEL < 1.5 THEN LINK FCV5554 STATUS IS ACTIVE AND LINK FCV5554 SETTING IS 120
RULE 3 ;Increased High Lift Production during Remote Reservoir Fill IF SYSTEM CLOCKTIME = 11:00 PM THEN PUMP HL-5440-VFD SPEED = 0.86 AND PUMP HL-5610-VFD SPEED = 0.86 AND LINK MFR SETTING IS 300 AND LINK TP-FCV STATUS IS CLOSED AND PUMP T-5410 STATUS IS CLOSED	RULE 3 ;Increased High Lift Production during Remote Reservoir Fill IF SYSTEM CLOCKTIME = 10:00 PM THEN PUMP HL-5440-VFD SPEED = 0.90 AND PUMP HL-5610-VFD SPEED = 0.90 AND LINK MFR SETTING IS 380 AND LINK TP-FCV STATUS IS CLOSED AND PUMP T-5410 STATUS IS CLOSED
RULE 4 ;Decreased High Lift Production after Remote Reservoir Fill IF SYSTEM CLOCKTIME = 4:30 AM THEN PUMP HL-5440-VFD SPEED = 0.82 AND PUMP HL-5610-VFD SPEED = 0.82 AND LINK TP-FCV STATUS IS ACTIVE AND LINK TP-FCV SETTING IS 65 ~~AND LINK MFR SETTING IS 285~~ AND PUMP T-5410 STATUS IS OPEN	RULE 4 ;Decreased High Lift Production after Remote Reservoir Fill IF SYSTEM CLOCKTIME = 6:00 AM THEN PUMP HL-5440-VFD SPEED = 0.85 AND PUMP HL-5610-VFD SPEED = 0.85 AND LINK TP-FCV STATUS IS ACTIVE AND LINK TP-FCV SETTING IS 65 AND PUMP T-5410 STATUS IS OPEN
RULE 5 ;Increase High Lift Production to meet System Demands IF SYSTEM CLOCKTIME = 8:00 AM THEN PUMP HL-5440-VFD SPEED = 0.85 AND PUMP HL-5610-VFD SPEED = 0.85 AND LINK MFR SETTING IS 395	RULE 5 ;Increase High Lift Production to meet System Demands IF SYSTEM CLOCKTIME = 7:00 AM THEN PUMP HL-5440-VFD SPEED = 0.85 AND PUMP HL-5610-VFD SPEED = 0.85 AND LINK MFR SETTING IS 390
RULE 6 ;Increase High Lift Production to meet evening Peak Demands IF SYSTEM CLOCKTIME = 3:00 PM THEN PUMP HL-5440-VFD SPEED = 0.81 AND PUMP HL-5610-VFD SPEED = 0.81 AND LINK MFR SETTING IS 290	RULE 6 ;Increase High Lift Production to meet evening Peak Demands IF SYSTEM CLOCKTIME = 3:00 PM THEN PUMP HL-5440-VFD SPEED = 0.81 AND PUMP HL-5610-VFD SPEED = 0.81 AND LINK MFR SETTING IS 300

RULE 7 ;North Park Reservoir Fill Valve Open and Pressure Sustaining Setting IF SYSTEM CLOCKTIME = 11:00 PM THEN LINK FCV-6551 STATUS IS ACTIVE AND LINK FCV-6551 SETTING IS 230	RULE 7 ;North Park Reservoir Fill Valve Open and Pressure Sustaining Setting IF SYSTEM CLOCKTIME = 11:00 PM THEN LINK FCV-6551 STATUS IS ACTIVE AND LINK FCV-6551 SETTING IS 200
RULE 8 ;North Park Reservoir Fill Valve Close IF SYSTEM CLOCKTIME = 2:00 AM THEN LINK FCV-6551 STATUS IS CLOSED	RULE 8 ;North Park Reservoir Fill Valve Close IF SYSTEM CLOCKTIME = 3:00 AM THEN LINK FCV-6551 STATUS IS CLOSED
RULE 13 ;Pine Street Reservoir Fill Valve Open IF SYSTEM CLOCKTIME = 2:00 AM THEN LINK PS_Res_Fill_Valve STATUS IS OPEN	RULE 13 ;Pine Street Reservoir Fill Valve Open IF SYSTEM CLOCKTIME = 3:00 AM THEN LINK PS_FCV STATUS IS ACTIVE AND LINK PS_FCV SETTING IS 200 *added Flow Control Valve
RULE 14 ;Pine Street Reservoir Fill Valve Close IF SYSTEM CLOCKTIME = 4:30 AM THEN LINK PS_Res_Fill_Valve STATUS IS CLOSED	RULE 14 ;Pine Street Reservoir Fill Valve Close IF SYSTEM CLOCKTIME = 7:00 AM THEN LINK PS_FCV STATUS IS CLOSED
RULE 21 ;PS2 LAG Pump Start during high demands (fire,main break) IF SYSTEM CLOCKTIME > 6:00 AM AND SYSTEM CLOCKTIME < 11:00 PM AND NODE PI6371 PRESSURE < 44 THEN PUMP M-6320 STATUS IS OPEN AND LINK FCV6311 STATUS IS CLOSED ; Turn off discharge control valve AND LINK 4725 STATUS IS OPEN ; Open bypass of discharge control valve	RULE 21 ;PS2 LAG Pump Start during high demands (fire,main break) IF SYSTEM CLOCKTIME > 7:00 AM AND SYSTEM CLOCKTIME < 11:00 PM AND NODE PI6371 PRESSURE < 44 THEN PUMP M-6320 STATUS IS OPEN AND LINK FCV6311 STATUS IS CLOSED ; Turn off discharge control valve AND LINK 4725 STATUS IS OPEN ; Open bypass of discharge control valve
RULE 22 ;PS2 LAG Pump close when pressures increase IF SYSTEM CLOCKTIME > 6:00 AM AND SYSTEM CLOCKTIME < 11:00 PM AND NODE PI6371 PRESSURE > 50 THEN PUMP M-6320 STATUS IS CLOSED AND VALVE FCV6311 SETTING IS 31 ;reset discharge control valve to original setting AND LINK 4725 STATUS IS CLOSED ; close bypass	RULE 22 ;PS2 LAG Pump close when pressures increase IF SYSTEM CLOCKTIME > 7:00 AM AND SYSTEM CLOCKTIME < 11:00 PM AND NODE PI6371 PRESSURE > 50 THEN PUMP M-6320 STATUS IS CLOSED AND VALVE FCV6311 SETTING IS 31 ;reset discharge control valve to original setting AND LINK 4725 STATUS IS CLOSED ; close bypass

RULE 28 ;North Park Pump Start and VFD Setting IF SYSTEM CLOCKTIME = 6:00 PM THEN PUMP M-6520-VFD STATUS IS OPEN AND PUMP M-6520-VFD SPEED = 0.87	RULE 28 ;North Park Pump Start and VFD Setting IF SYSTEM CLOCKTIME = 7:00 PM THEN PUMP M-6520-VFD STATUS IS OPEN AND PUMP M-6520-VFD SPEED = 0.87
RULE 30 ; Adam Street BPS lag pump closes and duty pump opens when pressures increase IF NODE TH_D1 PRESSURE > 52 THEN PUMP TH1 STATUS IS CLOSED AND PUMP TH2 STATUS IS OPEN	RULE 30 ; Adam Street BPS lag pump closes and duty pump opens when pressures increase IF NODE TH_D1 PRESSURE > 52 AND SYSTEM CLOCKTIME > 10:00 PM AND SYSTEM CLOCKTIME < 7:00 AM THEN PUMP TH1 STATUS IS CLOSED AND PUMP TH2 STATUS IS OPEN

New Rules

	RULE 31 ;Black Bear Ridge Tank Fill Valve Open IF SYSTEM CLOCKTIME = 10:00 PM THEN LINK FCV_ET STATUS IS ACTIVE AND LINK FCV_ET SETTING IS 105 AND LINK TO_BBR STATUS IS OPEN AND PUMP BBR_P1 STATUS IS OPEN AND PUMP BBR_P2 STATUS IS OPEN
	RULE 32 ;Black Bear Ridge Tank Fill Valve Closed IF SYSTEM CLOCKTIME = 7:00 AM THEN LINK FCV_ET STATUS IS CLOSED AND LINK TO_BBR STATUS IS CLOSED AND PUMP BBR_P1 STATUS IS CLOSED AND PUMP BBR_P2 STATUS IS CLOSED
	RULE 33 ;Black Bear Ridge Tank Fill Valve Overflow Control IF TANK BBR_ET LEVEL > 12.1 THEN LINK FCV_ET STATUS IS CLOSED AND LINK TO_BBR STATUS IS CLOSED AND PUMP BBR_P1 STATUS IS CLOSED AND PUMP BBR_P2 STATUS IS CLOSED
	RULE 34 ;Adam Street BPS IF SYSTEM CLOCKTIME = 10:00 PM THEN PUMP TH2 STATUS IS CLOSED AND PUMP TH3 STATUS IS OPEN AND LINK AS_PRV SETTING IS 70
	RULE 39 ;Black Bear Ridge Tank Fill Valve Open IF SYSTEM CLOCKTIME = 4:00 PM

	THEN LINK FCV_ET STATUS IS ACTIVE AND LINK FCV_ET SETTING IS 25 AND LINK TO_BBR STATUS IS OPEN AND PUMP BBR_P1 STATUS IS OPEN
	RULE 40 ;Black Bear Ridge Tank Fill Valve Closed IF SYSTEM CLOCKTIME = 7:00 PM THEN LINK FCV_ET STATUS IS CLOSED AND LINK TO_BBR STATUS IS CLOSED AND PUMP BBR_P1 STATUS IS CLOSED
	RULE 41 ;Black Bear Ridge Tank Fill Valve Overflow Control IF TANK BBR_ET LEVEL > 13.7 THEN LINK FCV_ET STATUS IS CLOSED AND LINK TO_BBR STATUS IS CLOSED AND PUMP BBR_P1 STATUS IS CLOSED

Pine Street Reservoir

- Added flow control valve to bypass uncertain pressure reducing valves

WTP

- Turn on all pump excess fire and main breaks back-up
- Adjusted Master Flow Valve to increase flow into system

Adam St Booster

- During filling, fire pump turned on and pressure reducing valve setting increased

APPENDIX C

Sanitary Sewer Design Sheet

SANITARY SEWER DESIGN SHEET - Black Bear Ridge Development 3,049 Homes + Commercial

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

q = Average daily per capita flow **350** L/Ca./Day

I = Unit of peak extraneous flow **0.28** L/s/ha

M = Harmon peaking factor (min = 2)

P = Population in 1000's

A = Area in hectares

Population Flows (Persons/Unit)

Single Family	3.0
Semi (3bdrm)	3.0
Townhouse/Apartment	2.5
Batchelor	1.6

Commercial Flows

Peaking Factor	1.00 (Note: GGG used no peaking factor)
Average Commercial Flows	5.00 L/m2d
Commercial Peak Flow	5.00 L/m2d
Floor Area = 30% Gross Area	

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

A = area of pipe in m^2

R = Hydraulic radius = A / P

P = Wetted perimeter

S = Slope (m/m)

n = Manning's friction coef.

Check

$Q_d \leq 0.8 \cdot (Pipe\ Capacity)$

$0.6 \leq V \leq 3.0$

use Actual V if d:D< **0.3**

CONTRIBUTES TO SIPHON

LOCATION				PEAK FLOW CALCULATION													PROPOSED SEWER									
DESCRIPTION	AREA	FROM	TO	RESIDENTIAL INDIVIDUAL Pop.	RESIDENTIAL INDIVIDUAL Area (A) (ha)	RESIDENTIAL CUMULATIVE Pop.	RESIDENTIAL CUMULATIVE Area (A) (ha)	Resid. Peaking Factor M	COMMERCIAL INDIVIDUAL Floor Area (A) (m2)	COMMERCIAL INDIVIDUAL Area (Ha)	COMMERCIAL CUMULATIVE Floor Area (A) (m2)	COMMERCIAL CUMULATIVE Area (Ha)	Pop. Flow $Q(_p)$ (L/s)	Commer. Flow $Q(_c)$ (L/s)	Peak Ex. Flow $Q(_i)$ (L/s)	Design Flow $Q(_d)$ (L/s)	Length (m)	Pipe Size (mm)	Type of Pipe	Grade (use m/m) (%)	Capacity, n = 0.013 (L/s)	Full Flow Velocity (m/s)	Ratio d:D	Actual Velocity at Q_d (m/s)	Check	q/Q
Settlers Ridge Phase 4	1A	SA25B	**SA 24**	33	0.87	33	0.87	4.35	0.00	0.00	0.00	0.00	0.6	0.0	0.2	0.8		525	PVC	0.18%	182.46	0.84	0.03	0.11	Velocity	0.5%
Settlers Ridge Phase 4	1A	SA 26	SA 27	13	0.28	13	0.28	4.40	0.00	0.00	0.00	0.00	0.2	0.0	0.1	0.3		200	PVC	0.70%	27.44	0.87	0.06	0.26	Velocity	1.1%
Settlers Ridge Phase 4	1A	SA 27	SA 28	50	1.01	63	1.29	4.29	0.00	0.00	0.00	0.00	1.1	0.0	0.4	1.5		200	PVC	0.40%	20.74	0.66	0.17	0.38	Velocity	7.0%
Settlers Ridge Phase 4	1A	SA 28	**SA 24**	16	0.46	79	1.75	4.27	0.00	0.00	0.00	0.00	1.4	0.0	0.5	1.9		200	PVC	0.40%	20.74	0.66	0.20	0.41	Velocity	9.0%
Settlers Ridge Phase 4	1A	**SA 24**	**SA 23**	18	0.54	130	3.16	4.21	0.00	0.00	0.00	0.00	2.2	0.0	0.9	3.1		525	PVC	0.19%	187.46	0.87	0.08	0.31	Velocity	1.7%
Settlers Ridge Phase 3	1A	SA 34	SA 33	52	1.21	52	1.21	4.31	0.00	0.00	0.00	0.00	0.9	0.0	0.3	1.2		200	PVC	0.70%	27.44	0.87	0.14	0.44	Velocity	4.5%
Settlers Ridge Phase 3	1A	SA 33	SA 32	24	0.61	76	1.82	4.27	0.00	0.00	0.00	0.00	1.3	0.0	0.5	1.8		200	PVC	0.40%	20.74	0.66	0.20	0.40	Velocity	8.8%
Settlers Ridge Phase 3	1A	SA 32	**SA 31**	25	0.60	101	2.42	4.24	0.00	0.00	0.00	0.00	1.7	0.0	0.7	2.4		200	PVC	0.40%	20.74	0.66	0.23	0.44	Velocity	11.6%
Settlers Ridge Phase 3	1A	SA 34	SA 35	24	0.70	24	0.70	4.37	0.00	0.00	0.00	0.00	0.4	0.0	0.2	0.6		200	PVC	0.70%	27.44	0.87	0.10	0.37	Velocity	2.3%
Settlers Ridge Phase 3	1A	SA 35	**SA 31**	28	0.40	52	1.10	4.31	0.00	0.00	0.00	0.00	0.9	0.0	0.3	1.2		200	PVC	0.40%	20.74	0.66	0.16	0.36	Velocity	5.9%
Settlers Ridge Phase 3	1A	**SA 31**	**SA 30**	0	0.00	153	3.52	4.19	0.00	0.00	0.00	0.00	2.6	0.0	1.0	3.6		200	PVC	0.40%	20.74	0.66	0.28	0.49	Velocity	17.3%
Settlers Ridge Phase 2	1A	SA 36A	SA 36	0	0.00	0	0	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.0	0.0		200	PVC	0.77%	28.78	0.92	0.00	0.00	Velocity	0.0%
Settlers Ridge Phase 2	1A	SA 36	**SA 30**	27	1.11	27	1.11	4.36	245.70	0.08	245.70	0.08	0.5	0.0	0.3	0.8		200	PVC	0.40%	20.74	0.66	0.13	0.32	Velocity	4.0%
Settlers Ridge Phase 2	1A	**SA 30**	SA 29A	42	1.05	222	5.68	4.13	0.00	0.00	245.70	0.08	3.7	0.0	1.6	5.3		200	PVC	0.40%	20.74	0.66	0.35	0.66	OK	25.8%
Settlers Ridge Phase 2	1A	SA 29A	SA 29B	24	1.05	246	6.73	4.11	0.00	0.00	245.70	0.08	4.1	0.0	1.9	6.0		200	PVC	0.40%	20.74	0.66	0.37	0.66	OK	29.0%
Settlers Ridge Phase 2	1A	SA 29B	**SA23**	15	1.05	261	7.78	4.10	0.00	0.00	245.70	0.08	4.3	0.0	2.2	6.6		200	PVC	0.40%	20.74	0.66	0.39	0.66	OK	31.6%
Settlers Ridge Phase 2	1A	**SA 23**	SA 22	15	0.37	406	11.31	4.02	0.00	0.00	245.70	0.08	6.6	0.0	3.2	9.8		525	PVC	0.18%	182.46	0.84	0.15	0.45	Velocity	5.4%
Settlers Ridge Phase 2	1A	SA 22	**SA 18**	15	0.38	421	11.69	4.01	0.00	0.00	245.70	0.08	6.8	0.0	3.3	10.2		525	PVC	0.18%	182.46	0.84	0.16	0.45	Velocity	5.6%
Settlers Ridge Phase 2	1A	SA 19	**SA 20**	33	1.09	33	1.09	4.35	0.00	0.00	0.00	0.00	0.6	0.0	0.3	0.9		200	PVC	0.70%	27.44	0.87	0.12	0.41	Velocity	3.2%
Settlers Ridge Phase 2	1A	SA 21	**SA20**	6	0.23	6	0.23	4.43	0.00	0.00	0.00	0.00	0.1	0.0	0.1	0.2		200	PVC	0.40%	20.74	0.66	0.05	0.15	Velocity	0.8%
Settlers Ridge Phase 2	1A	**SA 20**	**SA 18**	6	0.27	45	1.59	4.32	0.00	0.00	0.00	0.00	0.8	0.0	0.4	1.2		200	PVC	0.40%	20.74	0.66	0.16	0.36	Velocity	5.9%
Settlers Ridge Phase 2	1A	**SA 18**	**SA9**	33	0.87	499	14.15	3.97	0.00	0.00	245.70	0.08	8.0	0.0	4.0	12.0		525	PVC	0.40%	272.00	1.26	0.14	0.63	OK	4.4%
Settlers Ridge Phase 2	1A	SA14	SA13	42	1.05	42	1.05	4.33	0.00	0.00	0.00	0.00	0.7	0.0	0.3	1.0		200	PVC	0.70%	27.44	0.87	0.13	0.42	Velocity	3.8%
Settlers Ridge Phase 2	1A	SA13	SA12	27	0.71	69	1.76	4.28	0.00	0.00	0.00	0.00	1.2	0.0	0.5	1.7		200	PVC	0.40%	20.74	0.66	0.19	0.39	Velocity	8.1%
Settlers Ridge Phase 2	1A	SA12	**SA11**	45	1.10	114	2.86	4.23	0.00	0.00	0.00	0.00	2.0	0.0	0.8	2.8		200	PVC	0.40%	20.74	0.66	0.24	0.46	Velocity	13.3%
Settlers Ridge Phase 2	1A	SA14	SA15	38	0.71	38	0.71	4.34	0.00	0.00	0.00	0.00	0.7	0.0	0.2	0.9		200	PVC	0.70%	27.44	0.87	0.12	0.41	Velocity	3.2%
Settlers Ridge Phase 2	1A	SA15	SA16	27	0.50	65	1.21	4.29	0.00	0.00	0.00	0.00	1.1	0.0	0.3	1.5		200	PVC	0.44%	21.76	0.69	0.17	0.39	Velocity	6.7%
Settlers Ridge Phase 2	1A	SA16	SA17A	17	0.36	82	1.57	4.27	0.00	0.00	0.00	0.00	1.4	0.0	0.4	1.9		200	PVC	0.44%	21.76	0.69	0.19	0.42	Velocity	8.5%
Settlers Ridge Phase 2	1A	SA17A	SA17B	12	0.15	94	1.72	4.25	0.00	0.00	0.00	0.00	1.6	0.0	0.5	2.1		200	PVC	0.44%	21.76	0.69	0.21	0.44	Velocity	9.7%
Settlers Ridge Phase 2	1A	SA17B	**SA11**	0	0.07	94	1.79	4.25	0.00	0.00	0.00	0.00	1.6	0.0	0.5	2.1		200	PVC	0.44%	21.76	0.69	0.21	0.44	Velocity	9.7%
Settlers Ridge Phase 2	1A	**SA11**	SA10	24	0.81	232	5.46	4.12	0.00	0.00	0.00	0.00	3.9	0.0	1.5	5.4		200	PVC	0.64%	26.24	0.84	0.31	0.84	OK	20.6%
Settlers Ridge Phase 2	1A	SA10	**SA9**	3	0.21	235	5.67	4.12	0.00	0.00	0.00	0.00	3.9	0.0	1.6	5.5		200	PVC	1.37%	38.39	1.22	0.25	0.87	OK	14.4%
Settlers Ridge Phase 2	1A	**SA9**	SA8	15	0.67	749	20.49	3.88	0.00	0.00	245.70	0.08	11.8	0.0	5.8	17.5		525	PVC	0.18%	182.46	0.84	0.21	0.53	Velocity	9.6%
Settlers Ridge Phase 2	1A	SA8	**SA8A**	9	0.33	758	20.82	3.87	0.00	0.00	245.70	0.08	11.9	0.0	5.9	17.8		525	PVC	0.18%	182.46	0.84	0.21	0.53	Velocity	9.7%
Settlers Ridge Phase 1	1A	**SA8A**	**SA-7**	0	0.00	758	20.82	3.87	0.00	0.00	245.70	0.08	11.9	0.0	5.9	17.8		525	PVC	0.18%	182.46	0.84	0.21	0.53	Velocity	9.7%
Settler Ridge Phase 1	1A	Princeton Place	**SA-7**	245	3.15	245	3.15	4.11	0.00	0.00	0.00	0.00	4.1	0.0	0.9	5.0		200	PVC	0.20%	14.67	0.47	0.40	0.47	Velocity	33.9%
Settlers Ridge Phase 1	1A	**SA-7**	**SA-6**	21	0.65	1024	24.62	3.79	0.00	0.00	245.70	0.00	15.7	0.0	6.9	22.6		525	PVC	0.18%	182.46	0.84	0.23	0.57	Velocity	12.4%
Settlers Ridge Phase 1	1A	**SA-6**	**SA-5**	24	0.71	1048	25.33	3.79	0.00	0.00	245.70	0.00	16.1	0.0	7.1	23.2		525	PVC	0.21%	197.08	0.91	0.23	0.61	OK	11.8%

SANITARY SEWER DESIGN SHEET - Black Bear Ridge Development 3,049 Homes + Commercial

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

q =	Average daily per capita flow	**350** L/Ca./Day
I =	Unit of peak extraneous flow	**0.28** L/s/ha
M =	Harmon peaking factor (min = 2)	
P =	Population in 1000's	
A =	Area in hectares	

Population Flows (Persons/Unit)

Single Family	3.0
Semi (3bdrm)	3.0
Townhouse/Apartment	2.5
Batchelor	1.6

Commercial Flows

Peaking Factor	1.00 (Note: GGG used no peaking factor)
Average Commercial Flows	5.00 L/m2d
Commercial Peak Flow	5.00 L/m2d
Floor Area = 30% Gross Area	

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

A = area of pipe in m^2
R = Hydraulic radius = A / P
P = Wetted perimeter
S = Slope (m/m)
n = Manning's friction coef.

Check

$Q_d \leq 0.8 \cdot (Pipe\ Capacity)$

$0.6 \leq V \leq 3.0$

use Actual V if d:D< **0.3**

CONTRIBUTES TO SIPHON

LOCATION: DESCRIPTION	AREA	FROM	TO	RESIDENTIAL INDIVIDUAL Pop.	RESIDENTIAL INDIVIDUAL Area (A) (ha)	RESIDENTIAL CUMULATIVE Pop.	RESIDENTIAL CUMULATIVE Area (A) (ha)	Resid. Peaking Factor M	COMMERCIAL INDIVIDUAL Floor Area (A) (m2)	COMMERCIAL INDIVIDUAL Area (Ha)	COMMERCIAL CUMULATIVE Floor Area (A) (m2)	COMMERCIAL CUMULATIVE Area (Ha)	Pop. Flow $Q_{(p)}$ (L/s)	Commer. Flow $Q_{(c)}$ (L/s)	Peak Ex. Flow $Q_{(i)}$ (L/s)	Design Flow $Q_{(d)}$ (L/s)	PROPOSED SEWER: Length (m)	Pipe Size (mm)	Type of Pipe	Grade (use m/m) (%)	Capacity, n = 0.013 (L/s)	Full Flow Velocity (m/s)	Ratio d:D	Actual Velocity at Q_d (m/s)	Check	q/Q
Settlers Ridge Phase 1	1A	Cavendish Eas	**SA-5**	189	3.61	189	3.61	4.16	0.00	0.00	0.00	0.00	3.2	0.0	1.0	4.2		200	PVC	0.20%	14.67	0.47	0.37	0.47	Velocity	28.6%
Settlers Ridge Phase 1	1A	Sheridan Way	**SA-5**	210	6.38	210	6.38	4.14	0.00	0.00	0.00	0.00	3.5	0.0	1.8	5.3		200	PVC	0.20%	14.67	0.47	0.42	0.47	Velocity	36.2%
Settlers Ridge Phase 1	1A	**SA-5**	SA-4	0	0.00	1447	35.32	3.69	0.00	0.00	245.70	0.00	21.6	0.0	9.9	31.5		525	PVC	0.20%	192.33	0.89	0.27	0.65	OK	16.4%
Settlers Ridge Phase 1	1A	SA-4	SA-3	3	0.22	1450	35.54	3.69	0.00	0.00	245.70	0.00	21.7	0.0	10.0	31.6		525	PVC	0.20%	192.33	0.89	0.27	0.66	OK	16.5%
Settlers Ridge Phase 1	1A	SA-3	SA-2	3	0.22	1453	35.76	3.69	0.00	0.00	245.70	0.00	21.7	0.0	10.0	31.7		525	PVC	0.20%	192.33	0.89	0.27	0.66	OK	16.5%
Settlers Ridge Phase 1	1A	SA-2	SA-1A	3	0.62	1456	36.38	3.69	0.00	0.00	245.70	0.00	21.8	0.0	10.2	32.0		525	PVC	0.20%	192.33	0.89	0.27	0.66	OK	16.6%
Settlers Ridge Trunk	1F	SA-1A	SA-2A	0	0.00	1456	36.38	3.69	0.00	0.00	245.70	0.00	21.8	0.0	10.2	32.0		525	PVC	0.20%	192.33	0.89	0.27	0.66	OK	16.6%
Settlers Ridge Trunk	1F	SA-2A	SA-3A	15	1.50	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-3A	SA-4A	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-4A	SA-5A	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-5A	SA-6A	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-6A	SA-7A	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-7A	SA-8A	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
Settlers Ridge Trunk	1F	SA-8A	MH-41	0	0.00	1471	37.88	3.69	0.00	0.00	245.70	0.00	22.0	0.0	10.6	32.6		525	PVC	0.40%	272.00	1.26	0.23	0.84	OK	12.0%
DeerField Park Phase 4	2A	MH-23	**MH-28**	303	8.90	303	8.90	4.08	0.00	0.00	0.00	0.00	5.0	0.0	2.5	7.5		250	PVC	0.29%	32.02	0.65	0.33	0.65	OK	23.4%
Deerfield Park Phase 3	2A	**MH-28**	**MH-27**	18	0.51	321	9.41	4.07	0.00	0.00	0.00	0.08	5.3	0.0	2.7	7.9		250	PVC	0.25%	29.73	0.61	0.35	0.61	OK	26.7%
Deerfield Park Phase 3	2A	**Gardiner Street**	**MH-27**	18	0.85	18	0.85	4.39	0.00	0.00	0.00	0.00	0.3	0.0	0.2	0.6		200	PVC	1.00%	32.80	1.04	0.08	0.38	Velocity	1.7%
Deerfield Park Phase 3	2A	**MH-27**	MH-29	36	0.87	375	11.13	4.04	0.00	0.00	0.00	0.08	6.1	0.0	3.1	9.3		250	PVC	0.25%	29.73	0.61	0.38	0.61	OK	31.2%
Deerfield Park Phase 3	2A	MH-29	MH-31	24	0.73	399	11.86	4.02	0.00	0.00	0.00	0.08	6.5	0.0	3.3	9.8		250	PVC	0.22%	27.89	0.57	0.41	0.57	Velocity	35.3%
Deerfield Park Phase 3	2A	MH-31	MH-32	24	0.41	423	12.27	4.01	0.00	0.00	0.00	0.08	6.9	0.0	3.5	10.3		525	PVC	0.18%	182.46	0.84	0.16	0.45	Velocity	5.7%
Deerfield Park Phase 1-2	2C	MH-32	MH-40	0	0.00	423	12.27	4.01	0.00	0.00	0.00	0.08	6.9	0.0	3.5	10.3		250	PVC	0.27%	30.90	0.63	0.40	0.63	OK	33.4%
Deerfield Park Phase 1-2	2C	MH-40	MH-41	0	0.00	423	12.27	4.01	0.00	0.00	0.00	0.08	6.9	0.0	3.5	10.3		250	PVC	1.27%	67.02	1.37	0.26	0.99	OK	15.4%
Deerfield Park Phase 1-2	2C	MH-41	MH-42	0	0.00	1894	50.15	3.60	0.00	0.00	0.00	0.08	27.7	0.0	14.1	41.7		525	PVC	3.21%	770.52	3.56	0.15	1.89	OK	5.4%
Cloverleaf Drive	2C	MH-42	**SAN MH10**	0	0.00	1894	50.15	3.60	0.00	0.00	0.00	0.08	27.7	0.0	14.1	41.7		525	PVC	3.21%	770.52	3.56	0.15	1.89	OK	5.4%
Cloverleaf Drive	2C	MH-SA9	**SAN MH10**	114	8.10	114	8.10	4.23	0.00	0.00	0.00	0.00	2.0	0.0	2.3	4.2		200	PVC	1.31%	37.54	1.19	0.22	0.79	OK	11.2%
Lowes	3B	**SAN MH10**	SAN MH #6	0	0.00	2008	58.25	3.58	21600.00	7.20	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.18%	260.50	0.92	0.29	0.70	OK	18.7%
Lowes	3B	SAN MH#6	SAN MH #7	0	0.00	2008	58.25	3.58	0.00	0.00	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.18%	260.50	0.92	0.29	0.70	OK	18.7%
Lowes	3B	SAN MH#7	SAN MH#8	0	0.00	2008	58.25	3.58	0.00	0.00	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.18%	260.50	0.92	0.29	0.70	OK	18.7%
Millenium Pkwy (East)	7G	SAN MH#8	MH-31	0	0.00	2008	58.25	3.58	0.00	0.00	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.18%	260.50	0.92	0.29	0.70	OK	18.7%
Millenium Pkwy (East)	7G	MH-31	MH-30	0	0.00	2008	58.25	3.58	0.00	0.00	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.20%	277.33	0.98	0.28	0.74	OK	17.6%
Millenium Pkwy (East)	7G	MH-30	**MH-29**	0	0.00	2008	58.25	3.58	0.00	0.00	21600.00	7.28	29.2	1.3	18.3	48.8		600		0.22%	289.30	1.02	0.28	0.76	OK	16.9%
Farnham	5A	SA18	SA17	427	12.39	427	12.39	4.01	0.00	0.00	0.00	0.00	6.9	0.0	3.5	10.4		250	PVC	0.33%	34.16	0.70	0.38	0.70	OK	30.5%
Maitland	5B (west)	SA17	SA16	0	0.00	427	12.39	4.01	31380.00	10.46	31380.00	10.46	6.9	1.8	6.4	15.2		250	PVC	0.31%	33.11	0.67	0.47	0.67	OK	45.8%
Maitland	5B (west)	SA16	SA15	0	0.00	427	12.39	4.01	0.00	0.00	31380.00	10.46	6.9	1.8	6.4	15.2		250	PVC	0.31%	33.11	0.67	0.47	0.67	OK	45.8%
Maitland	5B (west)	SA15	SA14	0	0.00	427	12.39	4.01	0.00	0.00	31380.00	10.46	6.9	1.8	6.4	15.2		250	PVC	0.31%	33.11	0.67	0.47	0.67	OK	45.8%
Maitland	5B (west)	SA14	SA13	0	0.00	427	12.39	4.01	0.00	0.00	31380.00	10.46	6.9	1.8	6.4	15.2		250	PVC	0.31%	33.11	0.67	0.47	0.67	OK	45.8%
Maitland	5B (west)	SA13	SA12	0	0.00	427	12.39	4.01	0.00	0.00	31380.00	10.46	6.9	1.8	6.4	15.2		250	PVC	0.31%	33.11	0.67	0.47	0.67	OK	45.8%
Maitland	5B (west)	SA12	**SA2**	0	0.00	427	12.39	4.01	0.00	0.00	31380.00	10.46	6.9	1.8	6.4	15.2		450	PVC	0.36%	171.06	1.08	0.20	0.66	OK	8.9%
Maitland	7D	SA1A	SA1B	0	0.00	0	0.00	4.50	7380.00	2.46	7380.00	2.46	0.0	0.4	0.7	1.1		250	PVC	1.41%	70.61	1.44	0.08	0.50	Velocity	1.6%
Maitland	7D	SA1B	**SA2**	0	0.00	0	0.00	4.50	0.00	0.00	7380.00	2.46	0.0	0.4	0.7	1.1		250	PVC	1.43%	71.11	1.45	0.08	0.50	Velocity	1.6%
Mineral Drive	7D	**SA2**	SA5	0	0.00	427	12.39	4.01	0.00	0.00	38760.00	12.92	6.9	2.2	7.1	16.3		450	PVC	0.93%	274.95	1.73	0.16	0.94	OK	5.9%
Mineral Drive	7D	SA5	SA6	0	0.00	427	12.39	4.01	8070.00	2.69	46830.00	15.61	6.9	2.7	7.8	17.5		450	PVC	0.31%	158.74	1.00	0.22	0.66	OK	11.0%

SANITARY SEWER DESIGN SHEET - Black Bear Ridge Development 3,049 Homes + Commercial

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

q =	Average daily per capita flow	350	L/Ca./Day
I =	Unit of peak extraneous flow	0.28	L/s/ha
M =	Harmon peaking factor (min = 2)		
P =	Population in 1000's		
A =	Area in hectares		

Population Flows (Persons/Unit)

Single Family	3.0
Semi (3bdrm)	3.0
Townhouse/Apartment	2.5
Batchelor	1.6

Commercial Flows

Peaking Factor	1.00 (Note: GGG used no peaking factor)
Average Commercial Flows	5.00 L/m2d
Commercial Peak Flow	5.00 L/m2d
Floor Area = 30% Gross Area	

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

A = area of pipe in m^2
R = Hydraulic radius = A / P
P = Wetted perimeter
S = Slope (m/m)
n = Manning's friction coef.

Check

$Q_d \le 0.8 \cdot (Pipe\ Capacity)$

$0.6 \le V \le 3.0$

use Actual V if d:D< 0.3

CONTRIBUTES TO SIPHON

LOCATION				PEAK FLOW CALCULATION													PROPOSED SEWER									
DESCRIPTION	AREA	FROM	TO	RESIDENTIAL INDIVIDUAL Pop.	RESIDENTIAL INDIVIDUAL Area (A) (ha)	RESIDENTIAL CUMULATIVE Pop.	RESIDENTIAL CUMULATIVE Area (A) (ha)	Resid. Peaking Factor M	COMMERCIAL INDIVIDUAL Floor Area (A) (m2)	COMMERCIAL INDIVIDUAL Area (Ha)	COMMERCIAL CUMULATIVE Floor Area (A) (m2)	COMMERCIAL CUMULATIVE Area (Ha)	Pop. Flow $Q_{(p)}$ (L/s)	Commer. Flow $Q_{(c)}$ (L/s)	Peak Ex. Flow $Q_{(i)}$ (L/s)	Design Flow $Q_{(d)}$ (L/s)	Length (m)	Pipe Size (mm)	Type of Pipe	Grade (use m/m) (%)	Capacity, n = 0.013 (L/s)	Full Flow Velocity (m/s)	Ratio d:D	Actual Velocity at Q_d (m/s)	Check	q/Q
Mineral Drive	7F	SA6	SA7	0	0.00	427	12.39	4.01	2130.00	0.71	48960.00	16.32	6.9	2.8	8.0	17.8		525	PVC	0.56%	320.39	1.48	0.16	0.79	OK	5.6%
Mineral Drive	7F	SA7	SA8	0	0.00	427	12.39	4.01	0.00	0.00	48960.00	16.32	6.9	2.8	8.0	17.8		675	PVC	0.16%	336.24	0.94	0.15	0.50	Velocity	5.3%
		SA8	SA9	0	0.00	427	12.39	4.01	15720.00	5.24	64680.00	21.56	6.9	3.7	9.5	20.2		675	PVC	0.16%	336.24	0.94	0.16	0.51	Velocity	6.0%
		SA9	SA10	0	0.00	427	12.39	4.01	0.00	0.00	64680.00	21.56	6.9	3.7	9.5	20.2		675	PVC	0.16%	336.24	0.94	0.16	0.51	Velocity	6.0%
		SA10	**MH29**	0	0.00	427	12.39	4.01	0.00	0.00	64680.00	21.56	6.9	3.7	9.5	20.2		675	PVC	0.16%	336.24	0.94	0.16	0.51	Velocity	6.0%
Millenium Pkwy (East)	7G	**MH-29**	MH-28	0	0.00	2435	70.64	3.52	35370.00	11.79	121650.00	40.63	34.7	7.0	31.2	72.9		600		0.18%	260.50	0.92	0.36	0.92	OK	28.0%
Siphon		MH-28	Siphon Inlet	0	0.00	2435	70.64	3.52	0.00	0.00	121650.00	40.63	34.7	7.0	31.2	72.9		600		0.18%	260.50	0.92	0.36	0.92	OK	28.0%
Siphon		Siphon Inlet	Siphon Outlet	0	0.00	2435	70.64	3.52	0.00	0.00	121650.00	40.63	34.7	7.0	31.2	72.9		600		0.18%	464.00	1.64	0.27	1.20	OK	15.7%
Siphon		Siphon Outlet	**MH-11**	0	0.00	2435	70.64	3.52	3000.00	1.00	124650.00	41.63	34.7	7.2	31.4	73.4		600		0.18%	260.50	0.92	0.36	0.92	OK	28.2%
Maitland Drive		SA22	**SA1(Moira Lea)**	0	0.00	0	0.00	4.50	12900.00	4.30	12900.00	4.30	0.0	0.7	1.2	2.0		350		0.20%	65.23	0.68	0.12	0.31	Velocity	3.0%
Moira Lee/ Canif Mills sub.	6B	SA5 (Moira Lea)	**SA1(Moira Lea)**	1411	40.90	1411	40.90	3.70	0.00	0.00	0.00	0.00	21.1	0.0	11.5	32.6		450	SDR35	0.25%	142.55	0.90	0.33	0.90	OK	22.9%
Pumping Station		**SA1(Moira Lea)**	**MH33**	0	0.00	1411	40.90	3.70	0.00	0.00	12900.00	4.30	21.1	0.7	12.7	34.5		450		0.20%	127.50	0.80	0.35	0.80	OK	27.1%
BBR			**3,049 Homes**	**9397**	**120.00**	9397	120.00	2.98	48900.00	27.00	48900.00	27.00	113.5	2.8	41.2	**157.5**		600	**Ave**	**0.22%**	**288.00**	**1.02**	**0.55**	**1.02**	**OK**	**57.8%**
Cannifton Rd	8B	MH40	MH39	553	10.53	9950	130.53	2.96	3000.00	1.00	51900.00	28.00	119.2	3.0	44.4	166.6	129.9	600		0.21%	278.01	0.98	0.56	0.98	OK	59.9%
		MH39	MH38	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	166.6	119.8	600		0.23%	294.47	1.04	0.54	1.04	OK	56.6%
		MH38	MH37	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	166.6	120.5	600		0.19%	268.34	0.95	0.57	0.95	OK	62.1%
		MH37	MH36	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	166.6	121.2	600		0.23%	294.47	1.04	0.54	1.04	OK	56.6%
		MH36	MH35	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	166.6	119.0	600		0.23%	294.47	1.04	0.54	1.04	OK	56.6%
		MH35	MH34	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	166.6	119.7	600		0.23%	294.47	1.04	0.54	1.04	OK	56.6%
		MH34	**MH33**	0	0.00	9950	130.53	2.96	0.00	0.00	51900.00	28.00	119.2	3.0	44.4	**166.6**	117.8	600		0.22%	288.00	1.02	0.55	1.02	OK	**57.8%**
																197.4		600	**Ave**	**0.29%**	**331.79**	**1.17**	**0.56**	**1.17**	**OK**	**59.5%**
Cannifton Rd	9A	**MH33**	MH32	158	3.00	11518	174.43	2.89	4830.00	1.61	69630.00	33.91	135.0	4.0	58.3	197.4	96.2	600		0.18%	259.78	0.92	0.66	0.92	OK	76.0%
		MH32	MH17	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	109.8	600		0.19%	264.81	0.94	0.65	0.94	OK	74.5%
		MH17	MH16	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	103.8	600		0.45%	411.89	1.46	0.49	1.46	OK	47.9%
		MH16	MH15	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	102.1	600		0.20%	277.33	0.98	0.63	0.98	OK	71.2%
		MH15	MH14	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	108.9	600		0.20%	276.65	0.98	0.63	0.98	OK	71.3%
		MH14	MH13	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	105.5	600		0.40%	389.79	1.38	0.50	1.38	OK	50.6%
		MH13	MH12	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	105.0	600		0.20%	275.28	0.97	0.63	0.97	OK	71.7%
		MH12	**MH11**	0	0.00	11518	174.43	2.89	0.00	0.00	69630.00	33.91	135.0	4.0	58.3	197.4	106.7	600		0.50%	435.90	1.54	0.47	1.54	OK	45.3%
Lywood St	9B	MH25	**MH19**	320	6.10	320	6.10	4.07	2580.00	0.86	2580.00	0.86	5.3	0.1	1.9	7.4		200		0.40%	20.74	0.66	0.41	0.66	OK	35.5%
Black Diamond Road	9C	MH27	**MH19**	0	0.00	0	0.00	4.50	4200.00	1.40	4200.00	1.40	0.0	0.2	0.4	0.6		200		0.40%	20.74	0.66	0.12	0.31	Velocity	3.1%
Lywood St	9D	MH18	**MH19**	0	0.00	0	0.00	4.50	2400.00	0.80	2400.00	0.80	0.0	0.1	0.2	0.4		200		0.40%	20.74	0.66	0.08	0.24	Velocity	1.7%
Black Diamond Road	9E	**MH19**	**MH11**	0	0.00	320	6.10	4.07	1560.00	0.52	10740.00	3.58	5.3	0.6	2.7	8.6		200		0.40%	20.74	0.66	0.45	0.66	OK	41.5%
Cannifton Rd	11A	**MH11**	**MH10**	0	0.00	14274	251.17	2.80	4200.00	1.40	209220.00	80.52	161.9	12.1	92.9	266.9		675		0.37%	514.07	1.44	0.51	1.44	OK	51.9%
Cannifton Rd		**MH10**	**MH9**	0	0.00	14274	251.17	2.80	0.00	0.00	209220.00	80.52	161.9	12.1	92.9	266.9		675		0.33%	479.21	1.34	0.53	1.34	OK	55.7%
Cannifton Rd		**MH9**	**MH8**	0	0.00	14274	251.17	2.80	0.00	0.00	209220.00	80.52	161.9	12.1	92.9	266.9		675		0.32%	478.47	1.34	0.53	1.34	OK	55.8%
Cannifton Rd		**MH7**	401	0	0.00	14274	251.17	2.80	0.00	0.00	209220.00	80.52	161.9	12.1	92.9	266.9		675		0.35%	497.30	1.39	0.52	1.39	OK	53.7%

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Designed:	John Landry
Checked:	Bryon Keene P.Eng.
Date:	February 28, 2022

Project:
Black Bear Ridge Development
3,049 Homes plus Commercial

APPENDIX E:
STORM SEWER CATCHMENT AREAS AND DESIGN SHEET

STORM SEWER DESIGN SHEET

Peak Runoff Estimate by Rational Method

$$Q = \frac{1}{360}\, C\, i\, A$$

Where:

- Q = Peak Flow in cms
- C = Runoff Coefficient
- i = Rainfall Intensity in mm/hr
- A = Area in hectares

Intensity for: **Belleville** **Station:** 6150689

$$i = A * T_c^{\,B}$$

Where:

- i = Rainfall Intensity in mm/hr
- T_c = Time of Concentration in hours

5-Year Parameters

- A = 26.5
- B = -0.677

Manning's Coef

- CSP 0.024
- RCP/PVC 0.013

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n}\, A\, R^{2/3}\, S^{1/2}$$

Where:

- A = area of pipe in m^2
- R = Hydraulic radius = A / P
- P = Wetted perimeter
- S = Slope (m/m)
- n = Manning's friction coef.

Check

$q \leq Q$

$V \leq 6\ m/s$

LOCATION			PEAK FLOW CALCULATION									PROPOSED SEWER									
STREET / CATCHMENT AREA ID	FROM	TO	CATCHMENT AREAS / RUNOFF COEFFICIENT 0.25	0.45	0.6	0.7	R.C. x A (ha)	CUM. R.C x A (ha)	TIME OF CONCENTRATION (min)	INTENSITY (mm/hr)	PEAK FLOW (m^3/s)	Pipe Size (mm)	Length (m)	Type of Pipe	Grade *(use m/m)* (%)	Capacity, n = 0.013 (m^3/s)	Full Flow Velocity (m/s)	Time of Flow (min)	Actual Velocity at Q_d (m/s)	q/Q	Check Capacity
Street C&D																					
207B	ST1, ST10, ST17	Outlet			8.57		5.14	5.14	15.0	67.7	0.968	825	1366.1	PVC	0.50%	1.015	1.90	11.99	2.16	0.95	OK
211B	ST34	Outlet			3.87		2.32	2.32	15.0	67.7	0.437	675	271.1	PVC	0.50%	0.594	1.66	2.72	1.82	0.74	OK
211G	ST22	Outlet			0.67		0.40	0.40	15.0	67.7	0.076	375	67.1	PVC	0.50%	0.124	1.12	1.00	1.17	0.61	OK
211D	ST27	Outlet			0.45		0.27	0.27	15.0	67.7	0.051	300	15.2	PVC	0.50%	0.068	0.97	0.26	1.06	0.74	OK
211E	ST29	Outlet			1.75		1.05	1.05	15.0	67.7	0.198	525	18.6	PVC	0.50%	0.304	1.40	0.22	1.49	0.65	OK
211F	ST32	Outlet			0.48		0.29	0.29	15.0	67.7	0.054	300	11.2	PVC	0.50%	0.068	0.97	0.19	1.08	0.79	OK
211A	ST42, ST52	Outlet			5.67		3.40	3.69	15.0	67.7	0.695	750	448.0	PVC	0.50%	0.787	1.78	4.19	2.01	0.88	OK
				Total Area =		**21.46**															
Street A																					
208C	ST95	Outlet			4.48		2.69	2.69	15.0	67.7	0.506	675	248.0	PVC	0.50%	0.594	1.66	2.49	1.87	0.85	OK
				Total Area =		**4.48**															
Street K																					
204A	ST85	Outlet			3.12		1.87	1.87	15.0	67.7	0.353	675	312.1	PVC	0.30%	0.460	1.29	4.04	1.42	0.77	OK
204C	ST84	Outlet			2.85		1.71	1.71	15.0	67.7	0.322	675	371.5	PVC	0.30%	0.460	1.29	4.81	1.39	0.70	OK
Street L																					
204B	ST83	Outlet			3.95		2.37	2.37	15.0	67.7	0.446	750	383.5	PVC	0.30%	0.610	1.38	4.63	1.51	0.73	OK
204D	ST82	Outlet			4.59		2.75	2.75	15.0	67.7	0.519	750	327.1	PVC	0.30%	0.610	1.38	3.95	1.55	0.85	OK
				Total Area =		**14.51**															

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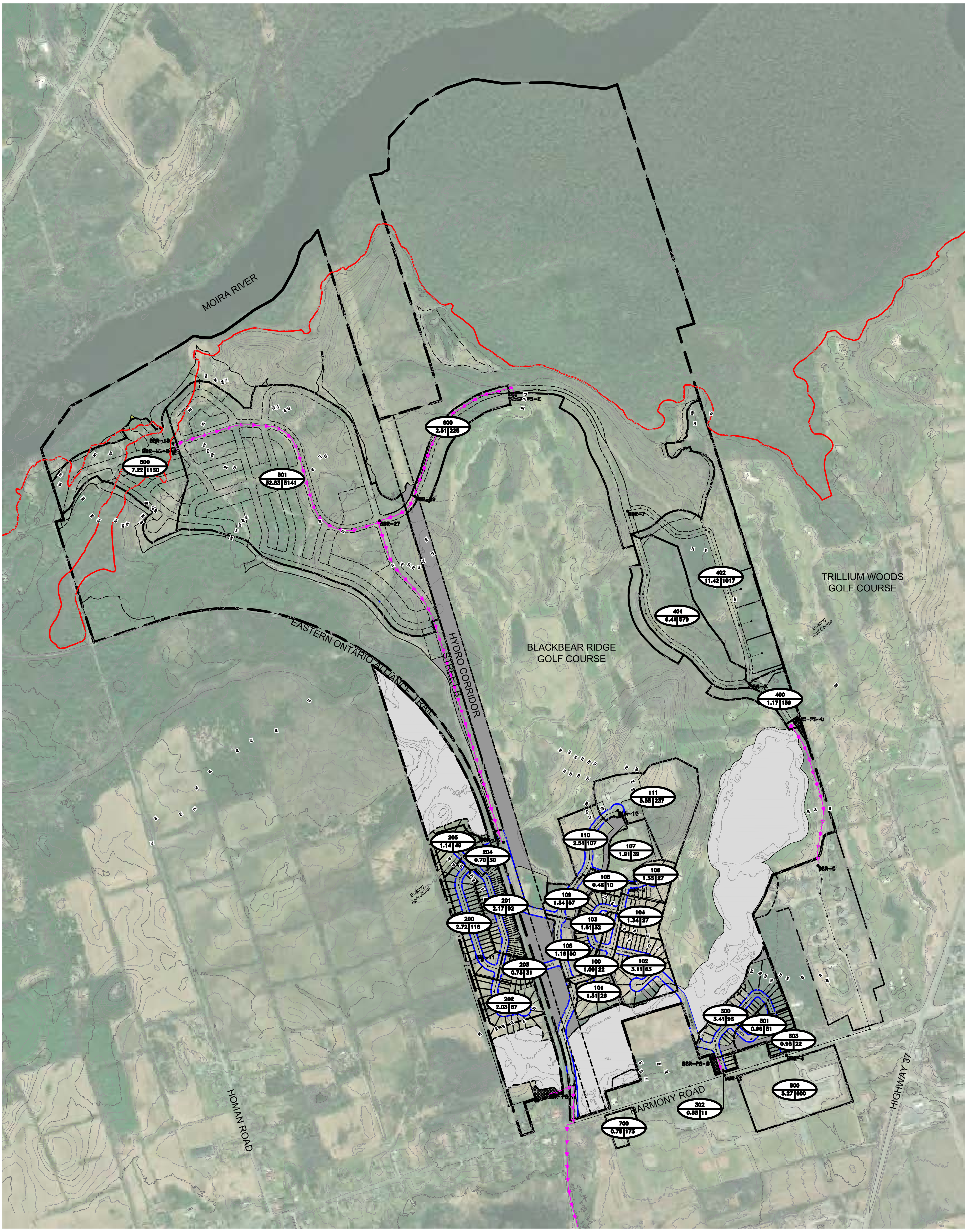
1-71 Millennium Parkway
Belleville, ON, K8N 4Z5
Ph. 613-969-1111
Fx. 613-969-8988
www.jewelleng.ca

Designed: Matthew Warner
Checked: Bryon Keene, P.Eng.
Date: July 16, 2024

Project:
Black Bear Ridge Development
South Portion



JEWELL ENGINEERING

GENERAL NOTES:

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE:

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.
- LIDAR CGVD 2013

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY
1	31/01/25	FIRST SUBMISSION	DFM

LEGEND

700 / 0.78 | 173	CATCHMENT AREA ID NUMBER
	AREA IN HECTARES (ha) / POPULATION
	ENVIRONMENTAL CONSTRAINTS
	SANITARY FORCE MAIN
	100 YEAR FLOOD LIMIT

N
W E
S

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
PROVINCE OF ONTARIO

BLACK BEAR RIDGE GP INC.
BLACK BEAR RIDGE VILLAGE

PRELIMINARY
SANITARY CATCHMENT AREA PLAN

DRAWN BY: DFM/AG
PROJECT NO: 2205260
DESIGNED BY: DFM/AG
DATE: February 2025
CHECKED BY: BK
CONTRACT NO:
APPROVED BY: BK
SCALE:
HORIZONTAL - 1:5000
VERTICAL -
DRAWING NO: SAN-2