FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT



2105 Headon Road, City of Burlington

Proposed Townhouses development

Report Prepared by:

EVERWEST ENGINEERING

205-1030 Upper James Street Hamilton,
Ontario L9C 6X6

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1 INTRODUCTION AND BACKGROUND

Everwest Engineering has been retained by JEC Properties to prepare this Functional Servicing & Stormwater Management Report for the redevelopment of 2105 Headon Road in the City of Burlington. The Site is bounded by a public school to the north, Headon Road to the south, Neighboring residential to the west and the Tuck Creek to the east. The site is currently a one storey brick dwelling, with a block of eight new three storey townhouses proposed for the lot. The total site area is approximately 0.4 ha, which includes the Tuck Creek regulated area to the east. Refer to legal survey by MMP land Surveyors ltd. dated September 2020.

2 STUDY PARAMETERS

This Functional Servicing and Stormwater Management Report has been prepared in coordination with regard to the following documents:

Ref. 1: Stormwater Management Design Guidelines, City of Burlington dated May 2020

Ref. 2: Water and Wastewater Linear Design Manual, version 6, dated NOV 2024.

Ref. 3: Region of Halton 2022 development charges background study for water, wastewater, roads & general services development charges

Ref. 4: Ministry of the Environment and Climate Change (MOECC) – Stormwater Management Practices Planning and Design Manual (Ministry of Environment, March 2003)

3 DESIGN POPULATION

The projected design population for the re-development is 19.5 people. Table 1 shows the population calculations.

Table 1– Population Density calculation

2105 Headon Road- Population		
Floor Area (ha)	**Equivalent Population Density (Person/Hectare)[1]**	**Population[2]**
0.075	260	19.5

[1] Equivalent population Density based on Table 2-1, 2.3 Equivalent Population Water and Wastewater Linear Design Manual, version 6, dated NOV 2024 (pg:6)

[2] Population= site area*Equivalent Population Density

4 WATER SUPPLY

4.1 Existing Conditions

The existing site has a one storey brick building and existing water lateral service location, size and invert unknown. There is an existing 600mmø watermain fronting the site and a 300mmø watermain at the far side of the property as per region of Halton drawing B-5754. Existing water lateral (public side) to be abandoned and detached at the main per region of Halton standards. The nearest fire hydrant to the property is situated in front of 2110 Headon Road, approximately 135 meters away.

4.2 Domestic water demand

The domestic and fire water demand for the Site was calculated based on the Region of Halton's Water and Wastewater Linear Design Manual Section 2 – Watermain Design. The relevant design criteria are summarized below:

Table 2 – Domestic Water Design

Pop. [1]	Average Consumption Rate (L/cap/d) [2]	Average Daily Demand (L/s) [3]	Max. Day Peaking Factor [4]	Max. Daily Demand (L/s) [5]	Max. Hour Peaking Factor (L/s) [4]	Max. Hourly Demand (L/s) [6]
19.5	265	0.059	2.25	0.1327	4.0	0.236

[1] Refer to Table 1
[2] Average Consumption Rate per Table B-1, water demand criteria section, Region of Halton 2022 development charges background study for water, wastewater, roads & general services development charges
[3] Average daily demand = population*average consumption rate
4 Peaking factors per Table 2-2 – Maximum hourly demand peaking factor in Water and Wastewater Linear Design Manual, dated Nov 2024
[5] Maximum daily demand = average daily demand*maximum daily peaking factor
[6] Maximum hourly demand= average daily demand*maximum hourly peaking factor

4.3 Proposed Water Connection

As shown in **Table 2**, the estimated average day demand will be 0.059 L/s, with the peak hour demand calculated to be 0.236L/s. Based on the design flow, a 150mmø water service connection to the existing 300mmø water main located on the far side of Headon Road will be required to support the proposed development. For each lot, a 19mmø service line will connect to the 150mmø water lateral to service the proposed 8 townhouse units. An on-site fire hydrant will be provided within the property to ensure a hydrant is located 45m from each building entrance. A 150mmø lead will be connected to the proposed 150mmø domestic service line to service the on-site fire hydrant. Fire hydrant will be installed and colored per the City of Burlington standards. The provided domestic watermain is expected to meet the required water demands. A hydrant flow test will be conducted to confirm that the existing pressure in the municipal system is adequate to service the development.

5 SANITARY SERVICING

5.1 Existing conditions

The site is located on Headon Road. There is an existing 450mmø sanitary sewer fronting the property as per Region of Halton drawing B-5754. The existing site has a one storey brick building with the existing sanitary lateral service location, size, and invert unknown (if one exists). The existing sanitary lateral (public side) needs to be abandoned and removed per region of Halton standards.

5.2 Sanitary Demand

Wastewater generation for the site was calculated based on the Water and Wastewater Linear Design Manual Halton Standards. **Table 3** summarizes the sanitary sewer discharge rates from the proposed site.

Table 3– Sanitary Discharge Flow Rate

Site Area (ha)	Population [1]	Peaking Factor [2]	Peak Flow (L/s) [3]	Infiltration Flow (L/s) [4]	Total Flow (L/s) [5]
0.075	19.5	4.38	0.44	0.021	0.461

[1] Population based on Table 1 above
[2] Peaking factor based on the Harmon Formula: PF = 1 + [14 / (4 + (P/1000)$^{0.5}$)], where P is population
[3] Peak flow = population * 450 L/cap/day * peaking factor
[4] Infiltration flow = 0.286*site area, Infiltration flow based on 0.286 L/s/ha per chapter 3 section 3.2.4.
[5] Total flow = Peak flow * Infiltration flow

5.3 Proposed Sanitary Connection

As shown in Table 3, the estimated total sanitary sewage flow generated by the proposed development is 0.46 L/s. Based on the post development design flow, a new 200mmø sanitary lateral connection at 2.0% discharging to the existing sanitary manhole on Headon Road is required. The proposed sanitary lateral has sufficient capacity to service the development. A downstream sanitary sewer capacity review was completed in coordination with the Region of Halton. The Region confirmed that the existing 450 mm diameter sanitary sewer on Headon Road has sufficient capacity to accommodate the proposed sewage flows from the development.

6 STORM WATER MANAGEMENT

6.1 City of Burlington Stormwater Management Requirements

Based on the City of Burlington's pre-submission application comment form, the following stormwater management (SWM) criteria will be applied to the site:

a. Quality Control (COB SWM 6.1.4.1 and 6.1.4.2):

• Enhanced 80.0% TSS removal achieved with a treatment train approach, to incorporate Low Impact Development strategies. An OGS will only be credited with 50.0% TSS removal.

• OGS CA-ETV certified units only. OGS to be sized to capture and treat a min. 90.0% of the runoff volume.

• Rooftop areas may be considered clean if the flows are separated from non-roof areas as per COB SWM 3.6 and 6.1.4.1.

b. Quantity Control (COB SWM 6.1.1.1):

- 100-year post development flows controlled to 5-year pre-development flow rates by providing on-site storage for all City of Burlington design storm events up to and including the 100-year.
- Discharge to a watercourse control post development to pre-development for the 2-year to the 100-year.
- Discharge to a storm sewer control either the 100-year post development or Regional Storm, whichever is greater to the 5-year pre-development.
- A max. pre-development imperviousness of 36.0% (C = 0.50) for re-development is to be used for the pre-development calculations.

c. Erosion Control (COB SWM 3.4 and 6.1.2):

• 25mm / 24hr min. with 48 hours preferred or retention of the 90 the percentile rainfall 27mm if a creek outlet proposed.

d. Water Budget (COB SWM 3.5 and 6.1.3):

• Strongly recommend the first 5mm of rainfall be retained and infiltrated or reused onsite.

e. Groundwater:

• If there is a foundation drainage collection system, flows must be accommodated in the quantity control for the site, and discharge must meet the Storm Sewer Discharge Bylaw 59-2022 Table 1 criteria, as amended.

f. External Flows:

• Identify all external flows and safely accommodate them under the developed conditions to avoid upstream flooding or ponding.

g. Overland Flow Route (COB SWM 5.3.2 and 6.1.1.3):

Provide the emergency overland flow route on the plans. Include the neighboring properties to ensure existing drainage patterns are being maintained.

6.2 Existing Condition

In the existing condition, the lot has split drainage with the front half of the lot draining towards Headon Road, and the remaining half draining towards the Tuck Creek. The lot is generally well drained with the fall across the site ranging between 2.0% and 5.0%. A review of the 2105 Headon Road as-built drawings provided by City of Burlington drawing number MH – 5, shows that there is an existing 600mmø storm sewer fronting the property. The existing conditions have been defined by two (2) catchment areas, Catchment's EX1, EX2. See **Table 4** and the Pre-Development Drainage Area Plan in Appendix A for details.

Table 4 – Existing Conditions Catchment Areas

Catchment ID	Description	Area (ha)	Runoff Coefficient, C
EX1	Site area draining towards Headon Road	0.18	0.43
EX2	Site area draining towards creek and neighbouring backyard	0.21	0.29
	Total	**0.40**	**0.35**

6.3 Proposed Conditions

The proposed grading plan maintains the existing conditions draining the front half towards Headon Road and the southeast side of the lot (*the top of bank buffer and Tuck Creek*) has been left undisturbed. A portion of the lot contains existing mature trees and will be maintained in the existing condition Two drainage areas, Catchments P1 and P2, have been identified in the proposed design. See **Table 5** and the Post-Development Drainage Area Plan in Appendix A for details.

Table 5 – Proposed Conditions Catchment Areas

Catchment ID	Description	Area (ha)	Runoff Coefficient, C
P1	Site area draining towards Headon Road	0.27	0.74
P2A	Site area draining towards creek and neighbouring backyard	0.13	0.25
	Total	**0.40**	**0.54**

6.4 Geotechnical Considerations

An infiltration test was completed by Soil-Mat Engineers and Consultants Ltd dated Aug 23, 2024. A letter from Soil-Mat Engineers and Consultants Ltd. is included in Appendix B. The letter details that “the native soils to consist of a thin layer of silty clay/clayey silt overburden soils with Queenston Shale bedrock at relatively shallow depths, on the order of approximately 1 to 2 meters below the existing grade or shallower.” Based on this information we can confirm that infiltration is not a viable solution in this location and have therefore designed an active control system to meet the City’s requirements.

6.5 Stormwater Management Design

The site was modeled using the modified rational method to determine runoff rates for both pre- and post-development conditions based on the new City standards published May 2020. The model was used to compare post- and pre-development runoff rates for the 5-year and 100-year storms. Pre-development runoff rates were determined for catchments EX1 and EX2, and post-development rates for catchments P1 and P2. Storage requirements for each storm event were then assessed, and a storage system was designed according to the results.

6.6 Quantity control

The design includes four catch basins along the internal road to collect runoff from the front half of the roof's and the surrounding surface area. Additionally, two catch basins are positioned on the north side to capture drainage from the rear half of the roof's and rear yards. The runoff collected by these basins is directed into a subsurface gallery located in the front yard. With the site runoff captured, a minimally sized 75mm diameter orifice control tube will provide stormwater control prior to the runoff being released into the existing storm sewer. As detailed in **Table 6** below and the modelling in Appendix C, 91.82m^3 of storage is required to control the post-development 100-year storm to the pre-development 5-year storm. A stormwater management system consisting of 112 EZ Storm half unit modules stacked on 184 full units, catch basins and, storm sewers provides up to 92.22m^3 of storage. The storage provided exceeds the required volume and provides spare capacity for the sump pump drainage. **Table 6** below details the model results.

Table 6– Stormwater Management Design Summary

Stormwater Management Design Summary				
Catchment	**Q Control (m^3/s)**	**100yr to 5yr (m^3)**	**Storage Required (m^3)**	**Storage Provided (m^3)**
EX1 - P1 (Flow towards Outlet)	0.023	91.82	91.82	92.22

A detailed copy of the stormwater modelling is included in Appendix C. The designed storage system is detailed on Drawings by C03 Everwest Engineering.

6.7 Quality Control

To provide stormwater quality control for Catchment P2, a jelly fish filter model JF4-1-1 has been proposed to be installed on the most downstream catchbasin manhole within the site prior to the runoff entering the SWM system. This location has been selected to treat the stormwater from the road and driveways prior to being released into the municipal storm sewer. The proposed road and driveways are the primary source of stormwater contamination on the site (from vehicle traffic), with the rest of the site consisting primarily of the building roof and vegetated areas. Catchment P2 consists of uncontrolled runoff flowing towards the Tuck Creek, therefore, no quality controls are provided for this catchment area. See *Appendix D* for Jelly fish filter specifications.

6.8 Stormwater Tank Maintenance Details

The chambers should be inspected every 6 months and after each major rainfall event during the first year to ensure that the system is free of any debris. In subsequent years, the system should be inspected semi-annually, or more if deemed necessary for this specific site. If the average depth of sediment exceeds 3 inches at the sump cleanout, maintenance should be performed. Maintenance should be executed using a vacuum pump truck to evacuate sediment and debris from system. The system should be flushed with clean water, with care taken to avoid extreme direct water pressures and is to be performed in dry weather. Material removed from the unit will be disposed of in a similar manner to that of other stormwater management facilities. The design specifies a litter trap to be installed in each catch basin (Enviropod Littatrap or equivalent). This trap will capture litter, debris, leaves, mulch, etc. that are conveyed into the catch basin. The homeowner is required to maintain this litter trap regularly by opening the catch basin lid, removing the trap, cleaning out the debris into a yard waste bag/bin, and replacing the trap in the catch basin. Regular maintenance of this trap will reduce the frequency of maintenance needed on the underground portion of the system and extend the lifetime of the system.

7 SEDIMENT AND EROSION CONTROL

In order to minimize erosion during the site grading and servicing period of construction, the following measures will be implemented:

- Install silt fencing along the outer boundary of the site to ensure that sediment does not migrate to the adjacent properties.
- Install a construction mud mat at the site entrance to minimize dirt being tracked into the municipal right of way.
- Install sediment control (silt sacks) in the proposed catchbasin and catchbasin manholes as well as the nearby existing catchbasins to ensure that no untreated runoff enters the existing conveyance system.
- Stabilize all disturbed or landscaped areas with hydro seeding/sodding to minimize the opportunity for erosion.

For details on the proposed erosion and sediment control for the proposed site, see the Grading & Servicing Plan included in the engineer drawings by Everwest Engineering.

8 CONCLUSION

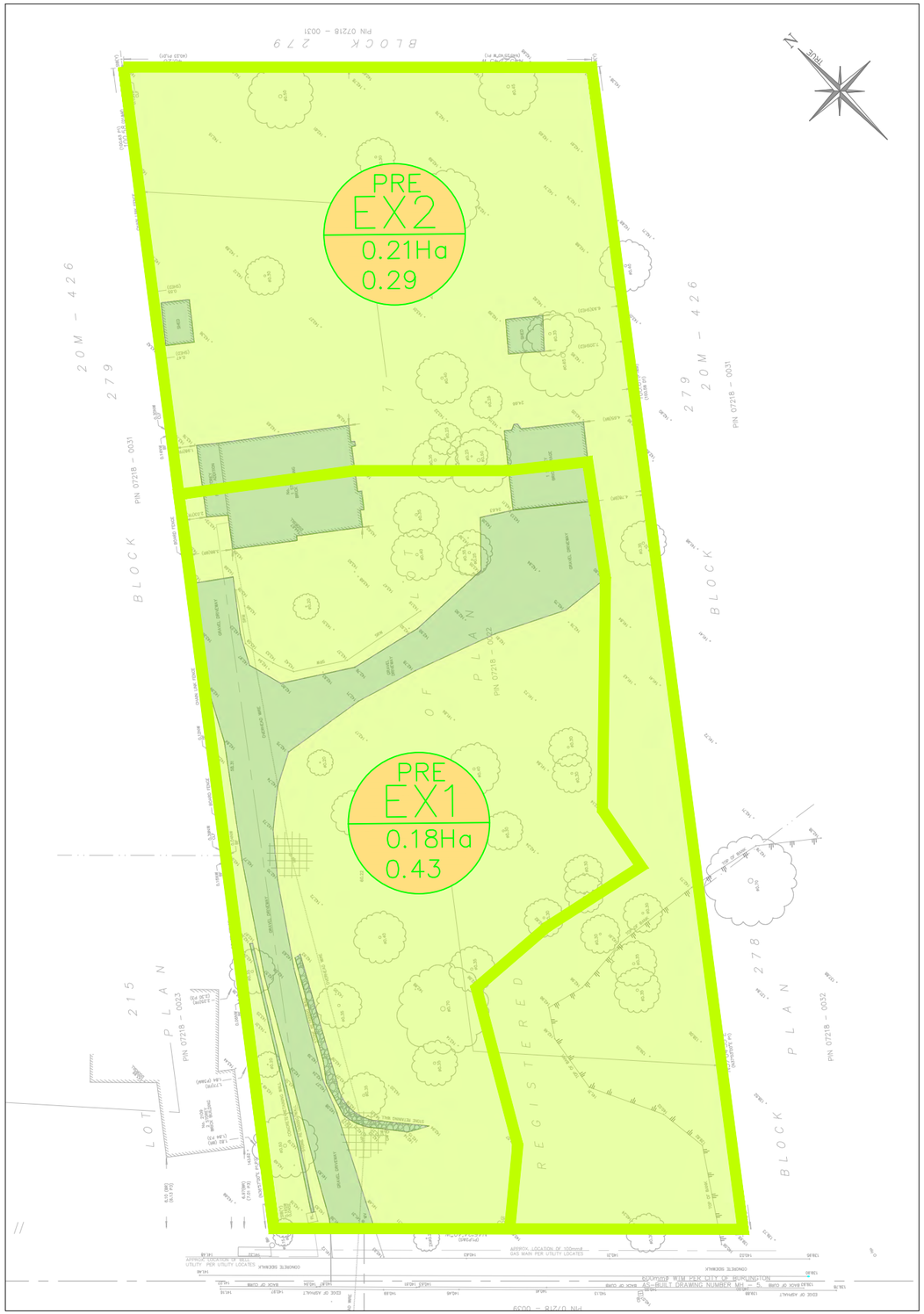
Based on the information provided herein, it is concluded that the proposed redevelopment of 2105 Headon Road can be constructed to meet the requirements of the City of Burlington and Region of Halton. Therefore, it is recommended that:

- The development is to be graded and serviced in accordance with the Grading Plan and Servicing Plans prepared by Everwest Engineering.
- Erosion and sediment controls need to be installed as described in this report to meet the City of Burlington's requirements.
- The proposed sanitary and water servicing system be installed as per the Servicing Plan and this report.
- A stormwater management system is proposed to provide post to pre stormwater control for the 5 and 100-year storms. The system designed exceeds the city of Burlington's minimum standards for stormwater management.

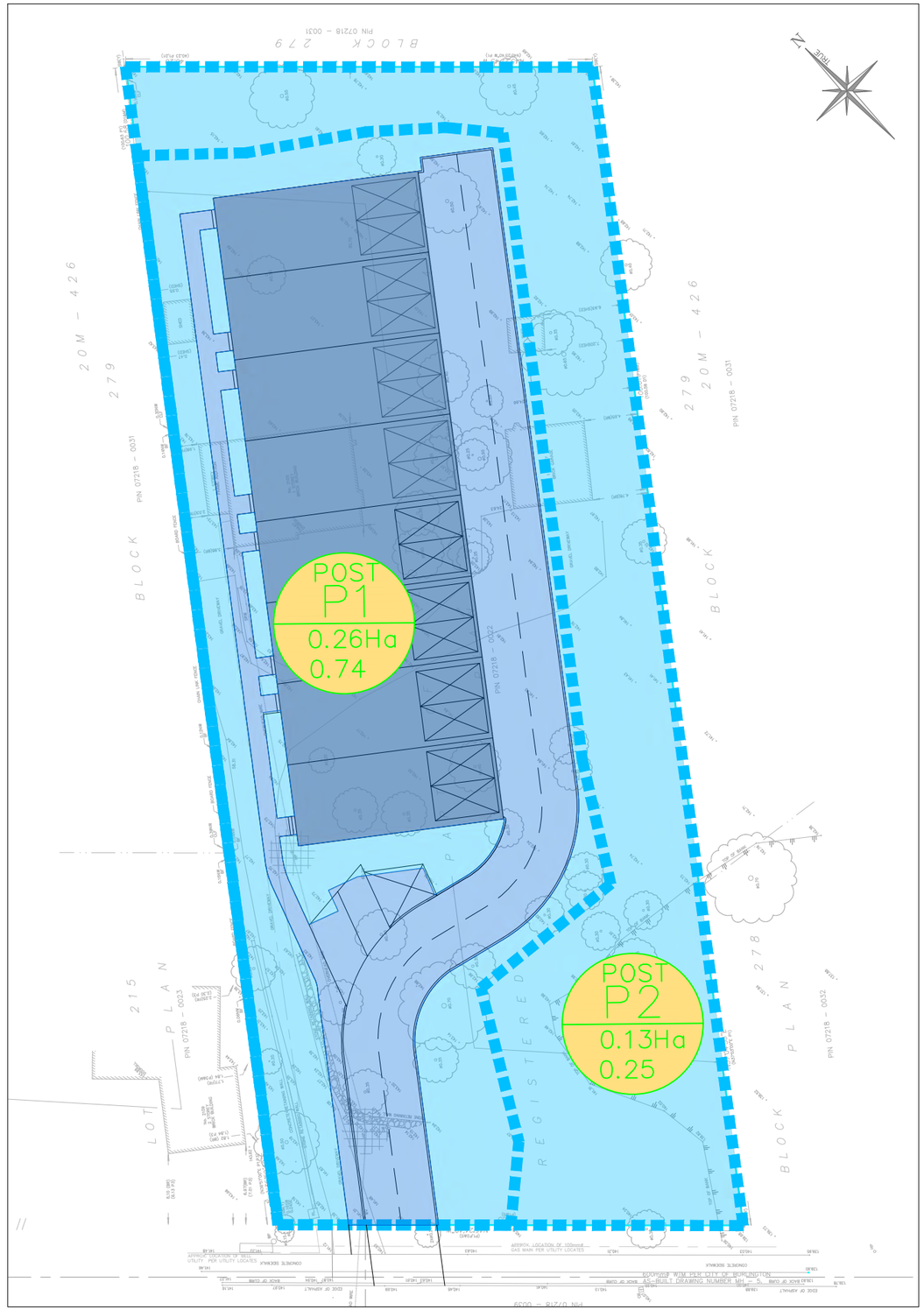
Wesley Taylor, P.Eng
Everwest Engineering
www.everwesteng.com
admin@everwesteng.com
t. 905 526-3404



APPENDIX A – FIGURE 1 –DRAINAGE BOUNDARY AREAS



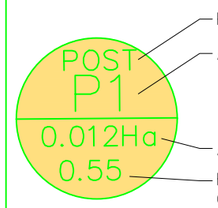
PRE-DEVELOPMENT STORMWATER DRAINAGE BOUNDARY



POST-DEVELOPMENT STORMWATER DRAINAGE BOUNDARY

LEGEND

- PRE-DEVELOPMENT DRAINAGE BOUNDARY
- PRE-DEVELOPMENT PERVIOUS AREA
- PRE-DEVELOPMENT IMPERVIOUS AREA
- POST-DEVELOPMENT DRAINAGE BOUNDARY
- POST-DEVELOPMENT PERVIOUS AREA
- POST-DEVELOPMENT IMPERVIOUS AREA



OVERLAND FLOW PATHWAY

Project

HEADON ROAD
BURLINGTON

Drawing

DRAINAGE
BOUNDARIES

EVERWEST ENGINEERING

Project No. 25-0105

Scale NTS

Dwg. FIG 1

APPENDIX B – GEOTECHNICAL REPORT

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

401 Grays Road · Hamilton, ON · L8E 2Z3

www.soil-mat.ca info@soil-mat.ca 905.318.7440 / 800.243.1922 (*toll free*) 905.318.7455

PROJECT NO.: SM 301961-G October 22, 2024

JEC PROPERTIES
5341 John Lucas Drive
Burlington, Ontario
L7L 6A8

Attention: Mohammad Abdullah

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
2105 HEADON ROAD
BURLINGTON, ONTARIO

Dear Mr. Abdullah,

Further to your authorisation, SOIL-MAT ENGINEERS & CONSULTANTS LTD. has completed the fieldwork, laboratory testing, and report preparation in connection with the above noted project. The scope of work was completed in general accordance with our proposal P301961, dated July 9, 2021 and revised on April 20, 2023. Our comments and recommendations based on our findings at the four [4] borehole locations are presented in the following paragraphs.

1. INTRODUCTION

We understand that the project will involve the construction a nine-unit townhouse block, along with installation of associated underground municipal services along an asphalt paved roadway, upon demolition of the existing structures located at 2105 Headon Road in Burlington, Ontario. The purpose of this geotechnical investigation work was to assess the subsurface soil and groundwater conditions, and to provide our comments and recommendations with respect to the design and construction of the proposed development, from a geotechnical point of view.

This report is based on the above summarised project description, and on the assumption that the design and construction will be performed in accordance with the applicable codes and standards. Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, this office must be consulted to review the new design with respect to the results of this investigation. It is noted that SOIL-MAT ENGINEERS has completed a Phase One ESA for the subject site, which has been reported under a separate cover.

2. PROCEDURE

A total of four [4] sampled boreholes were advanced at the locations illustrated in the attached Drawing No. 1, Borehole and Slope Profile Location Plan. The boreholes were advanced using continuous flight power auger equipment on August 23, 2024 under the direction and supervision of a staff member of SOIL-MAT ENGINEERS & CONSULTANTS LTD., to termination at depths of approximately between 4.8 to 6.3 metres below the existing ground surface.

Upon completion of drilling, monitoring wells were installed in Borehole Nos. 1 and 4 to allow for the future monitoring of the groundwater level. The monitoring wells consisted of 50-millimetre PVC pipe, screened in the lower 3.0 meters. The monitoring wells were encased in well filter sand up to approximately 0.3 metres above the screened portion, then with bentonite 'hole plug' to the surface and fitted with a protective steel 'stick up' casing. The remaining boreholes were backfilled in general accordance with Ontario Regulation 903, and the ground surface was reinstated even with the surrounding grade.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of ASTM test specification D1586, Standard Penetration Resistance Testing. After undergoing a general field examination, the soil samples were preserved and transported to the SOIL-MAT laboratory for visual, tactile, and olfactory classifications. Routine moisture content tests were preformed on all soil samples recovered from the borings, with hand penetrometer testing conducted on all cohesive samples. Additionally, two [2] selected soil samples were subject to laboratory grain size

The boreholes were located in the field by representatives of SOIL-MAT ENGINEERS, based on accessibility over the site and clearance of underground utilities. The ground surface elevation at the borehole locations has been referenced to a site specific geodetic benchmark, noted as the south corner of an existing dwelling, as illustrated in the Drawing No. 1, Borehole Location Plan. The benchmark is noted to have an elevation of 143.62 metres as per MMP survey drawing, Project No. 20-152, dated September 22, 2020, forwarded to our office.

Details of the conditions encountered in the boreholes, together with the results of the field and laboratory tests, are presented in Log of Boreholes Nos. 1 to 4 inclusive, following the text of this report. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made while drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed at the exact depths of geological change.

Additionally, a representative slope profile was measured, based on accessibility in the area of slope on the south side of the property, from the existing fence line, down to relatively flat area on the south side. The location of this slope profile is illustrated on the attached Drawing No. 2, Slope Profile Location Plan, while the slope profile itself is illustrated on the attached Drawing No. 3, Slope Profile A-A. It is noted that due to limited access over the slopes from fallen trees over the slope, this was the only accessible area to measure a profile, and based on our visual assessment of the slope, is considered representative of the slope section nearest the development.

A Slope Stability Rating Chart as per the Ontario Ministry of Natural Resources publication "Geotechnical Principles for Stable Slopes" [Geotechnical Principles publication] was completed for the slope at the worst-case scenario at Slope Profile A-A, and indicated a Rating Value of 32. This corresponds to a slight potential for slope instability. A copy of the Slope Stability Rating Charts have been appended to the end of this report.

3. SITE DESCRIPTION AND SUBSURFACE CONDITIONS

The subject site is consists of an existing residential dwelling with associated granular driveway and parking area, located at 2105 Headon Road in Burlington, Ontario. The subject site is surrounded by residential properties with CH Norton Public School located on north side of the site. The project area is relatively flat around the existing house structure, with gently sloping down to Headon Road on south side, and down to the valley lands of Tuck Creek on south and east side of the property with an elevation difference of approximately 4 metres.

The subsurface conditions encountered at the borehole locations are summarised as follows:

Topsoil

A surficial veneer of topsoil approximately 200 millimetres in thickness was encountered at Borehole Nos. 3 and 4. It is noted that the depth of topsoil may vary across the site and from the depths encountered at the borehole locations, and would be expected to be thicker in areas of heavy vegetation. As such, a conservative approach should be taken in estimating topsoil quantities across the site. It is also noted that the term 'topsoil' has been used from a geotechnical point of view, and does not necessarily reflect the materials nutrient content or ability to support plant life.

Pavement Structure

Borehole Nos. 1 and 2 were advanced through the existing granular surfaced driveway, which was found to consist of approximately 200 millimetres of compact granular material.

Silty Clay/Clayey Silt

Native silty clay/clayey silt was encountered beneath the topsoil or pavement structure in all the borehole locations. The native cohesive soils were reddish brown in colour, contained trace to some sand, trace gravel, occasional cobbles, and were generally very stiff to hard in consistency. The native soils were proven to termination at depths of between approximately 5.2 to 6.3 metres below existing ground surface. It is noted that the upper levels of the native soils generally had a weathered or reworked appearance in upper levels, likely due to weathering and having been subjected to freeze/thaw cycles. It is also noted that while not explicitly identified during our investigation, some fill deposits associated with grading, construction of existing structures, etc., may be present across the site.

Grain size analyses were conducted on two [2] selected samples of the native soils encountered. The results of this testing can be found appended to the end of this report, and are summarized as follows:

TABLE A – GRAIN SIZE ANALYSES

Sample ID	Depth	% Clay	% Silt	% Sand	% Gravel	Hydraulic Conductivity, k [cm/s]
BH1 SS4	2.6 m	34	46	16	4	10^{-7}
BH4 SS3	1.8 m	39	47	12	2	10^{-7}

The field and laboratory testing demonstrate the native soils to predominantly consist of Silt and Clay with some Sand and trave Gravel. According to the Unified Soil Classification System (USCS), the soils are classified as M.L. – clayey silts with slight plasticity to inorganic silts. The silty clay/clayey silt soils encountered have a low hydraulic conductivity, on the order of 10^{-7}cm/sec, and would be considered effectively impermeable.

A review of available published information [Quaternary Geology of Ontario, Southern Sheet Map 2556] indicate the subsurface soils to consist of clay to silt-textured till, consistent with our experience in the area, the above analyses, and our observations during drilling.

Groundwater Observations

All the boreholes were recorded as open and 'dry' upon completion of drilling. It is noted that insufficient time would have passed for the static groundwater level to stabilise in the open boreholes.

As noted above, monitoring wells were installed in Borehole Nos. 1 and 4 to allow for future measurements of the groundwater level. The details of the monitoring well installation, as well as the groundwater measurements taken, have been summarized as follows.

TABLE B: SUMMARY OF GROUNDWATER MEASUREMENTS

BH/MW#	Ground Surface Elevation (m)	MW Depth (m)	Screened Interval (m)	September 9, 2024		October 3, 2024	
				Water Depth (m)	Water Elevation (m)	Water Depth (m)	Water Elevation (m)
1	142.78	6.1	3.1 – 6.1	2.3	140.5	2.8	140.0
4	142.86	6.1	3.1 – 6.1	5.3	137.6	4.7	138.1

Note: It is noted that, the above elevations are referenced to a site specific temporary benchmark with an assigned elevation of 100 meters for convenience.

Based on the conditions encountered at the borehole locations it is anticipated that the relatively shallow groundwater levels observed in Borehole No. 1 may be artificially shallow, influenced by permeable seams. The available groundwater data to date indicates a groundwater level on the order of perhaps 4 to 5 metres below the existing ground surface, at elevations of approximately between 137.5 to 138.5 metres, varying with the physical topography, and would be expected to fluctuate seasonally. These groundwater depths are below the anticipated depths of construction. Regardless, shallower 'perched' deposits within permeable seams, etc., should be expected, especially during the 'wet' times of the year.

4. SLOPE CONDITIONS AND STABILITY ASSESSMENT

As noted above, a representative profiles of the subject slope, identified as Slope Profile A-A, was measured in the area of slope on the south side of the property, from the existing fence line, down to the valley lands on the south side. The subject slope on the south side of the property was measured to be approximately 4.5 metres in height, flat near the existing fence line sloping down to the south with an overall inclination of about 1.8 horizontal to 1 vertical. The ground surface was noted to be predominately vegetated with young to mature trees with exposed roots, and some patches of grass

and scrub vegetation. No evidence of significant surficial movements, failure scars, tension cracks, or other signs of global slope instability were observed at the time of our site visit.

As with all slopes, there is a reduction in shearing resistance attributed to the effects of freezing and thawing, wetting and drying, burrowing animals, etc. With time, the surface of the slope will degenerate and tend to reach equilibrium within its stress and ambient environment, including vegetative cover. However, this degeneration of the slope angle is a very slow process as is evident by the present condition of the existing slope.

A stability analysis of the subject slope was performed with a computerized modelling program [SLOPE/W 2021] utilising multiple methods of analysis [Ordinary, Bishop and Janbu, Morgenstern-Price, etc.] and considering different slip planes and centres of rotation, as well as normal and elevated groundwater conditions, to determine the minimum factors of safety for the slope.

Soil properties for the subsurface soils have been conservatively attributed based on the soil strata encountered in the boreholes advanced. In this case the subsurface soils have been modelled as native silty clay/clayey silt, and have been conservatively assigned a unit weight of γ = 19.5 kN/m^3, an angle of internal friction of ϕ = 32 degrees, and cohesion of c = 5 kPa.

Based on our analyses using the conservative soil parameters noted above, the subject slope was found to have minimum factors of safety, with respect to the top of stable slope location (delineated through the Ministry of Natural Resources method noted below) on the order of 1.88 or greater for normal groundwater conditions, and on the order of 1.72 or greater under elevated groundwater conditions. Sample results of these analyses are attached to the end of this report.

The Ministry of Natural Resources “Geotechnical Principles for Stable Slopes” publication, Table 7.2 lists a minimum Factor of Safety of 1.3 to 1.5 for Active Land Use properties [habitable or occupied structures near the slope]. As the calculated factors of safety were found to be within or exceed this range, the subject slope would be considered stable in the short and long term, with respect to the long-term top of stable slope.

4.1 TOP OF STABLE SLOPE

The top of stable slope location (based on the Ministry of Natural Resources method) is determined by the application of an erosion allowance at the toe and a stable slope inclination through the slope. In this case, considering the condition of the toe of the slope, and the absence of any watercourse within 15 metres of the toe of the slope, an erosion allowance of 0 metres would be considered appropriate. Table 4.3 of the Ministry of Natural Resources publication "Geotechnical Principles for Stable Slopes" indicates stable slope inclinations through glaciolacustrine clays and silts in the range of 2.0 to 4.0 horizontal to 1.0 vertical. Based on the conditions encountered at the borehole location, a stable slope inclination of 2.0 horizontal to 1 vertical has been applied for the very stiff to hard silty clay/clayey silt soils. Applying this stable slope inclination and erosion allowance results in a long-term top of stable slope location approximately 0.6 metres "uphill" of the physical crest, as illustrated on Drawing No. 1, Borehole and Slope Profile Location Plan.

4.2 CONSTRUCTION CONSIDERATIONS

It is our opinion that the existing slope is considered to be stable in the long-term, and that the construction of the proposed townhouse and associated laneway uphill of the established long-term top of stable slope location, would have no negative impact on the stability of the subject slope, from a geotechnical point of view.

The following recommendations should be considered in the proposed construction:

- Excavated soil and/or heavy construction equipment should not be placed or travel near the top of stable slope location. In this regard it is recommended that a silt fence be placed along, or slightly 'uphill' from, the top of stable slope for the duration of construction. This will act as a barrier for construction activity and also prevent sediment runoff during construction.
- Any drainage towards the slope should be in a controlled fashion, such as sheet flow through well-established grass or vegetation, so as to not alter the natural drainage over the slope or create concentrated flows onto the slope.
- Vegetation on the slope should be protected during construction and all existing trees on the slope maintained. Uphill of the crest of the slope the vegetation should be repaired/improved with the establishment of new deep-rooted vegetation post construction, where required, in accordance with the site landscape plan.

5. EXCAVATIONS

Excavations for the installation of foundations and underground services are anticipated to extend to depths of up to approximately 2 to 3 metres below the existing grade. Excavations through the native silty clay/clayey silt soils should be relatively straightforward, with the sides remaining stable for the short construction period at slopes of up to 60 degrees to the horizontal. Where wet seams are encountered, during periods of extended precipitation, or where excavations extend below the static groundwater level, the sides of excavations should be expected to 'slough in' to as flat as 3 horizontal to 1 vertical, or flatter. Notwithstanding the foregoing, however, all excavations must comply with the current Occupational Health and Safety Act and Regulations for Construction Projects. Excavation slopes steeper than those required in the Safety Act must be supported and a senior geotechnical engineer from this office should monitor the work. With respect to the Safety Act, the competent native silty clay/clayey silt soils would be considered as Type 2 soil.

As noted above the static groundwater level is conservatively anticipated at depths of approximately 4 to 5 metres below the existing ground surface, below the anticipated depths of construction. Regardless, some infiltration of water from permeable seams, as well as from surface runoff, should still be anticipated, especially during the 'wet' times of the year. The rate of infiltration should be sufficiently low, such that it should be possible to control via conventional construction dewatering techniques for excavations extending near to slightly below the static groundwater level. More water should be expected when connections are made to existing services. Surface water should be directed away from the excavations.

Any utility poles, light poles, etc. located within 3 metres of the top of an excavation slope should be braced to ensure their stability. Likewise, temporary support might be required for other existing above and below ground structures, including existing underground services, depending on their proximity to the trench excavations.

6. BACKFILL CONSIDERATIONS

The excavated materials will primarily consist of the native silty clay/clayey silt soils encountered in the boreholes, as described above, as well as fill materials associated with existing structures. The native soils are generally considered suitable for use as engineered fill, trench backfill, etc., provided they are free of organics, debris, or other deleterious material, and that their moisture contents can be controlled to within 3 percent of their standard Proctor optimum moisture content. Some selective sorting to remove organics, debris, etc. from fill deposits associated with existing structures, should be expected to be required.

It is noted that the cohesive soils encountered are not considered to be free draining, and should not be used where this characteristic is required. It is also noted that these soils present difficulties in achieving effective compaction where access with compaction equipment is restricted. The silty clay/clayey silt soils are generally considered to be near to slightly 'dry' of their standard Proctor Optimum moisture content. Some moisture conditioning may be required depending upon the weather conditions at the time of construction. It is noted that these soils will become nearly impossible to compact when wet of its optimum moisture content. Any material that becomes wet to saturated should be spread out to allow to dry, or removed and discarded, or utilised in non-settlement sensitive areas.

We note that where the backfill material is placed near or slightly above its optimum moisture content, the potential for long term settlements due to the ingress of groundwater and collapse of the fill structure is reduced. Correspondingly, the shear strength of the 'wet' backfill material is also lowered, thereby reducing its ability to support construction traffic and therefor impacting construction. If the soil is well dry of its optimum value, it will appear to be very strong when compacted, but will tend to settle with time as the moisture content in the fill increases to equilibrium condition. The cohesive soils may require high compaction energy to achieve acceptable densities if the moisture content is not close to its standard Proctor optimum value. It is therefore very important that the placement moisture content of the backfill soils be within 3 percent of their standard Proctor optimum moisture content during placement and compaction to minimise long term subsidence [settlement] of the fill mass. Any imported fill required in service trenches or to raise the subgrade elevation should have its moisture content within 3 percent of its optimum moisture content and meet the necessary environmental guidelines.

A representative of SOIL-MAT should be present on-site during the backfilling and compaction operations to confirm the uniform compaction of the backfill material to project specification requirements. Close supervision is prudent in areas that are not readily accessible to compaction equipment, for instance near the end of compaction 'runs', and around the foundation walls. Any structural fill required, backfill within service trenches, areas to be paved, etc., should be placed in loose lifts of 300 millimetres maximum thickness, and compacted with the heaviest available appropriate equipment, i.e. pad-toe roller for cohesive soils, smooth drum roller for granular materials, to a minimum of 98 per cent standard Proctor maximum dry density [SPMDD], with monitoring and testing be a representative of SOIL-MAT ENGINEERS on a full time basis. A method should be developed to assess compaction efficiency employing the on-site compaction equipment and backfill materials during construction.

7. MANHOLES, CATCH BASINS AND THRUST BLOCKS

Properly prepared bearing surfaces for manholes, valve chambers, etc. in the native competent soils, stabilised where required, will be practically non-yielding under the anticipated loads. Proper preparation of the founding soils will tend to accentuate the protrusion of these structures above the pavement surface if compaction of the fill around these structures is not adequate, causing settlement of the surrounding paved surfaces. Conversely, the pavement surfaces may rise above the valve chambers and around manholes under frost action. To alleviate the potential for these types of differential movements, free-draining, non-frost susceptible material should be employed as backfill around the structures located within the paved roadway limits, and compacted to a minimum of 98 per cent of its SPMDD.

The thrust blocks in the native soils may be conservatively sized as recommended by the applicable Ontario Provincial Standard Specification conservatively using a horizontal allowable bearing pressure of up to 150 kPa [~3,000 psf]. Any backfill required behind the blocks should be a well-graded granular product and should be compacted to a minimum of 98 per cent of its SPMDD.

8. TOWNHOUSE CONSTRUCTION

It is anticipated that the proposed structure will have a founding level on the order of approximately 2 to 3 metres below the existing ground surface. The native soils encountered at these depths are considered capable of supporting bearing capacities of 200 kPa [~4,000 psf] SLS and 300 kPa [~6,000 psf] ULS or more, however such higher bearing capacities are not expected to be required. Depending on the depths of any fill deposits encountered, some sub-excavation of unsuitable materials below founding elevations may be required. Where such fill deposits are encountered, the founding level may be raised from the competent native soils up to the design founding elevation via the placement of lean-mic [~5 MPa] concrete, or other methods. This would be best assessed at the time of construction, if required. The founding surfaces must be hand cleaned of any loose or disturbed material, along with any ponded water, immediately prior to placement of foundation concrete.

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of soil type and the tolerable deflections for serviceability, respectively. Evidently, the bearing capacity would be lower for very settlement sensitive structure and larger for more flexible buildings.

The support conditions afforded by the native soils and/or engineered fill are generally not uniform across the building footprint, nor are the loads on the various foundation elements. As such, the provision of nominal steel reinforcement should be considered in the concrete foundations. Such reinforcement will act to reduce the potential for cracking in the foundation walls due to minor settlements, heaving, shrinkage, etc. and will assist in resisting the earth pressures generated against the foundation walls by the backfill. This nominal reinforcement is an economical approach to the reduction or prevention of costly foundation repairs after completion and later in the life of the buildings. Such nominal reinforcement would typically consist of two continuous 15M steel bars placed in the footings [directly below the foundation wall] and a similar two continuous 15M steel bars placed approximately 300 millimetres from the top of the foundation walls. The reinforcing bars should be bent to reinforce all corners and under basement windows, and be provided with sufficient overlap at staggered splice locations. At 'steps' in the foundations and at window locations, the reinforcing steel should be bent to follow the step diagonally, rather than at 90 degrees, to maintain the continuous tensile capacity of the reinforcement.

All footings exposed to the environment must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation to protect against frost penetration. This frost protection, or equivalent, would also be required if construction were undertaken during the winter months. All footings must be proportioned to satisfy the requirements of the Ontario Building Code.

In areas where it will be necessary to provide adjacent footings at different founding elevations, the lower footing should be constructed before the higher footing is constructed, if possible, and the higher footing should be set below and imaginary line drawn up from the lower footing at 10 horizontal to 7 vertical. This practice will limit stress transfer from the higher footings to lower footings.

With foundations designed as outlined above and as required by the Ontario Building Code, and with careful attention paid to construction detail, total and differential settlements should be well within normally tolerated limits of 25 and 20 millimetres respectively, for the type of building and occupancy expected.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this geotechnical investigation report, and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the borehole locations.

9. PAVEMENT DESIGN CONSIDERATIONS

All areas to be paved should be stripped of all organic or otherwise unsuitable materials. The exposed subgrade should be proof rolled with 3 to 4 passes of a loaded tandem truck in the presence of a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., immediately prior to the placement of the sub-base material. Any areas of distress revealed by this or other means must be sub-excavated and replaced with suitable backfill material. Alternatively, the soft areas may be stabilized by placing coarse crushed stone and 'punching' it into the soft areas. Where the subgrade condition is poorer it may be necessary to implement more aggressive stabilisation methods, such as the use of coarse aggregate [50-millimetre clear stone, 'rip rap', etc.] 'punched' into the soft areas, or the use of stabilizing geogrid and a geofabric separator. The need for the treatment of softened subgrade will be reduced if construction is undertaken during the dry summer months and careful attention is paid to the compaction operations. The fill over shallow utilities cut into or across paved areas such as telephone, hydro, gas, etc. must also be compacted to 98 per cent of its SPMDD.

Good drainage provisions will optimize the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and mitigate softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved areas.

The most severe loading conditions on the subgrade typically occur during the course of construction, therefore precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic. This may include restricting certain areas from construction traffic, or 'bulking' up the access roadways with additional granular sub-base layers during import of granular materials, spreading out/reusing the additional granular material once import is complete.

If construction is conducted under adverse weather conditions, additional subgrade preparation may be required. During wet weather conditions such as during the Fall, Winter and Spring months, it should be anticipated that additional subgrade preparation will be required, such as additional depth of OPSS Granular 'B', Type II (crushed limestone bedrock) sub-base material. It is also important that the sub-base and base granular layers of the pavement structure be placed as soon as possible after exposure, preparation, and approval of the exposed subgrade.

Where roads are to be assumed by the City of Burlington the pavement structure should confirm to the appropriate municipal standard. The suggested pavement structures outlined in Table C below may also be considered. These are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-site soils and past experience. The outlined pavement structure may be expected to have an approximate fifteen to twenty year life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

TABLE C – TYPICAL SUGGESTED PAVEMENT STRUCTURES

LAYER DESCRIPTION	COMPACTION REQUIREMENTS	LIGHT DUTY SECTIONS	HEAVY DUTY [TRUCK ROUTE]
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	Min. 92 % Marshall MRD	40 millimetres	40 millimetres
Binder Course OPSS HL 8	Min. 92 % Marshall MRD	50 millimetres	65 millimetres
Base Course OPSS Granular 'A'	100% SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular 'B', Type II	100% SPMDD	300 millimetres	450 millimetres

* Marshall MRD denotes Maximum Relative Density.
* SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the anticipated traffic, a reduced light duty asphalt structure consisting of 65 millimetres of HL3 surface course may also perform sufficiently. This would be reasonable in areas subjected only to light vehicles such as cars for parking. Such a structure may have a reduced lifespan if subjected to heavier vehicles, and would also not allow for 'mill and pave' type operations for future rehabilitation.

To minimize segregation of the finished asphalt mat, the asphalt temperature must be maintained uniform throughout the mat during placement and compaction. All too often, significant temperature gradients exist in the delivered and placed asphalt with the cooler portions of the mat resisting compaction and presenting a honeycomb surface. As the spreader moves forward, a responsible member of the paving crew should monitor the pavement surface, to ensure a smooth uniform surface. The contractor can mitigate the surface segregation by 'back-casting' or scattering shovels of the full mix material over the segregated areas and raking out the course particles during compaction operations. Of course, the above assumes that the asphalt mix is sufficiently hot to allow the 'back-casting' to be performed.

10. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The material in it reflects SOIL-MAT ENGINEERS' best judgement in light of the information available at the time of preparation. The subsurface descriptions and borehole information are intended to describe conditions at the borehole locations only. It is the contractors' responsibility to determine how these conditions will affect the scheduling and methods of construction for the project. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SOIL-MAT ENGINEERS accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

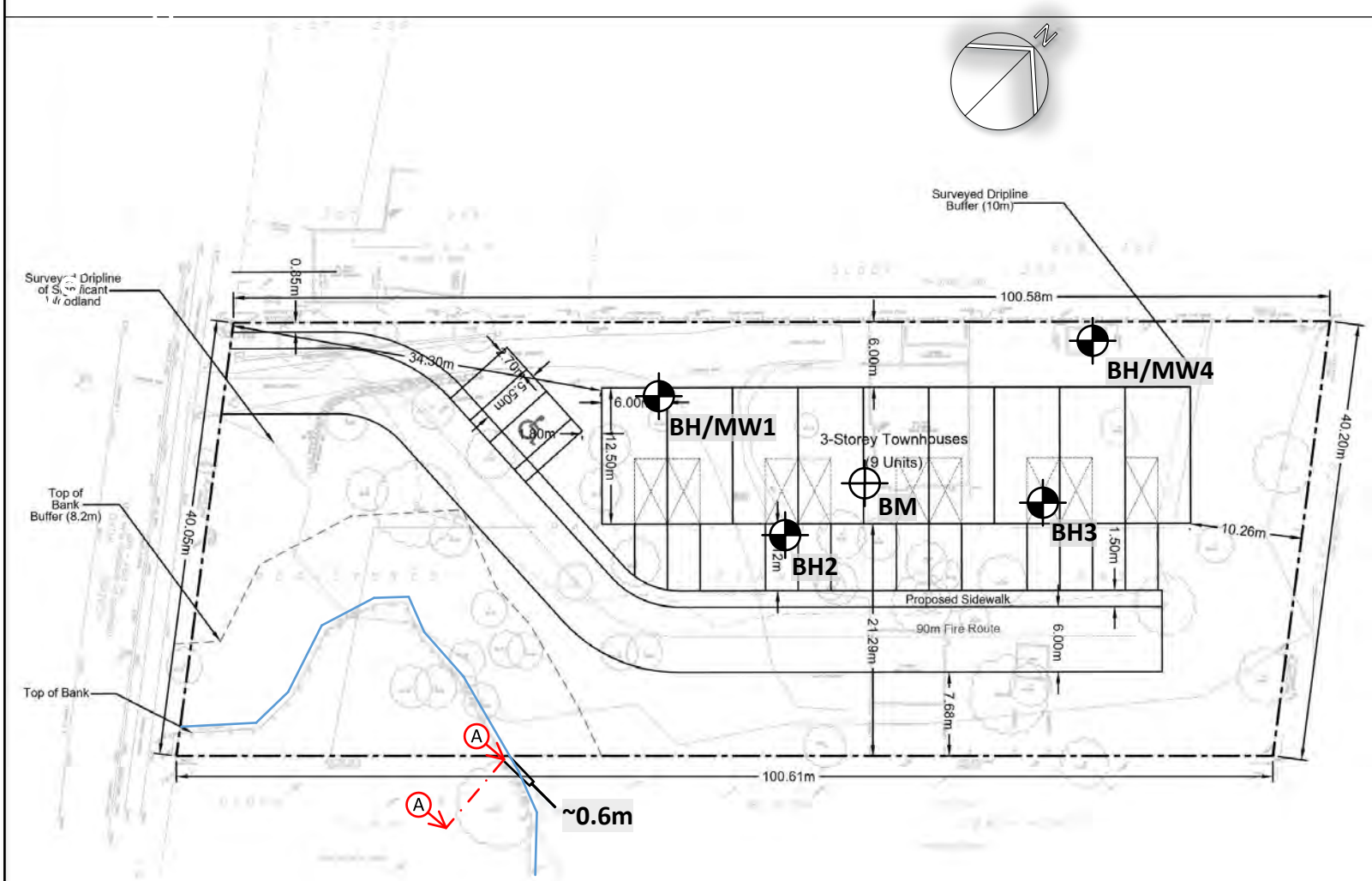
Ishan Chauhan, B.Eng., EIT.
Junior Engineer



Kyle Richardson, P. Eng.
Review Engineer

Enclosures: Drawing No. 1, Borehole and Slope Profile Location Plan
Log of Borehole Nos. 1 to 4, inclusive
Drawing No. 2, Slope Profile A-A
Slope Stability Rating Chart
Slope Stability Analysis
Grain Size Analysis

Distribution: JEC Properties [pdf]



LEGEND

- BH# — Borehole Location
- BM — Geodetic Benchmark
  Southwest corner of an existing house
  Geodetic elevation of 143.62 m.
- Slope Profile Location
- Long Term Top of Stable Slope Location

NOTES

1. This drawing should be read in conjunction with SOIL-MAT ENGINEERS & CONSULTANTS LTD. Report No. SM 301961-G.
2. Borehole, Slope profile and Top of Stable Slope locations are approximate.

SOIL-MAT

ENGINEERS & CONSULTANTS LTD.

Geotechnical Investigation
2105 Headon Road
Burlington, Ontario

Borehole and Slope Profile Location Plan

Project No. SM 301961-G

Date: October, 2024

Drawn: IC | Checked: KR

SM 301961-G Borehole and Slope Profile Location Plan

Drawing No. 2

Log of Borehole No. 1

Project No: SM-301961-G
Project: Proposed Townhouses
Location: 2105 Headon Road, Burlington
Client: JEC Properties

Project Manager: Kyle Richardson, P. Eng
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4803444
E: 596073

Depth (ft / m)	Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	142.78	Ground Surface							
0 / 0		**Pavement Structure** Approximately 200 millimetres of compact granular material.							
0 / 0		**Silty Clay/Clayey Silt** Reddish brown, some gravel, occasional to frequent cobbles, stiff to hard.	SS	1	16,8,7,6	15			
3 / 1			SS	2	5,5,7,13	12		>4.5	
6 / 2			SS	3	9,17,23,20	40		>4.5	
9 / 3			SS	4	6,12,18,24	30		>4.5	
12 / 4			SS	5	9,15,20,23	35		>4.5	
15 / 5			SS	6	27,50/4"	100		>4.5	
20 / 6	136.50		SS	7	50/1"	100			
		End of Borehole							

NOTES:

1. Borehole was advanced using hollow stem auger equipment on August 23, 2024 to termination at a depth of 6.3 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

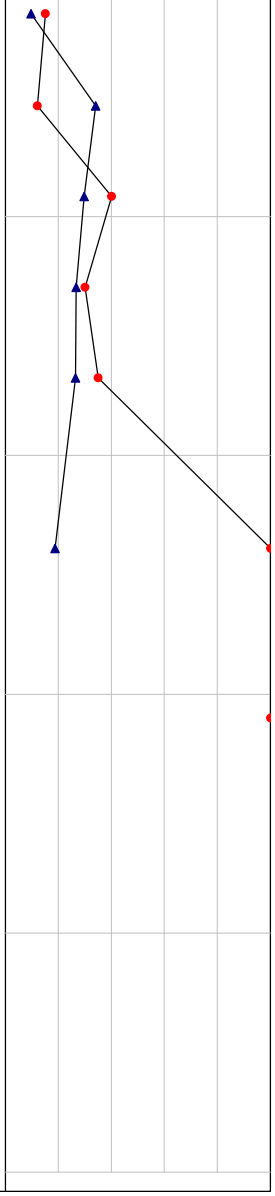
3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

September 9, 2024 - 2.28 metres

Moisture Content w% (10 20 30 40)

Standard Penetration Test blows/300mm (20 40 60 80)



Drill Method: Hollow Stem Auger
Drill Date: August 23, 2024
Hole Size: 200 Millimetres
Drilling Contractor: Elements

Soil-Mat Engineers & Consultants Ltd.
401 Grays Road, Hamilton, Ontario, L8E 2Z3
T: 905.318.7440, TF: 800.243.1922, F: 905.318.7455
www.soil-mat.ca E: info@soil-mat.ca

Datum: Geodetics
Field Logged by: AS
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 2

Project No: SM-301961-G
Project: Proposed Townhouses
Location: 2105 Headon Road, Burlington
Client: JEC Properties

Project Manager: Kyle Richardson, P. Eng
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4803436
E: 596088

Depth (ft / m)	Elevation (m)	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	142.87	Ground Surface								
0–3 / 0–1		**Pavement Structure** Approximately 200 millimetres of compact granular material.		SS	1	9,5,3,2	8			
3–6 / 1–2		**Silty Clay/Clayey Silt** Reddish brown, some gravel, stiff to hard.		SS	2	3,5,6,8	11		>4.5	
6–10 / 2–3				SS	3	7,15,29,23	35		>4.5	
10–15 / 3–4				SS	5	10,14,20,20	34		>4.5	
15 / 4.57	138.07			SS	6	50/6"	100		>4.5	
		End of Borehole								

Moisture Content w% (10 20 30 40)

Standard Penetration Test blows/300mm (20 40 60 80)

NOTES:

1. Borehole was advanced using solid stem auger equipment on August 23, 2024 to termination at a depth of 4.8 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Auger
Drill Date: August 23, 2024
Hole Size: 200 Millimetres
Drilling Contractor: Elements

Soil-Mat Engineers & Consultants Ltd.
401 Grays Road, Hamilton, Ontario, L8E 2Z3
T: 905.318.7440, TF: 800.243.1922, F: 905.318.7455
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Datum: Geodetics
Field Logged by: AS
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 3

Project No: SM-301961-G
Project: Proposed Townhouses
Location: 2105 Headon Road, Burlington
Client: JEC Properties

Project Manager: Kyle Richardson, P. Eng
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4803458
E: 596095

Depth (ft / m)	Elevation (m)	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	143.09	Ground Surface								
0 / 0		**Topsoil** Approximately 200 millimetres of topsoil.		SS	1	3,4,4,4	8			
3 / 1		**Silty Clay/Clayey Silt** Reddish brown, some gravel, occasional cobbles, very stiff to hard.		SS	2	5,13,18,24	31		>4.5	
6 / 2				SS	3	5,18,20,23	38		>4.5	
10 / 3				SS	5	8,20,30,30	50		>4.5	
16 / 5	137.90			SS	6	6,31,33,50	64		>4.5	
17 / 5		End of Borehole								

NOTES:

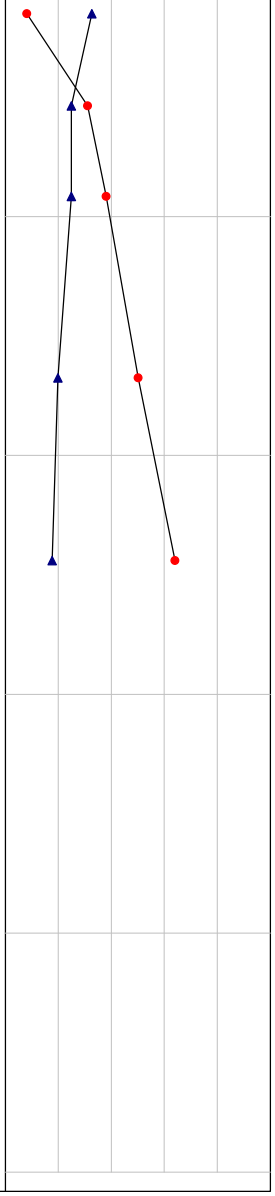
1. Borehole was advanced using solid stem auger equipment on August 23, 2024 to termination at a depth of 5.2 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Moisture Content w% (10 20 30 40)

Standard Penetration Test blows/300mm (20 40 60 80)



Drill Method: Solid Stem Auger
Drill Date: August 23, 2024
Hole Size: 200 Millimetres
Drilling Contractor: Elements

Soil-Mat Engineers & Consultants Ltd.
401 Grays Road, Hamilton, Ontario, L8E 2Z3
T: 905.318.7440, TF: 800.243.1922, F: 905.318.7455
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Datum: Geodetics
Field Logged by: AS
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 4

Project No: SM-301961-G
Project: Proposed Townhouses
Location: 2105 Headon Road, Burlington
Client: JEC Properties

Project Manager: Kyle Richardson, P. Eng
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4803461
E: 596108

Depth (ft / m)	Elevation (m)	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0 / 0	142.86	Ground Surface								
0 / 0	142.60	**Topsoil** Approximatley 200 millimetres of topsoil.								
1 / 0		**Silty Clay/Clayey Silt** Reddish brown, some gravel, occasional cobbles, stiff to hard.		SS	1	3,7,6,6,	13			
2 / 1				SS	2	8,17,23,24	40		>4.5	
3 / 1				SS	3	8,15,18,18	33		>4.5	
4 / 2										
5 / 2				SS	5	10,15,21,19	36		>4.5	
6 / 3										
7 / 3				SS	6	50/6"	100		>4.5	
8 / 4										
9 / 4										
10 / 5										
11 / 5										
12 / 6										
13 / 6										
14 / 7										
15 / 8										
16 / 8										
17 / 9										
18 / 9										
19 / 10										
20 / 10	136.50			SS	7	50/1"	100			
21 / 11		End of Borehole								

Moisture Content w% (10 20 30 40)

Standard Penetration Test blows/300mm (20 40 60 80)

NOTES:

1. Borehole was advanced using hollow stem auger equipment on August 23, 2024 to termination at a depth of 6.3 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

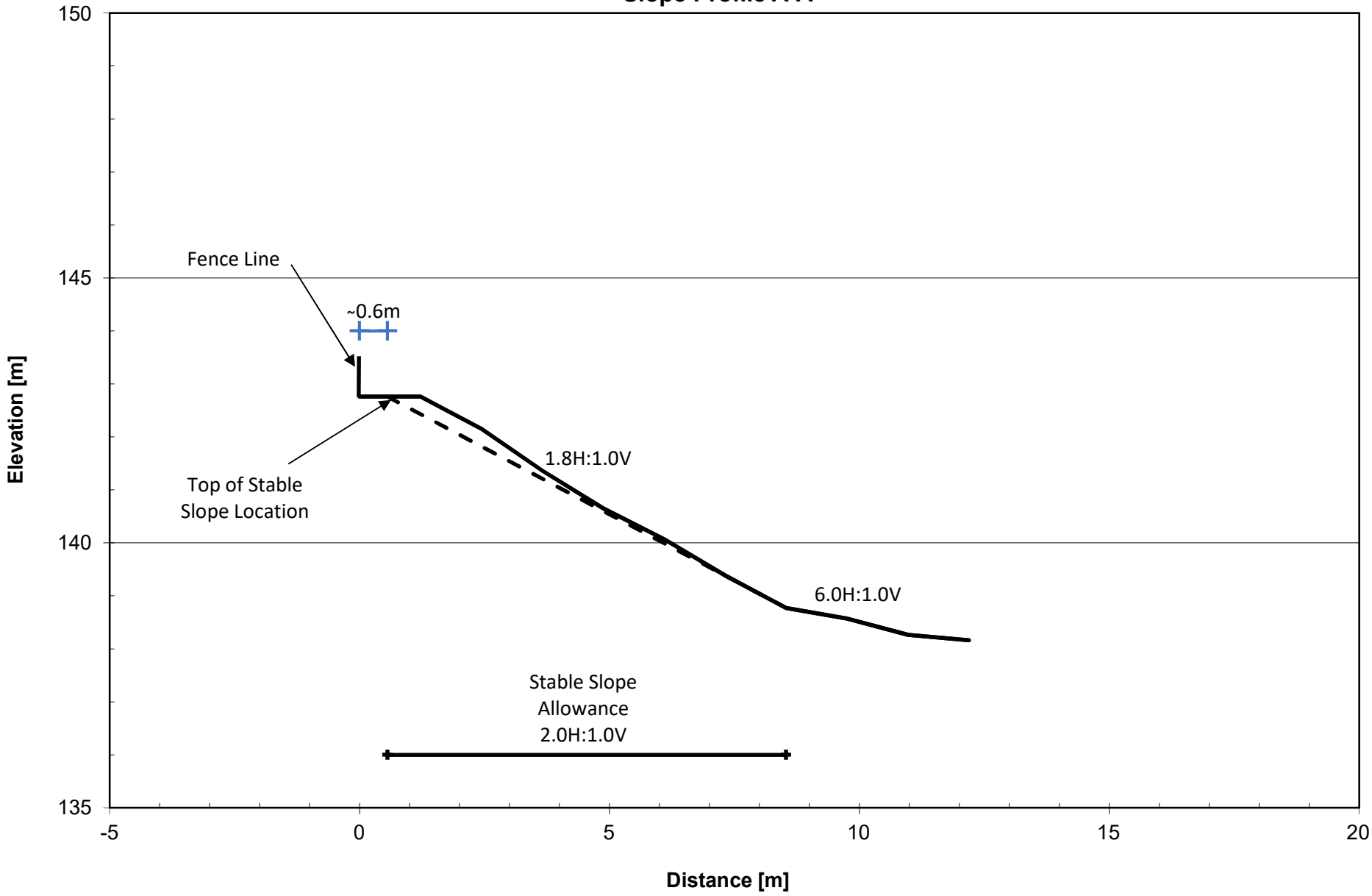
September 9, 2024 - 5.3 metres

Drill Method: Hollow Stem Auger
Drill Date: August 23, 2024
Hole Size: 200 Millimetres
Drilling Contractor: Elements

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Field Logged by: AS
Checked by: KR
Sheet: 1 of 1

Slope Profile Section
2105 Headon Road
Burlington, Ontario
Slope Profile A-A



Project No.: SM 301961-G

DRAWING No. 2

TABLE 8.1 - SLOPE STABILITY RATING CHART

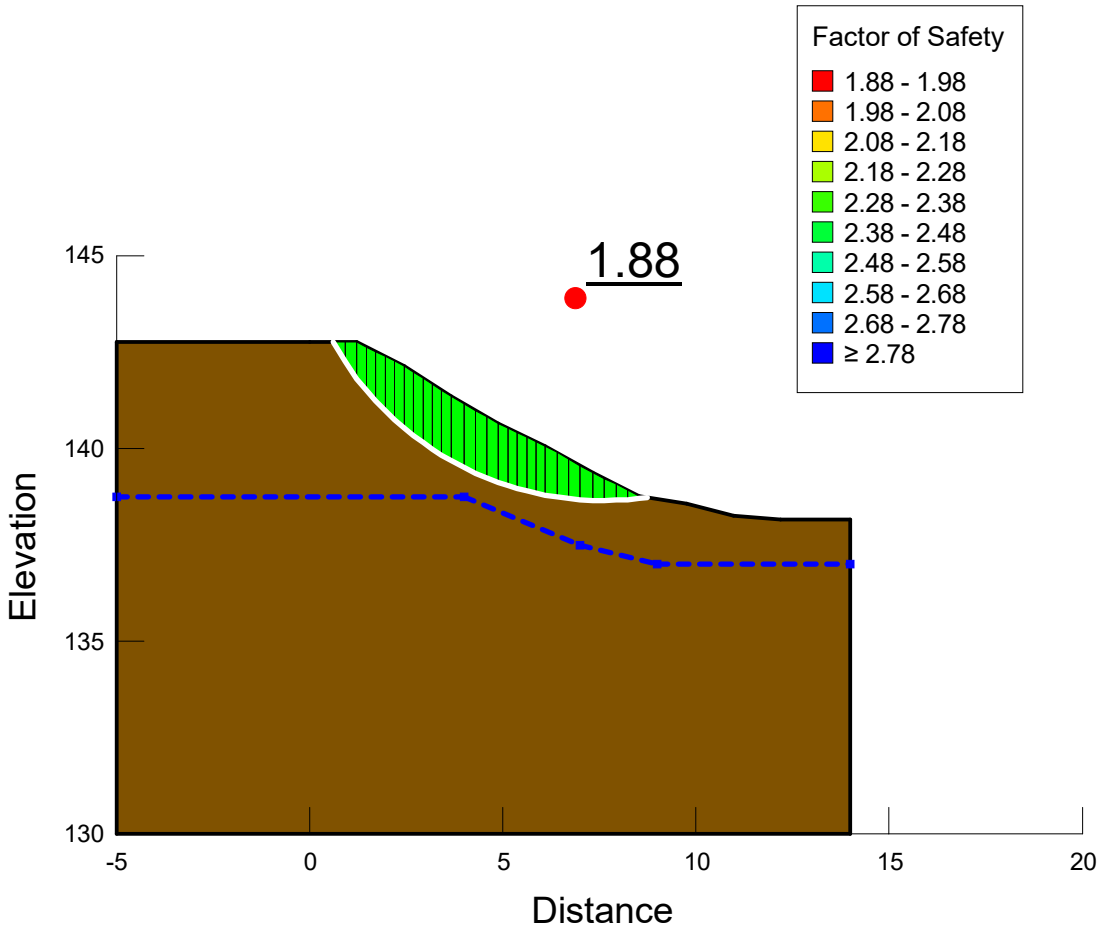
Site Location: 2105 Headon Road, Burlington
Property Owner:
Inspected By: I.C.

File No. SM 301961-G
Inspection Date: October 17, 2024
Weather:

				Rating Value	
1.	**SLOPE INCLINATION**				
		degrees	**horiz. : vert.**		
	a)	18 or less	3 : 1 or flatter	0	
	b)	18 - 26	2 : 1 to more than 3 : 1	6	16
	c)	more than 26	steeper than 2 : 1	16	
2.	**SOIL STRATIGRAPHY**				
	a)	Shale, Limestone, Granite (Bedrock)		0	
	b)	Sand, Gravel		6	
	c)	Glacial Till		9	12
	d)	Clay, Silt		12	
	e)	Fill		16	
	f)	Leda Clay		24	
3.	**SEEPAGE FROM SLOPE FACE**				
	a)	None or Near bottom only		0	
	b)	Near mid-slope only		6	0
	c)	Near crest only or, From several levels		12	
4.	**SLOPE HEIGHT**				
	a)	2 m or less		0	
	b)	2.1 to 5 m		2	
	c)	5.1 to 10 m		4	2
	d)	more than 10 m		8	
5.	**VEGETATION COVER ON SLOPE FACE**				
	a)	Well vegetated; heavy shrubs or forested with mature trees		0	
	b)	Light vegetation; Mostly grass, weeds, occasional trees, shrubs		4	0
	c)	No vegetation, bare		8	
6.	**TABLE LAND DRAINAGE**				
	a)	Table land flat, no apparent drainage over slope		0	
	b)	Minor drainage over slope, no active erosion		2	2
	c)	Drainage over slope, active erosion, gullies		4	
7.	**PROXIMITY OF WATERCOURSE TO SLOPE TOE**				
	a)	15 metres or more from slope toe		0	0
	b)	Less than 15 metres from slope toe		6	
8.	**PREVIOUS LANDSLIDE ACTIVITY**				
	a)	No		0	0
	b)	Yes		6	
	SLOPE INSTABILITY RATING		**RATING VALUES TOTAL**	**INVESTIGATION REQUIREMENTS**	**TOTAL** 32
1.	Low potential		< 24	Site inspection only, confirmation, report letter.	
2.	Slight potential		25-35	Site inspection and surveying, preliminary study, detailed report.	
3.	Moderate potential		> 35	Boreholes, piezometers, lab tests, surveying, detailed report.	

NOTES: a) Choose only one from each category; compare total rating value with above requirements.
b) If there is a water body (stream, creek, river, pond, bay, lake) at the slope toe; the potential for toe erosion and undercutting should be evaluated in detail and, protection provided if required.

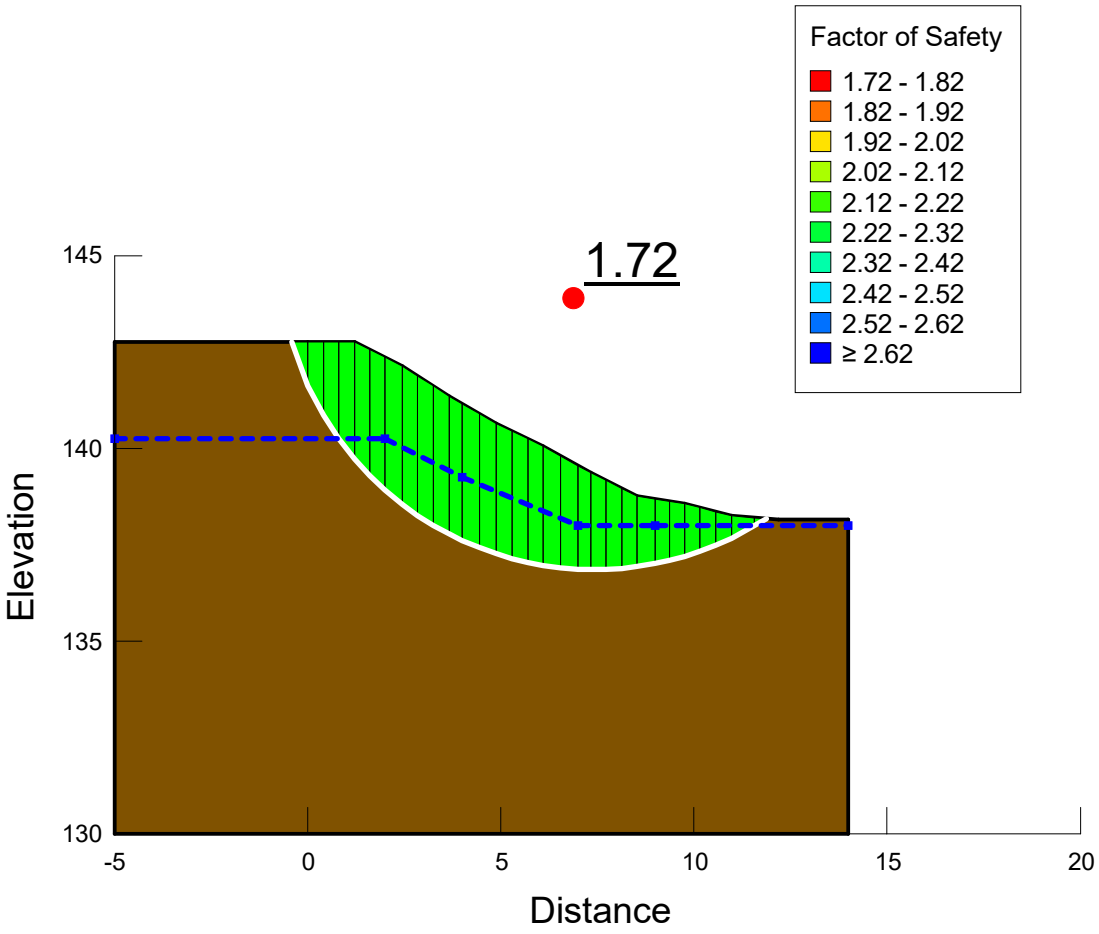
Elevated Groundwater Condition
Slope Profile A-A
2105 Headon Road
Burlington, Ontario



Name: Silty Clay/Clayey Silt
Unit Weight: 19.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 32 °

Slope Stability Analysis - 2105 Headon Road, Burlington
Slope Profile A-A (Normal Groundwater)
October 2024

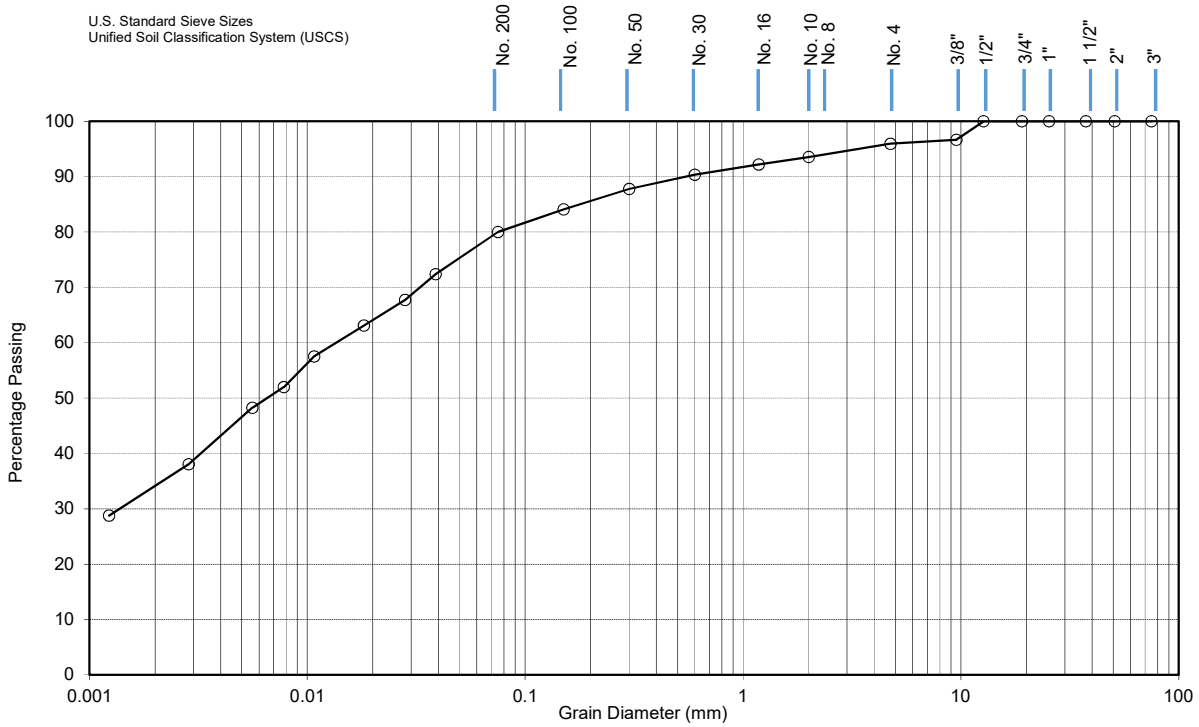
Elevated Groundwater Condition
Slope Profile A-A
2105 Headon Road
Burlington, Ontario



Name: Silty Clay/Clayey Silt
Unit Weight: 19.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 32 °

Slope Stability Analysis - 2105 Headon Road, Burlington
Slope Profile A-A (Elevated Groundwater)
October 2024

Mechanical & Hydrometer Analyses



CLAY	SILT	FINE	MEDIUM	COARSE	FINE	COARSE
		SAND			GRAVEL	

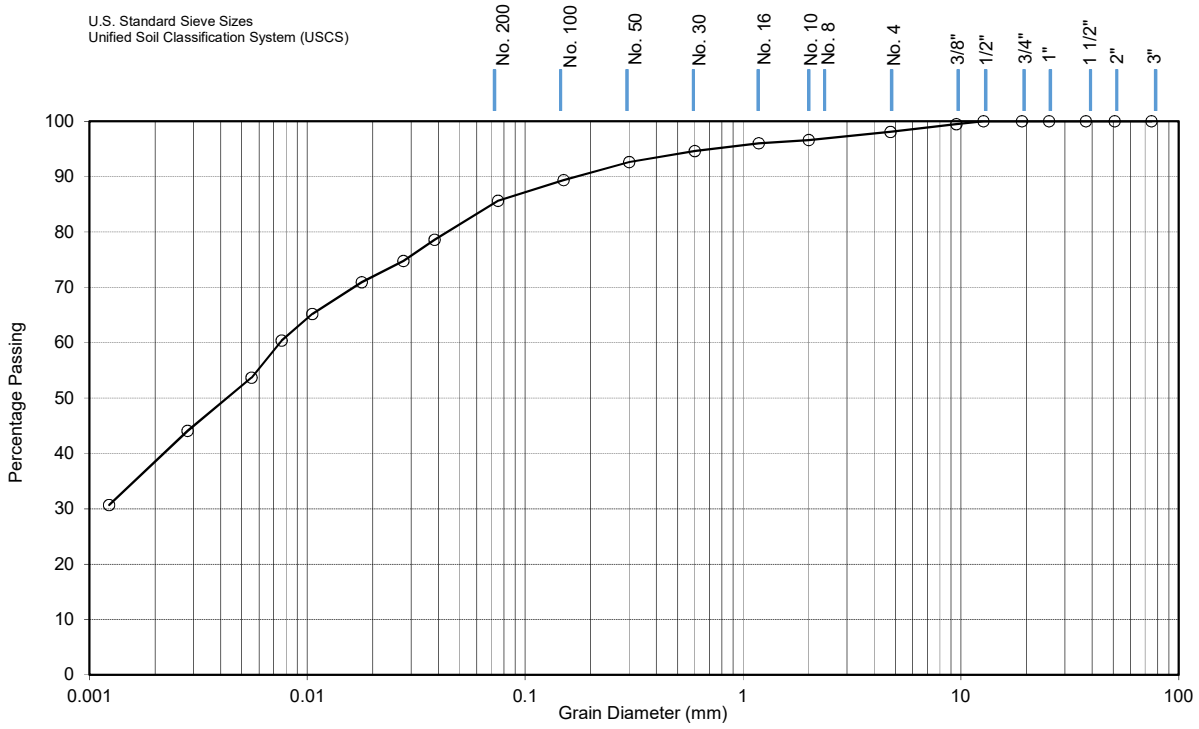
Lab No.:	**24-325**	Notes: Depth : 7.5'			
Borehole No.:	**1**				
Sample No.:	**4**				
CLAY [%]:	**34**	Soil Description: **Brown Clayey Silt w/ some Sand and a trace of Gravel**			
SILT [%]:	**46**	**M.L. - Clayey silts with slight plasticity, inorganic silts and very fine sands, clayey fine sands**			
SAND [%]:	**16**				
GRAVEL [%]:	**4**	Estimated Infiltration Rate [mm/hr] :	**< 10**	Estimated Permeability, k [cm/s]:	**10^{-7}**
D_{10} (Effective Diam. in mm):	**0.0003**	Coefficient of Uniformity C_U:	**46.7**	Coefficient of Curvature C_C:	**0.5**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

2105 Headon Road, Burlington ON

September 2024	**Grain Size Analysis No. 1**	**Project No.: SM 301961-T**

Mechanical & Hydrometer Analyses



CLAY	SILT	FINE	MEDIUM	COARSE	FINE	COARSE
		SAND			GRAVEL	

Lab No.:	**24-326**	Notes: Depth : 5'			
Borehole No.:	**4**				
Sample No.:	**3**				
CLAY [%]:	**39**	Soil Description: **Brown Silt and Clay w/ some Sand and a trace of Gravel**			
SILT [%]:	**47**	**M.L. - Inorganic silts and very fine sands to C.L. - Inorganic clays of low to medium plasticity**			
SAND [%]:	**12**				
GRAVEL [%]:	**2**	Estimated Infiltration Rate [mm/hr] :	**< 10**	Estimated Permeability, k [cm/s]:	**10^{-7}**
D_{10} (Effective Diam. in mm):	**0.0004**	Coefficient of Uniformity C_U:	**19.0**	Coefficient of Curvature C_C:	**0.6**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

2105 Headon Road, Burlington ON

September 2024	**Grain Size Analysis No. 2**	**Project No.: SM 301961-T**

APPENDIX C – STORMWATER MANAGEMENT CALCULATIONS

EVERWEST ENGINEERING

2501 Headon Road

25-0105

Date:2026.03
Design: GKK

Stormwater Management Input Data

Storm Data

Parameters: **City of Burlington IDF Parameters**

Time of Concentration: **10 min**

City of Burlington IDF Parameters						
Return Period	**A**	**B**	**C**	**i (mm/hr)**	**Storm Factor**	**Factored Intensity (mm/hr)**
2 yr	681.5	6.0	0.780	78.39		78.39
5 yr	802.0	5.0	0.764	101.31		101.31
10 yr	918.3	5.0	0.763	116.31		116.31
25 yr	1066.0	5.0	0.762	135.38	1.10	148.9183
50 yr	1172.3	5.0	0.761	149.30	1.20	179.1552
100 yr	1281.3	5.0	0.761	163.18	1.25	203.9712

*Rainfall intensity determined according to City of Burlingtons stormwater Mangement Design Guidelines, Appendix B-Current Rainfall IDF Curves, Intensity-Duration-frequency Values, pg.71.

Pre-Development Weighted Runoff Coefficient

Catchment EX1			
Uncontrolled (Towards Eventual Outlet)			
	C	**A**	**RC**
Pervious	0.25	1416.0	0.188
Impervious	0.95	467.9	0.236
Total		**1883.9**	**0.43**
Area in Ha		**0.188**	

Catchment EX2			
Uncontrolled			
	C	**A**	**RC**
Pervious	0.25	2035.8	0.239
Impervious	0.95	93.9	0.042
Total		**2129.7**	**0.29**
Area in Ha		**0.213**	

Total Site Area	**4013.6**	0.72

Post-Development Weighted Runoff Coefficient

Catchment P1			
Controlled			
	C	**A**	**RC**
Pervious	0.25	835.0	0.078
Impervious	0.95	1851.0	0.655
Total		**2686.0**	**0.74**
Area in Ha		**0.269**	

Catchment P2			
Uncontrolled			
	C	**A**	**RC**
Pervious	0.25	1327.6	0.250
Impervious	0.95	0.0	0.000
Total		**1327.6**	**0.25**
Area in Ha		**0.133**	

Total Site Area	**4013.6**	0.99

EVERWEST ENGINEERING

2501 Headon Road

25-0105

Date:2026.03

Design: GKK

Stormwater Management Calculations
100yr Post to 5yr Pre

City of Burlington IDF Parameters				
Return Period	**A**	**B**	**C**	**i (mm/hr)**
5 yr	802.04	5	0.764	101.3123
100 yr	1281.34	5	0.761	203.97

5yr Storm		
Catchment EX1		
C	0.43	
i	101.31	mm/hr
A	0.188392	ha
Q (pre)	**0.0228**	**m3/s**

100yr Storm		
Catchment P1 - Controlled		
C	0.74	
i	203.97	mm/hr
A	0.2686	ha
Q (post)	**0.1127**	**m3/s**

Outlet Rate		
Q available	0.0228	m3/s
Q orifice	0.0197	m3/s

Refer to Orifice Design Sheet for orifice design sizing

Infiltration Chamber Volume Determination				
T_d	i	T_d	Q_{post}	S_d
min	mm/hr	sec	m3/s	m3
10	203.97	600	0.113	55.81
20	138.27	1200	0.076	73.96
30	107.04	1800	0.059	82.83
40	88.41	2400	0.049	87.70
50	75.89	3000	0.042	90.35
60	66.83	3600	0.037	91.58
70	59.93	4200	0.033	91.82
80	54.49	4800	0.030	91.34
90	50.06	5400	0.028	90.30
100	46.39	6000	0.026	88.82
150	34.49	9000	0.019	77.01
200	27.88	12000	0.015	60.81
350	18.36	21000	0.010	0.35
		Required Storage	**91.82**	**m3**

Equations

Intensity (i)

$$i_{100} = \frac{A}{(t_d + B)^C}$$

Peak Flow (Q)

$$Q = 0.00278 * C * i_{(td)} * A$$

Storage Volume (m3)

$$S_d = Q_{post} * T_d - \frac{Q_{infiltration} * (td+tc)}{2}$$

EVERWEST ENGINEERING

2501 Headon Road

25-0105

Date:2026.03

Design: GKK

Orifice Sizing Design

Orifice Tube Sizing Design		
Orifice Type	Tube	
Invert Elevation	139.13	
Diameter of Orifice Tube	75	mm
Area of Orifice (A)	0.004	m2
Orifice Coefficient (Cd)	0.8	

Head Calculation		
Orifice Centroid Elevation	139.17	m
High Water Elevation	140.75	m**
Upstream Head	1.58	m

Flow Calculation		
Discharge Flow (Q)	Q = (Cd)*(A)*(2gh)^0.5	
Discharge Flow	**0.0197**	**m3/s**

EVERWEST ENGINEERING

2501 Headon Road
25-0105

Date:2026.03
Design: GKK

Stormwater Management Calculations
Post Development Area P1 Infiltration Gallery Sizing

EZ STORM - Chamber Dimensions		
Length	0.80	m
Width	0.80	m
Height	0.66	m
Void Ratio	**96**	**%**

Gallery Design

Stormwater Storage System Design								
	Stormtank M20		**Clearstone Bedding**		**Catch Basin**		Sewers	
Area of Base	**58.80**	m^2	**76.25**	m^2	**0.36**	m^2	0.05	m^2
Depth	1.27	m	1.57	m**	0.85	m	45.00	m
Void Ration	0.96		0.4		1.00		1.00	
Volume	**71.69**	m^3	**18.01**	**m^3**	**0.31**	**m^3**	**2.21**	**m^3**
Total Volume Provided	**92.22**	m^3						
Required Volume	**91.82**	m^3						
Safety Factor	**1.0**							

** Total depth of system including stone below, above, and around the stormpack. The volume calculation subtracts the volume of the stormpack within the stone.

APPENDIX D – JELLY FISH SPECIFICATIONS

ON-LINE
Jellyfish Filter Sizing Report

Project Information

Date	Friday, June 5, 2026
Project Name	2105 Headon Rd.
Project Number	25-0105
Location	Burlington

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.
Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF4-2-1-L1 is recommended to meet the water quality objective by treating a flow of 12.6 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 34 years of HAMILTON A rainfall data for this site. This Jellyfish Model has a Peak Flow Capacity of 83.4 L/s. This model has a sediment capacity of 142 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Peak Capacity (L/s)	Bypass MAW	Sediment Capacity (kg)
JF4-2-1-L1	2	1	1.2	12.6	83.4	Yes	142

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance inverval is designed to be a minimum of 12 months, but this will vary significantly with on-line desings based on storm event conditions, site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.
Please see ***www.ImbriumSystems.com*** *for more information.*

Thank you for the opportunity to present this information to you and your client.

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

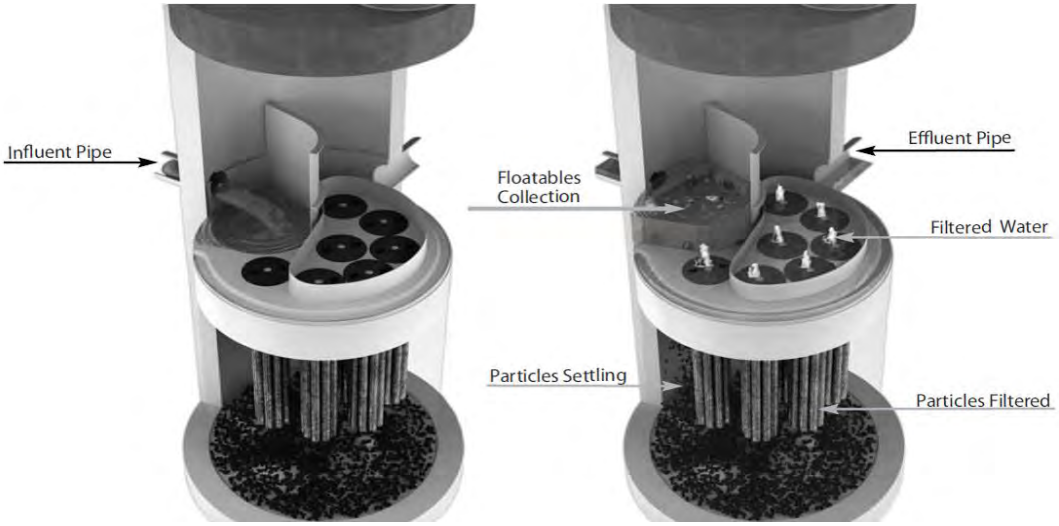
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 59% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

The Jellyfish filter has been field tested on an urban site with 25 TARP qualifying rain events and field monitored according to the TARP field test protocol, demonstrating:

- A median TSS removal efficiency of 89%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 59%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Friday, June 5, 2026
Project Name:	2105 Headon Rd.
Project Number:	25-0105
Location:	Burlington

Designer Information

Company:	Everwest Engineering
Contact:	Jithendar Ravi
Phone #:	

Notes

Rainfall

Name:	HAMILTON A
State:	ON
ID:	3194
Record:	1970 to 2003
Co-ords:	43°10.N'N, 79°56.W'N

Drainage Area

Total Area:	0.27 ha
Runoff Coefficient:	0.68

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 34 years of HAMILTON A rainfall data:	**5.1 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 953 m³, with a suspended sediment concentration of 100 mg/L.	**95 kg***
Peak Flow	The flow which must pass through the unit, includes	**74.5 L/s**

** Indicates that sediment loading is the limiting parameter in the sizing of this Jellyfish system.*

Recommendation

The Jellyfish Filter model JF4-2-1-L1 is recommended to meet the water quality objective by treating a flow of 12.6 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 34 years of HAMILTON A rainfall data for this site. This Jellyfish Model has a Peak Flow Capacity of 83.4 L/s. This model has a sediment capacity of 142 kg, which meets or exceeds the estimated average annual sediment load.

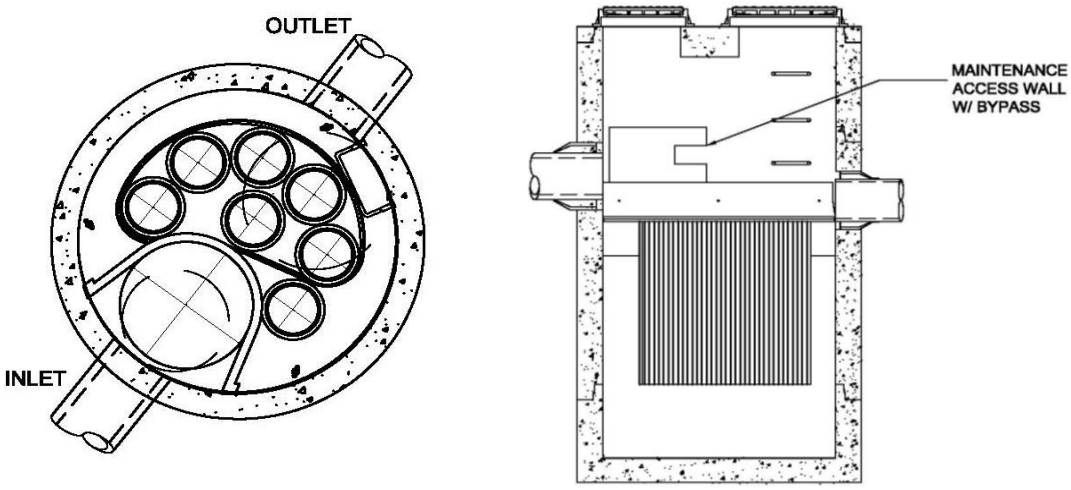
Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Bypass MAW	Addt'l Sump (ft)	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1-L0	1	1	No	0	1.2	2313	0.34	379	7.6	85
JF4-1-1-L1	1	1	Yes	1	1.2	2661	0.34	379	7.6	85
JF4-2-1-L0	2	1	No	0	1.2	2313	0.34	379	12.6	142
JF4-2-1-L1	**2**	**1**	**Yes**	**1**	**1.2**	**2661**	**0.34**	**379**	**12.6**	**142**
JF6-3-1-L0	3	1	No	0	1.8	5205	0.79	848	17.7	199
JF6-3-1-L1	3	1	Yes	1	1.8	6003	0.79	848	17.7	199
JF6-4-1-L0	4	1	No	0	1.8	5205	0.79	848	22.7	256
JF6-4-1-L1	4	1	Yes	1	1.8	6003	0.79	848	22.7	256
JF6-5-1-L0	5	1	No	0	1.8	5205	0.79	848	27.8	313
JF6-5-1-L1	5	1	Yes	1	1.8	6003	0.79	848	27.8	313
JF6-6-1-L0	6	1	No	0	1.8	5205	0.79	848	28.6	370
JF6-6-1-L1	6	1	Yes	1	1.8	6003	0.79	848	28.6	370
JF8-6-2-L0	6	2	No	0	2.4	9252	1.42	1469	35.3	398
JF8-6-2-L1	6	2	Yes	1	2.4	10675	1.42	1469	35.3	398
JF8-7-2-L0	7	2	No	0	2.4	9252	1.42	1469	40.4	455
JF8-7-2-L1	7	2	Yes	1	2.4	10675	1.42	1469	40.4	455
JF8-8-2-L0	8	2	No	0	2.4	9252	1.42	1469	45.4	512
JF8-8-2-L1	8	2	Yes	1	2.4	10675	1.42	1469	45.4	512
JF8-9-2-L0	9	2	No	0	2.4	9252	1.42	1469	50.5	569
JF8-9-2-L1	9	2	Yes	1	2.4	10675	1.42	1469	50.5	569
JF8-10-2-L0	10	2	No	0	2.4	9252	1.42	1469	50.5	626
JF8-10-2-L1	10	2	Yes	1	2.4	10675	1.42	1469	50.5	626
JF10-11-3-L0	11	3	No	0	3.0	14456	2.21	2302	63.1	711
JF10-11-3-L1	11	3	Yes	1	3.0	16678	2.21	2302	63.1	711
JF10-12-3-L0	12	3	No	0	3.0	14456	2.21	2302	68.2	768
JF10-12-3-L1	12	3	Yes	1	3.0	16678	2.21	2302	68.2	768
JF10-12-4-L0	12	4	No	0	3.0	14456	2.21	2302	70.7	796
JF10-12-4-L1	12	4	Yes	1	3.0	16678	2.21	2302	70.7	796
JF10-13-4-L0	13	4	No	0	3.0	14456	2.21	2302	75.7	853
JF10-13-4-L1	13	4	Yes	1	3.0	16678	2.21	2302	75.7	853
JF10-14-4-L0	14	4	No	0	3.0	14456	2.21	2302	78.9	910

JF10-14-4-L1	14	4	Yes	1	3.0	16678	2.21	2302	78.9	910
JF10-15-4-L0	15	4	No	0	3.0	14456	2.21	2302	78.9	967
JF10-15-4-L1	15	4	Yes	1	3.0	16678	2.21	2302	78.9	967
JF10-16-4-L0	16	4	No	0	3.0	14456	2.21	2302	78.9	1024
JF10-16-4-L1	16	4	Yes	1	3.0	16678	2.21	2302	78.9	1024
JF10-17-4-L0	17	4	No	0	3.0	14456	2.21	2302	78.9	1081
JF10-17-4-L1	17	4	Yes	1	3.0	16678	2.21	2302	78.9	1081
JF10-18-4-L0	18	4	No	0	3.0	14456	2.21	2302	78.9	1138
JF10-18-4-L1	18	4	Yes	1	3.0	16678	2.21	2302	78.9	1138
JF10-19-4-L0	19	4	No	0	3.0	14456	2.21	2302	78.9	1195
JF10-19-4-L1	19	4	Yes	1	3.0	16678	2.21	2302	78.9	1195
JF12-20-5-L0	20	5	No	0	3.6	20820	3.2	2771	113.6	1280
JF12-20-5-L1	20	5	Yes	1	3.6	24012	3.2	2771	113.6	1280
JF12-21-5-L0	21	5	No	0	3.6	20820	3.2	2771	113.7	1337
JF12-21-5-L1	21	5	Yes	1	3.6	24012	3.2	2771	113.7	1337
JF12-22-5-L0	22	5	No	0	3.6	20820	3.2	2771	113.7	1394
JF12-22-5-L1	22	5	Yes	1	3.6	24012	3.2	2771	113.7	1394
JF12-23-5-L0	23	5	No	0	3.6	20820	3.2	2771	113.7	1451
JF12-23-5-L1	23	5	Yes	1	3.6	24012	3.2	2771	113.7	1451
JF12-24-5-L0	24	5	No	0	3.6	20820	3.2	2771	113.7	1508
JF12-24-5-L1	24	5	Yes	1	3.6	24012	3.2	2771	113.7	1508
JF12-25-5-L0	25	5	No	0	3.6	20820	3.2	2771	113.7	1565
JF12-25-5-L1	25	5	Yes	1	3.6	24012	3.2	2771	113.7	1565
JF12-26-5-L0	26	5	No	0	3.6	20820	3.2	2771	113.7	1622
JF12-26-5-L1	26	5	Yes	1	3.6	24012	3.2	2771	113.7	1622
JF12-27-5-L0	27	5	No	0	3.6	20820	3.2	2771	113.7	1679
JF12-27-5-L1	27	5	Yes	1	3.6	24012	3.2	2771	113.7	1679

Jellyfish® Filter

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter. However, note that inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Online Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance. Alternative driving head values are only possible with off-line configuration.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation.
- The Jellyfish Filter can not accommodate multiple inlet pipes in an on-line configuration.
- Typical systems conform to the following pipe orientations:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Maximum Inlet Pipe Diameter (mm)
1.2	**62º**	**150**	**300**
1.8	59º	200	300
2.4	52º	250	375
3.0	48º	300	450
3.6	40º	300	450

- Outlet pipes to be one size larger than inlet pipes on new projects.
- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head caclulations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged condtions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish vault units can be designed.

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft^2 (0.142 lps/m^2).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow

event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Conseal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the

local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL
In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after is has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

5.2.1 The manufacturer shall provide an Owner's Manual upon request.

5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

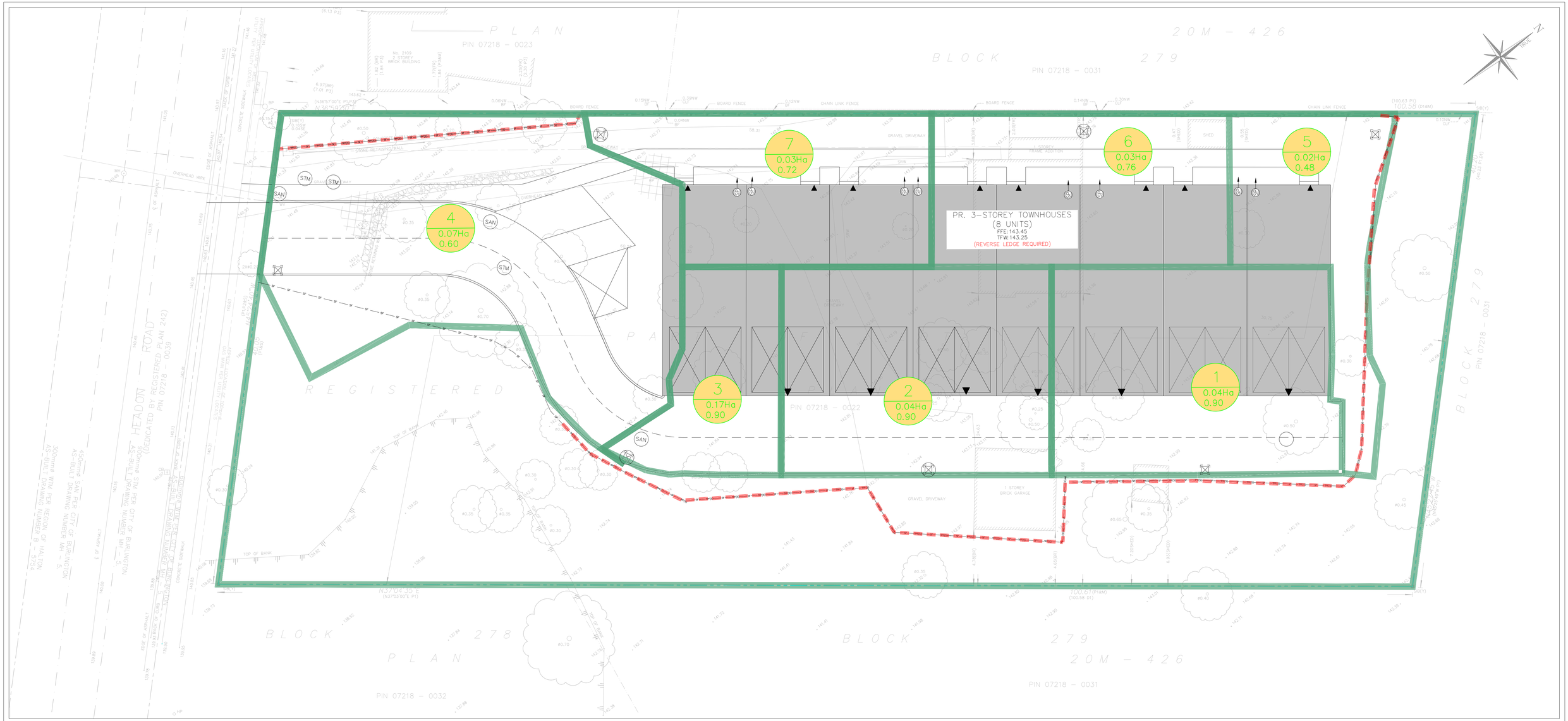
APPENDIX E – STM DESIGN SHEET

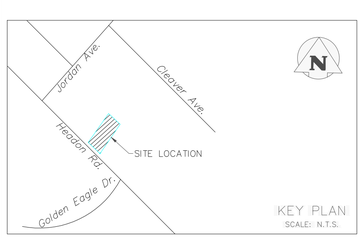
Computed by: GKK
Revised by:
Checked by: WST
Version: SWM Brief
Date: 2025.03

City of Burlington
2105 Headon Road
Project Number: 25-0105
Storm Sewer Design Calculations
5-Year Storm Event

Design Characteristics			
Storm Event	**5 year minor event**		
$i = A / (T + B)^C$	A	=	802.0
	B	=	5.000
	C	=	0.764
Velocities	V_{min} =	0.90	m/s
	V_{max} =	3.65	m/s
Flow	Q	=	αCiA
Manning's n Factor	n	=	0.013
Modification Factor	α	=	2.780
Maximum Capacity	85%		

From MH	To MH	Area Number	Incremental			Cumulative		Time of Concentration			Rainfall	Flow	Proposed Sewer Design						
			Area (ha)	Coefficient c	A*c	Area (ha)	A*c	T_{start} (min)	$T_{insection}$ (min)	T_{end} (min)	Intensity, i (mm/hr)	Q (L/s)	Length (m)	Diameter (mm)	Material	Grade [%]	Capacity (L/s)	Capacity %	Velocity (m/s)
CB1	CBMH2	1	0.042	0.90	0.04	0.042	0.038	10.00	0.22	10.22	101.31	10.6	22.4	250	PVC	2.00	84.1	12.6%	1.71
CBMH2	CBMH3	2	0.040	0.90	0.04	0.082	0.074	10.22	0.24	10.45	100.20	20.5	24.2	250	PVC	2.00	84.1	24.4%	1.71
CBMH3	MH4	3	0.017	0.90	0.02	0.099	0.089	10.45	0.18	10.63	99.03	24.5	18.0	250	PVC	2.00	84.1	29.2%	1.71
CB5	MH4	4	0.070	0.60	0.04	0.070	0.042	10.00	0.25	10.25	101.31	11.9	17.9	250	PVC	1.00	59.5	20.0%	1.21
MH4	STORAGE					0.170	0.131	10.00	0.04	10.04	101.31	37.0	4.5	250	PVC	2.00	84.1	44.0%	1.71
CB8	CBMH7	5	0.022	0.48	0.01	0.022	0.011	10.00	0.23	10.23	101.31	3.0	23.6	250	PVC	2.00	84.1	3.6%	1.71
CBMH7	CBMH6	6	0.032	0.76	0.02	0.054	0.035	10.23	0.38	10.61	100.14	9.8	39.4	250	PVC	2.00	84.1	11.6%	1.71
CBMH6	STORAGE	7	0.031	0.72	0.02	0.085	0.057	10.61	0.01	10.63	98.26	15.6	1.5	250	PVC	2.00	84.1	18.5%	1.71
STORAGE	ORIFICE TUBE											17.3	5.0	75	PVC	2.00	ORIFICE TUBE		
ORIFICE TUBE	OGS											17.3	3.0	250	PVC	1.50	72.8	23.8%	1.48
OGS	MH 10											17.3	1.2	300	PVC	1.50	118.4	14.6%	1.68
MH10	OUTLET											17.3	11.1	300	PVC	1.50	118.4	14.6%	1.68

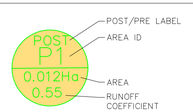




LEGEND

- PROPERTY LINE
- EX. SANITARY / STORM SEWER
- EX. WATERMAIN
- EXISTING FENCE
- EXISTING GRADE
- EXISTING HYDRO POLE
- PR. SANITARY SEWER
- PR. STORM SEWER
- PR. WATER SERVICE
- ROOF/ OVERHANG
- PR. CURB STOP
- PR. CATCH BASIN

- ×195.00 PROPOSED GRADE
- ×194.86BS PROPOSED BOTTOM OF SWALE GRADE
- ×194.92 GRADE TAG (TO MATCH EXISTING)
- 2.0% PROPOSED MINOR FLOW DIRECTION
- 2.0% PROPOSED GRASSED SWALE
- 1.5% PROPOSED GRASSED SWALE C/W UNDERDRAIN
- PROPOSED 3:1 SLOPE AND BOTTOM OF SLOPE
- OVERLAND FLOW PATHWAY
- PROPOSED TREE PROTECTION FENCE
- SILT FENCE PER OPSD 219.130
- SILT FENCE PER MODIFIED SILT FENCE DETAIL ON C02
- 3.00m TREE PROTECTION FENCE DIMENSION



SURVEY NOTES:

SURVEY COMPLETED BY: MMP LAND SURVEYORS & MAPPERS.
DATED: SEPTEMBER 22, 2020.

GEODETIC BENCHMARK:

CITY OF BURLINGTON BNCH MARK NO.410
ELEVATION = 140.082 METRES CGVD 1928:1978 ADJUSTMENT
BRASS PLAQUE LOCATED ON SOUTHEAST END OF CONCRETE CULVERT ON HEADON ROAD OPPOSITE GOLDEN EAGLE DRIVE AND 300 METRES ABOVE CONCRETE SIDEWALK AND 0.20 METRES NORTH OF SOUTHERLY END OF CONCRETE ABUTMENT.

DRAWING NOTES:

THIS DRAWING IS THE EXCLUSIVE PROPERTY OF EVERWEST ENGINEERING AND THE REPRODUCTION OF ANY PART OF IT WITHOUT PRIOR WRITTEN CONSENT OF THIS OFFICE IS STRICTLY PROHIBITED.

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, LEVELS, AND DATUMS ON SITE AND REPORT ANY DISCREPANCIES OR OMISSIONS TO THIS OFFICE PRIOR TO CONSTRUCTION.

THIS DRAWING IS TO BE READ AND UNDERSTOOD IN CONJUNCTION WITH ALL OTHER PLANS AND DOCUMENTS APPLICABLE TO THIS PROJECT. DO NOT SCALE THIS DRAWING.

ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

METRIC:

DISTANCES SHOWN ON THIS PLAN ARE IN METERS AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

DRAFT ISSUE

NOT FOR CONSTRUCTION

No.	ISSUE / REVISION	YYYY/MMM/DD
3A	ISSUED FOR COORDINATION	2026/MAY/06
3	ISSUED FOR SUBMISSION	2026/MAR/26
2A	ISSUED FOR SUBMISSION	2025/JUN/13
1	ISSUED FOR SUBMISSION	2025/MAY/26
0	ISSUED FOR SUBMISSION	2025/MAR/24

Project
2105 HEADON ROAD,
BURLINGTON, ON

Drawing
STM SEWER DESIGN SHEET

EVERWEST ENGINEERING
e. info@everwesteng.com
t. 905 526-3404
w. www.everwesteng.com

Drawn	JLT	Design	GKK	Project No.	25-0105
Check	WST	Scale	1:150	Sheet	2 of 3
				Dwg.	C02