Functional Servicing and Stormwater Management Report

Chedoke Browlands Development
Hamilton, Ontario
Project TPB188083; Client name: Valery Homes

Prepared for:

Valery Homes
2140 King Street East, Hamilton, ON L8K 1W6

9/18/2020

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1.0 Introduction

Valery Homes (Valery) is proposing a residential development at the Chedoke Browlands site located at 801, 820, 828, 855, 865 and 870 Scenic Drive in the City of Hamilton. The site is bounded to the south by Scenic Drive, and to the north by the Niagara Escarpment. The site is also generally bisected into two (2) areas (west and east) by the existing creek (Chedoke Creek tributary) which drains an existing stormwater management (SWM) facility to the south.

The subject property is the site of the former Hamilton Sanitorium, which was historically used as a treatment facility for tuberculosis patients. The site was previously owned by the Chedoke Health Corporation, and was ultimately sold, and acquired by Deanlee Management Inc (Deanlee) some time prior to 2007. Deanlee proposed to develop a portion of the site into a residential development, including medium and high-density usages. A number of technical studies were completed in support of this development plan at that time (which are discussed as relevant in subsequent sections of this report). It is understood that the previously proposed development was ultimately resolved through a settlement at the Ontario Municipal Board (OMB).

Valery has since acquired the site, and is currently proposing a modified development for the site. As per the OMB Minutes of Settlement, Valery is required to make a revised submission to the City of Hamilton (City) to reflect the currently proposed change in usage from the previous submission. Urban Solutions, acting as representatives for Valery, undertook a formal consultation meeting with City staff on February 14, 2018 (as documented in City File FC-18-004, dated March 1, 2018). It should also be noted that the existing creek which bisect the site is a regulated feature (and also incorporates an existing online stormwater management quantity control/flood control facility) and as such will require permitting approval through the Hamilton Conservation Authority (HCA).

The currently proposed concept by Valery, as reflected in the attached supporting material, is generally divided based on the two halves of the site. For the west half of the site, a total of 260 residential units are proposed. Two connected 8 storey buildings are proposed (204 units in total) along with 56 separate townhouse units, accessible via a proposed private roadway which travels around the perimeter of the west site. For the east half of the site, a total of 370 residential units are proposed, through a series of multi-storey buildings. Two connected 8-storey buildings will have a total of 216 units, adjacent to the creek block. A single 4-storey building (38 units) and a connected 5-storey building (116 units) are proposed along Scenic Drive. The existing heritage building (Long & Bisby) is to be preserved in the current concept.

Wood Environment & Infrastructure Solutions (Wood) has been retained by Valery to support the planned revised submission and associated Official Plan Amendment and Zoning By-Law Amendment applications for the subject property, through the completion of supporting engineering and servicing analyses related to water, sanitary, stormwater management, and related considerations. The purpose of this report is to outline a functional servicing strategy for the site with respect to these considerations, and provide the associated supporting technical analyses.

It is recommended that the current FSR and supporting materials be reviewed in conjunction with the architectural and mechanical engineering drawings (by others) to ensure a complete understanding of the proposed servicing connectivity.

2.0 Existing and Proposed Servicing

2.1 Sanitary Services

2.1.1 Existing Sanitary Services

An existing 375 mm diameter sanitary sewer (approximately 0.5% slope) is currently located along Scenic Drive beginning just east of San Pedro Drive and draining westerly across the entire west site frontage then continuing west (ref Fig 1 and record drawings in Appendix A). As per available record drawings, the most upstream portion of the sanitary sewer (between Sanatorium Road and San Pedro Drive) is understood to connect to a private sanitary sewer line further upstream, which services the upstream private development site (20.9 ha as per the Sanitary Drainage Area Plan from the adjacent Tiffany Development; a copy of the drawing has been included in Appendix A). Other external contributing sanitary drainage areas are noted in Figure 2-1 . The extents of the proposed Chedoke Browlands development are indicated in blue on the Figure.

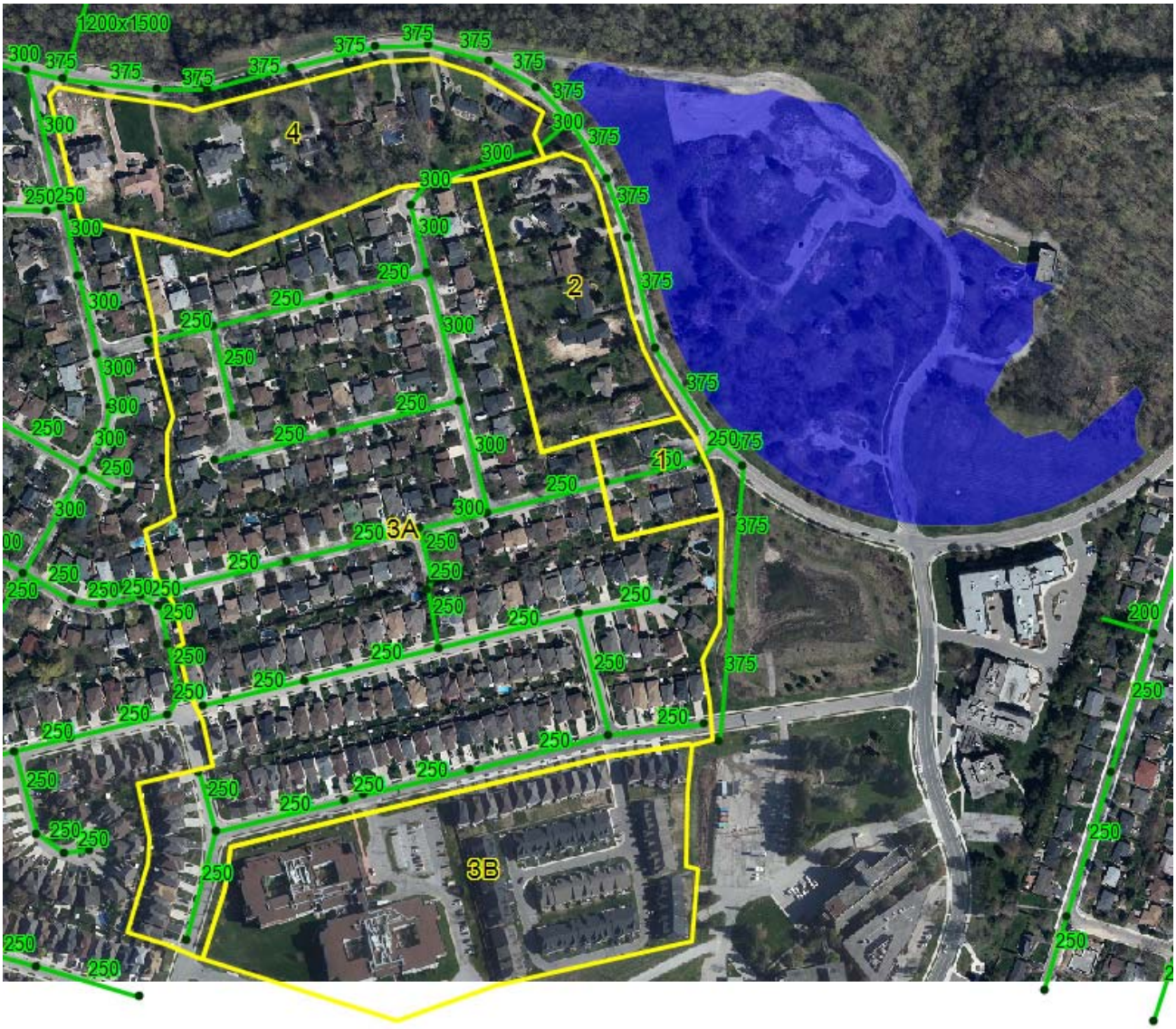


Figure 2-1: External Sanitary Drainage Areas

Additional external contributing areas include:

- Area 1: 0.5 ha (7 properties) – San Pedro Drive
- Area 2: 1.7 ha (7 properties) – Scenic Drive between San Pedro Drive and Trail from San Paulo Dr
- Area 3A: 13.6 ha (190 properties) – area contributes to 300 mm sewer on trail from San Paulo Dr
- Area 3B: 4.5 ha – area contributes to 300 mm sewer on trail from San Paulo Dr
- Area 4: 3.0 ha (10 properties) – Scenic Drive between Trail and Trunk Sewer over escarpment

There is currently no sanitary sewer along Scenic Drive across the frontage of the east site (refer to Figures 1 & 2, attached).

Of note for the east site is an existing sanitary sewer which currently services the existing Heritage Building (refer to Figure 2, attached) and connects to an existing 375 mm diameter sanitary sewer system which runs down the escarpment and through the Chedoke Golf Course lands. It should be noted however, that the records of the existing sanitary service within the Browlands site, tributary to the 375 mm sanitary system, indicate the local system consists of 150 mm diameter pipe built in 1917, with no information with respect to the pipe slope. At a minimum, this 150 mm sanitary service would have to be replaced with 200 mm pipe at 0.5% slope (as per current City of Hamilton standards) provided that it would be acceptable to continue discharging to the existing 375 mm diameter sanitary system, which also dates back to the early 1900's. Given the age of this system, it is considered unlikely that the City of Hamilton would agree to continue to use this service, and more likely that a connection to Scenic Drive would be preferable.

2.1.2 Proposed Sanitary Services

2.1.2.1 West Site

The west site sanitary flows are proposed to be conveyed to the existing 375 mm diameter sanitary sewer along Scenic Drive with a portion of the flows being connected at the west end of the west site and the remaining portion being directed to the east end of the site (ref. Figure 5).

Internal to the west site, the sanitary sewers are proposed to be located within the road areas for the west and south roads however, due to the extents of the proposed underground parking structure within the east road, the sanitary sewer in that location will need to be located in the rear yard areas adjacent to the creek block. Basement floor elevations for all townhome units have been set high enough to permit gravity flow sanitary laterals to the proposed sanitary mains.

The sanitary flows for the two apartment buildings are proposed to be internally conveyed by piping within the upper areas of the underground parking structure; this piping has been designed by the mechanical engineer (refer to separate materials by KNYMH and others). The apartment building sanitary piping will flow by gravity and be connected to the proposed 200 mm diameter sanitary main at two (2) points along the west road. Sanitary flows however for the underground parking level floor drains, will have to be pumped to the sanitary system (refer to Figure 5).

The total estimated average sanitary flow for the two (2) apartment buildings and 56 townhouse units for the West Site have been calculated to be 2.98 litres/sec based on the Ontario Building Code – Part 8 (refer to Appendix D). Sanitary pipe sizing calculations have been included in Appendix D, consistent with City of Hamilton Standards and based on Equivalent Persons Population of 360 litres/person/day.

2.1.2.2 East Site

Given the concerns with the existing 375 mm diameter sanitary system through the Chedoke Golf Course lands discussed in Section 2.1.1, it is recommended that the sanitary flows for the east site ultimately be

conveyed to the existing 375 mm diameter sanitary sewer within Scenic Drive. However, as this system is not currently extended across the frontage of the east site, it will be necessary to extend this system easterly with 250 mm sanitary piping along Scenic Drive for approximately 205 m in order to connect to the site sanitary systems.

The construction of the new 250 mm diameter sanitary sewer along Scenic Drive will necessitate non-standard construction configuration due to the elevation of the existing 750 mm diameter storm pipe which crosses Scenic Drive approximately 12 m east of the current most easterly sanitary manhole. From the latest as-built information (ref. Appendix A), it appears that it will be necessary to install the new 250 mm diameter sanitary sewer on the east side of the existing manhole (invert to invert), with the existing 375 mm diameter pipe on the west side. Typically, City standards require the pipes to align obvert to obvert. In using this non-standard approach, it appears that the clearance of the proposed 250 mm diameter sanitary pipe below the existing 750 mm diameter storm sewer will be approximately 100 mm which is below the typical minimum threshold of 150 mm (refer to detail on Figure 6).

It is recommended that the existing 750 mm diameter storm pipe be exposed (daylighted) at the point of crossing with the proposed 250 mm diameter sanitary sewer extension such that a more accurate elevation of this pipe can be determined and assurances can be made that the proposed 250 mm diameter sanitary pipe will fit beneath it. Should it be determined that the 250 mm diameter sanitary sewer will not fit below with adequate clearance, then other alternatives may have to be considered such as:

- Using stronger (higher pipe class) 250 mm diameter watermain pipe for the first section from the existing manhole to the first new manhole to the east and using concrete bedding around this pipe at the point of crossing with the existing 750 mm diameter storm pipe
- Substituting the proposed 250 mm diameter pipe with twin 200 mm diameter pipes for the first section from the existing manhole to the first new manhole to the east to create an additional 50 mm of clearance
- Consider the possibility of removing and reinstalling the 750 mm storm at a higher elevation, or replacement with twin pipes of a smaller diameter

Should none of the above prove to be satisfactory, it may be feasible to have the 250 mm diameter sanitary sewer pass above the existing 750 mm diameter storm, though this would result in the road cover over the 250 mm diameter pipe being approximately 1.4 m which is also not desirable and may not provide adequate depth to service the east site. As such, an approach which involves the sanitary sewer extension passing below the 750 mm diameter storm sewer is considered preferable.

Internal to this east site, the sanitary flows for the apartment buildings will be conveyed by piping within the upper areas of the underground parking structure. This piping has been designed by the mechanical engineer (refer to separate materials by KNYMH and others). The apartment building sanitary piping is proposed to flow by gravity then be connected to the proposed 200 mm diameter sanitary laterals near the south limits at two (2) points. Sanitary flows however for the underground parking level floor drains will have to be pumped to the sanitary system.

The existing Heritage Building is proposed to be serviced for sanitary flows by disconnecting from the existing 150 mm diameter lateral (which in turn drains to the 375 mm diameter sanitary system through the Chedoke Golf Course lands) and re-directing flows by a lift pump and forcemain to the new sanitary system being installed within the underground parking structure of the apartment buildings (refer to Figure 6).

The total estimated average sanitary flow for the five (5) apartment buildings and existing Heritage Building has been calculated to be 3.74 litres/sec based on the Ontario Building Code – Part 8 (refer to Appendix D). Sanitary pipe sizing calculations have also been included in Appendix D, consistent with City of Hamilton Standards and based on Equivalent Persons Population of 360 litres/person/day.

2.1.2.3 Impact to Downstream System

An analysis of the receiving sanitary sewer system has been undertaken, to confirm whether the additional flows generated by the proposed development would be expected to result in any negative impacts off-site. The lumped external sanitary drainage areas presented in Figure 2-1, have been applied, in conjunction with the external sanitary sewershed mapping completed for the previous Tiffany Development (a copy of that plan is included in Appendix A). There is some contradicting information with respect to the sanitary drainage areas and population counts for the area south of Redfern and east of Chedmac; the most conservative (larger) population and densities have therefore been applied for this area accordingly. For all other detached residential areas, the measured sanitary drainage area has been applied in conjunction with the City of Hamilton's recommended population density for single detached properties of 60 ppha (as per Section E.1.4 of the City's Comprehensive Development Guidelines and Financial Policies Manual). A sanitary sewer design sheet (including flows from the proposed development) are included in Appendix D.

The simulated results indicate that the majority of the existing sanitary sewer along Scenic Drive would be unaffected by the proposed development flows, with the section of 375 mm sanitary sewer (0.5% slope) up to the trail from San Paulo Drive indicating capacities of between 58 and 65% of full flow. The section of sewer system beyond this point however (where the 300 mm diameter sanitary sewer from San Paulo Drive connects) receives additional flows and also flattens to an average slope of approximately 0.2%. This slope continues from this point along Scenic Drive to the outlet over the escarpment at Goulding Avenue (approximately 380 m west of the trail connection point). Based on the simulated results, the two sections of sanitary sewer immediately downstream would be at 165% and 151% of the theoretical full flow capacity respectively. By comparison, if all proposed site flows were removed, the two sections of sanitary sewer in question would remain surcharged, at 130% and 120% of the theoretical full flow capacity respectively, suggesting this to be an existing potential capacity constraint. The surcharge in this area would directly affect the ten (10) downstream properties fronting on Scenic Drive, however tailwater conditions from surcharge could impact upstream areas as well. Based on the record drawings, at the roadway sag point near Sanatorium Road, there would be approximately 3.0 m of cover from the roadway to the top of the sanitary sewer. Further downstream (westerly), the cover continues to increase and thus the impacts of sanitary system surcharge would be reduced.

Notwithstanding, Wood has requested a copy of the currently approved sanitary sewer hydraulic model (MIKE Urban) from the City of Hamilton to further verify the simulated capacity in the existing sanitary sewers to be connected to, and to further assess the potential impact of the additional flows from the proposed development. The modelling files have as of the time of writing not yet been received. Depending on the outcomes of this supplemental analysis, there may also be value in completing a sanitary sewer flow monitoring program, to validate/calibrate the modelling and better assess potential impacts. This would confirm whether or not the theoretical flows are in fact reasonable, and how actual measured flows compare to theory.

In the event that off-site sanitary sewer upgrades (along Scenic Drive) are determined to be required, a cost sharing approach with the City would need to be discussed, particularly given that the downstream sanitary sewers are currently indicated as over capacity prior to the addition of any flows from the proposed development.

2.2 Water Services

2.2.1 Existing Water Services

Currently along the south side of Scenic Drive, opposite the west site, is an existing 200 mm diameter watermain, extending from San Pedro Drive, west to the western limits of the site (ref. Figures 1 and 2 and record drawings in Appendix A). Between Sanatorium Road and San Pedro Drive, there is an existing 300 mm diameter watermain along the north side. East of Sanatorium Road for 90 m to the east, is an existing 300mm watermain along the south side of Scenic Drive

The existing Heritage Building according to City records is currently serviced by a 76 mm diameter water service which is connected to a 150 mm diameter watermain system. It is not clear at this time if this existing system is currently operating, however it is expected that this system would be abandoned as part of the west site development (refer to Figures 1 & 2) and thus would not be available for servicing.

Watermain hydraulic analyses are located in Appendix G of this report.

2.2.2 Proposed Water Services

2.2.2.1 West Site

Internal to the west site the watermains are for the most part proposed to be installed behind the curb in keeping with standard practice for townhome condominium sites. The bulk of the west site is proposed to be serviced with 250 mm diameter watermain with a small portion of the site being serviced by 300 mm diameter watermain on the east site. Five (5) fire hydrants are proposed at spacings of approximately 110 m in keeping with City of Hamilton standards for medium density residential development.

The two (2) apartment buildings are proposed to be serviced with a 300 mm diameter water service to enter a utility room at the south west corner of the underground parking structure (refer to Figure 5). The final sizing will be determined by the Mechanical Engineer in conjunction with the water and fire protection needs of the final building design and pressure and flow testing of the existing and upgraded watermain system (to be completed at the detailed design stage).

In order to meet fire flow demand requirements for both the west and east sites, it is necessary to twin the existing 300 mm diameter watermain along the north side of Scenic Drive from Sanatorium Drive to San Pedro Drive (refer to Figures 5 & 6 and Appendix G for supporting hydraulic analyses). Cost sharing for this upgrade should be discussed with the City of Hamilton, as this was not originally anticipated to be required to service the proposed development.

The estimated water usage for the west site has been calculated to be 2.98 litres/sec based on the Ontario Building Code – Part 8 (refer to Appendix D). These calculations will need to be further refined by the Mechanical Engineer at the detailed design stage when fixture counts are confirmed.

2.2.2.2 East Site

Internal to the east site all of the water servicing, with the exception of the existing Heritage Building, is internal to the underground parking structure, and will be designed by the Mechanical Engineer at the detailed design stage. The existing Heritage Building is proposed to be serviced for water by the abandonment of the existing 76 mm diameter water service, and provision of a new water service proposed to be connected to the new watermain system to be installed within the underground parking structure of the apartment building (refer to Figure 6). The final sizing for the Heritage Building water service will be determined by the Mechanical Engineer in conjunction with the water and fire protection

needs of the final building design and pressure and flow testing of the existing and upgraded watermain system (to be completed at the of detailed design stage).

The five (5) apartment buildings are proposed to be serviced with a 300 mm diameter water service to enter a utility room at the south mid-point of the underground parking structure. The final sizing will be determined by a mechanical engineer in conjunction with water and fire protection needs of the final building design and pressure and flow testing of the existing and upgraded watermain system (to be completed at the detailed design stage). Two (2) fire hydrants are proposed, as located by the architect (KNYMH and others) in keeping with OBC requirements.

In order to meet fire flow demand requirements for both the west and east sites, it is necessary to twin the existing 300 mm diameter watermain along the north side of Scenic Drive from Sanatorium Drive to San Pedro Drive (refer to Figures 5 & 6 and Appendix G for supporting hydraulic analyses). Cost sharing for this upgrade should be discussed with the City of Hamilton, as this was not originally anticipated to be required to service the proposed development.

The estimated water usage for the west site has been calculated to be 3.74 litres/sec based on the Ontario Building Code – Part 8 (refer to Appendix D). These calculations will need to be further refined by the Mechanical Engineer at the detailed design stage when fixture counts are confirmed.

2.3 Storm Services

2.3.1 Existing Storm Services

An existing storm sewer system (ref. Figures 1 and 2, attached) is currently located along Scenic Drive beginning at San Pedro Drive and draining westerly across the entire west site frontage then continuing west of the site limits. The size of the storm pipes range from 375mm (at San Pedro Drive) to 1350 mm in diameter (at the western site limits).

There is currently no storm sewer system along the frontage of the east site within Scenic Drive east of San Pedro Drive. There is a box culvert and 750 mm diameter storm pipe (outlet for the upstream SWM Facility) which cross Scenic Drive and outlet to the creek block, Separately, there are two (2) catch basins with leads that cross Scenic Drive and outlet into the east site.

2.3.2 Propsed Storm Services

2.3.2.1 West Site

Storm water runoff for the two (2) apartment buildings and associated area above the underground parking structure is proposed to be collected and conveyed with an internal piping system within the upper areas of the underground parking structure. This internal storm system will be designed by the Mechanical Engineer (KNYMH and others) and will outlet from the underground parking structure at three (3) points due to length and grade restrictions. Two (2) of these points will discharge to SWM Facility #1 (wet pond) in the creek bock, and the other point which outlets to the proposed storm system located along the rear yards of the townhome units along the west road. This storm system then discharges to SWM Detention Unit #2 (underground storage tank - refer to Figure 5).

All storm piping external to the underground parking structure has been conservatively sized for the 100-year storm event to be consistent with the internal piping sizing (by KNYMH and others; as per OBC) and to eliminate any concerns with respect to overland flow route. Sizing has been completed using the Rational Method (refer to Appendix E).

Stormwater quality and quantity control is discussed further in Section 4.0 of this report.

2.3.2.2 East Site

Storm water runoff for the five (5) apartment buildings and associated area above the underground parking structure is proposed to be collected and conveyed with an internal piping system within the upper areas of the underground parking structure. This internal storm system will be designed by the Mechanical Engineer and will outlet from the underground parking structure at eight (8) points given constraints regarding internal servicing (maintaining headroom clearance) and the need to direct flows to specific locations. Three (3) of these outlets are to discharge to a storm piping system to be located external to the underground parking structure along the south limits of the site. This storm system will then convey the flows to SWM Detention Unit #3 (underground storage tank) proposed to be located in the south west corner of the site. In addition to these three (3) outlets, proposed SWM Detention Unit #3 also receives flows from two (2) other outlets discharging from the south west portion of the building.

Flows from the north most corner of underground parking structure are proposed to discharge to SWM detention Unit #4 (underground storage tank) and the remaining two (2) outlets are proposed to discharge to the east woodlot area, via a proposed bioswale (refer to Figure 6), in order to maintain water to this area and the associated sensitive/significant plant species (refer to ecological submissions for further details).

All storm piping external to the underground parking structure has been conservatively sized for the 100-year storm event to be consistent with the internal piping sizing (by KNYMH and others; as per OBC) and to eliminate any concerns with respect to overland flow routes. Sizing has been completed using the Rational Method (refer to Appendix E).

Storm water runoff from the existing Heritage Building will be discharged overland west and east, as it does currently.

As discussed in Section 2.3.1, there is no existing storm piping system within Scenic Drive along the frontage of the east site, but there are two (2) existing catch basins with leads that cross Scenic Drive and currently outlet into the east site. With the development of the east site, it will be necessary to collect these flows with a new municipal storm sewer system along Scenic Drive, and convey them westerly to a suitable outlet. This will require the installation of approximately 195 m of new storm sewer piping with a suggested connection to the existing 750 mm diameter storm sewer, which then discharges to the creek block. The specific requirements of this municipal storm sewer are expected to be completed at the detailed design stage. Given that this additional storm sewer system would service solely municipal drainage, it is expected that the costs of this service would be incurred by the City of Hamilton, however cost sharing as part of the overall reconstruction of Scenic Drive would need to be discussed.

Stormwater quality and quantity control is discussed further in Section 4.0 of this report.

3.0 Existing and Proposed Grading

3.1 Existing Grading

The topographic survey completed by A.J. Clarke and Associated Limited (dated February 16, 2016) has been applied for the assessment of existing grading and the development of proposed grading for the site. This information has been supplemented by additional topographic survey data for the east woodlot, and additional/updated legal boundary survey data, as completed by A.T. McLaren Limited (2019/2020).

Much of the existing surface runoff for the west site currently drains west and north toward the escarpment, while a portion of the site drains east into the existing creek (refer to Figure 1).

For the east site, lands west of the closed Sanatorium Road ROW drain west into the existing creek. Lands east of Sanatorium Road generally drain east and north into the woodlot, with the remaining north-east portion draining north and west toward the escarpment (refer to Figure 2).

Pre-development drainage boundaries are discussed in further detail as part of the stormwater management analyses, as per Section 4 of the current report.

3.2 Proposed Grading

3.2.1 West Site

Generally, the proposed grading for the west site was guided by the existing geotechnical information. Specifically, the underside of footings for the underground parking structure should be above the rock elevations identified in the Geotechnical Report which show rock elevations ranging from 189.5 to 190.1 m (refer to Appendix B).

For hard surface areas, proposed grades are generally 1.0% or greater, and 2.0% or greater for grassed areas in keeping with City of Hamilton standards. Rear yard grading for the west road townhomes is designed to meet the existing ground with 4H:1V slopes within the area to be dedicated as a park block to the City. Rear yards along the east road townhomes are designed to surface discharge towards the proposed storm water management facility (refer to Figure 3).

3.2.2 East Site

Similar to the west site, the east site proposed grading was also guided by the objective that the underside of footings for the underground parking structure be above the rock elevations identified in the Geotechnical Report, which show rock elevations ranging from 189.4 to 191.8 m (refer to Appendix B).

For hard surface areas, proposed grades are generally 1.0% or greater and 2.0% or greater for grassed areas in keeping with City of Hamilton standards (refer to Figure 4).

3.2.3 Creek Block

Based on the constraints associated with the existing creek (extents and proximity to proposed development and SWM measures, erosion risk, etcetera), it has been proposed to re-grade and re-generate the creek form using contemporary natural channel principles, while also maintaining the existing flood control/quantity control function of the existing online SWM facility.

Based on a preliminary review of constraints, it was confirmed that the erosion hazard limit (meander belt) would be the governing requirement for establishing the geometry of the creek block design, as flood storage and flood hazard limits could more easily be refined and accommodated. A meander belt assessment and natural channel design have been undertaken separately by GEO Morphix Ltd; a copy of

the reporting is included in Appendix C. Overall, it was confirmed by GEO Morphix that the meander belt limit would be expected to be 30 m; this width has been applied accordingly to determine the grading of the channel block base. Overbank side slopes of between 2 and 10% have been proposed. The channel block side slopes have been graded at stable slopes of approximately 3H:1V to contain flood flows. The proposed reconstructed channel will have an overall grade of approximately 0.9%, which ensures that both upstream and downstream culvert inverts are matched. Detailed hydraulic modelling and flood levels are discussed further in Section 4.

Stormwater management facilities have been proposed on either side of the creek block; a wet pond SWM facility on the west side, and an underground storage tank on the east side. Overbank and grading in these areas has been adjusted, as required, to accommodate these proposed features, including stable slopes in the case of the SWM facility, and sufficient ground cover in the case of the underground storage tank, while matching into the proposed grades of the proposed adjacent development units. SWM measures are discussed in further detail in Section 4.

4.0 Stormwater Management

4.1 Background Information

A number of previous hydrologic and hydraulic studies have been completed which are considered relevant to the currently proposed site development; this includes works both for the development site itself, as well as larger-scale studies for adjacent areas or roadways. A brief summary of relevant reports is presented chronologically as part of this section.

- *Preliminary Engineering Reports – Chedoke Browlands – Deanlee Management Inc. (A.J. Clarke and Associates Ltd., July 2007)*

  This report was prepared to support the proposed re-development of the property to a residential development. The 2007 report reviewed existing municipal services (water, sanitary, and storm) and outlined the proposed high-level approach for connection. With respect to stormwater management, a MIDUSS hydrology model was developed to assess pre-development and post-development peak flows from the site and determine approximate quantity control requirements. Two (2) conceptual SWM facilities (presumably dry ponds) were proposed as part of the reporting, with simulated 100-year peak storage volumes of 1,036 and 1,266 m^3 for the west and east areas respectively (2,302 m^3 total). Stormwater quality control was proposed to be provided by oil/grit separator units, given the small contributing drainage areas. It is understood that the SWM strategy proposed as part of this reporting was ultimately incorporated and verified by the subsequent analyses completed by Dillon Consulting Limited as part of its subsequent "Sanatorium Road Realignment Flood and Erosion Impact Assessment", and were also part of the development submission ultimately resolved through a hearing at the Ontario Municipal Board (OMB).

- *Mountview Neighbourhood Storm Drainage Study and Stormwater Management Facility Review – City of Hamilton (McCormick Rankin Corporation, May 2008)*

  The study was undertaken to review flooding complaints in the Mountview Neighbourhood, approximately bounded by Stone Church Road to the south, the escarpment to the north, Scenic Drive to the West, and Upper Paradise Road to the east. The capacity of the storm drainage system in this area was investigated, along with potential new stormwater management facilities. An XP-STORM model was noted to have been developed to assess the storm sewer and combined sewer systems. Part B of the reporting mentions that a SWMHYMO model was also used to assess "the impacts of some of the detention storage alternatives", although details are not provided. The study suggested expansion of the existing online SWM facility on the Sanatorium Lands (referred to as Pond 2 – Outlet 3), however this recommendation was revised as part of the subsequent Sanatorium Road Study.

- *Meander Belt Width Assessment – Chedoke Creek (Scenic & Sanatorium) – Draft Report (Parish Geomorphic, September 2009)*

  This report was completed for A.J. Clarke and Associates in support of the overall development submission for Deanlee Management, in order to determine the meander belt limit for the unnamed tributary of Chedoke Creek which drains through the subject site. Based on its review, a preliminary meander belt width of 41.0 m, plus an additional erosion setback of 8.2 m was identified (total meander belt width of 49.2 m). Notably, this initial meander belt width has been reviewed and a revised meander belt width of 30.0 m determined as part of the current submission (GEO Morphix Ltd – refer to Appendix C).

- *City of Hamilton Sanatorium Road Realignment Flood and Erosion Impact Assessment (Dillon Consulting Limited, June 2010)*

  Dillon Consulting Limited completed the Sanatorium Road Reconstruction and Stormwater Improvements Class EA ESR, subsequent to the completion of the Mountview Neighbourhood Study, with the intent to incorporate the recommendations of that study into its work. A copy of this original report was not available (report was completed in approximately late 2007), however a presentation from PIC #1 (available on the City's website) indicates that the online SWM facility within the subject lands was to remain unchanged. The preferred road layout also indicates a round-about at the intersection of Scenic Drive and Sanatorium Road, at the limits of the currently proposed development.

  A Part II Order ("bump-up") was submitted by Deanlee Management to the Ministry of the Environment in March 2008 to express concerns regarding issues related to the development site that were not addressed through the 2007 study (floodlines, flood/erosion control, etcetera). As such, this additional (2010) study was undertaken to address the requirements of the Part II order, which included assessment of the proposed SWM strategy for the site (as per the 2007 A.J. Clarke report – two dry SWM ponds).

  Through this study, a continuous simulation hydrologic model (HEC-HMS – built upon the subcatchment data and information from the Mountview Study XP-STORM model, as well as the Visual OTTHYMO modelling completed by Dillon as part of the initial EA ESR), was developed to assess erosion risk. It was concluded that the previously proposed SWM strategy for the development (as per A.J. Clarke and Associates Ltd, July 2007) was sufficient to address erosion risk. In addition to the preceding, the Visual OTTHYMO model developed for the original EA was used to assess the expected changes in flood flows and flood levels for the Regional Storm Event. Simulated Regional Storm flows of 4.8 m^3/s were indicated for existing conditions, with a peak level of 190.6 m within the online flood control facility. For the Proposed 1 scenario (preferred SWM solution from original EA), the simulated peak flow at outlet 3 (i.e. the creek at the escarpment) would increase slightly to 5.0 m^3/s, with a corresponding water level of 190.75 m in the online flood control facility. For the Proposed 2 scenario (preferred SWM solution from the original EA and proposed site SWM strategy as per the 2007 A.J. Clarke assessment), the peak flow would increase to 5.2 m^3/s (maximum water level of 191.0 m). The conclusions do not suggest any further quantity control measures. It is further noted that the simulated increase in floodlines could be mitigated through a number of potential alternatives, including the use of online storage for the development itself, as noted below:

  "*Flexibility in the choice of SWM strategies for the Chedoke Browlands that mitigate the increase in floodlines on the Chedoke Browlands including a submission by Deanlee Development to eliminate the new SWM facilities that form part of the Deanlee Development SWM strategy for the proposed 2 conditions and the use of the existing SWM pond (Scenic Drive pond) for quantity control may be considered by the City. Detailed information and supporting documentation satisfactory to the City and review agencies during the final design stage would need to be provided by Deanlee Development for these alternative SWM strategies for the Chedoke Browlands*". (Dillon, 2010)

  The City of Hamilton provided copies of the HEC-HMS and Visual OTTHYMO models from this study to Wood for the current study. However, the HEC-HMS modelling could not be opened and are assumed to be corrupted. Numerous attempts were made to contact Dillon Consulting to obtain a revised copy of the modelling, however Dillon ultimately advised that the modelling files could not be located. The Visual OTTHYMO modelling has been successfully used for the current study, however it should be noted that the model represents the Regional Storm only, and is also truncated for all drainage areas upstream of the site (i.e. a direct hydrograph is inputted to the modelling, rather than

actual subcatchment and routing elements). As such, the modelling could not be applied to simulate other storm events.

- *Sanatorium Road EA SWM Review Summary Memo (Dillon Consulting Limited, December 1, 2015)*

  This memorandum was prepared by Dillon Consulting to summarize the work completed for the Sanatorium Road EA. It re-affirms that there are no expected negative impacts from the original preferred SWM strategy on the site, and that no concerns were received by Deanlee Management with respect to the revised EA analyses.

- *Browlands – Hydraulic Analysis Related to the Floodline Elevations of the Existing Watercourse (S. Llewellyn & Associates Limited, November 4, 2016)*

  This technical memorandum was prepared to review floodline constraints and opportunities with respect to the subject watercourse on the site, and its potential impacts to the proposed site development. S. Llewellyn & Associates (SLA) extracted topographic survey data (as per A.J. Clarke and Associates Ltd, February 16, 2016) as the basis for the model geometry, and also utilized peak flow data for the Regional Storm Event (Hurricane Hazel), as available in the 2010 Sanatorium Road Study (Dillon Consulting Ltd), namely 4.8 m^3/s for Existing Conditions and 5.2 m^3/s for "Proposed 2" conditions. A copy of the existing conditions cross-section location plan (non-georeferenced) and associated simulated floodplain extents, are included in Appendix A for reference. A copy of the HEC-RAS modelling has also been obtained, and used as part of subsequent assessments in the current study.

  For proposed conditions (i.e. with the anticipated development in place), some modifications to the channel geometry are indicated, as the revised floodlines are altered; however, a detailed comparison of grading changes was not provided. Overall, the floodlines are generally reduced at the upstream limits and increased at the downstream limits, including notes that the downstream area is to be re-graded to provide storage (unclear if this is flood control or riparian). An approximate development limit is indicated based on the revised floodplain mapping.

4.2 Site Reconnaissance

A site reconnaissance was completed by Wood staff on January 8, 2019. Selected photographs are included in Appendix F. The focus of the site reconnaissance was on the existing creek, and in particular the culvert connections at both the upstream and downstream ends.

The on-line flood control facility control structure was measured as a rectangular concrete opening with an arch/rounded top. The concrete opening was measured to be 1.15 m wide, and 1.05 m high. The arch/rounded top accounts for approximately 0.15 to 0.20 m of the total height. The CSP arch culvert behind the control structure is slightly less than this opening, 1.6 m wide, by 1.1 m high. An equivalent rectangular opening for the control structure would therefore be approximately be 1.15 m wide by 0.95 m high.

The downstream end of the CSP arch culvert is perched from the escarpment rock on a degraded concrete structure. The bottom of the downstream end of the CSP arch culvert is also heavily corroded, such that low flows drain through the bottom of the openings in the CSP culvert, and in turn through cracks in the concrete structure below, down to the base of the escarpment rock.

4.3 Existing Conditions

4.3.1 Watercourse

As noted in Section 4.1, a number of previous studies and hydrologic/hydraulic models have been prepared for the existing watercourse. The most recently completed and available HEC-RAS modelling has been utilized for the estimation of existing flood storage and water levels within the creek corridor, namely that developed by SLA (November 2016), based on the topographic survey of the site completed by A.J. Clarke (February 2016). Note that there is only one (1) hydraulic structure between Sanatorium Road and Scenic Drive, a minor pedestrian bridge in poor condition. This structure has been excluded from the modelling given its nominal function and for consistency with subsequent proposed conditions modelling and to ensure that volume calculations do not reflect any backwater conditions (i.e. riparian storage).

For reference purposes, the originally estimated Regional Storm Floodline for existing conditions, and the associated cross-section location plan (non-georeferenced) are included in Appendix A for reference purposes. Given the proposed reconstruction of the existing watercourse under proposed conditions, it is not considered necessary to re-generate or re-plot the existing conditions floodline. What is considered of particular concern is the existing available flood control/quantity control storage, and the associated peak discharge.

The SLA modelling utilized the Regional Storm flows available in the Dillon (2010) study, namely 4.8 m^3/s for existing conditions and 5.2 m^3/s for the "Proposed 2" conditions (i.e. with upstream EA recommended measures in place along with the previously proposed SWM strategy for the Browlands site). Of note, fixed downstream boundary conditions in the HEC-RAS model were determined using Culvert Master, based on the previous peak inflows and the 1630 x 1120 mm CSP arch culvert dimensions. This approach neglects the dimensions of the actual control structure (as discussed in Section 4.2; refer to photos in Appendix F), and also results in water levels that differ from those used in the hydrologic modelling rating curve used by Dillon (2010). For existing conditions, this results in a difference of approximately 0.08 m (190.60 m in Dillon, 190.68 m in SLA), and for "Proposed 2" conditions, this results in a difference of approximately 0.17 m (191.00 m in Dillon and 191.17 m in SLA).

As noted, given the particular focus on replication of available flood control/quantity control storage, and the need to ensure a consistent approach between existing and proposed conditions, both the hydrologic modelling (Visual OTTHYMO) and hydraulic modelling (HEC-RAS) for the Regional Storm have been revisited and updated. The rating curve for existing conditions has been reviewed using a two-fold approach:

- Development of a revised stage-discharge curve based on the actual existing concrete control structure at the downstream limits of the creek (i.e. at the escarpment/Sanatorium Road)
- Development of a stage-storage curve for the existing watercourse based on the HEC-RAS modelling (as per SLA, 2016)

As noted in Section 4.2, an equivalent rectangular opening for the control structure is approximately 1.15 m wide by 0.95 m high. Design Chart 5.39 (Inlet Control: Box Culvert) from the MTO's Drainage Management Manual has been used to develop the depth – discharge relationship, using Scale 2 (headwall with no bevels). An equivalent fit has been developed using Manning's Equation for low flow and orifice flow for high flow, with adjustments to the slope and Manning's Roughness and orifice coefficient values to ensure a good fit to the nomograph generated data. Details are provided in Appendix F.

In conjunction with the preceding, the previously developed HEC-RAS modelling by others (SLA, 2016) has been used to develop a stage-storage curve for the existing watercourse. Version 4.10 of the model has been used, consistent with the original modelling. Minor adjustments have been made to the received modelling, by adjusting the bank station and overbank roughness. A series of fixed boundary condition levels have been applied in conjunction with minor flows to ensure that calculated storages reflect a constant, fixed water level.

Results are provided in Appendix F. The rating curve has been developed to an elevation of 191.5 m, which is approximately the elevation at which spill over Sanatorium Road would be expected to occur. Overall, the lower portion of Wood's updated rating curve compares well with the previous (Dillon, 2010) curve. There is greater deviation at higher elevations, as Wood's updated calculations indicate lower storage volumes and lower discharges at comparable elevations.

As a verification, the input hydrographs from the Regional Storm Visual OTTHYMO modelling (Dillon, 2010) have been inputted into the revised rating curve (Wood, 2020) to confirm Regional Storm storage volumes, discharges, and flooding levels. A discrepancy between the values presented in Table 4.3 of the 2010 Sanatorium Road EA (2010 – refer to Appendix A) and the received modelling files has been noted however; refer to Table 4.3.1 for the comparison.

Table 4.3.1: Simulated Regional Storm Peak Flow (m^3/s) Comparison for Online Flood Control Facility

Modelling Scenario	Source	Inflow (m^3/s)	Outflow (m^3/s)
Existing	Dillon (Report)	5.8	4.8
	Dillon (Visual OTTHYMO)	5.46	4.65
	Difference	**-0.34**	**-0.15**
Proposed 1	Dillon (Report)	6.0	5.0
	Dillon (Visual OTTHYMO)	5.65	5.19
	Difference	**-0.35**	**+0.19**
Proposed 2	Dillon (Report)	6.3	5.2
	Dillon (Visual OTTHYMO)	5.66	4.83
	Difference	**-0.64**	**-0.37**

The reason for the discrepancy is not clear. Overall, the "Proposed 2" scenario (which should reflect the inclusion of the previous development site SWM strategy) generates the highest overall inflows, as would be expected. In order to be conservative and also provide a consistent comparison to the proposed modifications to the online SWM facility (as described in Section 4.4.1), this hydrograph has been applied. The updated rating curve developed by Wood has been used in place of the original (Dillon, 2010) rating curve. Results are presented in Table 4.3.2.

Table 4.3.2: Simulated Regional Storm Peak Flow (m^3/s) Comparison for Online Flood Control Facility for "Proposed 2" Inflow Scenario				
Source	**Inflow (m^3/s)**	**Peak Storage (m^3)**	**Peak WSE (m)**	**Outflow (m^3/s)**
Dillon (Report)	6.3	6,126	191.00	5.2
Wood	5.66	4,864	190.93	4.58
Difference	**-0.64**	**-1,262**	**-0.07**	**-0.62**
Dillon (Visual OTTHYMO)	5.66	3,933	190.65	4.83
Wood	5.66	4,864	190.93	4.58
Difference	**0**	**+931**	**+0.28**	**-0.25**

Given the noted discrepancy between the previously reported values (report as compared to modelling), differences are evident as compared to the updated rating curve results from Wood. The simulated peak storage volume is less than that indicated in the previous reporting, but greater than that using the actual hydrologic modelling. Overall, the updated results generated by Wood have been used as the basis of comparison to proposed channel conditions, in order to ensure consistency.

4.3.2 Site

In order to develop updated quantity control volumes and structures for proposed conditions, appropriate pre-development peak flow targets are required, to ensure that the City of Hamilton's requirements for post-development to pre-development peak flow control (for the 2 through 100-year return periods), are achieved. As noted previously, pre-development hydrologic modelling was completed previously by A.J. Clarke as part of its 2007 assessment. The SWM strategy for the (then) proposed development form (based on providing sufficient flood control to achieve the simulated pre-development peak flow targets), was subsequently assessed as part of the Sanatorium Road assessment work by Dillon. Given that these control rates were defined as part of the previous higher level study (and also subsequently included as part of the OMB settlement), and that there has been no change in land use for the site since that time, it is considered reasonable and appropriate to utilize the previously generated pre-development peak flow targets for the current study. A copy of the pre-development drainage boundaries and simulated peak flows from that study (A.J. Clarke, 2007) are included in Appendix A for reference. The simulated peak flows to the primary creek are also presented in Table 4.3.3; peak flows to the east woodlot are presented in Table 4.3.4.

Table 4.3.3: Simulated Pre-Development Peak Flows to Creek (from A.J. Clarke, 2007)			
Return Period (Years)	**West Site to Creek Areas 110 and 120 (m^3/s)**	**East Site to Creek Areas 210 and 220 (m^3/s)**	**Total Site to Creek Not Including Creek Block (m^3/s)**
2	0.086	0.060	0.146
5	0.133	0.095	0.228
10	0.171	0.120	0.291
25	0.223	0.164	0.389
50	0.266	0.201	0.467
100	0.312	0.242	0.554

Table 4.3.4: Simulated Pre-Development Peak Flows to East Woodlot (from A.J. Clarke, 2007)	
Return Period (Years)	**East Woodlot Areas 230 and 240 (m^3/s)**
2	0.047
5	0.076
10	0.104
25	0.162
50	0.209
100	0.286

4.4 Proposed Conditions

4.4.1 Watercourse

As noted in Section 3.2.3, the design of the watercourse/creek corridor is governed by the requirement for the meander belt allowance and associated erosion hazard setback. A 30 m meander belt has been estimated by GEO Morphix Ltd (refer to Appendix C); the toe of the valley slope for the proposed grading has been designed on this basis. GEO Morphix has also designed the plan form of the low flow channel. The creek profile has been designed on the basis of matching upstream channel inverts at Scenic Drive, and matching the existing downstream invert of the CSP arch culvert (and associated control structure). The top of valley slope has been set as 191.3 m, on the basis of the previously estimated Regional Storm Water Surface Elevation for the "Proposed 2" scenario of 191.0 m (as per Dillon, 2010) with an additional 0.3 m of freeboard.

Notwithstanding the preceding, as part of the revised watercourse design, the riverine/flood control storage volume is expected to be notably increased as compared to existing conditions, due to the implementation of a consistent 30 m meander belt width. In addition, it has been proposed to re-align and re-construct the downstream portion of the existing outlet culvert, given the proposed channel re-alignment, which would involve re-locating the centerline of creek away from the toe of the slope, as is the case under existing conditions. Given the poor condition of the downstream CSP arch culvert, a full replacement of the outlet culvert may be warranted; this should be considered further as part of the subsequent detailed design effort. Given that the existing culvert would have been municipal infrastructure, cost sharing with the City of Hamilton should be considered for the replacement.

As part of the preceding, the outlet control structure will need to be reconstructed. As an initial comparison, it has been assumed that a control structure with the same dimensions as the existing structure would be re-constructed.

An updated stage-storage-discharge relationship for the proposed creek corridor and associated flood control facility has been generated for the preceding two (2) scenarios, using the same approach applied for existing conditions as described in Section 4.3.1:

- Development of a stage-storage curve for the proposed watercourse (based on the updated grading and HEC-RAS modelling prepared by Wood)
- Utilization of the same stage-discharge relationship for existing conditions as per Section 4.3.1 (1.15 m wide by 0.95 m high)

Cross-section locations for the updated proposed conditions hydraulic model are presented on Figure 12. The same Regional Storm hydrograph as utilized for the existing conditions analysis (i.e. "Proposed 2" –

with upstream EA recommended SWM and previously proposed on-site SWM) has been routed through the updated rating curve for proposed conditions. Results are presented in Table 4.4.1.

Table 4.4.1: Simulated Regional Storm Peak Flow (m^3/s) Comparison for Online Flood Control Facility for "Proposed 2" Inflow Scenario with Proposed Creek Grading and Replicated Control Structure

Source	Inflow (m^3/s)	Peak Storage (m^3)	Peak WSE (m)	Outflow (m^3/s)
Existing	5.66	4,864	190.93	4.58
Proposed with Existing Structure	5.66	8,358	190.59	4.28
Difference	**0**	**3,494**	**-0.34**	**-0.30**

As expected, the flood control storage volume under proposed conditions is increased, which reflects the incorporation of the 30 m meander belt width and associated cut in the creek corridor. As the stage-discharge relationship is maintained from existing conditions (based on the assumed replication of the existing control structure), both the peak water surface elevation and peak outflow are reduced. Based on the simulated proposed water surface elevation of 190.59 m, there would be a large freeboard to the spill elevation of 191.3 m (0.71 m).

Proposed conditions Regional Storm floodplain limits using the preceding have been generated using the HEC-RAS modelling, and are presented in Figure 12. The peak water surface elevation from Table 4.4.1 has been applied as a fixed boundary condition. As per conventional practice when developing floodlines for online storage reservoirs, the attenuated peak discharge has been applied for the reach section. However, in this case the modelling is not sensitive to the peak flow applied, as the flood level is dictated by the boundary condition water level (190.59 m).

Further optimization to the configuration of the outlet control structure could potentially be considered as part of the subsequent detailed design phase, however this may be of limited benefit. Further optimization to creek grades could also be considered (i.e. to raise creek block grades to reduce the freeboard to a more typical 0.3 m). Notwithstanding, changes to the creek corridor grading would be predicated on a) replacement of the full length of the former Sanatorium Road/Niagara escarpment culvert crossing to raise grades and b) re-calculation of the associated meander belt width (which would be expected to increase with flatter grades). As such, it is considered that further creek grading modifications would similarly be of limited value.

As part of the detailed design phase, there may be value in incorporating a flood storage "overflow" in the event of debris or other blockage of the primary outlet control structure. This could take the form of one or more ditch inlets at the downstream limits, connected into the storm sewer downstream of the control structure. The bottom of the grates would be expected to be set somewhere between 190.59 m and 191.0 m.

The capacity of the former Sanatorium Road culvert (i.e. online flood control outlet at the Niagara Escarpment edge) under proposed conditions has also been assessed. The currently proposed approach would involve a replacement of the upstream section of the existing 1.6 m x 1.1 m CSP arch culvert, and replacement with a proposed 1500 mm diameter circular concrete pipe. As noted previously, given the poor structural condition of the most downstream section of CSP arch culvert, it may be worthwhile to consider replacement of the entire section with 1500 mm diameter circular concrete pipe. The slope of the existing CSP arch culvert is not known given issues accessing the downstream invert for topographic survey. Previous drawings and other materials have listed the slope of the culvert as 2.2%, however the

source of this slope calculation is unknown. As such, the slope of the proposed section of 1500 mm circular concrete pipe has been conservatively designed at a grade of 0.5% only. Based on manufacturer information, a 1.63 m x 1.13 m CSP arch is a standard size, with an end area of 1.44 m^2, which would be very close to a 1350 mm diameter circular equivalent. As such, this sizing has been used for culvert capacity analysis calculations, as included in Appendix F.

The results based on both the unattenuated Regional Flow (5.66 m^3/s) and proposed attenuated Regional Flow (4.28 m^3/s) have both been presented for comparison purposes in Table F1 (refer to Appendix F). the results indicate that the proposed 1500 mm diameter concrete pipe would be at 86% of full flow capacity for the attenuated Regional Storm, and slightly surcharged (113% of full flow) for the unattenuated flow. A slight surcharge is considered acceptable given the magnitude of the storm event in question. For the existing section of CSP arch culvert (at the assumed 2.2% slope as noted previously), the culvert would be just under full flow capacity for the attenuated Regional Storm flow (99.8%) and at a greater surcharge for the unattenuated flow (132%).

4.4.2 Site

4.4.2.1 Model Selection

The previous site-specific hydrologic modelling (A.J. Clarke, 2007) utilized MIDUSS; however, this is a proprietary and relatively simplistic hydrologic modelling platform. As noted, the analysis completed for the Sanatorium Road study (Dillon, 2010) utilized both HEC-HMS and Visual OTTHYMO. The HEC-HMS modelling was used primarily for erosion analyses, and ultimately, no working copy of the model could be obtained either from the City or Dillon. The Visual OTTHYMO model is also restricted to the simulation of the Regional Storm Event only, as external flows are truncated As such, it is considered that the hydrologic simulation of currently proposed post-development conditions could be completed in any modelling platform, however a preference is for open-source software which also has the ability to leverage hydraulic modelling if required. As such, PCSWMM has been selected for the current study.

PCSWMM is a graphical user interface and pre-processor for the well-known US EPA-SWMM model. It is a combined hydrologic/hydraulic model which is capable of assessing complex hydraulic conditions, including dual drainage (minor/major system) and surcharge and reverse flow. Version 5.1.012 of the EPA-SWMM engine has been used for the current assessment.

4.4.2.2 Hydrologic Modelling Parameters

The more detailed subcatchment boundaries developed for the sizing of the on-site storm sewer system (Figures 7 and 8) have been aggregated to represent the overall contributing drainage areas to the various proposed outfalls on the site. Hydrologic modelling subcatchments are presented in Figure 11. A total of twelve (12) subcatchments has been included in the simulation, including for the proposed development site itself, as well as some for other permeable areas of the site to enable a comparison to previous pre-development flow targets.

Key hydrologic modelling input parameters within PCSWMM include:

- Drainage area (directly measured within PCSWMM)
- Imperviousness (SWMM typically uses directly connected imperviousness only, however imperviousness can be partially routed over pervious land segments if required)
- US SCS Curve Number (based on soils and adjusted to account for non-directly connected imperviousness)
- Depression storage
- Overland flow length

- Overland flow slope
- Overland flow roughness

Imperviousness has been calculated directly based on the proposed form of development. Buildings, roadways, and hard surfaces (including paving stone areas) have all been assumed to be completely impervious, and all directly connected imperviousness.

Based on the completed geotechnical investigation by Landtek (refer to Appendix B), the surficial soils on the site consist primarily of silty clay and clayey silt, which is typical for the area. Consistent with these findings, and the approach applied by Wood for other hydrologic investigations within the City, an SCS Soil Class of C has been assumed for drainage from the site itself. A corresponding Curve Number of 74 has been applied for all pervious (i.e. grassed) areas.

Base values of 1 mm and 5 mm have been assumed for depression storage of the impervious and pervious land segments respectively. Overland flow roughness values of 0.013 and 0.25 have been applied for the impervious and pervious land segments. A uniform slope of 1% has been applied for all subcatchments based on a review of the proposed site grading.

Overland flow lengths and slopes have been directly measured from available data. Overland flow lengths typically range from a minimum of 10 m to a maximum of 150 m for true sheet flow (as recommended in EPA-SWMM Applications Manual, July 2009). Flow lengths have been estimated through measurement for the proposed subcatchments. In general, given the small drainage areas, the model would be expected to have minimal sensitivity to changes in overland flow length within the preceding range of values.

A table presenting all applied subcatchment parameters is presented in Table 4.4.2.

Table 4.4.2: Applied Subcatchment Parameters

Subcatchment ID	Drainage Area (ha)	Overland Flow Length (m)	Imperviousness (%)
S01A	1.07	45	78.8
S01B	0.43	30	85.3
S01C	0.42	20	23.5
S02	1.09	40	65.8
S03A	0.15	20	88.2
S03B	0.25	30	96.6
S04	0.19	20	95.9
S05	0.65	50	88.0
S06A	0.28	35	94.2
S06B	0.21	50	0.0
S07	0.06	20	0.0
S08	1.39	40	0.0

Rainfall inputs have been simulated using a 6-hour Chicago design storm distribution, developed using the City of Hamilton's approved 2004 intensity-duration-frequency (IDF) data from the Mount Hope gauge.

4.4.2.3 SWM Sizing

In order to ensure that the City of Hamilton's requirements for post-development to pre-development peak flows are achieved, quantity control storage will be required. As per the outcomes of the

Sanatorium Road EA (Dillon, 2010), the provision of peak flows controls for the 2 through 100 year storm events (as per A.J. Clarke, 2007) was determined to be sufficient to address any potential erosion risk to downstream receivers, as per the HEC-HMS modelling completed as part of that study. As such, no additional erosion controls have been incorporated into the currently proposed SWM sizing design. Low Impact Development Best Management Practices (LID BMPs) have been considered, however the site is generally constrained by its proximity to the escarpment edge and height of bedrock. As such, on site infiltration is likely of limited benefit but could be considered further as part of the subsequent detailed design phase where feasible.

Consideration has been given to the potential to utilize the existing online SWM facility within the creek block to provide the required storage, as suggested as a possibility in the conclusion to the Sanatorium Road EA (Dillon, 2010). Notwithstanding, this would require a complete hydrologic model for the upstream contributing drainage area. As noted previously, Wood made multiple attempts to obtain such modelling but was only successful in obtaining a Regional Storm simulation (Visual OTTHYMO), which truncates external drainage areas (no subcatchment elements – direct input of a hydrograph from an external source). The re-generation of a new hydrologic model for the contributing subwatershed is considered beyond the scope of the current assessment. Further, it is considered that the proposed SWM strategy for the site should generally be consistent with that advanced in the previous site analyses (A.J. Clarke, 2007), given that this strategy was ultimately tested and approved by the Sanatorium Road EA (Dillon, 2010), and that this approach was also subsequently part of the OMB settlement. As such, in order to achieve the pre-development peak flow targets from the site (as discussed in Section 4.3.2), a series of discrete storage reservoirs have been proposed. A total of four (4) reservoirs have been proposed, as summarized in Table 4.4.3.

Table 4.4.3: Proposed Quantity Control Storage Reservoirs

ID	Drainage Area (ha)	Imperviousness (%)	Type of Facility	Peak 100-Year Storage Volume (m^3)
1	1.92	68%	Wet Pond	1,299
2	1.09	66%	Underground Storage Tank	175
3	1.06	90%	Underground Storage Tank	552
4	0.19	96%	Underground Storage Tank	69

All of the above noted storage reservoirs have been located outside of the updated/proposed Regional Storm Floodplain extents (as per Figure 12, attached).

The preceding storage reservoirs have been explicitly modeled in PCSWMM, with a stage-surface area relationship to define the storage element, and a series of one or more orifices to define the discharge characteristics. Rating curves for each of the SWM facilities, based on the simulated outflows for the 2 through 100-year storm events (including discharges through the orifice controls), are included in Appendix F.

In general, the proposed storage reservoirs have been located and sized based on the available space within the proposed site and associated contributing drainage areas. The proposed wet pond has been selected based on the largest contributing drainage area and the grading constraints of the site and creek corridor. In other locations, the contributing drainage areas generally do not warrant a surface pond feature, or in many cases, such a facility would be infeasible based on the existing site layout and grading. For the underground storage reservoir, extents and depths have been estimated based on Wood's

experience with manufacturer requirements for similar systems, such as the ADS Stormtech Chambers (or equivalent). Based on this experience, systems tend to be approximately 1.8 m in total depth, and the surface area extents (in m^2) are generally equal to the total storage volume (in m^3), given that aggregate void space is used for storage in addition to the open chamber units. These assumptions have been implemented into the current analysis; the exact extents and details of the sub-surface chambers would need to be confirmed as part of the detailed design phase. A minimum ground cover of 1.2 m should be implemented for all sub-surface chamber to ensure sufficient cover for loading and frost heave is maintained.

In addition to the preceding, two (2) additional outfalls to the east woodlot have been proposed. A minor discharge of site flow is proposed from the eastern portion of the site (S07), which will consist of permeable/grassed surface cover, and is therefore considered to be minor and clean. Discharge is also proposed to the existing swale within the woodlot from the internal storm sewer servicing (S06A). This outlet has been implemented based on the recommendations of the terrestrial ecology assessment of the woodlot (refer to reporting by Dougan & Associates), which has noted that there are several significant plant species in this area which require water for sustainability. The discharge from this outlet has attempted to preserve the same "post to pre" peak flow approach; a bioswale has been proposed at the outlet of this area however to provide some infiltration/filtration and water quality treatment prior to discharge.

Simulated peak flow results under post-development conditions at nodes of interest are presented in Table 4.4.4. A comparison to pre-development peak flow targets (A.J. Clarke, 2007, previously presented in Table 4.3.4 and 4.3.5) at nodes of interest is presented in Table 4.4.5. SWM facility rating curves are included in Appendix F.

Table 4.4.4: Simulated Peak Flows at Nodes of Interest for Post-Development Conditions (m^3/s)

Location	Simulated Peak Flow (m^3/s) for Specified Return Period (Years)					
	2	5	10	25	50	100
SWM 1 Inflow	0.268	0.398	0.493	0.616	0.710	0.801
SWM 1 Outflow	0.012	0.015	0.017	0.018	0.019	0.021
SWM 2 Inflow	0.147	0.217	0.269	0.338	0.391	0.442
SWM 2 Outflow	0.069	0.090	0.154	0.220	0.302	0.358
SWM 3 Inflow	0.194	0.279	0.335	0.405	0.459	0.510
SWM 3 Outflow	0.018	0.022	0.045	0.086	0.105	0.124
SWM 4 Inflow	0.038	0.053	0.063	0.076	0.086	0.095
SWM 4 Outflow	0.015	0.019	0.021	0.024	0.026	0.028
Total from SWM to Creek	0.114	0.146	0.237	0.348	0.452	0.531
East Woodlot Swale Outflow (S06A + S06B)	0.055	0.080	0.099	0.123	0.142	0.161

Table 4.4.5: Comparison of Simulated Peak Flows at Nodes of Interest for Various Scenarios (m^3/s)

Location	Scenario	Simulated Peak Flow (m^3/s) for Specified Return Period (Years)					
		2	5	10	25	50	100
Total from SWM to Creek	Pre-Development (AJC, 2007)	0.146	0.228	0.291	0.389	0.467	0.554
	Post-Development (Wood, 2020)	0.114	0.146	0.237	0.348	0.452	0.531
	Difference	**-0.032**	**-0.082**	**-0.054**	**-0.041**	**-0.015**	**-0.023**
East Woodlot Swale Outflow	Pre-Development (AJC, 2007)	0.047	0.076	0.104	0.162	0.209	0.286
	Post-Development (Wood, 2020)	0.055	0.080	0.099	0.123	0.142	0.161
	Difference	**+0.008**	**+0.004**	**-0.005**	**-0.039**	**-0.067**	**-0.125**

As evident from Table 4.4.3, the general SWM strategy involves a greater degree of quantity control at SWM facilities 1, 3 and 4, and a somewhat reduced degree of quantity control at SWM facility 2, which is constrained by the available space for sub-surface storage in that location. The results presented in Table 4.4.4 indicate that the proposed SWM strategy would ensure that the total post-development discharge for areas to the existing creek would be less than the pre-development peak flow targets for all storm events between the 2 and 100 year return periods.

For discharges to the east woodlot, the simulated results indicate that peak flows would be slightly increased for the 2 and 5 year storm events, but decreased for the 10 year storm event and greater. The preceding does not include any crediting of the proposed bioswale attenuative function at the storm sewer outlet to the swale. It is expected that a bioswale feature can be reasonably designed to address the minor difference in peak flows in this case as part of the subsequent detailed design stage. Further discussions with the terrestrial ecology team may also be warranted to confirm the sensitivity of the significant plant species in the swale area, and whether or not the minor increase in peak flows noted above would have any expected impact.

4.4.2.4 Quality Control

Overall

The proposed development site drains to a tributary of Chedoke Creek, which is a tributary to Cootes Paradise and the Hamilton Harbour. As per the City's Comprehensive Development Guidelines, this area requires "Enhanced" water quality treatment as per the Ministry of the Environment's 2003 SWM Planning and Design Manual (i.e. 80% average annual removal of total suspended solids (TSS)).

The proposed stormwater quality control strategy for the site is comprised of a number of different measures, depending on the location in question:

- Wet Pond Stormwater Management Facility
- Oil/Grit Separator Units
- Isolator Row/Separator Row in Underground Storage Chambers
- Bioswales

The Cultec Separator Row is a dedicated row within the sub-surface chamber system that the inflows are routed through first. It is specifically designed to provide water quality treatment and settling of suspended solids as well as other contaminants. This feature is typically then cleaned out in an analogous way to an oil/grist separator, with a specific attachment to a hydrovac hose. It is an approved technology under the Canadian Environmental Testing Verification (CA-ETV) program. As per the information available on the CA-ETV website, this system was tested to provide between 65 and 80% TSS removal, depending on the flow rate. Any proposed oil/grit separator units should also be based on units which are certified through the CA-ETV program; any sizing should also be based on the CA-ETV particle size distribution (PSD).

Outlet 1

For outlet 1, a wet pond SWM facility has been proposed. It should be noted that typically wet pond SWM facilities are only recommended for drainage areas greater than 5 ha to sustain the permanent pool and to ensure that there is sufficient continuous inflow to prevent stagnant conditions. Given the high imperviousness associated with the proposed development, and the implementation of storm sewer inlets at both ends of the SWM facility, it is considered that the wet pond will function adequately. An impermeable liner may be considered at the detailed design stage to ensure maintenance of the permanent pool, notwithstanding the low permeability of on-site soils (silty clays and clayey silts).

The wet pond permanent pool volume has been sized based on the requirements for Enhanced Treatment (80% average annual TSS removal) as per Table 3.2 of the 2003 SWM Planning and Design Manual. With an estimated contributing imperviousness of 68%, the water quality volume would be 220 m^3 per hectare, of which 180 m^3/ha would be permanent pool. Given the estimated contributing drainage area of 1.92 ha, this would equate to a minimum permanent pool volume of 346 m^3. The preliminary proposed wet pond has an estimated permanent pool volume of 620 m^3 which well exceeds the minimum required, at a depth of 1 m (as per the minimum average depth requirement from the 2003 SWM Planning and Design Manual). The preliminary grading includes consideration for a 3 m bench of 5H:1V sloping on either side of the permanent pool as per the 2003 guidelines; all other grades will be a maximum of 3H:1V. It is expected that the design of the wet pond SWM facility may be further adjusted and refined as part of the subsequent detailed design phase.

Outlet 2

Outlet 2 (north-west outlet on site) receives a total of 1.09 ha at 65.8% imperviousness), which includes townhouse roof areas as well as the northern section of site roadway. Given the size of the contributing drainage area and the amount of roadway drainage included, an oil/grit separator has been proposed to provide pre-treatment of the contributing flow. An Imbrium/Stormceptor EF6 unit (1800 mm inside diameter) or equivalent has been proposed. Sizing calculations are included in Appendix F. Based on the sizing calculations, the proposed unit would provide 61% average annual TSS removal for the contributing drainage area. To address the remaining required water quality treatment, it is recommended that the proposed downstream underground chamber system include the isolator row/separator row technology to ensure that the combined water quality treatment exceeds 80% as required.

Outlet 3

An oil/grit separator is proposed to provide pre-treatment for the largest contributing drainage area to Outlet 3, namely subcatchment S05 (0.65 ha at 88.0% impervious). An Imbrium/Stormceptor EF6 unit (1800 mm inside diameter) or equivalent has been proposed. Sizing calculations are included in Appendix F. Based on the sizing calculations, the proposed unit would provide 62% average annual TSS removal for the contributing drainage area.

The other two (2) contributing areas to outlet 3/underground storage tank 3 are smaller (S03A – 0.15 ha at 88.2% imperviousness, and S03B – 0.25 ha at 96.6% imperviousness) and consist primarily of roof area, which is generally considered as "clean" runoff. As such, stand-alone oil/grit separators have not been proposed for these outlets.

Secondary water quality treatment for S05 and primary water quality treatment for S03A and S03B is proposed to be provided through the implementation of an isolator row/separator row. As noted previously, CA-ETV testing suggests this system can provide between 65 and 80% TSS removal depending on the flow rate. This does not account for the secondary water quality treatment associated with filtration/infiltration through the primary open bottom stormwater chambers. Given the pre-treatment provided by the OGS unit for subcatchment S05, and the proportion of the remaining contributing drainage area to Tank 3 which consists of roof drainage, it is considered that the isolator row/separator row would be sufficient to achieve the required overall 80% TSS removal rate.

Outlet 4

Outlet 4 drains a relatively small total area (0.19 ha at 95.9% imperviousness). Of the total estimated imperviousness, the vast majority of the contributing drainage area is actually building roof area, which is generally considered "clean" discharge; the balance represents parking lot area (8 spots). Given the preceding, an oil/grit separator is not considered warranted in this case. Quality control is proposed to be provided through the implementation of an isolator row/separator row. As noted previously, CA-ETV testing suggests this system can provide between 65 and 80% TSS removal depending on the flow rate. This does not account for the secondary water quality treatment associated with filtration/infiltration through the primary open bottom stormwater chambers. Overall, it is considered that these two measures combined would be sufficient to achieve the required 80% average annual TSS removal.

As part of the subsequent detailed design effort (including co-ordination with the mechanical engineer), it may be possible to separate out the storm sewer servicing for the parking lot area from the building drainage, such that the isolator row/separator row can be focused upon the more sediment laden runoff from the parking area. This would ensure that treatment closer to the upper end of the estimated treatment level (i.e. 80% average annual TSS removal) be achieved.

East Woodlot

For the outlet to the east woodlot, as noted a bioswale feature is proposed to provide filtration and water quality treatment of the stormwater discharge. The specifics of the proposed bioswale would be developed at the detailed design stage, however it assumed given the relatively small direct drainage area in this case (0.28 ha from the development), the bioswale can easily be sized to achieve the required 80% TSS removal target.

5.0 Additional Considerations

5.1 Staging/Phasing and Construction Management

Based on the preliminary staging/phasing plan for the site as proposed by Valery Homes, Buildings A and B on the east site (total of 254 units) would be part of Phase 1 of construction (refer to site plans by others). In order to support and service this initial phase of development, it will be necessary to also undertake the following support works:

- At a minimum, re-construct the most downstream portion (65 m +\-) of the existing creek (to the point where the existing and proposed creek alignments match in plan view), including replacement of the control structure and section of culvert (including consideration for a full culvert replacement and incorporation of ditch inlet(s)). Reconstruction/re-grading around proposed SWM tank 3 would also be required, however surface grades in this area would approximately match to existing grades.
- Scenic Drive would need to be re-constructed between San Pedro Drive and a point approximately 100 m east of Sanatorium Drive (approximately 270 m in total). The works would include the extension of the existing sanitary sewer to service the site, twinning of the existing 300 mm diameter watermain, and implementation of a new municipal storm sewer service for the roadway (which is expected to be urbanized in this section). As per the Sanatorium Road EA (2010), a new round-about is also proposed for the intersection of Scenic Drive and Sanatorium Road (and the new Phase 1 entrance). It may be preferable to defer construction of this feature until the completion of Phase 2 given construction vehicle access; this would need to be reviewed with the City of Hamilton.

It is expected that the primary proposed entrance would also serve as the primary construction entrance during Phase 1 and 2. A secondary construction access to the east may also be required during Phase 1. It is expected that the remainder of the eastern site would be used for temporary construction stockpiling and storage during Phase 1 of construction.

Material excavation (cut) will be required to support the proposed construction, including the underground parking garage structures, building foundations, and services. Arterial roads should be used for haul routes to the extent possible. Given the site location and proposed Phase 1 access, the expected haul route would be along Sanatorium Road to Mohawk Road, and from there to the Lincoln Alexander Parkway, via either Mohawk Road/Golf Links Road or Garth Street.

It is expected that excavation will occur with mostly typical construction equipment (backhoes and excavators), since as noted in Section 3.2, the grading design for the site has been designed such that rock excavation is minimized to the extent possible. Some localized rock excavation may be required, including for services and SWM storage tanks along the escarpment edge; in such cases excavation using jackhammers and other vibratory equipment would be required. Such work should be confined to typical working hours to limit the disturbance to adjacent residents.

Ultimately, it is expected that the City of Hamilton will require a more detailed staging, phasing and construction management plan as part of the future detailed design submissions. In conjunction with this, it is expected that a preliminary construction cost estimate will be required by the City of Hamilton. Upon completion of detailed design work, a construction schedule should be developed for review with the City. In addition, residents in the adjacent area should be notified of the pending construction work including timelines for Phase 1 as well as subsequent phases.

5.2 Erosion and Sediment Control

Erosion and Sediment Controls should be implemented as part of Phase 1 and 2, as well as all subsequent phases of construction. Particular efforts should be taken to avoid the transportation of construction phase sediment towards both the creek and the woodlot. Sediment control fencing should be implemented and regularly maintained during the construction phase along the site perimeter and the toe of slope. Silt traps should be implemented in all catchbasins and inlets. A mud mat should also be implemented at all construction accesses, with additional measures taken with respect to dust and sediment control as required (including application of calcium chloride and/or water). Adjacent roadways should be checked regularly and cleaned as required.

Within the creek block, any exposed slopes should be quickly re-vegetated through the application of hydroseeding or terraseeding, with a nurse crop. Erosion control blankets may be required during the germination period. Temporary rock check dams may be required at any surface storm sewer outlets, such as the outlet from SWM tank 3, which would be constructed as part of the Phase 1 works.

6.0 Summary and Recommendations

A proposed functional servicing approach has been developed for the proposed site, in order to address the currently understood requirements for water, wastewater, stormwater, and grading. Servicing and grading plans have been proposed for both the west and east sites, as well as detailed grading for the reconstructed creek block.

Overall, the proposed strategy is considered reasonable to address the applicable requirements, subject to review by the City of Hamilton and Hamilton Conservation Authority. It is expected that further refinement will occur as part of the subsequent detailed design phase.

The following considerations should be noted based on the currently completed work:

Sanitary Sewer Servicing

1. The proposed sanitary servicing strategy will involve an extension of the existing sanitary sewer, preferably beneath the existing 750 mm diameter storm sewer, which will result in a low vertical clearance (approximately 100 mm). It is recommended that the top of the 750mm storm pipe at the point of crossing with the proposed 250mm sanitary pipe be exposed and the elevation accurately determined. The invert of the existing 375mm sanitary pipe on the west side of the most easterly sanitary manhole on Scenic Drive should also be accurately determined, and the vertical clearance between the top of the proposed 250mm sanitary pipe and the existing 750mm storm pipe be assessed for adequacy.
2. The preliminary analysis of the existing sanitary sewer system indicates that the proposed development would increase flows to a section of currently theoretically surcharged sanitary sewer along Scenic Drive between the western limit of Sanatorium Road and Goulding Avenue (380 m +\-), due the flat pipe slope in this area (0.2%). Additional confirmation through review of the City's MIKE Urban model is recommended. Supplementary sanitary sewer flow monitoring may also be warranted to definitively confirm actual flows in the section in question and update model calibration/validation. In the even that off-site pipe upgrades along Scenic Drive are required a cost sharing arrangement with the City of Hamilton should be considered, given that based on the preliminary analyses, the downstream sections of sewer are over-capacity prior to the addition of any flows from the development site.

Watermain Servicing

3. In order to meet the water servicing requirements for the site, it has been proposed to twin a section of the existing 300 mm diameter watermain along Scenic Drive in the vicinity of San Pedro Drive. It is expected that this construction would be completed in conjunction with the extension of the sanitary sewer on Scenic Drive. Cost sharing for the watermain upgrade with the City of Hamilton should be considered in conjunction with the overall roadway reconstruction works.

Storm Sewer Servicing, Creek Block and Stormwater Management

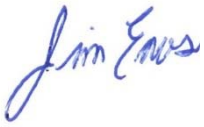
4. A new municipal storm sewer system is recommended along Scenic Drive between Sanatorium Road and the existing creek, in order to collect drainage from external areas and to provide drainage for the roadway itself, which is expected to be urbanized as part of the future reconstruction. The design of the storm sewer system would be completed at the detailed design stage. The costs for this storm sewer are expected to be the responsibility of the City of Hamilton, given that it would service exclusively municipal drainage, however it is expected that this would be reviewed as part of the overall cost sharing arrangement for the reconstruction of Scenic Drive.

5. The proposed creek block is proposed to be re-graded to accommodate the calculated meander belt width of 30 m, with a natural channel design to be implemented. As part of this re-construction, the culvert outlet at the former Sanatorium Road (Niagara Escarpment) is to be at a minimum, partially replaced with a 1500 mm diameter circular concrete pipe to accommodate the creek re-alignment and to avoid the existing bank. A complete replacement of the culvert outlet should however be considered, given the poor condition of the downstream limits of the existing CSP arch culvert. Given that this culvert was municipal infrastructure (crossing of Sanatorium Road), a cost sharing arrangement for the proposed replacement/upgrade should be considered. The existing online flood facility control structure is currently proposed to be replaced "like for like", which will ensure that existing conditions flood control volumes and discharges are achieved. As part of the reconstruction, a flood relief outlet may be considered at the downstream limits (ditch inlet(s)) in the event of debris blockage.

We trust the preceding to be satisfactory.

Respectfully submitted,

Wood Environment & Infrastructure Solutions,
a division of Wood Canada Limited



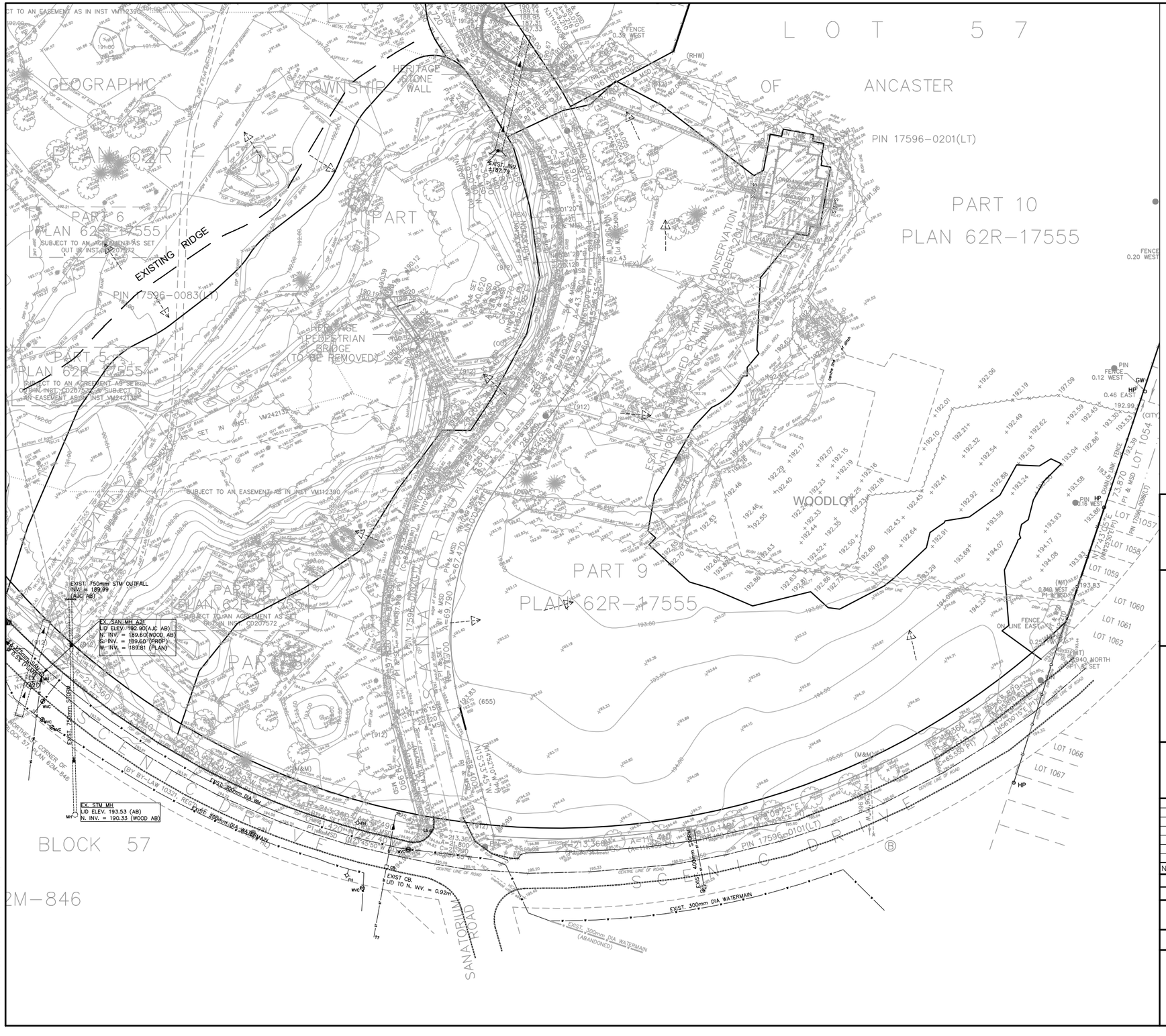


Per: Jim Enos, C.E.T.
Senior Technologist

Per: Matthew Senior, M.A.Sc., P. Eng.
Senior Water Resources Engineer

MJS\JWE\rbs


SANATORIUM ROAD
ESTABLISHED BY BY-LAW 66-244 REG'D AS INST.25862AB
PIN 17596-0101(LT)
ESA LIMIT ESTABLISHED BY HAMILTON CONSERVATION AUTHORITY AND CITY OF HAMILTON, OCTOBER 2019
PART LOT 5
REGISTERED PLAN No 699
LOT 6
LOT 7
LOT 8
LOT 9
LOT 10
LOT 11
LOT 12
CRESCENT ROAD
SAN PEDRO DRIVE
PLAN 62R-7555
PART 6
PART 5
PART 4
PART 3
PART 2
PART 1
PIN 17596-0267(LT)
PIN 17596-0083(LT)
PIN 17596-0084(LT)
PIN 17596-0102(LT)
SUBJECT TO AN EASEMENT AS IN INST VM112390
SUBJECT TO AN AGREEMENT AS IN INST CD207572
SUBJECT TO AN AGREEMENT AS SET OUT IN INST CD207572
EXISTING RIDGE
HERITAGE STONE WALL
TOP OF BROW
CHAIN LINK FENCE
FENCE 4.52 EAST
FENCE 7.04 EAST
FENCE 4.07 EAST
PEDESTRIAN BRIDGE (TO BE REMOVED)
EXIST. 450mm STORM @ 0.5% (PLAN)
EXIST. 375mm SANITARY @ 0.5% (PLAN)
EXIST. 200mm DIA WATERMAIN
EXIST. 750mm STM OUTFALL INV. = 189.99 (AJC AB)
EX. STM MH
LID ELEV. 190.95(AB)
N. INV. = 184.84 (PLAN)
S. INV. = 185.52 (PLAN)
EX. SAN MH
LID ELEV. 191.15(AB)
N. INV. = 186.84 (PLAN)
S. INV. = 187.61 (PLAN)
EX. STM MH
LID ELEV. 192.41(AB)
W. INV. = 189.02 (PLAN)
EX. SAN MH
LID ELEV. 192.50(AB)
N. INV. = 189.06 (PLAN)
E. INV. = 189.25 (PROP)
S. INV. = 189.45 (PLAN)
W. INV. = (PLAN)
EX. SAN MH #2
LID ELEV. 192.90(AJC AB)
N. INV. = 189.60(WOOD AB)
E. INV. = 189.60 (PROP)
S. INV. = 189.61 (PLAN)
W. INV. = 189.81 (PLAN)
EX SAN INV. 187.73
BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m
LEGEND
EXISTING OVERLAND FLOW DIRECTION
KEY PLAN
TRUE NORTH
CONSTRUCTION NORTH
CHEDOKE
BROWLANDS
VALERY HOMES
EXISTING CONDITIONS
PLAN (WEST)
No Date Drawn Appr'd Revisions
Design JE Checked PP
Drawn ML Checked JE
Scale 1:500
Date JULY 2020
Contract No.
Consultant File No. TPB188083
Drawing No. FIGURE 1
wood.
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Last Saved By: jim.enos
Plotted: 2020-08-22
Last Saved: 2020-08-22



KEY PLAN

LEGEND

EXISTING OVERLAND FLOW DIRECTION

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

EXISTING CONDITIONS PLAN (EAST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale: 1:500

Date: JULY 2020

wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 2

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Last Saved By: jim.enos
Plotted: 2020-06-22
Last Saved: 2020-06-22



KEY PLAN

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE
BROWLANDS
VALERY HOMES

FUNCTIONAL SITE
GRADING PLAN (WEST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale: 1:500

Date: JULY 2020

wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 3

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Plotted: 2020-08-22
Last Saved: 2020-08-22

KEY PLAN

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH
CONSTRUCTION NORTH

CHEDOKE
BROWLANDS
VALERY HOMES

FUNCTIONAL SITE
GRADING PLAN (EAST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale 1:500

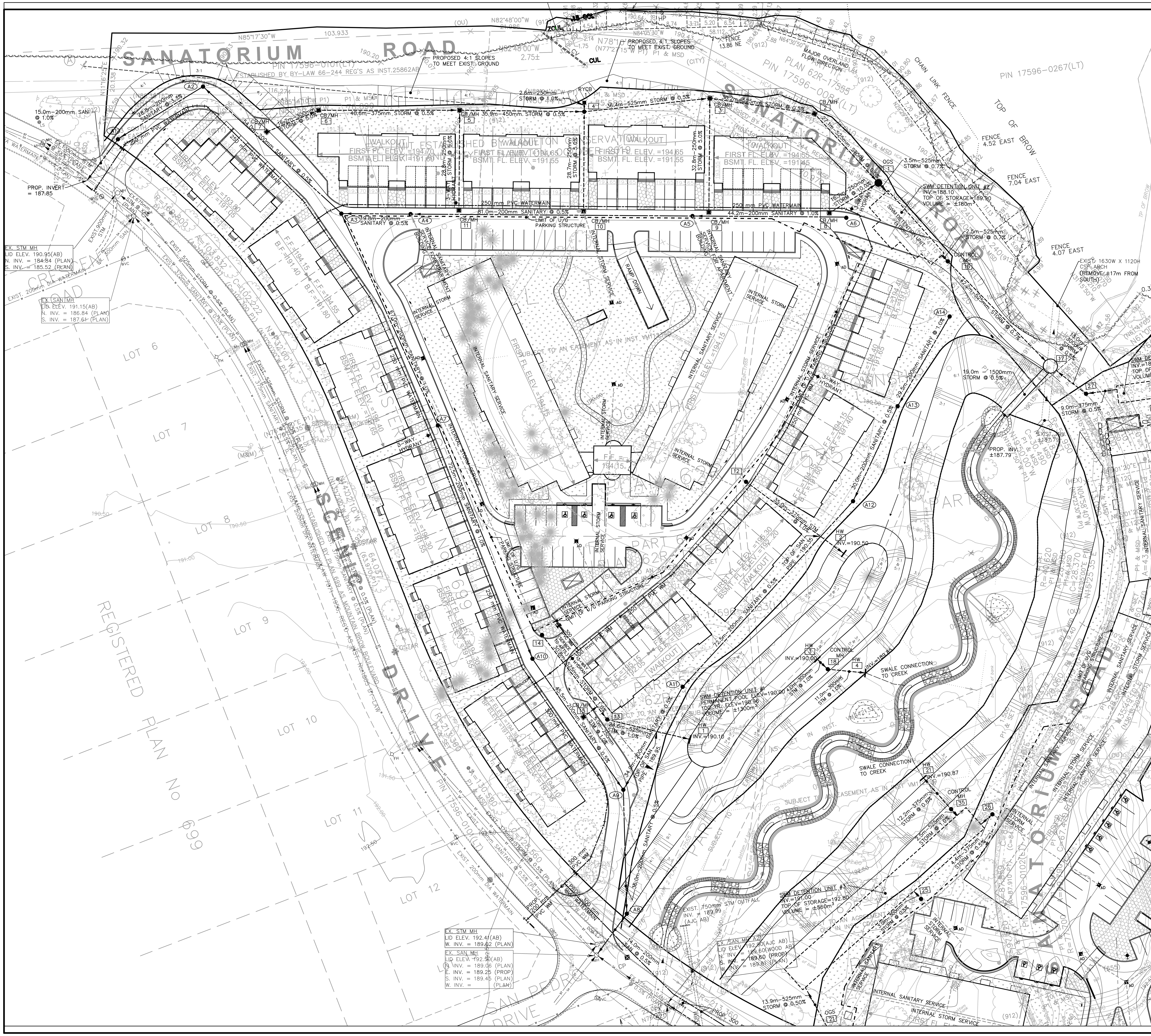
Date JULY 2020

wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 4



KEY PLAN

STORM SUMMARY SCHEDULE

No.	LID ELEV.	N. INV.	S. INV.	W. INV.	E. INV.	REMARKS
OGS 1	191.40	188.20	188.15	188.70	–	ST–EF6
CBMH2	190.55	–	188.34	188.37	–	
CBMH3	190.55	–	189.10	188.55	188.52	
MH4	191.00	189.02	189.30	188.82	188.74	
CBMH5	190.80	–	189.30	189.08	189.00	
CBMH6	191.00	–	–	189.37	189.29	
CBMH7	192.65	–	–	–	190.18	
CBMH8	193.10	–	–	–	190.37	
CBMH9	193.15	190.74	–	–	–	
CBMH10	193.25	190.74	192.15	–	–	BY OTHERS
CBMH11	193.25	190.74	–	–	–	
MH12	193.60	191.40	190.85	–	–	
MH13	191.75	190.38	190.35	–	–	SUMP INV. 190.00
MH14	193.55	191.75	190.69	–	–	BY OTHERS
CBMH15	193.20	190.80	–	–	–	
MH16	192.10	188.08	188.05	–	–	CONTROL MH
MH17	191.70	EX 187.70±	EX 187.70±	187.75	187.93	
MH18	191.30	189.96	189.96	–	–	CONTROL MH
RYCB1	190.70	–	189.05	–	–	
HW 1	–	–	190.10	–	–	
HW 2	–	–	190.50	–	–	
HW 3	–	–	190.00	–	–	
HW 4	–	–	189.85	–	–	

SANITARY SUMMARY SCHEDULE

No.	LID ELEV.	N. INV.	S. INV.	W. INV.	E. INV.	REMARKS
MHA1	190.95	–	–	188.00	188.05	
MHA2	192.40	–	189.08	189.00	–	
MHA3	193.30	NW189.35	SE189.93	–	189.43	ROCK 189.80
MHA4	193.42	–	192.05	189.53	189.56	BY OTHERS
MHA5	193.20	–	191.90	189.97	190.00	BY OTHERS
MHA6	193.15	–	–	190.45	–	ROCK 189.50
MHA7	193.63	190.60	190.60	–	–	
MHA8	192.25	–	–	189.32	189.37	
MHA9	193.25	189.60	189.55	–	189.60	
MHA10	193.43	189.86	189.83	–	–	
MHA11	191.60	–	–	189.77	189.80	
MHA12	192.00	–	–	190.17	190.20	
MHA13	192.20	–	–	190.35	190.38	
MHA14	192.40	–	–	190.68	–	

BENCH MARK NOTE:
BENCHMARK No: 10–06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH | CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

FUNCTIONAL SITE SERVICING PLAN (WEST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale 1:500

Date JULY 2020

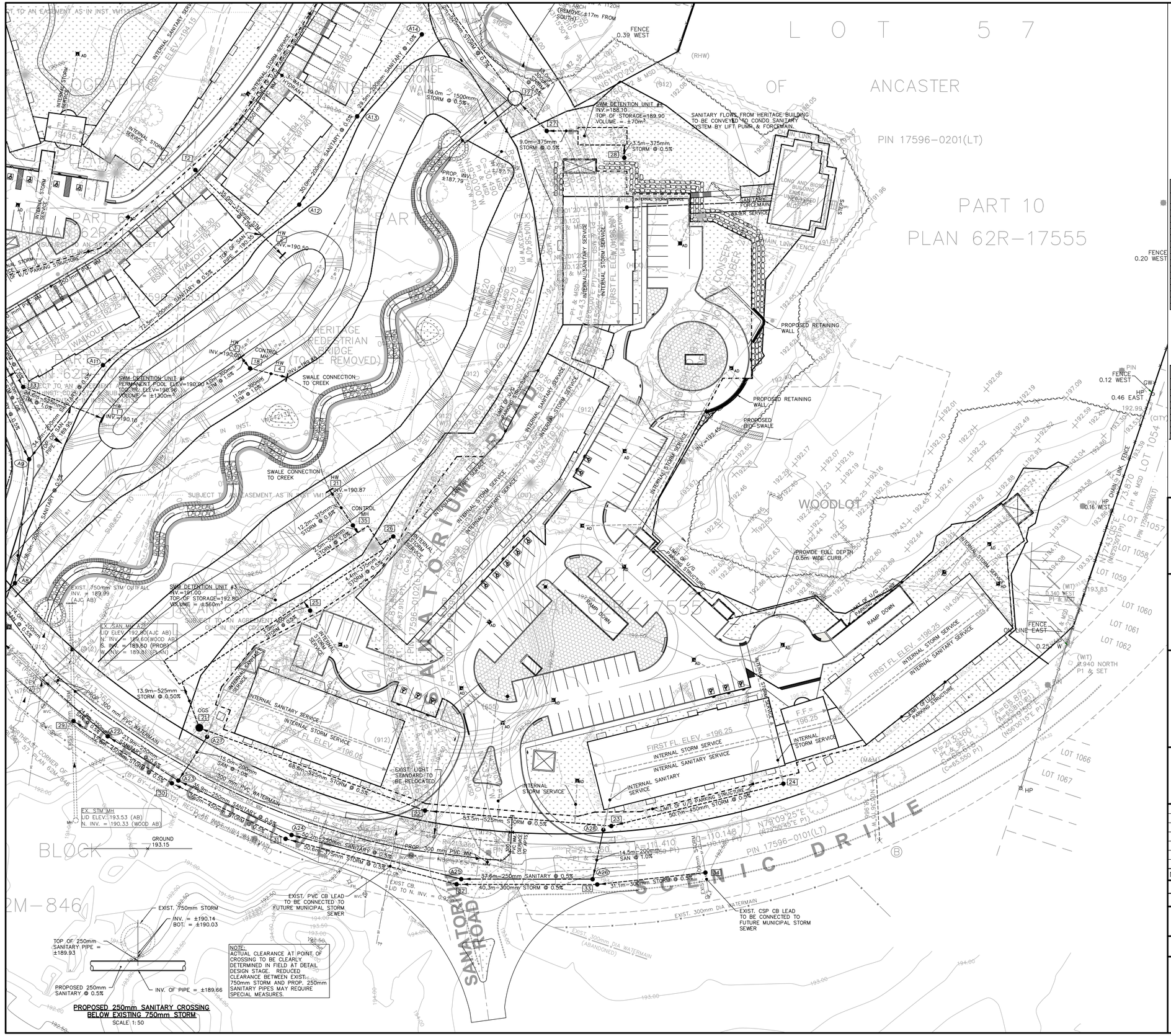
Contract No.

wood.

Consultant File No. TPB188083

Drawing No. FIGURE 5

Path: P:\2018\Projects\TPB188083 – Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Functional Site Servicing Plans.dwg
Plotted: 2020-09-18 Plotted By: mike.loncar
Last Saved: 2020-09-18 Last Saved By: mike.loncar



KEY PLAN

STORM SUMMARY SCHEDULE

No.	LID ELEV.	N. INV.	S. INV.	W. INV.	E. INV.	REMARKS
MH17	191.70	EX. 187.70±	EX. 187.70±	187.75	187.93	
OGS 21	194.15	–	–	191.12	191.07	STC EF6
MH22	195.05	–	–	191.42	191.45	
MH23	195.35	–	–	191.72	191.80	
MH24	196.00	193.80	–	192.06	–	BY OTHERS
MH25	194.30	EX. 194.16±	–	191.02	194.30	BY OTHERS
MH26	194.40	–	–	194.10	191.03	– BY OTHERS
MH27	191.85	–	–	188.00	188.05	CONTROL MH
MH28	191.70	188.12	193.05	–	–	BY OTHERS
MH29	193.15	EX. 190.46±	EX. 190.466±	–	190.46	
MH30	193.90	–	–	190.63	190.66	
MH31	194.70	–	–	190.84	190.92	
MH32	195.10	–	–	191.18	191.25	
MH33	195.35	–	–	191.45	191.48	
MH34	195.60	–	???	191.64	–	BY OTHERS
MH35	194.20	–	–	190.97	190.97	CONTROL MH
HW21	–	–	–	190.87	–	

SANITARY SUMMARY SCHEDULE

No.	LID ELEV.	N. INV.	S. INV.	W. INV.	E. INV.	REMARKS
EXMH21	192.90	189.60EX	189.60	189.81EX	–	
MH22	193.40	–	–	189.73	189.76	
MH23	193.90	189.96	–	189.88	189.91	
MH24	194.70	–	–	190.09	190.12	
MH25	195.10	–	–	190.37	190.40	
MH26	195.30	190.66	–	190.58	–	
MH27	194.20	193.20	190.11	–	–	BY OTHERS
MH28	195.40	193.85	190.80	–	–	BY OTHERS

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION WEST SIDE OF SANATORIUM ROAD SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

FUNCTIONAL SITE SERVICING PLAN (EAST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale: 1:500

Date: JULY 2020

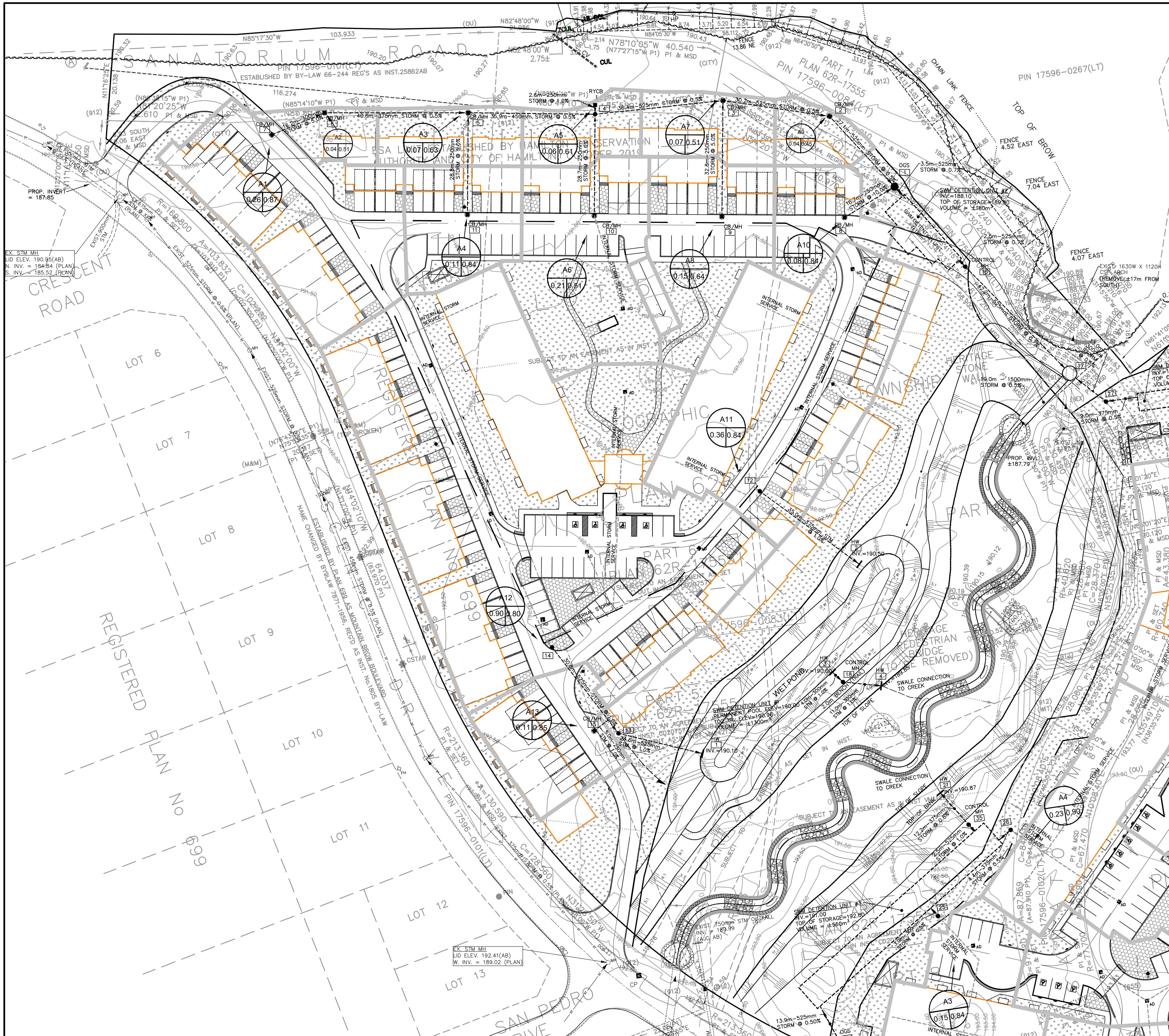
wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 6

Plotted: 2020-09-18
Last Saved: 2020-09-18
Plotted By: mike.loncar
Last Saved By: mike.loncar
Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LDI\FSR\Functional Site Servicing Plans.dwg



KEY PLAN

AREA IDENTIFIER

A2

0.22 | 0.77

AREA (ha)

COEFFICIENT OF IMPERVIOUSNESS

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

FUNCTIONAL STORM DRAINAGE AREA PLAN (WEST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE
Scale	1:500		
Date	JULY 2020		

wood.

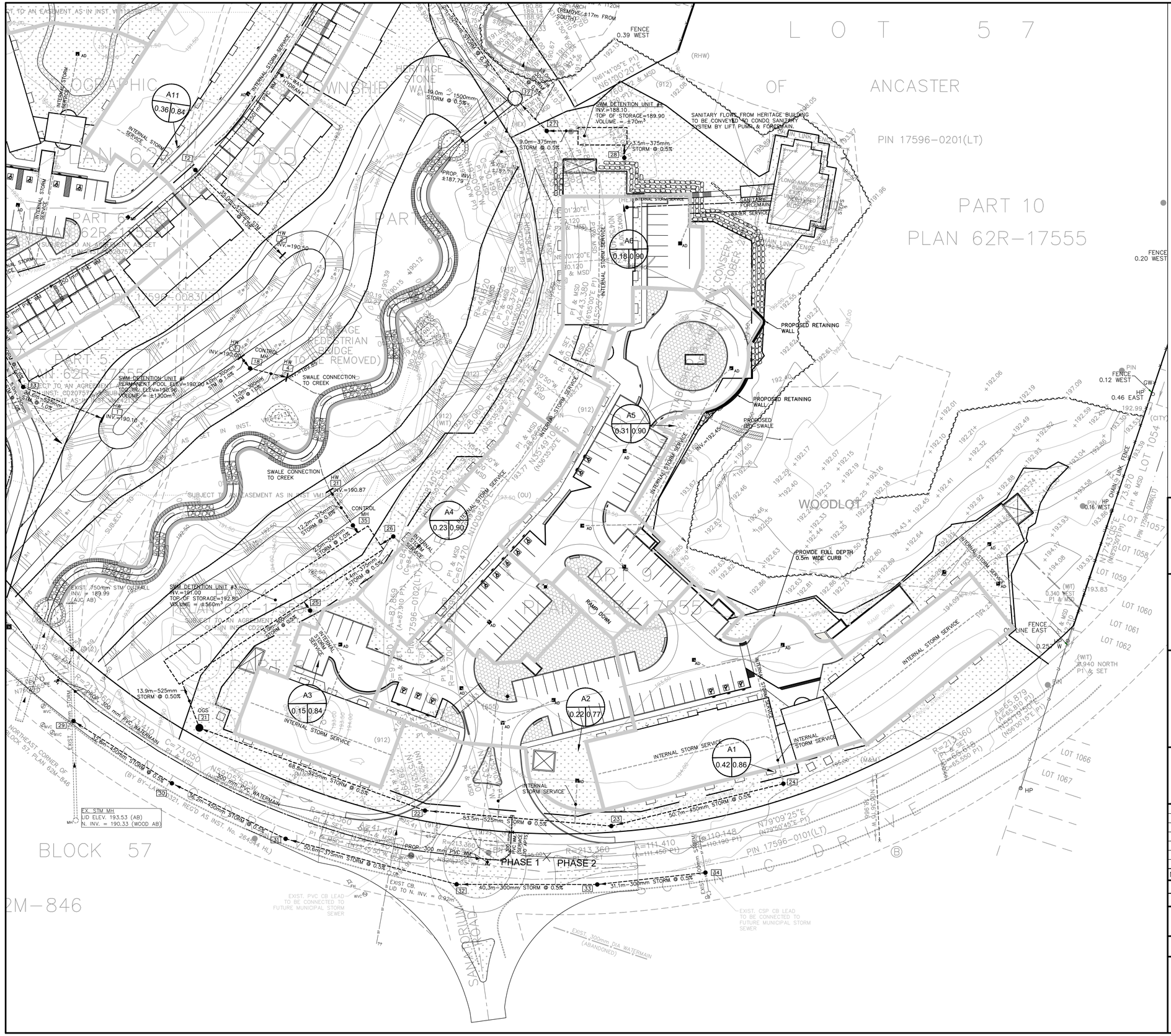
Contract No.

Consultant File No.
TPB188083

Drawing No.
FIGURE 7

Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Functional Storm Drainage Area Plans.dwg

Plotted: 2020-08-22 Plotted By: jim.enos
Last Saved: 2020-08-22 Last Saved By: jim.enos



KEY PLAN

AREA IDENTIFIER
A2
AREA (ha) 0.22 0.77 COEFFICIENT OF IMPERVIOUSNESS

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION WEST SIDE OF SANATORIUM ROAD SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH | CONSTRUCTION NORTH

CHEDOKE
BROWLANDS
VALERY HOMES

FUNCTIONAL STORM DRAINAGE AREA PLAN (EAST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale 1:500

Date JULY 2020

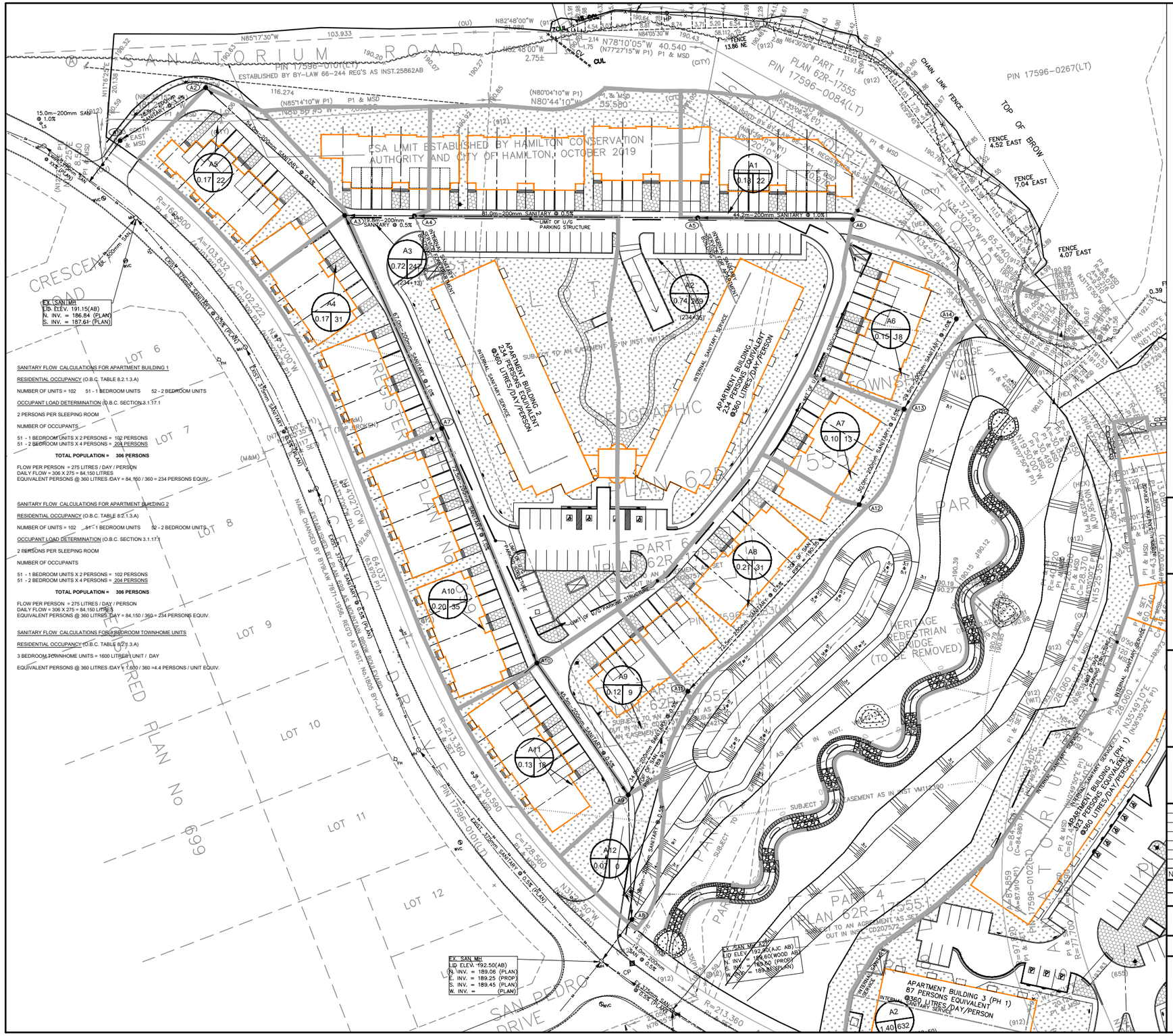
wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 8

Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LDI\FSR\Functional Storm Drainage Area Plans.dwg
Plotted: 2020-08-22 Plotted By: jim.enos
Last Saved: 2020-08-22 Last Saved By: jim.enos



Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Functional Sanitary Drainage Area Plans.dwg

Plotted: 2020-08-22 Plotted By: jim.enos
Last Saved: 2020-08-22 Last Saved By: jim.enos

CHEDOKE BROWLANDS
VALERY HOMES

FUNCTIONAL SANITARY DRAINAGE AREA PLAN (WEST)

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale 1:500

Date JULY 2020

wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 9

SANITARY FLOW CALCULATIONS FOR APARTMENT BUILDINGS (PHASE 1)

RESIDENTIAL OCCUPANCY (O.B.C. TABLE 8.2.1.3.A)

NUMBER OF UNITS = 254 127 - 1 BEDROOM UNITS 127 - 2 BEDROOM UNITS

OCCUPANT LOAD DETERMINATION (O.B.C. SECTION 3.1.17.1

2 PERSONS PER SLEEPING ROOM

NUMBER OF OCCUPANTS

127 - 1 BEDROOM UNITS X 2 PERSONS = 254 PERSONS
127 - 2 BEDROOM UNITS X 4 PERSONS = 508 PERSONS

TOTAL POPULATION = 762 PERSONS

FLOW PER PERSON = 275 LITRES / DAY / PERSON
DAILY FLOW = 762 X 275 = 290,550 LITRES
EQUIVALENT PERSONS @ 360 LITRES /DAY = 209,550 / 360 = 582 PERSONS EQUIV.

SANITARY FLOW CALCULATIONS FOR HERITAGE BUILDING

ASSUMED OFFICE OCCUPANCY

FOR OFFICE AREA OF 600m²

600m² / 9.3m² PER PERSON = 65 PEOPLE

FLOW = 275 LITRES / DAY / PERSON
TOTAL DAILY FLOW = 65 x 275 = 17,875 LITRES/DAY
EQUIVALENT PERSONS @ 360 LITRES / DAY = 18,875/ 360 = 50 PERSONS EQUIV.

SANITARY FLOW CALCULATIONS FOR APARTMENT BUILDINGS (PHASE 2)

RESIDENTIAL OCCUPANCY (O.B.C. TABLE 8.2.1.3.A)

NUMBER OF UNITS = 116 58- 1 BEDROOM UNITS 58 - 2 BEDROOM UNITS

OCCUPANT LOAD DETERMINATION (O.B.C. SECTION 3.1.17.1

2 PERSONS PER SLEEPING ROOM

NUMBER OF OCCUPANTS

58 - 1 BEDROOM UNITS X 2 PERSONS = 116 PERSONS
58 - 2 BEDROOM UNITS X 4 PERSONS = 232 PERSONS

TOTAL POPULATION = 348 PERSONS

FLOW PER PERSON = 275 LITRES / DAY / PERSON
DAILY FLOW = 348 X 275 = 95,700 LITRES
EQUIVALENT PERSONS @ 360 LITRES /DAY = 95,700 / 360 = 266 PERSONS EQUIV.

KEY PLAN

AREA IDENTIFIER
A1
0.42 | 31
AREA (ha) — PERSONS EQUIVALENT (360 LITRES/DAY)

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH | CONSTRUCTION NORTH

CHEDOKE
BROWLANDS
VALERY HOMES

FUNCTIONAL SANITARY
DRAINAGE AREA PLAN
(EAST)

No	Date	Drawn	Appr'd	Revisions	
Design		JE	Checked		PP
Drawn		ML	Checked		JE

Scale 1:500

Date JULY 2020

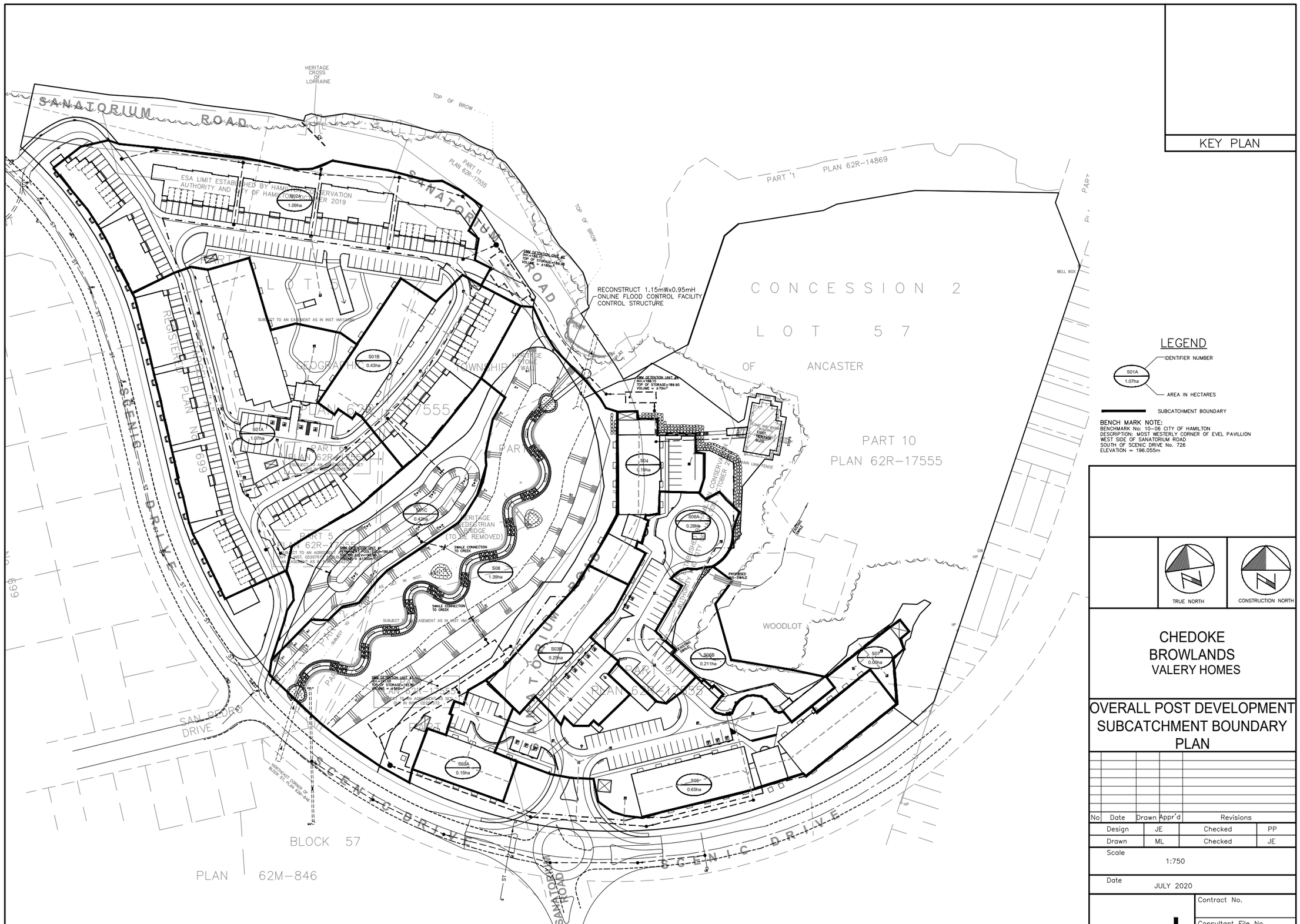
wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 10

Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Functional Sanitary Drainage Area Plans.dwg
Plotted By: jim.enos
Last Saved By: jim.enos
Plotted: 2020-06-22
Last Saved: 2020-06-22



KEY PLAN

LEGEND

S01A 1.07ha — IDENTIFIER NUMBER; AREA IN HECTARES

SUBCATCHMENT BOUNDARY

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

OVERALL POST DEVELOPMENT SUBCATCHMENT BOUNDARY PLAN

No	Date	Drawn	Appr'd	Revisions

Design	JE	Checked	PP
Drawn	ML	Checked	JE

Scale: 1:750

Date: JULY 2020

wood.

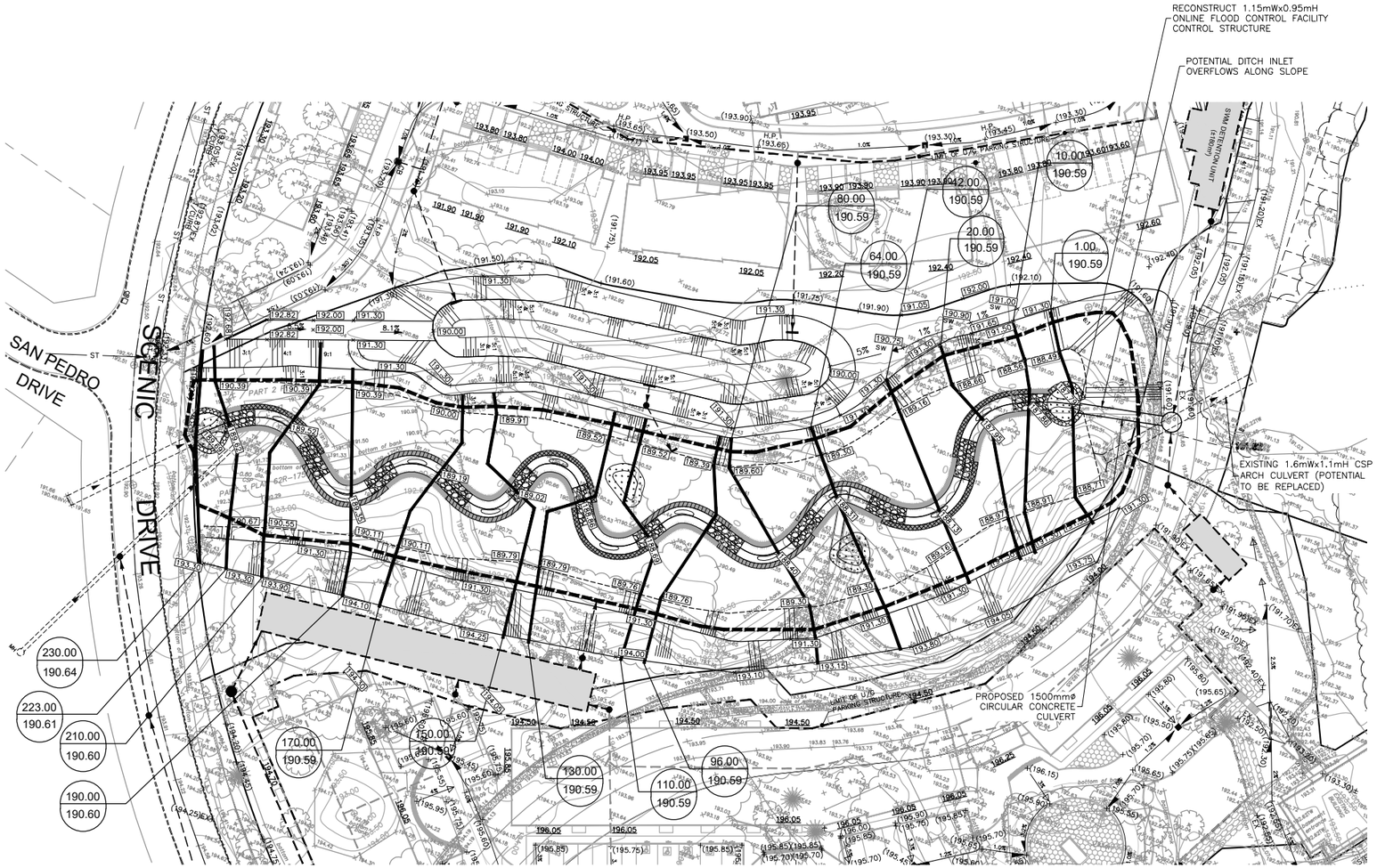
Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 11

Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Overall Post-Development Subcatchment Boundary Plan.dwg

Platted: 2020-09-08 Platted By: mary.kelly
Last Saved: 2020-09-08 Last Saved By: mary.kelly



KEY PLAN

LEGEND

- × 191.30 — PROPOSED GRADE
- × 191.83 — EXISTING GRADE A.J. CLARKE & ASSOCIATES LIMITED, FEBRUARY 16, 2016
- × 192.55 — EXISTING GRADE A.T. McLAREN LTD. 2016
- 192.05 — PROPOSED ELEVATION GRADE AT DWELLING
- 192.00 — EXISTING CONTOURS
- 20.00 / 190.00 — CROSS SECTION NUMBER
- CROSS SECTION LOCATION
- PROPOSED REGIONAL STORM FLOOD ELEVATION (m)
- PROPERTY LINE
- REGIONAL STORM FLOODLINE

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH

CONSTRUCTION NORTH

CHEDOKE
BROWLANDS
VALERY HOMES

POST DEVELOPMENT FLOODPLAIN MAPPING PLAN

No	Date	Drawn	Appr'd	Revisions

Design	MS	Checked	MS
Drawn	ML	Checked	MS

Scale: 1:500

Date: JULY 2020

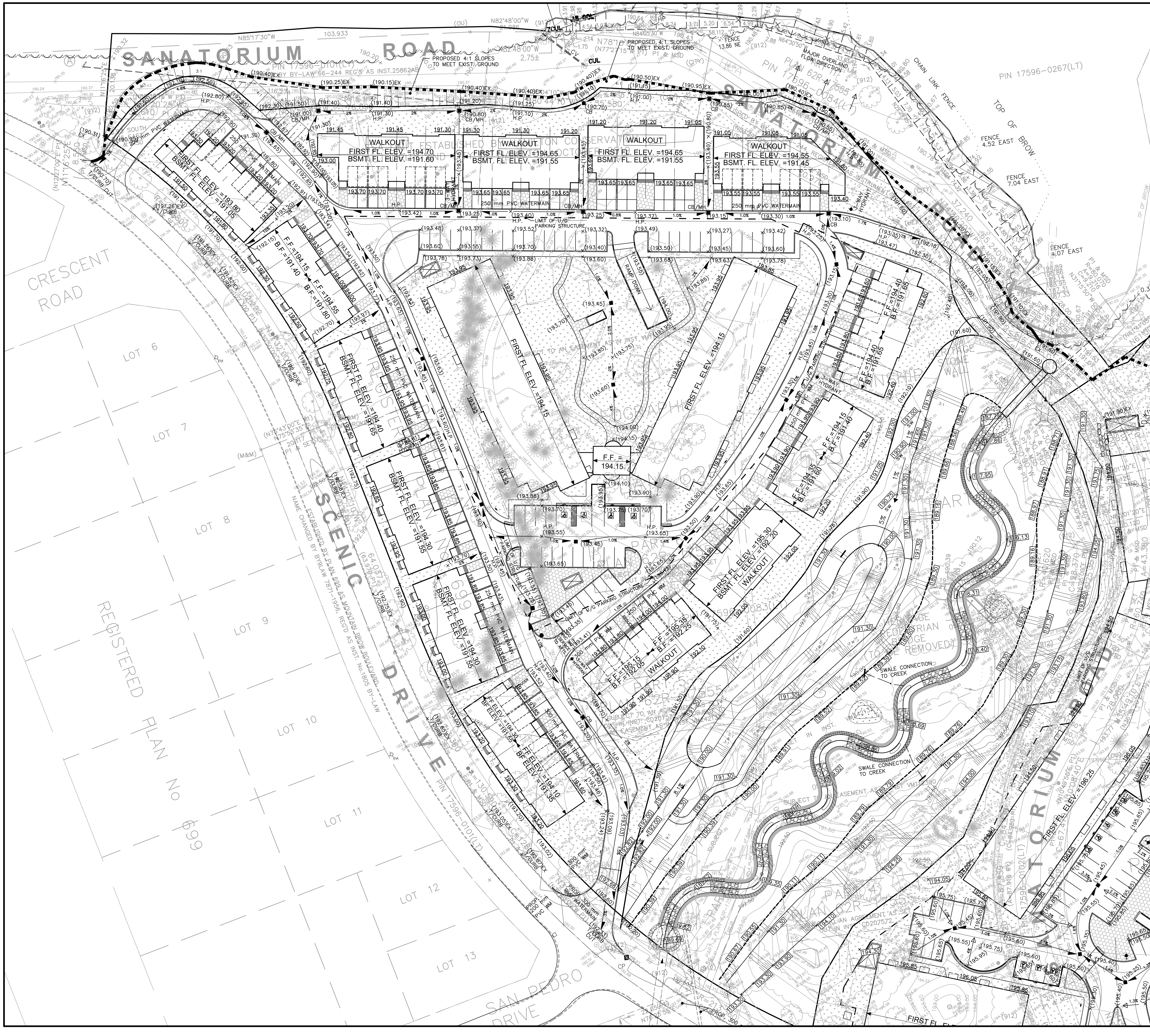
wood.

Contract No.

Consultant File No. TPB188083

Drawing No. FIGURE 12

Plotted: 2020-09-08 Plotted By: mary.kelly
Last Saved: 2020-09-08 Last Saved By: mary.kelly
Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\TSR\Post-Development Floodplain Mapping Plan.dwg



Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENG\01_CAD\02_DWGS\01_LD\FSR\Functional Limit Of Disturbance Plans.dwg

Plotted: 2020-09-03 Plotted By: mike.loncar
Last Saved: 2020-09-01 Last Saved By: mike.loncar

KEY PLAN

LIMIT OF DISTURBANCE PLAN

BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION
WEST SIDE OF SANATORIUM ROAD
SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m

TRUE NORTH | CONSTRUCTION NORTH

CHEDOKE BROWLANDS
VALERY HOMES

FUNCTIONAL LIMIT OF DISTURBANCES PLAN (WEST)

No	Date	Drawn	Appr'd	Revisions

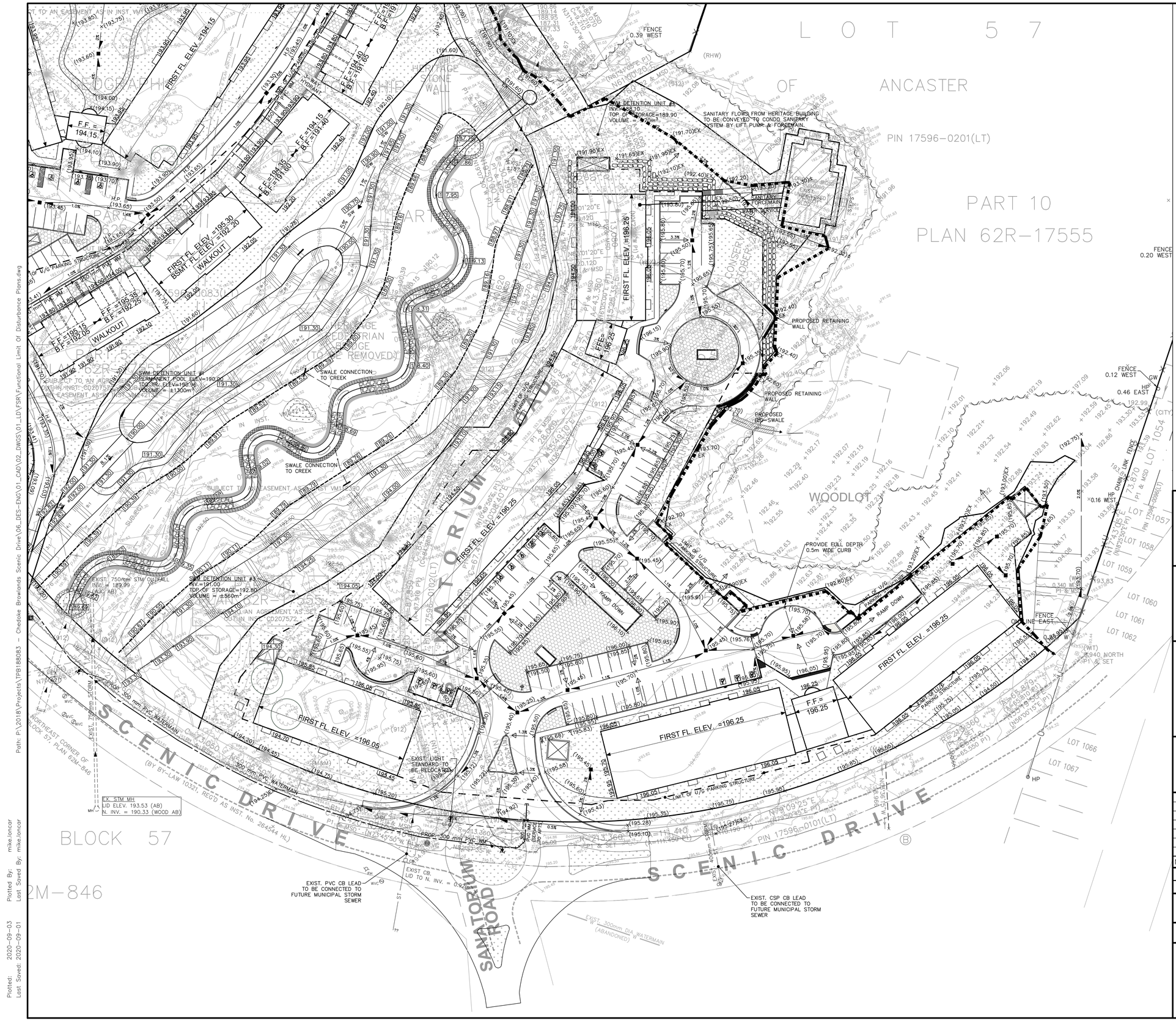
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Drawn	ML	Checked	JE
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Date	JULY 2020		

wood.

Contract No.

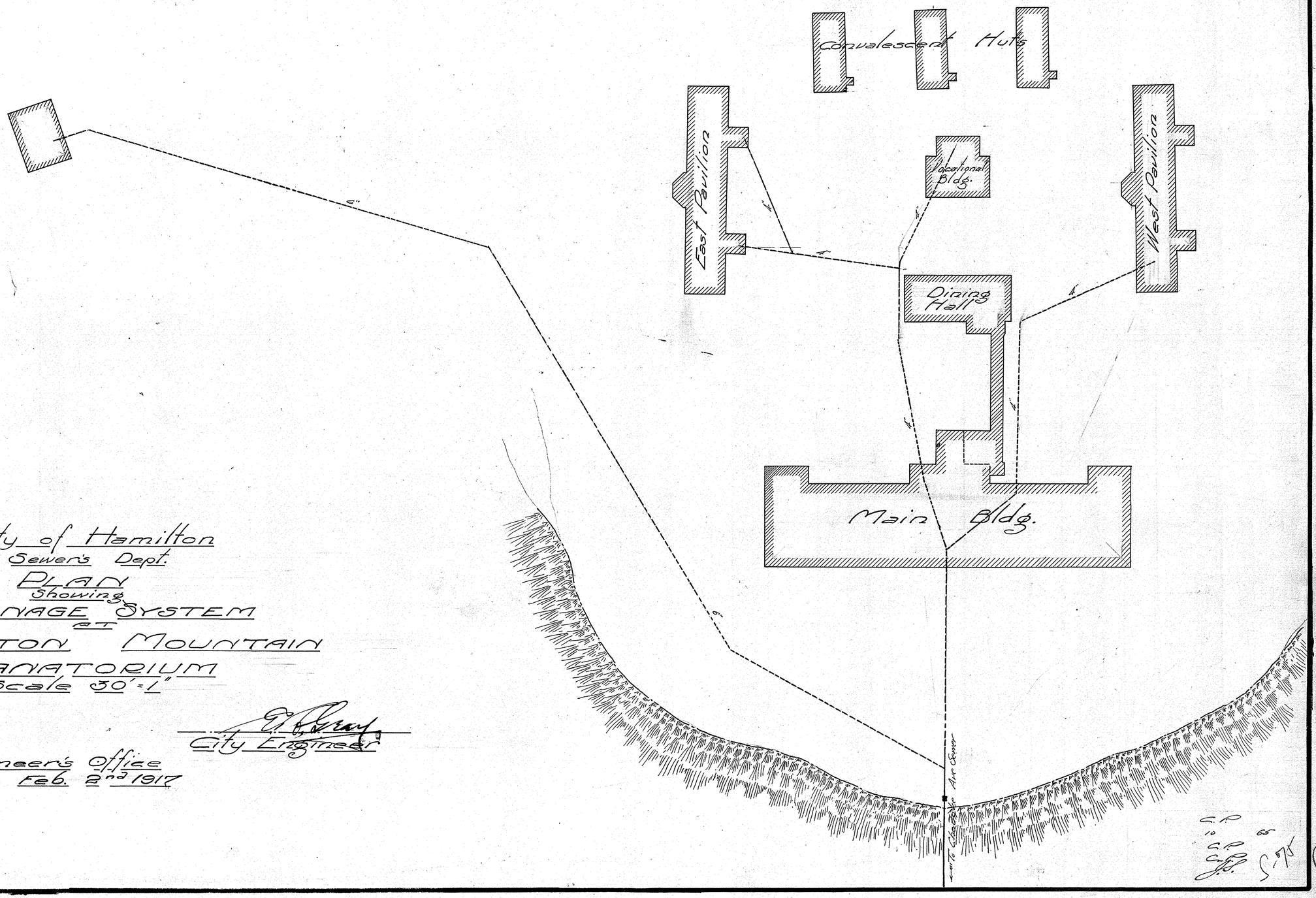
Consultant File No. TPB188083

Drawing No. FIGURE 13


SCENIC DRIVE
SANATORIUM ROAD
LOT 57
PART 10
PLAN 62R-17555
PIN 17596-0201(LT)
PIN 17596-0101(LT)
BLOCK 57
62M-846
WOODLOT
SWM DETENTION UNIT #1
PERMANENT POOL ELEV.=190.80
100 YR. ELEV.=192.90
VOLUME = ±1300m³
SWM DETENTION UNIT #2
INV.=188.10
TOP OF STORAGE=189.90
VOLUME = ±70m³
SWM DETENTION UNIT #3
INV.=191.00
TOP OF STORAGE=192.80
VOLUME = ±560m³
SANITARY FLOWS FROM HERITAGE BUILDING TO BE CONVEYED TO CONDO SANITARY SYSTEM BY LIFT PUMP & FORCEMAIN
SANITARY FORCEMAIN
WATER SERVICE
FIRST FL. ELEV.=196.25
FIRST FL. ELEV.=196.05
FIRST FL. ELEV.=195.30
BSMT FL. ELEV.=192.20
PROPOSED RETAINING WALL
PROPOSED BIO-SWALE
SWALE CONNECTION TO CREEK
EXIST. 750mm STM OUTFALL
INV.= 189.99 (AB)
EXIST. STM MH
LID ELEV. 193.53 (AB)
N. INV. = 190.33 (WOOD AB)
EXIST. LIGHT STANDARD TO BE RELOCATED
PROP. 300 mm PVC WM
EXIST. 300mm DIA WATERMAIN (ABANDONED)
EXIST. PVC CB LEAD TO BE CONNECTED TO FUTURE MUNICIPAL STORM SEWER
EXIST. CSP CB LEAD TO BE CONNECTED TO FUTURE MUNICIPAL STORM SEWER
EXIST. CB. LID TO N. INV. = 0.92
PROVIDE FULL DEPTH 0.5m WIDE CURB
RAMP DOWN
LIMIT OF U/G PARKING STRUCTURE
FENCE 0.39 WEST
FENCE 0.20 WEST
FENCE 0.12 WEST
FENCE 0.46 EAST
FENCE 0.16 WEST
FENCE LINE EAST
HERITAGE STONE WALL
PEDESTRIAN BRIDGE (TO BE REMOVED)
CONSERVATORY
HP
GW
CHAIN LINK FENCE
LIMIT OF DISTURBANCE PLAN
BENCH MARK NOTE:
BENCHMARK No: 10-06 CITY OF HAMILTON
DESCRIPTION: MOST WESTERLY CORNER OF EVEL PAVILLION WEST SIDE OF SANATORIUM ROAD SOUTH OF SCENIC DRIVE No. 726
ELEVATION = 196.055m
KEY PLAN
TRUE NORTH
CONSTRUCTION NORTH
CHEDOKE
BROWLANDS
VALERY HOMES
FUNCTIONAL LIMIT OF DISTURBANCES PLAN (EAST)
No Date Drawn Appr'd Revisions
Design JE Checked PP
Drawn ML Checked JE
Scale 1:500
Date JULY 2020
Contract No.
Consultant File No. TPB188083
Drawing No. FIGURE 14
wood.
Path: P:\2018\Projects\TPB188083 - Chedoke Browlands Scenic Drive\06_DES-ENGY\01_CAD\02_DWGS\01_LD\FSR\Functional Limit Of Disturbance Plans.dwg
Plotted By: mike.loncar
Last Saved By: mike.loncar
Plotted: 2020-09-03
Last Saved: 2020-09-01

APPENDIX A:

Background Information



City of Hamilton
Sewers Dept.
PLAN
Showing
DRAINAGE SYSTEM
AT
HAMILTON MOUNTAIN
SANATORIUM
Scale 30'=1"

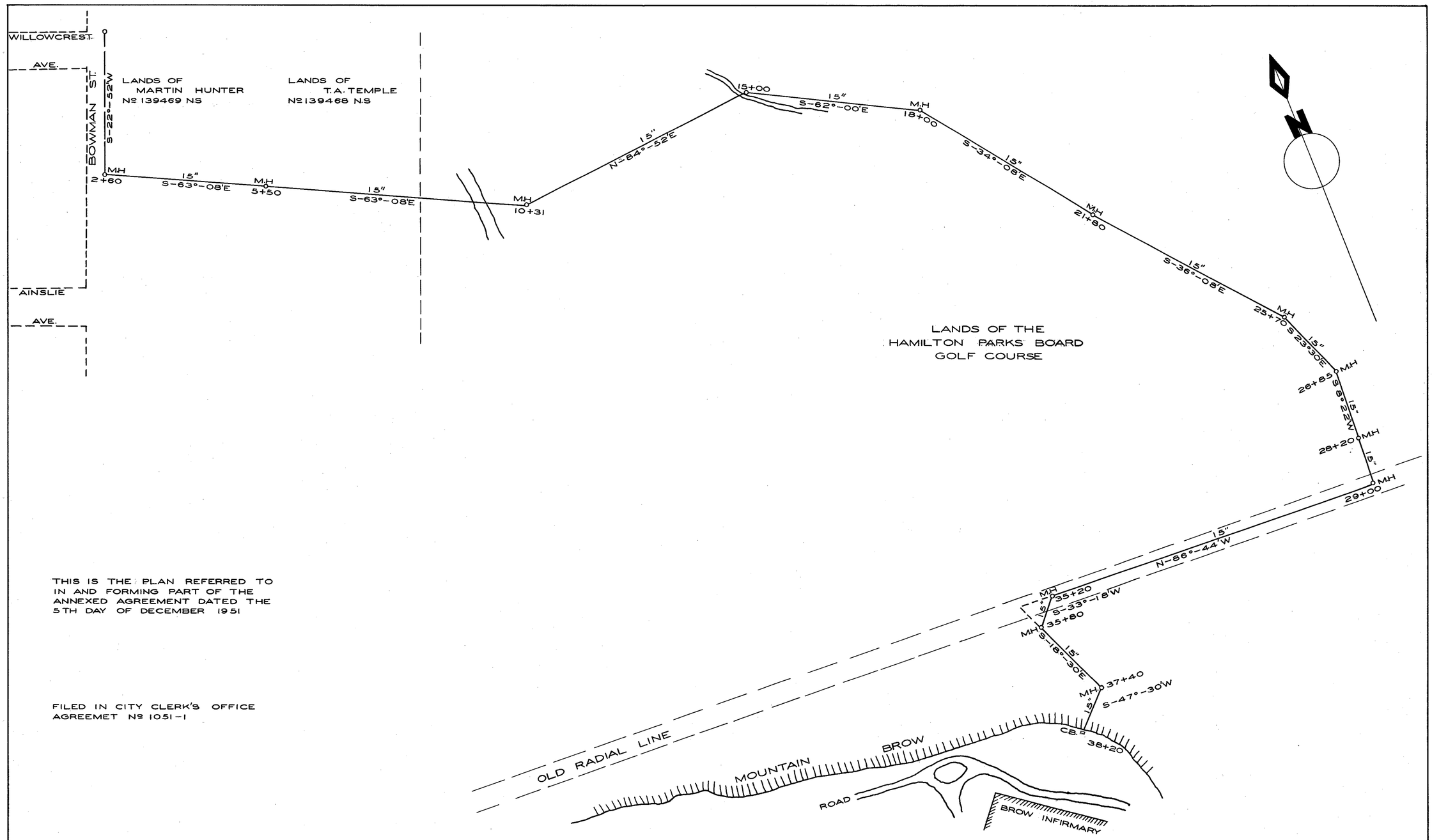
C. L. Darby
City Engineer

City Engineer's Office
Feb. 2nd 1917

C.P. 10 65
C.P.
C.P.

S.75

1/2



THIS IS THE PLAN REFERRED TO IN AND FORMING PART OF THE ANNEXED AGREEMENT DATED THE 5TH DAY OF DECEMBER 1951

FILED IN CITY CLERK'S OFFICE
AGREEMET № 1051-1

ORIGINAL PLAN PREPARED
BY: H.A. LUMSDEN
PROFESSIONAL ENGINEER

PLAN
SHOWING ROUTE
OF
SANITARY SEWER
FOR
MOUNTAIN SANATORIUM
HAMILTON

SCALE 1"-100' NOV. 3 1948
TRACED DEC. 20 1962

PLAN № S-437 SEWERS

26012 25272

SAN PAULO DRIVE

SAN PEDRO DRIVE

MILLER AVENUE

9.0m @ 0.44% 250mm SAN.

EASEMENT

250mm SAN. 41.9m @ 0.53%

EXTERNAL AREA 'B' 1.16 / 62

HEPBURN CRESCENT

GEM COURT

TIFFANY (PHASE 2)

EXTERNAL AREA 'A' 2.93 / 101

JEWEL STREET

CEDARDALE

250mm SAN. @ 1.29%

250mm SAN. 63.0m @ 1.29%

250mm SAN. 103.0m @ 0.49%

250mm SAN. 61.0m @ 0.92%

250mm SAN. @ 0.5%

250mm SAN. 63.5m @ 0.52%

250mm SAN. 100.8m @ 0.50%

250mm SAN. 68.0m @ 0.97%

250mm SAN. 89.5m @ 0.5%

375mm SAN.

375mm PRIVATE SAN 105.0mm @ 0.5%

375mm PRIVATE SAN @ 0.7%

MH A13

MH A12

MH A11

SCENIC DRIVE

BLOCK 57
EXTENDED DETENTION POND No.2

SANATORIUM DRIVE

REDFERN AVENUE

FUTURE AQUATIC CENTRE

LANDS OF CHEDOKE HEALTH CORP. (CHEDOKE–McMASTER HOSPITAL)

EXTERNAL AREA 'C' 20.9 / 106

1.248 / 111

1.916 / 110

0.87 / 101

0.971 / 125

1.44 / 125

0.535 / 125

0.986 / 62

0.944 / 62

0.529 / 62

0.277 / 62

0.476 / 62

0.335 / 62

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56

KEYPLAN N.T.S.

CHEDOKE EXPRESSWAY

CHATEAU COURT

SCENIC DRIVE

SAN PEDRO DR.

SAN ANTONIO DR.

SAN DIEGO PL.

MAGNOLIA DRIVE

CHEDMAC DRIVE

SANATORIUM DRIVE

MOHAWK ROAD WEST

PROPOSED SUBDIVISION

PHASE 1

PHASE 2

LEGEND

0.33 / 62 — AREA IN HECTARES (TOTAL) / POPULATION DENSITY (PERSONS/HECTARES) (AVERAGE)

0.971 / 125 — SUB AREA IN HECTARES / POPULATION DENSITY (PERSONS/HECTARES)

DRAINAGE AREA BOUNDARY

SUB AREA BOUNDARY

EXTERNAL SANITARY DRAINAGE AREA PLAN

SCALE 1:2500

SAN DIEGO COURT

SAN PEDRO DRIVE

EXTERNAL AREA 'B' 1.16 / 62

HEPBURN CRES.

250mm SAN. 63.0m @ 1.33%

EXTERNAL AREA 'A' 2.93 / 101

1.12 / 62

REDFERN AVE.

250mm SAN. 63.5m @ 0.52%

1.81 / 125

AQUATIC CENTRE

CHEDMAC DRIVE

ARENA

CERTIFICATE OF COMPLETION

THIS IS TO CERTIFY THAT THE WORK SHOWN ON THIS DRAWING RELATED TO UNDERGROUND WORKS REQUIRED BY THE REGIONAL MUNICIPALITY OF HAMILTON–WENTWORTH HAS BEEN COMPLETED IN THE FIELD INDICATED WITH ALL VARIATIONS AND CORRECTIONS SHOWN, AS PER THE SUBDIVISION AGREEMENT.

DECEMBER 01, 1998 — DATE

A. J. CLARKE AND ASSOCIATES LTD.

FILE: DRASAN.DWG

LICENSED PROFESSIONAL ENGINEER C. GIAMMARCO PROVINCE OF ONTARIO

AS–CONSTRUCTED

1. AS–CONSTRUCTED OCT. 1998

99-S-1348

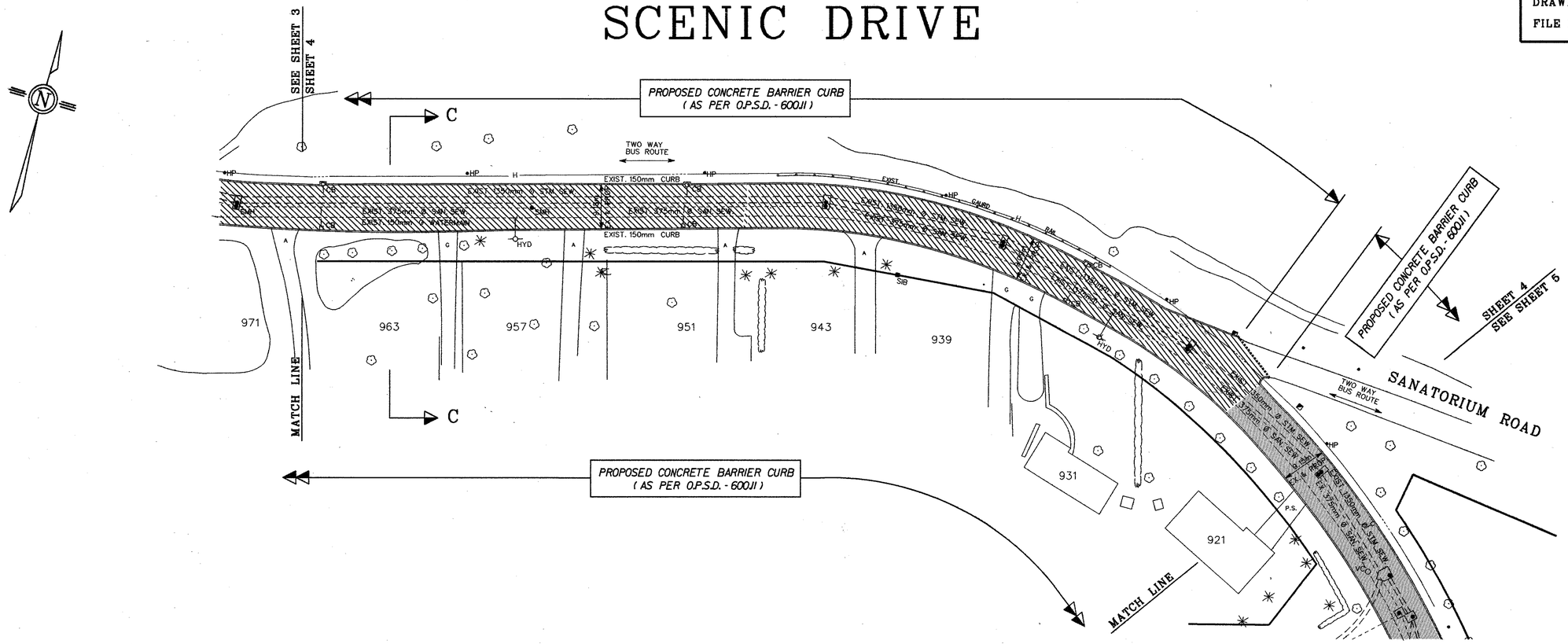
A. J. Clarke and Associates Ltd.
ONTARIO LAND SURVEYORS • CONSULTING ENGINEERS

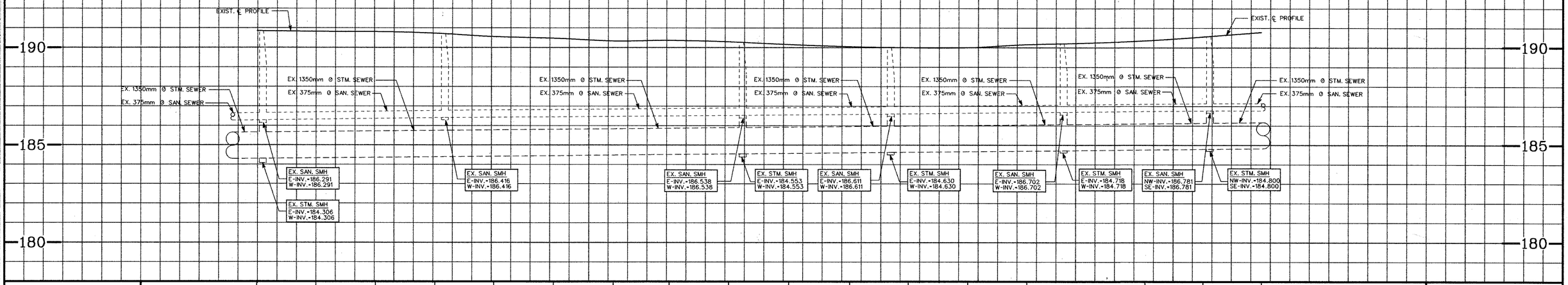
CITY OF HAMILTON	DESIGN: C. GIAMMARCO
	DRAWN: T. LAM
TIFFANY (PHASE 1)	SCALE: 1=750
SANITARY DRAINAGE AREA PLAN	DATE: JUNE 1997
	DWG: K–1485–10

TIFFANY\ENG\DRASAN.DWG Apr 28, 1998 – 17:03:40

CONTRACT No. RHW-93-20 (H)	SHEET No.
DRAWING No. 93-H-9	4 of 10
FILE No. 800-93 - WALKS	

SCENIC DRIVE





EXISTING GUTTER ELEVATIONS, NORTH SIDE	190.67	190.57	190.59	190.50	190.41	190.26	190.24	190.13	190.07	190.02	189.91	189.79	189.71	190.00	190.30	190.39	190.68	EXISTING GUTTER ELEVATIONS, NORTH SIDE
EXISTING GUTTER ELEVATIONS, SOUTH SIDE	190.70	190.67	190.63	190.49	190.39	190.36	190.23	190.14	190.04	190.05	189.96	189.88	189.82	189.99	190.14	190.27	190.40	190.71	EXISTING GUTTER ELEVATIONS, SOUTH SIDE
EXISTING ℄ OF ROAD PROFILE ELEVATIONS	190.85	190.82	190.81	190.73	190.56	190.48	190.34	190.37	190.30	190.16	190.05	189.99	189.98	190.16	190.24	190.36	190.56	190.79	EXISTING ℄ OF ROAD PROFILE ELEVATIONS
℄ CHAINAGE	8+11.19	8+26.39	8+41.68	8+56.90	8+72.01	8+87.35	9+02.72	9+17.09	9+32.33	9+47.77	9+63.42	9+78.47	9+93.75	10+08.91	10+24.14	10+39.22	10+54.10	10+69.10	℄ CHAINAGE

No.	REVISIONS	INITIAL	DATE

DRAWN BY: U.G. & L.B. DATE: JUNE 16, 1993

REFERENCE MATERIAL:

SCALES
HORIZONTAL 1:500 (0, 5m, 10m, 20m)
VERTICAL 1:100 (0, 1m, 2m, 4m)

JP 0.C.G.S. RD

DIRECTOR

COMMISSIONER OF TRANSPORTATION AND ENVIRONMENTAL SEVICES

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

TRANSPORTATION AND ENVIRONMENTAL SERVICES GROUP

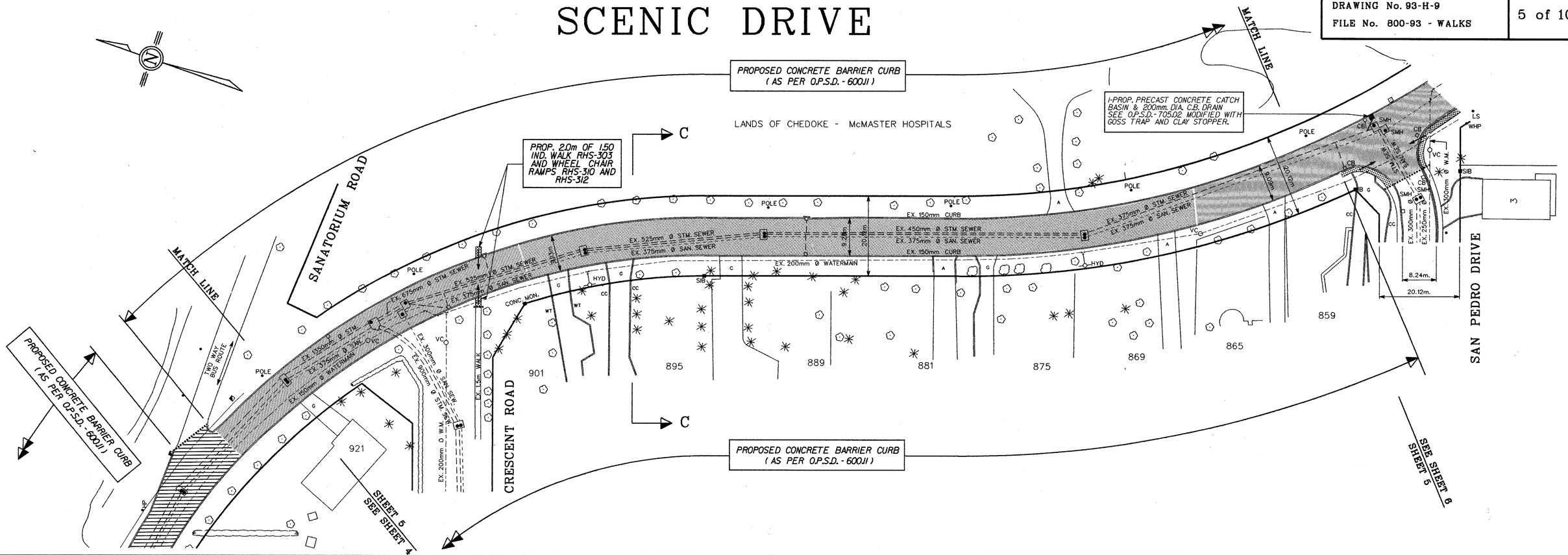
SCENIC DRIVE
PROPOSED ROAD CURB, SIDEWALK RECONSTRUCTION AND ROAD RESURFACING
FROM : WEST CITY LIMITS
TO : WEST 32nd STREET

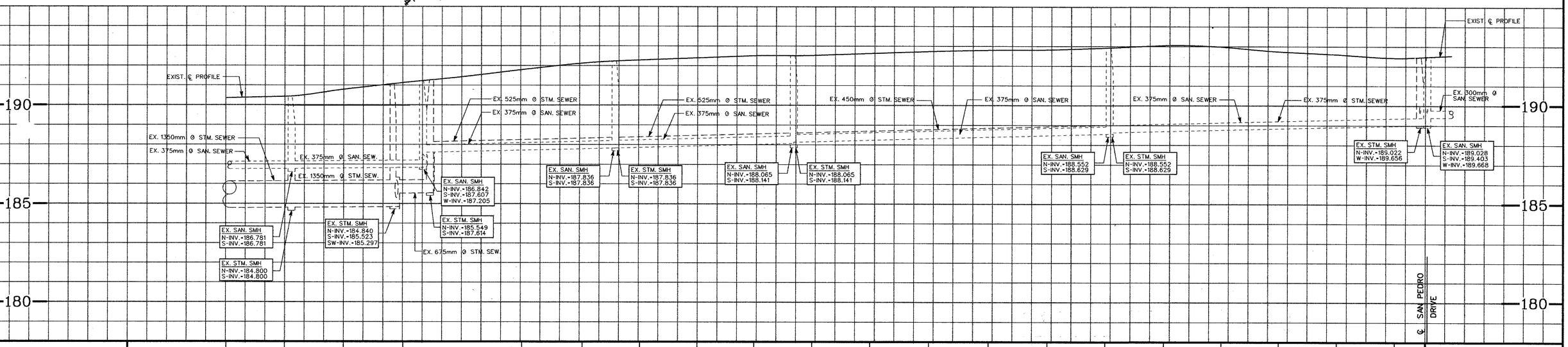
4. SCENIC DR.
93-H-9

5. Scenic Dr.

CONTRACT No. RHW-93-20 (H)	SHEET No.
DRAWING No. 93-H-9	5 of 10
FILE No. 800-93 - WALKS	

SCENIC DRIVE





EXISTING GUTTER ELEVATIONS, EAST SIDE		190.30	190.39	190.68	190.96		191.65	192.08	192.24	192.30	192.41	192.48	192.59	192.74	192.79	192.76	192.90	192.87	192.78	192.52	192.41	192.26		EXISTING GUTTER ELEVATIONS, EAST SIDE
EXISTING GUTTER ELEVATIONS, WEST SIDE		190.27	190.40	190.71		191.32	191.74	192.07	192.28	192.33	192.41	192.45	192.57	192.60	192.76	192.48	192.88	192.98	192.91		192.51	192.52		EXISTING GUTTER ELEVATIONS, WEST SIDE
EXISTING ℄ OF ROAD PROFILE ELEVATIONS	190.24	190.36	190.56	190.79	191.13	191.44	191.80	192.13	192.29	192.40	192.51	192.53	192.63	192.73	192.80	192.82	192.92	193.05	192.98	192.74	192.60	192.43		EXISTING ℄ OF ROAD PROFILE ELEVATIONS
℄ CHAINAGE	10-24.14	10-39.22	10-54.10	10-69.10	10-84.22	10-99.36	11-14.49	11-29.86	11-44.44	11-59.04	11-73.48	11-88.25	12-02.94	12-17.92	12-33.08	12-48.01	12-62.73	12-77.62	12-92.26	13-06.82	13-21.55	13-36.31		℄ CHAINAGE

No.	REVISIONS	INITIAL	DATE

DRAWN BY: U.C. & L.B. DATE: JUNE 16, 1993

REFERENCE MATERIAL:

SCALES

HORIZONTAL 0 5m 10m 20m 1:500

VERTICAL 0 1m 2m 4m 1:100

JP 56-RO

DIRECTOR

COMMISSIONER OF TRANSPORTATION AND ENVIRONMENTAL SERVICES

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

TRANSPORTATION AND ENVIRONMENTAL SERVICES GROUP

SCENIC DRIVE

PROPOSED ROAD CURB, SIDEWALK RECONSTRUCTION AND ROAD RESURFACING

FROM : WEST CITY LIMITS

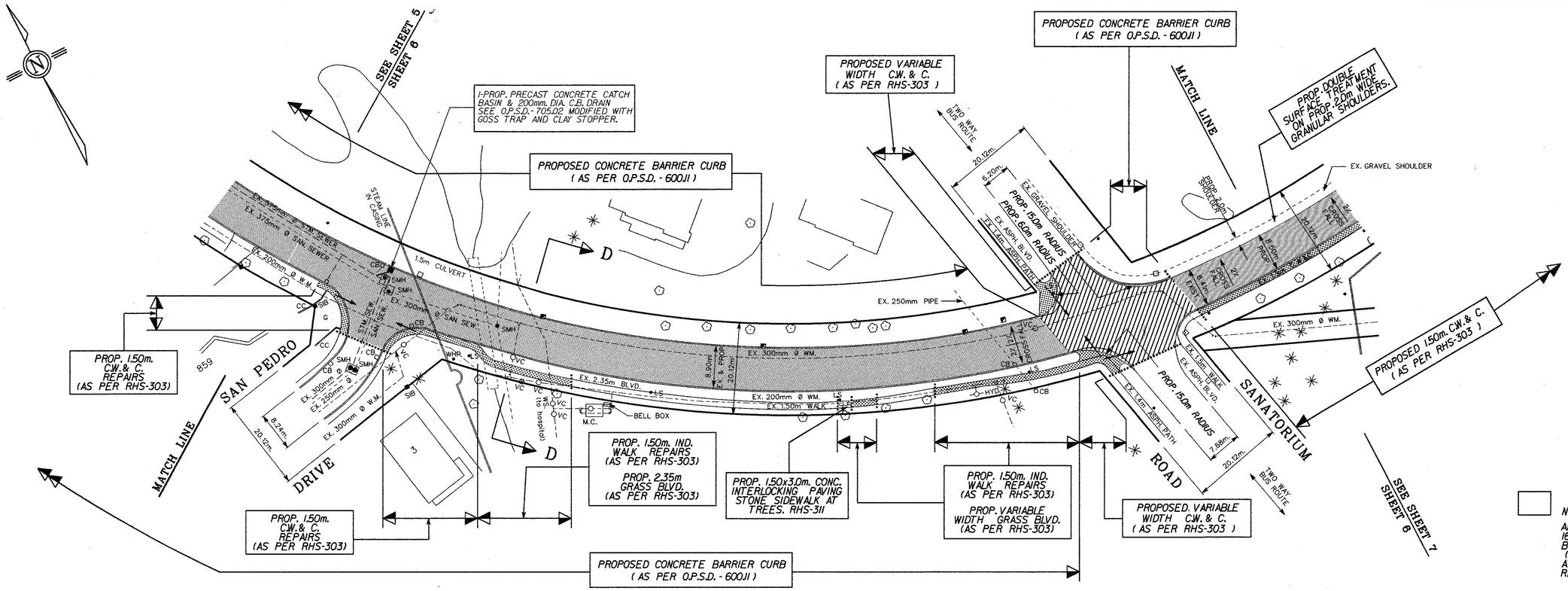
TO : WEST 32nd STREET

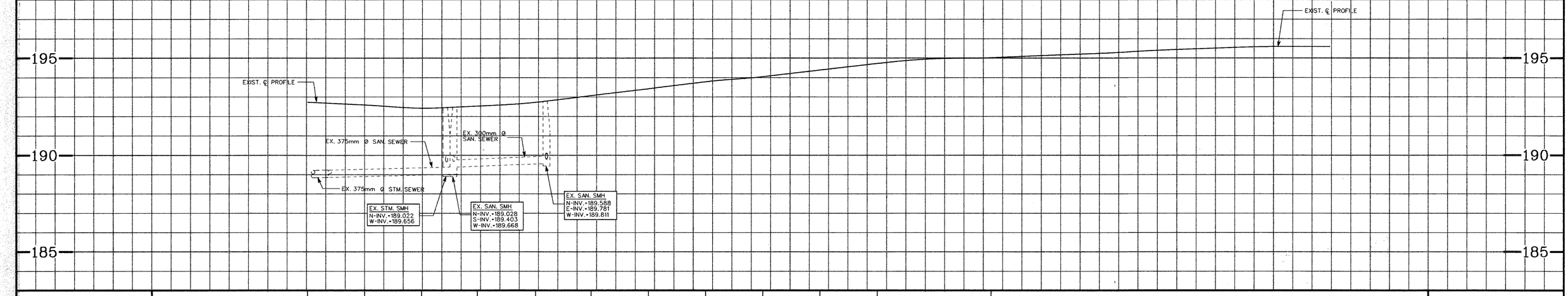
93 H 9

Scale 0k.

CONTRACT No. RHW-93-20 (H)	SHEET No.
DRAWING No. 93-H-9	6 of 10
FILE No. 800-93 - WALKS	

SCENIC DRIVE





EXIST. GUTTER ELEVATIONS, NORTH EAST SIDE	192.52	192.41	192.26	192.36	192.57	192.86	193.23	193.55	193.87	194.22	194.60	194.88											EXIST. GUTTER ELEVATIONS, NORTH EAST SIDE
EXIST. GUTTER ELEVATIONS, SOUTH WEST SIDE	192.51	192.52	192.55	193.04	193.33	193.68	194.01	194.38	194.75	194.92	194.93	195.52	195.61	195.61									EXIST. GUTTER ELEVATIONS, SOUTH WEST SIDE
EXISTING ℄ OF ROAD PROFILE ELEVATIONS	192.74	192.60	192.43	192.49	192.54	192.73	193.07	193.43	193.79	194.05	194.39	194.73	194.97	195.02	195.14	195.25	195.40	195.51	195.59				EXISTING ℄ OF ROAD PROFILE ELEVATIONS
℄ CHAINAGE	13+06.82	13+21.55	13+36.31	13+45.23	13+50.74	13+65.77	13+80.15	13+94.95	14+09.74	14+24.51	14+39.30	14+54.15	14+68.92	14+83.55	14+97.94	15+12.54	15+26.92	15+41.57	15+56.25				℄ CHAINAGE

No.	REVISIONS	INITIAL	DATE

DRAWN BY: U.G. & L.B. DATE: JUNE 16, 1993

REFERENCE MATERIAL:

SCALES
HORIZONTAL 1:500 (0 5m 10m 20m)
VERTICAL 1:100 (0 1m 2m 4m)

DIRECTOR for COMMISSIONER OF TRANSPORTATION AND ENVIRONMENTAL SEVICES

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

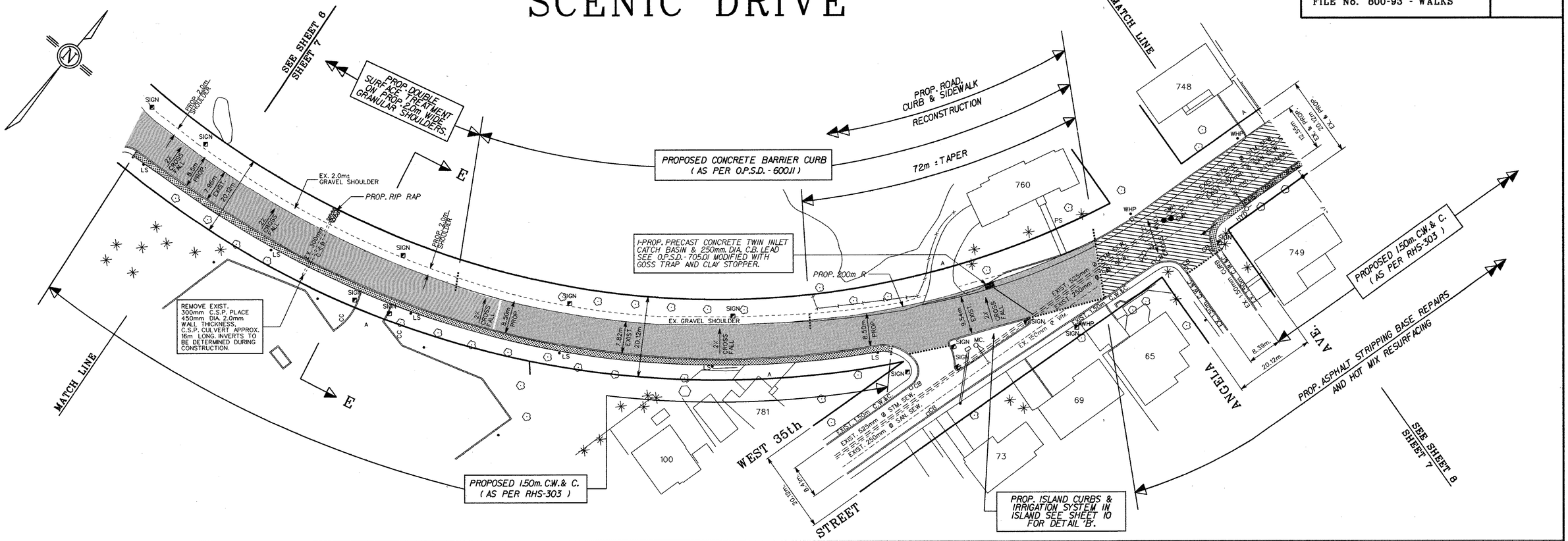
TRANSPORTATION AND ENVIRONMENTAL SERVICES GROUP

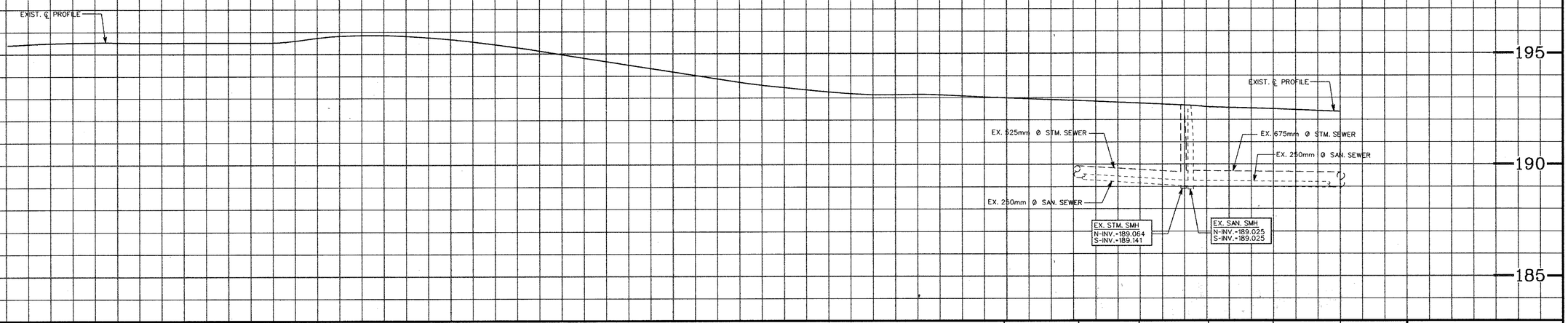
SCENIC DRIVE
PROPOSED ROAD CURB, SIDEWALK RECONSTRUCTION AND ROAD RESURFACING
FROM : WEST CITY LIMITS
TO : WEST 32nd STREET

93-H-9

CONTRACT No. RHW-93-20 (H)	SHEET No.
DRAWING No. 93-H-9	7 of 10
FILE No. 800-93 - WALKS	

SCENIC DRIVE





EXIST. GUTTER ELEVATIONS, NORTH WEST SIDE																	192.89	192.76	192.60	192.43	192.36	192.30		EXIST. GUTTER ELEVATIONS, NORTH WEST SIDE
EXIST. GUTTER ELEVATIONS, SOUTH EAST SIDE	195.52	195.61	195.61	195.58	195.72	195.90	195.93	195.63	195.16	194.69	194.18	193.68	193.42	193.17	193.04	192.67	192.57	192.50	192.42	192.30		EXIST. GUTTER ELEVATIONS, SOUTH EAST SIDE
EXISTING ℄ OF ROAD PROFILE ELEVATIONS	195.40	195.51	195.59	195.59	195.67	195.81	195.82	195.57	195.17	194.69	194.22	193.75	193.40	193.16	193.15	193.00	192.88	192.75	192.64	192.58	192.49	192.37		EXISTING ℄ OF ROAD PROFILE ELEVATIONS
℄ CHAINAGE	15+26.92	15+41.57	15+56.25	15+71.16	15+85.97	16+00.89	16+15.44	16+30.37	16+45.31	16+60.38	16+75.49	16+90.59	17+05.43	17+20.51	17+35.66	17+50.97	17+66.07	17+81.29	17+93.18	17+96.88	18+11.42	18+26.57		℄ CHAINAGE

No.	REVISIONS	INITIAL	DATE

DRAWN BY: U.C. & L.B. DATE: JUNE 16, 1993

REFERENCE MATERIAL:

SCALES
0 5m 10m 20m
HORIZONTAL 1:500
0 1m 2m 4m
VERTICAL 1:100

JP ccg LB BB S.S. RD

DIRECTOR

for COMMISSIONER OF TRANSPORTATION AND ENVIRONMENTAL SEVICES

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

TRANSPORTATION AND ENVIRONMENTAL SERVICES GROUP

SCENIC DRIVE

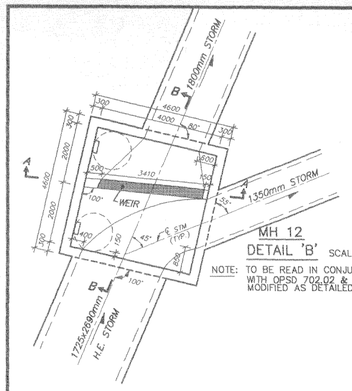
PROPOSED ROAD CURB, SIDEWALK RECONSTRUCTION AND ROAD RESURFACING

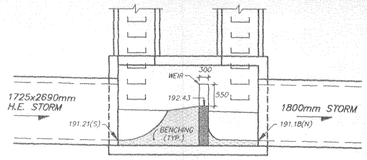
FROM : WEST CITY LIMITS

TO : WEST 32nd STREET

7. SCENIC DR. 93 H 9

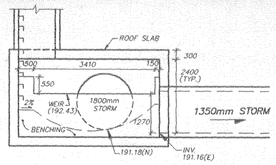
BLOCK 57



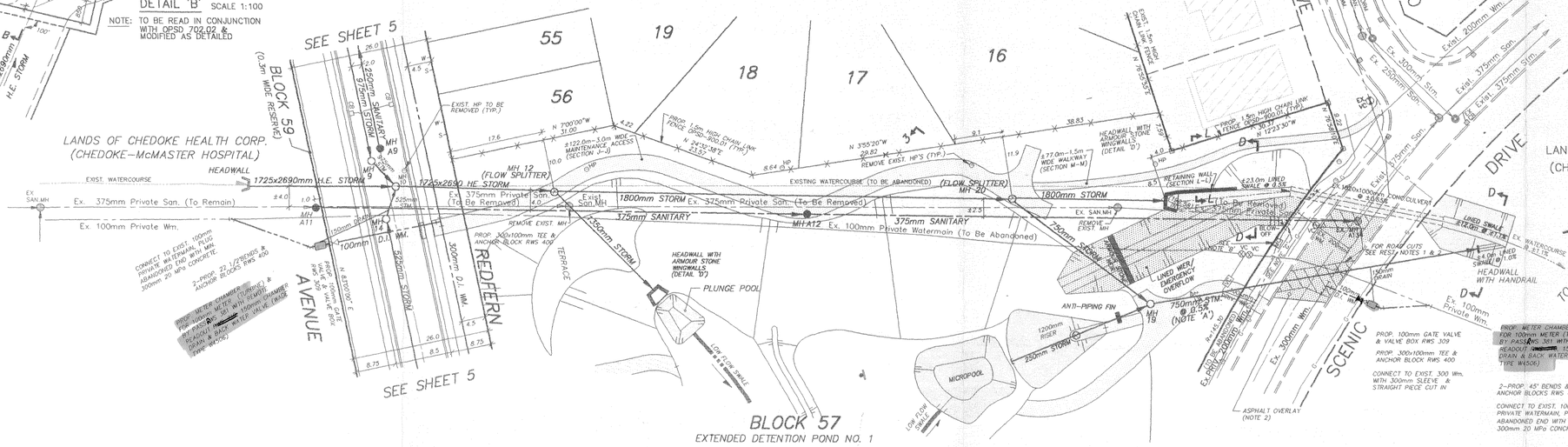


SECTION B–B

NOTE: WEIR REINFORCING TO CONSIST OF SIZE 15 REBAR @ 300mm O/C EACH WAY, EACH FACE

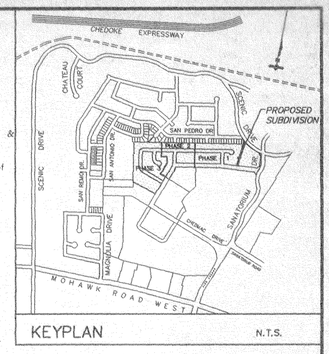


SECTION A–A



SCENIC DRIVE RESTORATION NOTES

1. Saw–cut pavement over trenches as required and backfill with Gron. 'A' material compacted to 95% SPD. Pavement to be restored to suit existing conditions (min. 40mm HM 3 & 85mm HL 8) as per RHS 501 & 502.
2. A lap joint (minimum 1.0m wide) or overlay areas as noted, shall be milled to a depth of 40mm, tack coated and overlaid with 40mm HM 3) as detailed.
3. All disturbed sidewalks, curb & gutters, etc. and all utilities shall be restored to suit original conditions.
4. All disturbed subdrains (if any) to be replaced and reconnected to suit original conditions.
5. All disturbed grassed boulevard areas to be fine graded and restored with sod on min. 100mm topsoil.
6. One lane of traffic shall be kept open at all times.
   All restoration shall be to the satisfaction of the City and the Region.

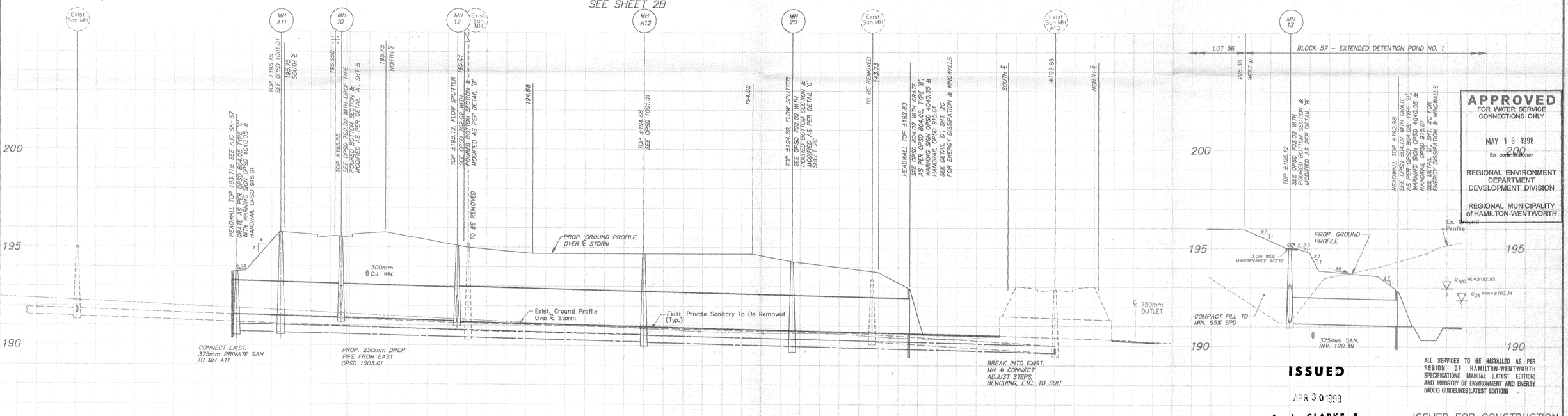


NOTES

'A' FOR PROFILE OF 750mm STORM (MH 20 – MH 19 – HEADWALL), POND SECTIONS, DETAILS, ETC. REFER TO SHEETS 2B&2C,

'B' EXIST. 150mm TURBINE METER, PIPING, ETC. IN METER CHAMBER TO BE SALVAGED IF POSSIBLE AND REINSTALLED IN METER CHAMBER AT SANATORIUM ROAD & REDFERN AVENUE.
EXIST. METER & VALVE CHAMBERS, VALVES, PIPING, ETC. TO BE REMOVED.

'C' DISCONNECT 200mm WATERMAIN AT BLOW–OFF, INSTALL PLUG & & ANCHOR BLOCK RWS 400, PLUG ABANDONED END WITH MIN. 300mm 20MPa CONCRETE.



APPROVED
FOR WATER SERVICE CONNECTIONS ONLY
MAY 13 1998
for commissioner
REGIONAL ENVIRONMENT DEPARTMENT
DEVELOPMENT DIVISION
REGIONAL MUNICIPALITY of HAMILTON–WENTWORTH

ISSUED
APR 30 1998
A. J. CLARKE & ASSOCIATES LTD.

ALL SERVICES TO BE INSTALLED AS PER REGION OF HAMILTON–WENTWORTH SPECIFICATIONS MANUAL (LATEST EDITION) AND MINISTRY OF ENVIRONMENT AND ENERGY (MOEE) GUIDELINES (LATEST EDITION)

ISSUED FOR CONSTRUCTION

Exist. 375mm Priv. San. @ ±0.7%	S 191.46± E 191.56± N 191.46±	Exist. 375mm Private Sanitary @ ±0.7% (To Remain)	±191.09 190.635	PROP. 92.0m–375mm SANITARY @ 0.5% P.V.C. PIPE CSA B182.2 SDR–35
	190.175 190.145	PROP. 105.0m–375mm SANITARY @ 0.5% P.V.C. PIPE CSA B182.2 SDR–35	±189.62 ±189.58	Exist. 375mm Priv. San. @ 0.5%
				SANITARY SEWER INVERT ELEVATIONS
Exist. Watercourse @ ±1.1%	±191.93	PROP. 28.0m–1725X2690mm H.E. STORM @ 0.5% R.C. PIPE CSA A257.2, HE–111 TYPE 50 CEMENT	S 191.391 W 192.056 E 192.441 N 191.361	PROP. 29.5m–1725X2690mm H.E. STM. @ 0.5% R.C. PIPE CSA A257.2, HE–111 TYPE 50 CEMENT
	S 191.213 E 191.163 N 191.183	PROP. 85.5m–1800mm STORM @ 0.5% R.C. PIPE CSA A257.2, 100–D TYPE 50 CEMENT	S 190.753 E 190.723 N 190.735	PROP. 30.0m–1800mm STM. @ 0.5% R.C. PIPE CSA A257.2, 100–D TYPE 50 CEMENT
190.605	PROP. LINED SWALE ±23.0m @ 0.5% (SECTION D–D)	±190.49	Exist. 1820x1000mm Conc. Box Culvert @ ±0.65%	±190.30
PROP. 12.0m @ 1.1% LINED SWALE (SECTION D–D)	±190.14	Exist. Watercourse @ ±1.1%	S 191.213 N 191.183 E 191.163	PROP. 27.5m–1350mm STORM @ 0.5%, R.C. PIPE CSA A257.2 100–D, TYPE 50 CEMENT
191.02	PLUNGE POOL (DETAIL 'E')	189.96		
				STORM SEWER INVERT ELEVATIONS
	REDFERN AVENUE		SCENIC DRIVE	LOW FLOW SWALE @ 0.2%

GENERAL NOTES

1 TENDERERS SHALL SATISFY THEMSELVES AS TO THE NATURE OF THE GROUND AND BID ACCORDINGLY.

2 ALL ROCK LINE INDICATIONS SHOWN ON THE PLAN MUST BE VERIFIED BY THE CONTRACTOR.

3 CONTRACTOR SHALL VERIFY LOCATIONS AND INVERTS OF ALL EXISTING SANITARY AND STORM SEWERS AND WATERMAINS, PRIVATE SEWER DRAINS AND WATER SERVICES, GASMAINS, CABLE TV, HYDRO AND TELEPHONE DUCTS, ETC., AT START OF CONSTRUCTION.

BENCH MARK

City of Hamilton Index No. 824
Elevation: 198.288m
Description: On west side of Nursing School on Sanatorium Road South of Scenic Drive 18m north of entrance.

NO	REVISION	BY	DATE
4	MH A11 SHIFTED TO ℄, STM. & SAN. ALIGNMENT REVISED	C.G.	FEB. 20'98
3	METER CHAMBERS & METERS REVISED	C.G.	FEB. 10'98
2	MH'S 12 & 20 ALIGNMENT REVISED	C.G.	OCT. 5'97
1	UPDATED	C.G.	SEPT '97

ENGINEER

LICENSED PROFESSIONAL ENGINEER 98–03–10 PROVINCE OF ONTARIO
ENGINEER

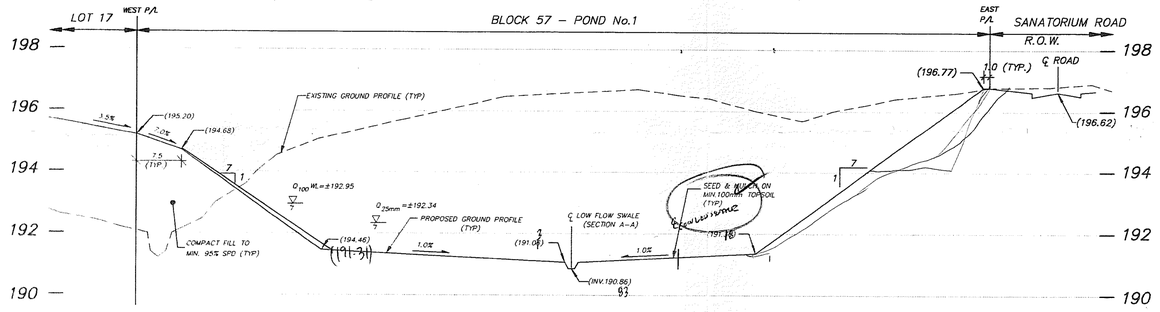
A. J. Clarke and Associates Ltd.
ONTARIO LAND SURVEYORS • CONSULTING ENGINEERS

CITY OF HAMILTON
TIFFANY (PHASE 1)
BLOCK 57

DESIGN: C. GIAMMARCO
DRAWN: T. LAM
SCALE: H 1:500 V 1:100
DATE: MAY 1997
DWG: K–1485–9

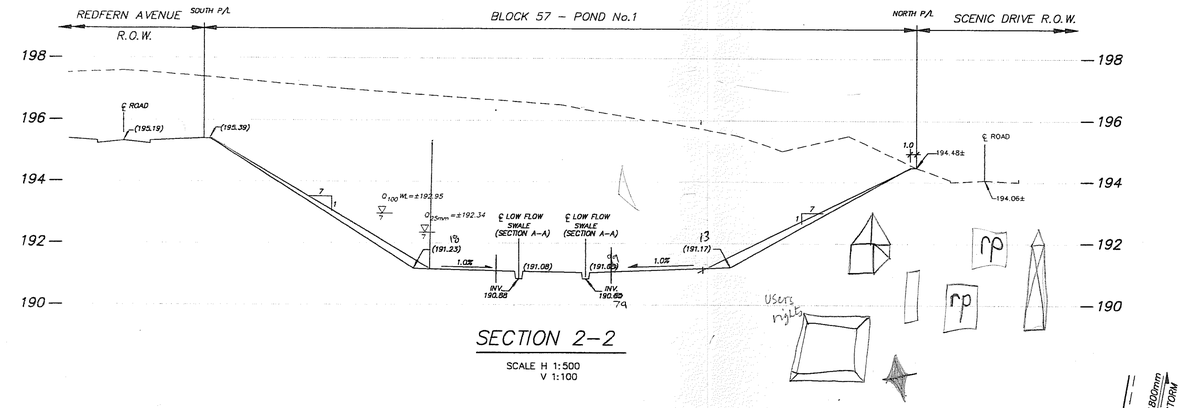
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LSP_1642



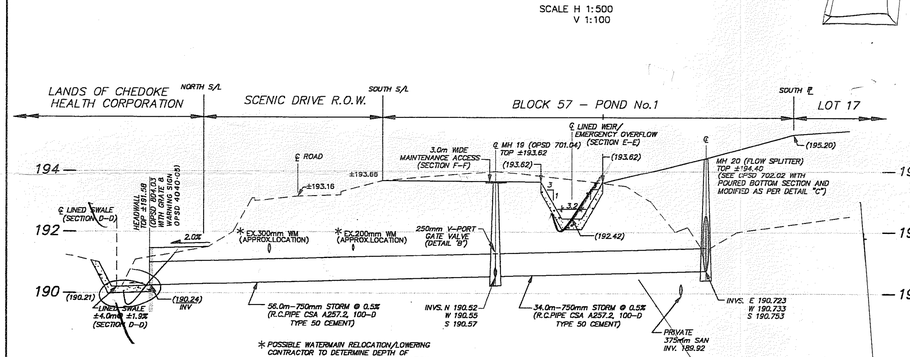
SECTION 1–1

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V 1:100



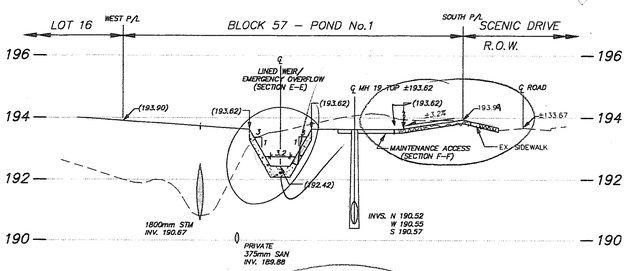
SECTION 2–2

SCALE H 1:500
V 1:100



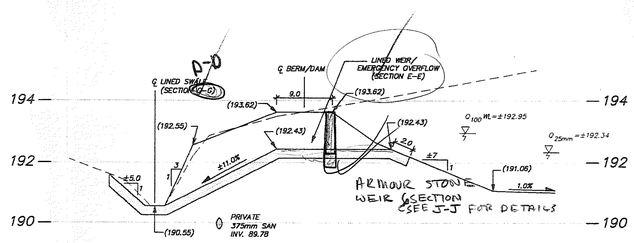
SECTION 3–3

SCALE H 1:500
V 1:100



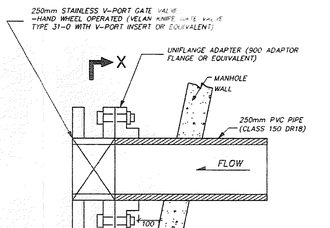
SECTION 4–4

SCALE H 1:500
V 1:100



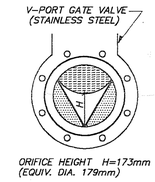
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V 1:100



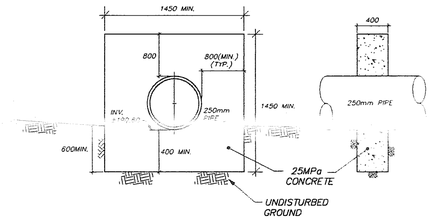
DETAIL 'B'

N.T.S.



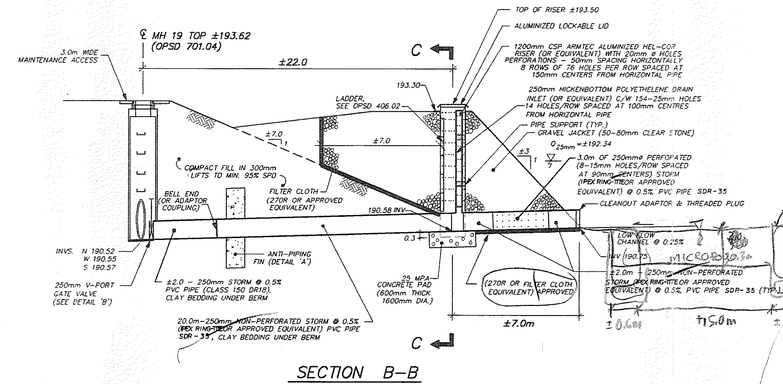
SECTION X–X

N.T.S.



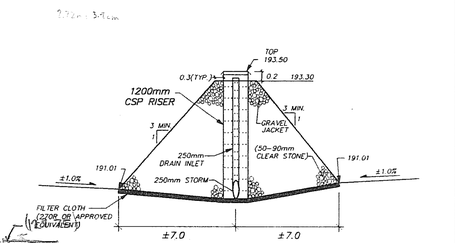
ANTI–PIPING FIN DETAIL 'A'

N.T.S.



SECTION B–B
QUALITY POND OUTLET

N.T.S.

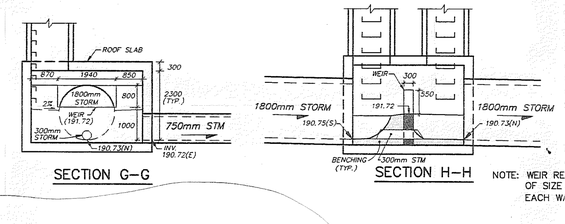


SECTION C–C

N.T.S.

SECTION A–A
LOW FLOW SWALE

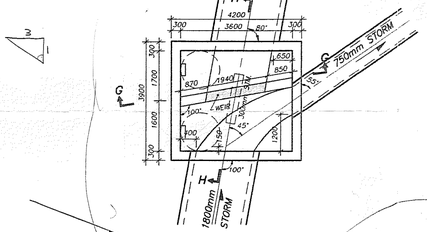
N.T.S.



SECTION G–G

SECTION H–H

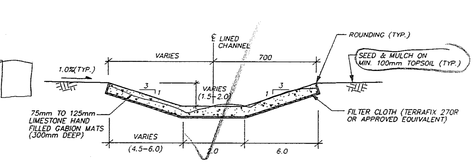
NOTE: WEIR REINFORCING TO CONSIST OF SIZE 15 REBAR @ 300mm O/C EACH WAY, EACH FACE



MH 20
DETAIL 'C'

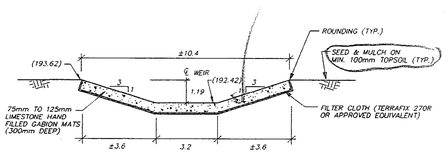
SCALE 1:100

NOTE: TO BE READ IN CONJUNCTION WITH OPSD 702.02 & MODIFIED AS DETAILED



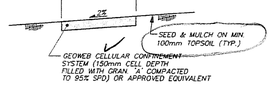
SECTION D–D
GABION LINED LOW FLOW SWALE

N.T.S.



SECTION E–E
GABION LINED WEIR/EMERGENCY OVERFLOW

N.T.S.



SECTION F–F
MAINTENANCE ACCESS

N.T.S.

ISSUED

JUL 23 1997

A. J. CLARKE & ASSOCIATES LTD.

BENCH MARK

City of Hamilton Index No. 824
Elevation: 198.288m
Description: On west side of Nursing School on Sanatorium Road South of Scenic Drive 18m north of entrance.

NOT ISSUED FOR CONSTRUCTION

A. J. Clarke and Associates Ltd.
ONTARIO LAND SURVEYORS • CONSULTING ENGINEERS

CITY OF HAMILTON	DESIGN: C. GIAMMARCO DRAWN: T. LAM/M. PIZZUTI
TIFFANY (PHASE 1)	SCALE:
BLOCK 57 EXTENDED DETENTION POND No.1 DETAILS	DATE: JUNE 1997 DWG: K–1485–2C

FILE: TIFFANY\ENG\SHT2.DWG Jun 24, 1997 – 10:54:39

Table 3 – Pre Development Peak Flows at existing outlets (m^3/s) West of existing Chedoke Creek				
Design Storm	**Area 110 Outlet 11**	**Area 120 Outlet 12**	**Area 130 Outlet 13**	**Total West**
2	0.010	0.076	0.062	0.148
5	0.027	0.117	0.096	0.229
10	0.042	0.145	0.121	0.291
25	0.061	0.181	0.155	0.378
50	0.077	0.210	0.182	0.448
100	0.096	0.241	0.201	0.513

Table 4 – Pre Development Peak Flows at existing outlets (m^3/s) East of existing Chedoke Creek				
Design Storm	**Area 210 Outlet 21**	**Area 220 Outlet 22**	**Area 230 [1] Outlet 23**	**Total East**
2	0.042	0.018	0.047	0.107
5	0.068	0.027	0.076	0.171
10	0.085	0.035	0.104	0.221
25	0.119	0.047	0.162	0.321
50	0.144	0.057	0.209	0.397
100	0.174	0.068	0.286	0.515

[1] Including External Area 240 from Columbia International College

Refer to Appendix 'E' for Pre Development Hydrological Analysis for areas west and east of the existing Chedoke Creek.

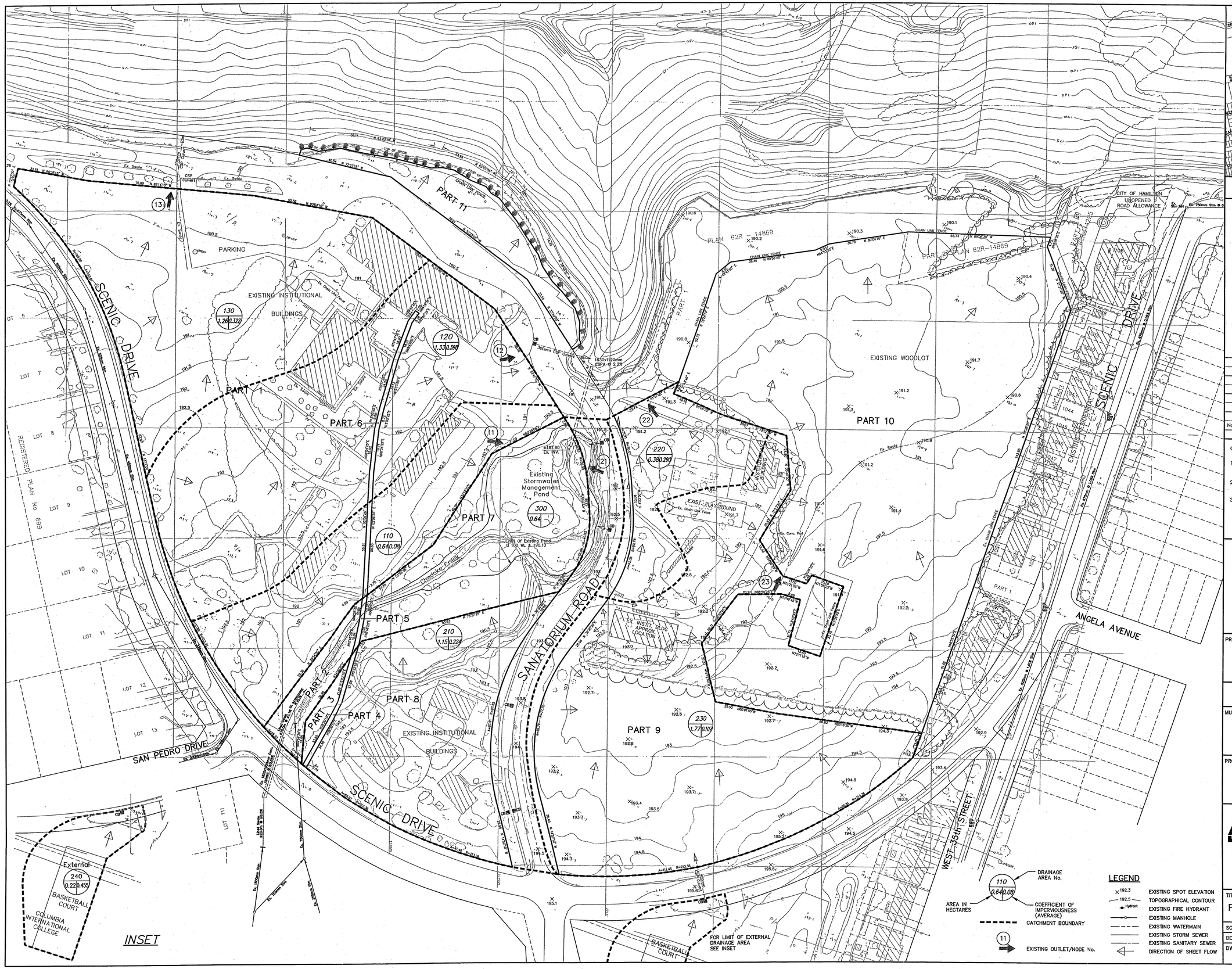
2.4.1.4 Post Development Conditions

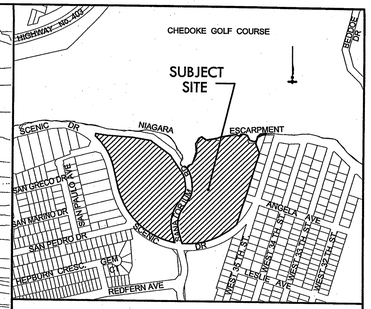
It is anticipated that under post development conditions the site will consist of residential medium density (e.g., townhouses) and high density (e.g., apartment buildings) development. An average coefficient of imperviousness for new development has been determined based on the City of Hamilton design standards, as follows:

Proposed Land Use	**Assumed Fraction**	**Coefficient of Imperviousness**
Medium Density Residential (60upha)	50%	0.60
High Density Residential (100upha)	50%	0.70
Average Coefficient of Imperviousness		**0.65**

Due to the existing Chedoke Creek traversing the site in the middle, and to be consistent with analysis conducted for existing conditions, the post development conditions have been analysed separately for west and east parts of the property. For post development Drainage Area No. 1,

CHEDOKE BROW LANDS





KEYPLAN N.T.S.

No.	Revision	Date	By

REVISIONS

GENERAL NOTES

1 TENDERERS SHALL SATISFY THEMSELVES AS TO THE NATURE OF THE GROUND AND BID ACCORDINGLY.

2 ALL ROCK LINE INDICATIONS SHOWN ON THE PLAN MUST BE VERIFIED BY THE CONTRACTOR.

3 CONTRACTOR SHALL VERIFY LOCATIONS AND INVERTS OF ALL EXISTING SANITARY AND STORM SEWERS AND WATERMAINS, PRIVATE SEWER DRAINS AND WATER SERVICES, GASMAINS, CABLE TV, HYDRO AND TELEPHONE DUCTS, ETC., AT START OF CONSTRUCTION.

N

B. Bednarska July 16, 2007 — ENGINEER

PROJECT OWNER:
DEANLEE MANAGEMENT

NOT ISSUED FOR CONSTRUCTION

MUNICIPALITY:
CITY OF HAMILTON

PROJECT NAME:
CHEDOKE BROWLANDS

A. J. Clarke and Associates Ltd.
SURVEYORS • PLANNERS • ENGINEERS
25 MAIN STREET WEST, SUITE 300
HAMILTON, ONTARIO L8P 1H1
Tel: 905 528-8761 Fax: 905 528-2289
email: ajc@ajclarke.com

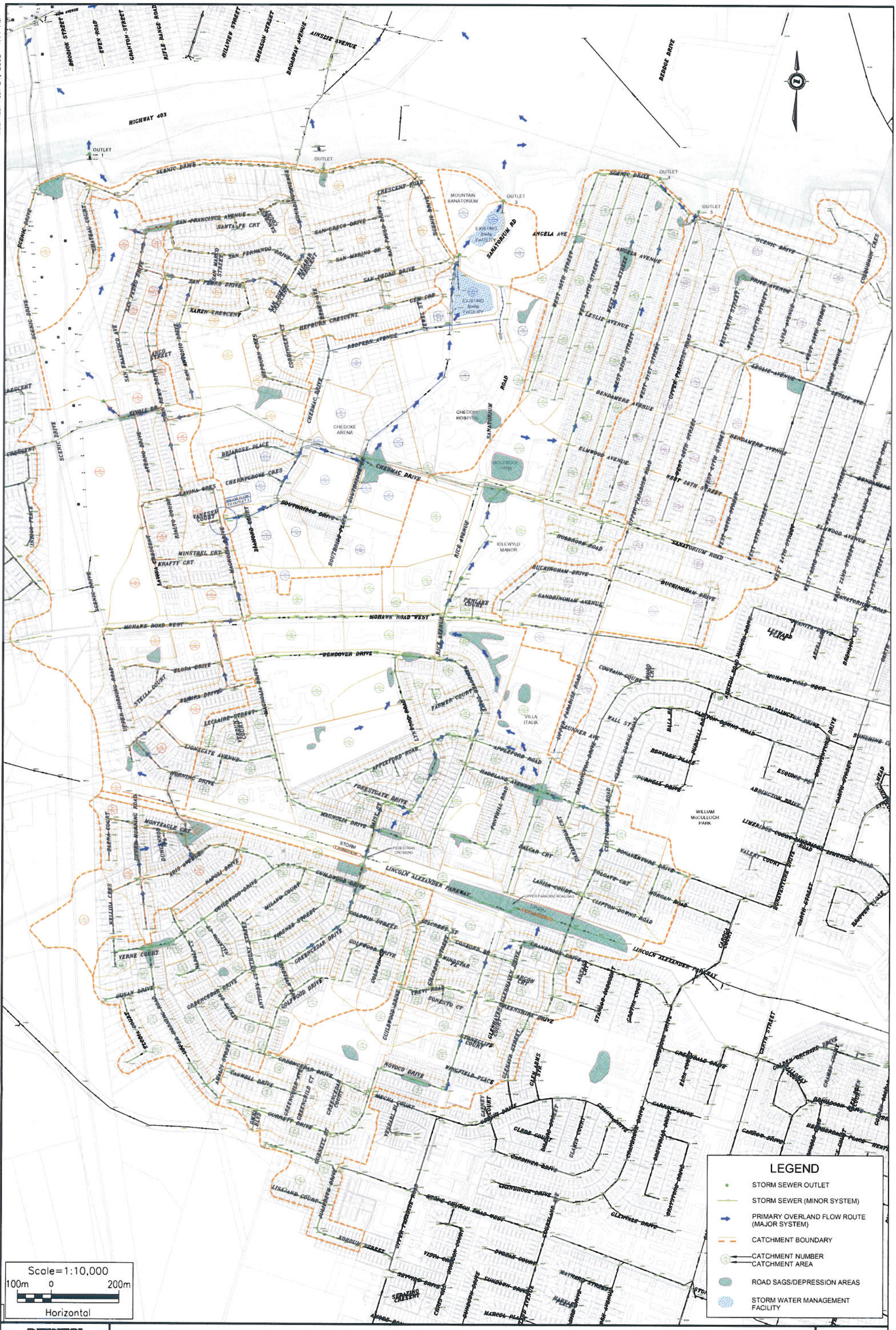
TITLE:
PRE-DEVELOPMENT DRAINAGE BOUNDARIES

SCALE: 1:750	DATE: JULY 2007
DESIGN: B. BEDNARSKA	DRAWN: M.P./A.G.
DWG: K-06-023	SHT: 2

CHEDOKE BROW LANDS

M:\Jobs\A to Z\Chedoke Browlands - Deanlee\engfiles\Dwgs\July 2007\PRE-DEVELOPMENT.dwg, 16/07/2007 11:54:25 AM

MODIFIED: Feb. 21, 2008 1:14 PM



DRAWING NAME: K:\DRAWINGS\6200-6399\6346-MOUNTVIEW\REPORT\PART B\K6346-EXHIBIT 4-DRAINAGE SYSTEM-MAJOR AND MINOR.DWG

MRC

DRAINAGE SYSTEM - MAJOR AND MINOR

MOUNTVIEW NEIGHBOURHOOD STORM DRAINAGE STUDY

EXHIBIT 4

MODIFIED: Feb. 21, 2008 1:23 PM



Scale=1:10,000
100m 0 200m
Horizontal

LEGEND

- STORM SEWER OUTLET
- STORM SEWER (MINOR SYSTEM)
- PRIMARY OVERLAND FLOW ROUTE (MAJOR SYSTEM)
- CATCHMENT BOUNDARY
- CATCHMENT NUMBER / CATCHMENT AREA
- ROAD SAGS/DEPRESSION AREAS
- STORM WATER MANAGEMENT FACILITY
- 1-12 GENERAL LOCATION OF FLOODING COMPLAINT(S)

MRC

FOCUS AREAS

MOUNTVIEW NEIGHBOURHOOD STORM DRAINAGE STUDY

EXHIBIT 5

DRAWING NAME: K:\DRAWINGS\6200-6399\6346-MOUNTVIEW\REPORT\PART B\K6346-EXHIBIT 5-FOCUS AREAS.DWG

MODIFIED: Feb. 21, 2008 1:55 PM

MOUNTAIN SANATORIUM

OUTLET 3

EXPAND SWM FACILITY TO PROVIDE ADDITIONAL STORAGE

POND 2
EXISTING SWM FACILITY

SANATORIUM RD

ANGELA AVE

SCENIC DRIVE

SAN PAUBO DRIVE

RETROFIT SWM FACILITY TO INCREASE STORAGE DEPTH

POND 1
EXISTING SWM FACILITY

GEM CRT

JEWEL STREET

REDFERN AVENUE

WEST 35TH STREET

WEST 34TH STREET

LESLIE AVENUE

"DRY" STORMWATER DETENTION FACILITY AND ENVIRONMENTAL ENHANCEMENT/ RECREATIONAL AREA

SANATORIUM ROAD

WEST 33RD STREET

MAJOR FLOW DIVERSION ALONG SANATORIUM ROAD (FUTURE ALIGNMENT)

CHEDOKE HOSPITAL

ELMWOOD AVENUE

MAINTAIN/REDEFINE DETENTION AREA

HOLBROOK PARK

CHEDMAC DRIVE

WEST 32ND STREET

"DRY" STORMWATER DETENTION FACILITY AND ENVIRONMENTAL ENHANCEMENT AREA

RICE AVENUE

IDLEWYLD MANOR

Scale=1:3,000
25m 0 50m
Horizontal

LEGEND

- STORM SEWER (MINOR SYSTEM)
- PRIMARY OVERLAND FLOW ROUTE (MAJOR SYSTEM)
- ROAD SAGS/DEPRESSION AREAS
- PROPOSED "DRY" STORMWATER DETENTION AREA
- PROPOSED MAJOR FLOW DIVERSION SEWER

MRC

SANATORIUM ROAD ALTERNATIVES

MOUNTVIEW NEIGHBOURHOOD STORM DRAINAGE STUDY

EXHIBIT 9

DRAWING NAME: K:\DRAWINGS\6200-6399\6346-MOUNTVIEW\REPORT\PART B\K6346-EXHIBIT 8+9-FLOOD CONTROL ALTERNATIVES.DWG

Technically Preferred Stormwater Alternative

The technically preferred alternative includes a combination of the following alternatives:

- Sanatorium Road/Chedmac Drive intersection:
  - New stormwater inlet at intersection
  - Road grading to direct overflows to Holbrook Park. Overflows to the park will be reduced in frequency and magnitude
  - Flow diversion structure in the existing storm sewer to direct some of the flow to the new Sanatorium Road storm sewer
- Drainage improvements at Holbrook Park:
  - Construct berm on north side of park to increase existing storage capacity and protect properties to the north from flooding
  - New controlled flow drainage inlet at north end of park and upgrade storm sewer connection to West 33rd Street sewer at Elmwood Avenue
  - Tile drainage in low-lying areas to improve drainage of the playing field
- New oversized storm sewer constructed along Sanatorium Road in conjunction with road improvements to convey the additional flow diverted from the Chedmac Road inlet
- Dry SMW detention facility.



Table 4.3
Summary of Visual OTTHYMO Modeling – Hurricane Hazel Event

Catchment ID	Drainage Area (ha)	Location	Hurricane Hazel/Regional Event		
			Peak Flow - Existing (m^3/s)	Peak Flow - Proposed 1 (m^3/s)	Peak Flow - Proposed 2 (m^3/s)
1001	193.3	Upstream Inflow Sanatorium Rd.	13.7	13.7	13.7
Diversion 1		Major System (To Holbrook Park)	1.2	3.2	3.2
		Minor System (To W. 33 or Diversion 2)	12.5	10.5	10.5
Diversion 2		Major System (To Tiffany Pond)	n/a	1.8	1.8
		Minor System (To W. 33)	n/a	8.7	8.7
6405	229.6	Outlet 4	17.0	14.7	14.8
1002	41.5	Upstream Inflow Redfern Ave.	5.3	5.3	5.3
104		Tiffany Pond Inflow	5.5	6.8	6.8
		Tiffany Pond Outflow	5.5	5.7	5.7
5004	Junction	All Catchments Tributary to Scenic Drive Pond	5.8	6.0	6.3
107	50.7	Scenic Drive Pond / Outlet 3	4.8	5.0	5.2

See Model Schematic – Appendix C for Catchment ID locations
Proposed 1 includes Sanatorium Road EA Works
Proposed 2 includes Sanatorium Road EA Works and Deanlee SWM Plan

4.2.4 Floodplain Impacts

In order to assess the floodplain impacts associated with the proposed works (i.e., Sanatorium Road EA and Deanlee Development), the existing Regional floodline was established based on the stage-storage-discharge relationship for the Scenic Drive Pond outlet (i.e., Outlet 3). This rating curve was extended using both the capacity of the modified outlet structure as described in the *SWM Report – Tiffany* and weir flow over Sanatorium Road (at Outlet 3). *Table 4.4* summarizes the Stage-Storage-Discharge relationship for the Scenic Drive Pond.

Table 4.4

Rating Curve for Scenic Drive Pond

Elevation (m)	Depth (m)	Discharge (m^3/s)	Storage (m^3)
187.9	0	0	0
188	0.1	0.07	4
188.5	0.6	1.04	70
189	1.1	2.5	190
189.5	1.6	3.38	420
190	2.1	4.1	1136
190.5	2.6	4.67	3034
191	3.1	5.22	6126

Under existing conditions the peak flow at this outlet for the Regional event, based on the Visual OTTHYMO modeling is 4.8 m^3/s. This flow corresponds to an elevation of approximately 190.6 m.

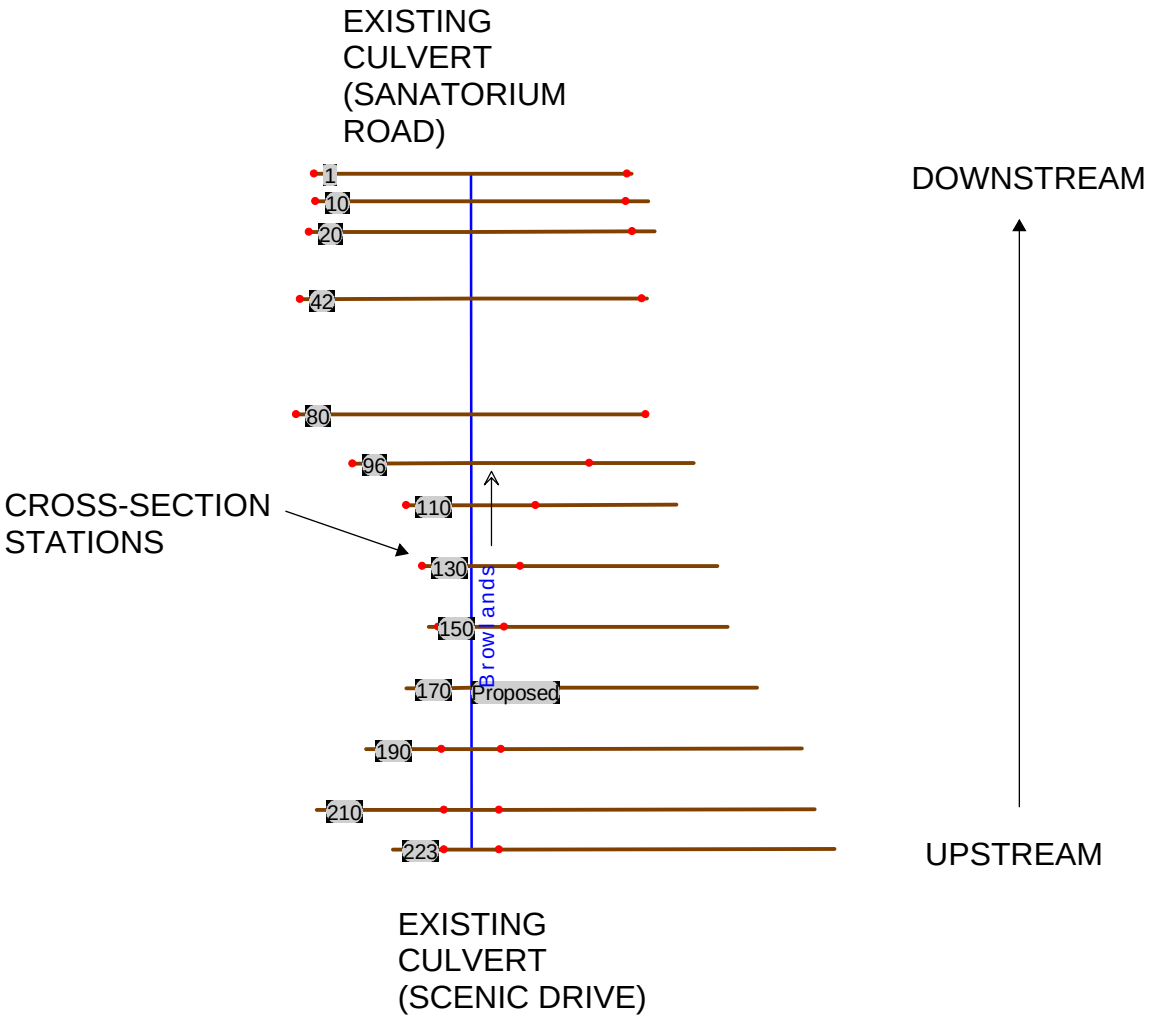
For proposed 1 conditions that include the preferred SWM solution from the Sanatorium Road Reconstruction and Stormwater Improvements Class EA, there is an increase in peak flows at Outlet 3 which in turn would result in an increase in flood elevation. This increase in peak flow translates to a floodplain elevation of approximately 190.75 m. The difference between the peak flow elevation under existing conditions of 190.6 m. and the peak flow floodplain elevation of approx. 190.75 m. under proposed 1 conditions is an increase of approx. 0.15 m +/-.

For proposed 2 conditions that include both the preferred SWM solution from the Sanatorium Road Reconstruction and Stormwater Improvements Class EA and the Deanlee Development SWM strategy, there is an increase in peak flows at Outlet 3 which in turn would result in an increase in flood elevation. This increase in peak flow translates to a floodplain elevation of approximately 191.0 m. The difference between the peak flow elevation under existing conditions of 190.6 m. and the peak flow floodplain elevation of approx. 191.0 m. under proposed 2 conditions is an increase of approx. 0.4 m +/-.

Proposed Conditions HEC-RAS Schematic

LEGEND:

- Watercourse
- Extent of cross-section
- Main channel boundary



None of the XS's are Geo-Referenced | Geo-Ref user entered XS | Geo-Ref interpolated XS | Non Geo-Ref user entered XS | Non Geo-Ref interpolated XS

LEGEND

Regional Floodline - Elevation = 190.60m - "Existing"

Center Line of Creek

N

EX. 37.5m - 1630x1120mm CSPA CULVERT @ 2.20%

EX. 27.8m - 1820x1000mm CONC. CULVERT @ 0.65%

PART 7 PLAN 62R-17555

PART 8 PLAN 62R-17555

PART 2 PLAN 62R-17555

PART 3 PLAN 62R-17555

PART 4 PLAN 62R-17555

PART 5 PLAN 62R-17555

PART 6 PLAN 62R-17555

CENTENNIAL ROAD

CENTENNIAL DRIVE

EXISTING REGIONAL FLOODLINE
16082 - Browlands
1:750

APPENDIX B:

Geotechnical Investigation (Landtek Ltd)

LANDTEK LIMITED
Consulting Engineers
205 Nebo Road, Unit 3
Hamilton, Ontario
Canada
L8W 2E1
Phone: 905-383-3733
Fax: 905-383-8433
engineering@landteklimited.com
www.landteklimited.com

Geotechnical Investigation
Proposed Residential Development
801, 820, 828, 855, 865, and 870 Scenic Drive
Hamilton, Ontario

Prepared for:

Valery (Chedoke Browlands) Developments Inc.
2140 King Street East
Hamilton, Ontario
L8K 1W6



Landtek File: 18211
November 9, 2018

EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	Valery (Chedoke Browlands) Developments Inc. is intending to develop the site identified as civil addresses 801, 820, 828, 855, 865, and 870 Scenic Drive in Hamilton, Ontario as residential properties. The proposed development is anticipated to comprise of approximately one hundred ten (110) medium density units and approximately twelve (12) apartment buildings in the order of 4 storeys to 12 storeys. The development is also anticipated to include access roads and paved parking areas. It is assumed that the residential properties will include a maximum of one level of basement.
Report Deliverables	The purposes of the Geotechnical Investigation were to confirm the subsurface conditions at the site and to provide design and construction recommendations with regards to building foundations, floor slabs, pavement structures, and subsurface drainage and utilities.

SITE DETAILS AND SETTING

UTM 17T Coordinates	588077, 4788454	**Site Area (approx.)**	10.7 hectares (107400 m^2)
Site Description	The project site is bound to the south and west by Scenic Drive, to the east by residential premises, and to the north by wooded areas. The site area is predominantly undeveloped, and is covered predominantly by unmaintained grasses, shrubs and woodlands, with the remainder of the site consisting of the existing Sanatorium Road, barren areas, and an abandoned, two-storey building located at the northeast section of the site.		
Geology	Topsoil was encountered at the ground surface. Underlying the surface material is fill material comprising predominantly of silt and clay, and native silty clay and clayey silt. Limestone bedrock was encountered at shallow depths across the site, underlying the fill material and native silty clay and clayey silt soil.		

ENGINEERING CONSIDERATIONS

Foundations	It is considered by Landtek that bearing conditions to support the proposed structures on concrete footings can be provided by the native soils.
Settlements	The general limiting of the total settlement of 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate for the native soils at the site. Where foundations are to be seated in bedrock, then the SLS condition will not govern their design.
Earthquake Considerations	The subject property is considered to be a 'B' Site Class
Floor Slabs	The subgrade conditions can adequately support the concrete floor slab on grade, provided that areas of softened soils are excavated to uncover, more competent soils underlying the soft sections.

CONSTRUCTION CONSIDERATIONS

Excavations	The subsurface soils to be encountered during excavation at the site are expected, in general, to behave as Type "3" soils. The limestone bedrock has strength characteristics that exceed Type 1 soils.
Dewatering	Details regarding the requirements of permits for dewatering during construction have been provided by Landtek under separate cover by the completion of a Hydrogeological Assessment at the site.
Material Reuse	The fill material, native soils and limestone bedrock encountered on site are considered from a geotechnical perspective as suitable for re-use as engineered backfill.
Pavements	The subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck to traverse the exposed subgrade, prior to the placement of pavement granular fill.

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APPENDICES

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Appendix B Symbols and Terms Used in the Report
Classification of Soils for Engineering Purposes
Appendix C Drawing 1-Site Plan Showing Borehole Locations
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Appendix D Environmental Analytical Report
Appendix E Geotechnical Laboratory Testing Results
Appendix F Site Photographs

1.0 INTRODUCTION

Landtek Limited (herein "*Landtek*") is pleased to submit the geotechnical investigation report for the proposed residential development at lands located at 801, 820, 828, 855, 865, and 870 Scenic Drive in Hamilton, Ontario. Authorization to proceed with the work was received from Mr. Paul Valeri on behalf of Valery (Chedoke Browlands) Developments Inc. in June of 2018.

Preliminary layout drawings provided to Landtek by UrbanSolutions Planning & Land Development Consultants Inc. (herein *"UrbanSolutions*") indicate that the proposed development is to comprise of approximately one hundred ten (110) medium density units and approximately twelve (12) apartment buildings in the order of 4 storeys to 12 storeys. The development is also anticipated to include access roads, paved parking areas, landscaped areas, a stormwater management pond, and full site servicing. It is assumed that the residential properties will include a maximum of one level of basement.

The primary objectives of this investigation were:

- To confirm the subsurface soil and groundwater conditions for foundation design and construction;
- Provide design and construction recommendations with regards to building foundations, floor slabs, pavement structures, and subsurface drainage and utilities; and,
- Assess the characteristics of the soils to be excavated and their suitability for reuse on site.

This report has been prepared for the Client, the nominated engineers, designers, and project managers pertaining to the proposed residential development at the site identified as 801, 820, 828, 855, 865, and 870 Scenic Drive in Hamilton, Ontario. Further dissemination of this report is not permitted without Landtek's prior written approval. Additional details of the limitations of this report are presented in Appendix A.

2.0 PROJECT AND SITE SETTING

2.1 Site Location and Description

The site is located in Hamilton, Ontario, and is centered at approximate grid reference 588077, 4788454 (UTM 17T coordinates). The approximate Geodetic elevation of the ground surface at the borehole locations ranges from 190.5 m to 194.7 m. The site area is approximately 107400 m^2 (10.74 hectares). The site location is presented in Figure 1.



***Figure 1**: Site location and surrounding area*

The project site is bound to the south and west by Scenic Drive, to the east by residential premises, and to the north by wooded areas. The site area is predominantly undeveloped, and is covered predominantly by unmaintained grasses, shrubs and woodlands, with the remainder of the site consisting of the existing Sanatorium Road, barren areas, and an abandoned, two-storey building located at the northeast section of the site.

2.2 Published Geology

Based on previous geotechnical experience for the area and a review of an existing geological publication for the site area, Ontario Geological Survey (OGS) Map P.993: "*Quaternary Geology of the Grimsby Area*", the site is underlain by glaciolacustrine deposits of clay and silt.

According to the OGS Map 2343 "*Paleozoic Geology of the Grimsby Area*", the superficial geology is underlain by blue-grey dolostone/limestone bedrock of the Lockport Formation. Previous geotechnical investigation undertaken in the vicinity of the site indicates that

limestone/dolostone bedrock sub-crops the site at a geodetic elevation of approximately 189 m. With a surface elevation at the site ranging from approximately 190.5 m to 194.7 m, the anticipated depth to bedrock is approximately 1.5 m to 5.7 m below existing ground level.

2.3 Hydrogeology and Hydrology

The hydrostratigraphy (i.e. the vertical sequence and horizontal extent of aquifers and aquitards) in the overburden and shallow bedrock generally follows the geologic layering. The more massive unfractured limestone and dolostone bedrock generally act as aquitards while the more permeable and fractured limestone and dolostone generally behave as aquifers.

Groundwater typically follows the general path of the surface water courses and flow to low lying and major surface water features from areas of higher gradients to areas of relative lower gradients. There appears to be a water divide at the Site, with groundwater flow interpreted to be northeast east of Sanatorium Road; and northwest west of Sanatorium Road.

3.0 FIELDWORK AND METHODOLOGY

Fieldwork undertaken at the site by Landtek included clearance of underground services, borehole drilling, rock coring and soil sampling, and field supervision. The geotechnical investigation was undertaken in conjunction with a hydrogeological investigation completed by Landtek for the site. A total of fifteen boreholes (boreholes BH1 to BH16) were drilled between June 25 and July 4, 2018. Boreholes BH10 and BH11 were not drilled due to access restrictions. All boreholes were logged using those standard symbols and terms defined in Appendix B. The borehole location plan, Drawing 1, and the borehole logs are provided in Appendix C.

The boreholes were drilled using a CME-75 track-mounted drilling rig equipped with continuous flight, hollow stem augers, and were advanced to practical auger refusal on bedrock, at depths of between approximately 0.7 m and 5.2 m below existing ground level. Boreholes BH2, BH6, BH7S, BH7D, BH12 and BH15 were further extended into the bedrock by either coring or air rotary drilling to depths of between 9.1 m and 15.5 m below existing ground level, and were completed as monitoring wells. Boreholes BH7S and BH7D were completed as nested wells.

Standard Penetration Tests (SPT's) and split spoon samples were taken during drilling at selected depths. Full time supervision of drilling, soil sampling, and rock coring operations was carried out by a representative of Landtek.

The monitoring wells consisted of new/sealed 50 mm polyvinyl chloride (PVC) screen with No.10 slots threaded onto a matching riser. The screens and riser were pre-threaded including o-ring seals such that no glues or solvents were used to connect the pipe sections. The annular space between the PVC well and the borehole was backfilled to approximately 0.6 m above the top of the screen section with sand pack, and then with bentonite to the existing ground surface. A J-Plug lockable air-tight cap was installed on all the risers. The borehole logs in Appendix C present the monitoring well installation details and the results of subsequent water monitoring visit undertaken on July 25 and August 31, 2018.

The soil samples and rock cores were then transported to Landtek`s in-house, Canadian Council of Independent Laboratories (CCIL) certified laboratory and visually examined to determine their classification. Moisture contents were carried out on all samples. Selected, composite samples were submitted to ALS Environmental to be analyzed for O. Reg. 153 Metal and Inorganic parameters to assist with determining materials reuse options, and Soil Corrosivity to assist with any protective requirements for buried concrete and metal infrastructure. Atterberg limits determination tests were conducted on selected soil samples. Ten rock core samples were submitted to Golder Associates for strength parameter (Uniaxial Compressive Strength and Point Load Index) testing.

The approximate geodetic elevations at the boreholes were established by Landtek relative to site measurements and with reference to the site topographical plan prepared by A.J. Clarke and Associates Ltd. and dated February 16, 2016.

4.0 SUBSURFACE CONDITIONS

4.1 Overview

The geology encountered across the site was as anticipated from Landtek's review of the published geological information for the site, as detailed in Section 2.2.

The borehole information is generally consistent with the geological data, and the native soil comprises a veneer of clay and silt deposits. Limestone bedrock was encountered underlying the overburden soils at depth of approximately 0.7 m and 5.2 m below existing ground level. The detailed borehole logs are presented in Appendix C, with the subsurface conditions encountered summarized in the following Table 1. Subsurface conditions encountered by the boreholes are discussed further in the sections following.

Table 1: Summary of Borehole Conditions Encountered

BH ID	Ground Elevation[1]	Surface Material	Superficial Soils Depth and (Description)		Bedrock Contact Depth
		Topsoil	Fill	Native Soil	
BH1	190.5 m	300 mm	-	0.3 m – 0.7 m (clayey silt)	0.7 m
BH/MW2	190.9 m	300 mm	-	0.3 m – 0.9 m (clayey silt)	0.9 m
BH3	190.7 m	-	0.0 m – 0.7 m (clayey silt)	0.7 m – 1.2 m (clayey silt)	1.2 m
BH4	192.6 m	300 mm	-	0.3 m – 2.5 m (silty clay/clayey silt)	2.5 m
BH5	191.5 m	250 mm	-	0.3 m – 0.6 m (silty clay)	0.6 m
BH/MW6	192.4 m	300 mm	-	0.3 m – 3.7 m (silty clay/clayey silt)	3.7 m
BH/MW7	193.5 m	300 mm	0.3 m – 2.3 m (silt)	2.3 m – 5.2 m (silty clay/clayey silt)	5.2 m
BH8	194.2 m	250 mm	-	0.3 m – 5.2 m (clayey silt/silty clay)	5.2 m
BH9	192.4 m	250 mm	-	0.3 m – 2.4 m (clayey silt)	2.4 m
BH/MW12	192.5 m	300 mm	-	0.3 m – 2.0 m (clayey silt)	2.0 m
BH13	192.2 m	250 mm	-	0.3 m – 2.3 m (clayey silt)	2.3 m
BH14	194.7 m	300 mm	-	0.3 m – 2.9 m (clayey silt)	2.9 m
BH/MW15	194.2 m	300 mm	0.3 m – 0.9 m (clayey silt)	0.9 m – 4.0 m (silty clay/clayey silt)	4.0 m
BH16	194.0 m	300 mm	0.3 m – 0.9 m (clayey silt)	0.9 m – 4.6 m (silty clay/clayey silt)	4.6 m

Notes:
1. Geodetic elevations are considered approximate.

4.2 Surface Materials

An approximately 250 mm to 300 mm thick layer of topsoil was encountered from ground surface in all boreholes, except borehole BH3.

4.3 Fill

Fill material comprising predominantly of silt and clay was encountered in boreholes BH3, BH/MW7, BH/MW15 and BH16, underlying the topsoil and at the ground surface (borehole BH3), and extends to depths of between approximately 0.7 m (borehole BH3) and 2.3 m (borehole BH/MW7) below existing ground level. The fill contains traces of organics, and is brown and occasionally grey in colour. Moisture contents in the fill range between 17 % and 32 %.

4.4 Silty Clay and Clayey Silt

Native silty clay and clayey silt deposits were encountered in all boreholes underlying the topsoil and fill material, and extend to depths of approximately 0.7 m (borehole BH1) and 5.2 m (borehole BH/MW7) below existing ground level. The silty clay and clayey silt is primarily brown and grey in colour, and is of a low to medium plasticity.

SPT “N” values for the silty clay and clayey silt were recorded ranging from 10 to 45 indicating the silty clay and clayey silt to be of a stiff to hard, but generally very stiff consistency. Moisture content testing results were recorded between 14 % and 29 %.

4.5 Bedrock

The borehole information indicates that bedrock contact, based on rock coring and auger penetration refusal, is at approximate Geodetic elevations ranging from 191.8 m (borehole BH14) to 188.3 m (borehole BH7).

The bedrock comprises a sedimentary limestone of the Lockport Formation. The limestone is grey and dark grey in colour, very finely crystalline, slightly porous and contains interbeds of shale. Strength testing performed on intact rock core specimens indicate rock strength of the limestone in the order 74.8 MPa to 177 MPa, indicating that the bedrock can be classified as having a strength grade of R4 to R5. This is representative of rock that is “strong” to “very strong” according to the Canadian Foundation Engineering Manual.

The Rock Quality Designation (RQD) values of the bedrock were in the order of 11 % to 38 % in the upper 3.5 m, indicating that the bedrock is of “very poor” to “poor” quality, and at depths of beneath 3.5 m, the RQD values were in the order of 70 % to 100 % indicating the bedrock to be of a “good” to “excellent” quality at depth.

4.6 Groundwater

Monitoring wells were installed in boreholes BH2, BH6, BH7S, BH7D, BH12 and BH15 to monitor the groundwater conditions. Groundwater level measurements were taken on July 25 and August 31, 2018, by which time the water levels were considered stable. Water level readings recorded in the monitoring wells during the visits are summarized in Table 2.

It should be noted that these water levels are not considered to reflect the long term stabilized water table. Groundwater conditions and surface water flow conditions are expected to vary according to the time of the year and seasonal precipitation levels. Water seepage is also expected from soil fissures above the water table.

Table 2: Summary of Water Level Measurements

BH ID	Borehole Elevation (m)	Measured Water Levels			
		July 25, 2018		August 31, 2018	
		Depth (mbgl*)	Elevation (m)	Depth (mbgl*)	Elevation (m)
BH/MW2	190.9	5.5	185.4	5.5	185.4
BH/MW6	192.4	4.5	187.9	4.3	188.1
BH/MW7S	193.5	3.0	190.5	3.3	190.2
BH/MW7D	193.5	2.9	190.6	3.3	190.2
BH/MW12	192.5	4.5	188.0	4.5	188.0
BH/MW15	194.2	3.4	190.8	4.6	189.6

* mbgl meters below ground level

5.0 FOUNDATION DESIGN CONSIDERATIONS

5.1 Foundations in Native Soils

The fill encountered in the boreholes is not considered suitable as a bearing stratum due to the high risk of unacceptable settlements. This given, it is considered by Landtek that bearing conditions to support any shallow seated concrete footings can be provided by the native clayey silt to silty clay soils and limestone bedrock. It is assumed that the residential properties will include either a ground-bearing floor slab or a maximum of one level of basement.

Table 3 summarizes the recommended geotechnical reactions at the Serviceability Limit State (herein "*SLS*") and factored geotechnical resistances at the Ultimate Limit State (herein "*ULS*") for the native soils and limestone bedrock. Where foundations are to be seated in bedrock, then the SLS condition will not govern their design as the stress required to induce the typical 25 mm settlement criteria at the SLS is anticipated to exceed the ULS. It should be noted that the determined design parameters have been determined by Landtek for the design stage only.

Where the bearing levels of the footings are at different design elevations, the footing base levels should be stepped along a line of 10H:7V, drawn upwards from the lowest footing, to avoid overlapping stresses.

Subsurface conditions can vary over relatively short distances and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. Therefore, a Geotechnical Engineer should be engaged during construction to examine the exposed sub-soil and bedrock quality and condition, and confirm the subsurface conditions are consistent with design assumptions. This is in compliance with field review requirements in the National Building Code, Volume 1, Clause 4.2.2.3.

Table 3: Recommended Limit State Foundation Design Values

Founding Elevation Range		Founding Stratum	Foundation Design Value	
Depth Range	Geodetic Elevation[5]		SLS [1,2]	ULS [3,4]
1.2 m to 5.2 m	192.3 m to 188.3 m	Clayey Silt/ Silty Clay	150 kPa	225 kPa
1.2 m to 5.2 m	192.3 m to 188.3 m	Limestone	-	2.5 MPa

Notes:
1. The National Building Code general safety criterion for the serviceability limit states is: SLS resistance ≥ effect of service loads.
2. Recommended SLS bearing values conform to Estimated Values based on soil types given in Tables K-8 and K-9 of the National Building Codes User's Guide.
3. The ULS resistance factor for shallow foundations is 0.5, as given in Table K-1 of the National Building Code User's Guide.
4. The National Building Code general safety criterion for the ultimate limit states is: factored ULS resistance ≥ effect of factored loads.
5. Geodetic elevations are considered approximate.

5.2 Frost Susceptibility

The native silt and clay soils encountered at shallow depths across the site are considered sensitive to water and frost, and their physical and mechanical properties are dependent on in-situ moisture content. As such, the founding soils at the site are considered to have a moderate to high frost susceptibility, being classified as Frost Group "*F4*" (Table 13.1 of the "*Canadian Foundation Engineering Manual*", 4th Edition). However, the identified depths for foundations, as given in Section 6.1 are considered to be below the maximum depth for frost penetration of 1.2 m in the Hamilton area.

Should any re-grading be required as part of the proposed development and adjacent to the new structures, it will be important to ensure that the associated exterior footings will have a minimum of 1.2 m of soil cover, or equivalent suitable insulation, for frost protection.

Concerns regarding frost protection to footings are more directed towards those seated within soils. Foundations in the limestone bedrock are generally deemed exempt from any frost protection requirements. This given however, consideration should be given to the use of non-frost susceptible materials as backfill for foundation wall excavations and the installation of foundation drainage in order to minimize the risk of adfreezing.

5.3 Settlement Considerations

Based on the outline information provided for the nature of the proposed redevelopment of the site, it is anticipated that the loads to be applied to the ground by any such structure will be generally moderate intensity. As such, associated settlements within the native soils are not expected to be large. Therefore, the general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate. Settlements for foundations seated within bedrock are to be deemed negligible (i.e. less than 15 mm).

5.4 Seismic Design Considerations

Based on the soils conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (herein "*OBC*"), the site is considered to be a 'B' Site Class. The acceleration and velocity-based site coefficients, F_a and F_v, should be determined from Tables 4.1.8.4.B. and 4.1.8.4.C. respectively of the OBC for the above recommended Site Class. The seismic design data given in Table 1.2 of Supplementary Standard SB-1 in Volume 2 of the OBC, for selected Municipal locations, should be used to complete the seismic analysis.

5.5 Existing Building Demolition

It is understood that the existing building located adjacent to the existing woodlot will be demolished prior to construction of the proposed residential structures. For the purposes of this report, it has been assumed that the existing structure and all associated substructures will be removed in full prior to development.

Should there be a need to fill excavations created by the demolition of the existing structure with engineered fill or unshrinkable backfill prior to commencing the proposed development, Landtek should be contacted to determine the most appropriate placement requirements of the fill material.

5.6 Waterproofing Considerations

The subsurface areas should be damp proofed and comply with the OBC requirements. As a minimum it is recommended that the damp proofing system include a Delta Drainage Board or MiraDrain 2000 series product, or an approved alternative, along with an asphalt based spray-on wall coating.

It is recommended that all subsurface areas including walls and floor slabs are appropriately waterproofed below any stabilized groundwater levels plus the required buffer zone (nominally 1.0 m to 1.5 m above the stabilized groundwater level.

6.0 FLOOR SLAB AND PERIMETER DRAINAGE CONSIDERATIONS

Based on the borehole soil conditions and preliminary design information provided to Landtek, it should be possible to construct the floor slab level using slab-on-grade methods. The subgrade support conditions are anticipated to be native clayey to silty soils and limestone bedrock, which should provide competent conditions for placing the vapour barrier material. However, after the subgrade has been prepared to the underfloor design elevation it is recommended that the area be assessed by Landtek to determine if there is a need for any remedial work.

Any required grade raising below floor slabs or localized, 'soft-spot' remediation to the subgrade should be completed using select subgrade material placed per Sections 9.0 and 11.0 of this report. The select subgrade materials are to be compacted to a recommended target compaction of 100 % Standard Proctor Maximum Dry Density (herein *"SPMDD"),* with no individual test below 98 % SPMDD.

It is recommended that a minimum 150 mm layer of clear 19 mm crushed quarried stone be used as the vapour barrier under the floor slab. The vapour barrier stone should meet the requirements of Ontario Provincial Standard Specifications (herein "*OPSS*") 1004 for 19 mm Type II clear stone. If a graded crushed stone is substituted for clear stone, the material should be limited to a maximum of 5 % fines (passing the 0.075 mm sieve).

The floor slab thickness should meet the specifications of the project based on anticipated floor loadings. The finished exterior ground surface should be sloped away from the buildings at a grade in the order of 2 %.

The concrete properties should meet the requirements of OPSS 1350. Contraction and isolation jointing practices should be in accordance with current Portland Cement Association recommendations, as given in the engineering bulletin "*Concrete Floors on Ground*", second edition, by R. E. Spears, and W. C. Panarese.

Perimeter drainage should be provided around all subsurface floor areas where water may accumulate. Underfloor drains may be required depending on excavation and groundwater seepage conditions. The drainage system should comply with the current OBC and associated amendments.

7.0 EARTH PRESSURE CONSIDERATIONS ON SUBSURFACE WALLS

The earth pressure, p, acting on subsurface walls at any depth, h, in metres below the ground surface assumes an equivalent triangular fluid pressure distribution and may be calculated using expression (1) below. It is assumed that granular material is used as backfill. Allowances for pressure due to compaction operations should be included in the earth pressure determinations and a value of 12 kPa is applicable for a vibratory compactor and granular material.

If the structure retaining soil can move slightly, the active earth pressure case can be used in determining the lateral earth pressure. For restrained structures and no yielding an “at rest” earth pressure condition should be used. The determination of the earth pressures should be based on the following expression:

$$p = K (\delta h + q) \qquad (1)$$

where:

p = the pressure in kPa acting against any subsurface wall at depth, h, in metres (feet) below the ground surface;

K = the at rest earth pressure coefficient considered appropriate for subsurface walls; OPSS 1010 Granular B Type 1 (pit-run sand and gravel) material has an effective angle of friction estimated to be 32° with a corresponding at rest earth pressure coefficient, K_o, of 0.45;

δ = the moist bulk unit weight of the retained backfill; 21.5 kN/m^3

and,

q = the value for any adjacent surcharge in kPa which may be acting close to the wall

h = the depth, in m, at which the pressure is calculated

Granular B backfill should meet OPSS 1010 Type I or Type II material specifications. The granular fill should be compacted to a minimum of 97 % SPMDD, or to the levels and backfilling procedures specified.

8.0 SUBSURFACE CONCRETE

8.1 Soil Corrosivity

A total of four selected soil samples were submitted to ALS Environmental for analysis of PH, soil conductivity and redox potential, and concentrations of sulphides, sulphates, and chlorides.

The American National Standards Institute (ANSI)/ American Water Works Association (AWWA) 10-point scoring method was used to determine the soil corrosivity potential. For each given soil sample, points were assigned to the different parameters to evaluate their contribution towards the corrosivity of soil. The test results are provided in Appendix D and are summarized in Table 4 below.

Table 4: Results of Soil Corrosivity Potential

Sample ID	Composite Sample Depth	Parameters Analysed	Measured Value	ANSI/AWWA Point Rating	Total ANSI/AWWA Points
BH4-SS1 and SS2	0.7 m – 2.0 m	Sulphide (%)	0.70	0.0	1.0
		pH (ph units)	7.63	0.0	
		Resistivity (ohm.cm)	7520	0.0	
		Redox Potential (mV)	232	0.0	
		Moisture (%)	19.0	1.0	
BH6-SS3 and SS4	2.3 m – 3.5 m	Sulphide (%)	1.38	0.0	1.0
		pH (ph units)	7.66	0.0	
		Resistivity (ohm.cm)	7690	0.0	
		Redox Potential (mV)	233	0.0	
		Moisture (%)	21.9	1.0	
BH9-SS1 and SS2	0.7 m – 2.0 m	Sulphide (%)	1.26	0.0	1.0
		pH (ph units)	7.62	0.0	
		Resistivity (ohm.cm)	7870	0.0	
		Redox Potential (mV)	233	0.0	
		Moisture (%)	15.4	1.0	
BH16-SS2 and SS3	1.5 m – 2.7 m	Sulphide (%)	0.32	0.0	9.0
		pH (ph units)	7.68	0.0	
		Resistivity (ohm.cm)	774	8.0	
		Redox Potential (mV)	238	0.0	
		Moisture (%)	21.0	1.0	

Corrosion protection for buried metallic improvements or steel are recommended, when a score of 10 points or greater is reported. Based on the total ANSI/AWWA values of ranging from 1.0 to 9.0, it is considered that the soils at the site have no corrosion potential towards buried metallic improvements such as cast iron alloy pipes. It should be noted that the analytical results only provide an indication of the potential for corrosion.

The contribution of chloride ions to soil corrosivity towards buried metallic improvements or steel structures is very significant. According to the Corrosion Guidelines (Caltrans, January 2015, version 2.1), a site is considered corrosive if, "*chloride concentration is 500 ppm or greater, sulphate concentration is 2,000 ppm or greater, or the pH is 5.5 or less*. "

In addition, the Canadian Standards Association (CSA) A23.1-14 "*Concrete materials and methods of concrete construction*", Table 3, "*Additional requirements for concrete subjected to sulphate attack*", states that design requirements for sulphate resistant concrete are only necessary when the water soluble sulphate content of the soil in which the concrete is to be embedded is greater than 0.1 % (1,000 mg/kg).

The representative soil samples at the site contain sulphate and chloride ion concentrations of up to 488 mg/kg and 515 µg/g respectively. This given, the soil environment within the native soils should be considered "*highly*" corrosive towards reinforced concrete.

8.2 Concrete Class Considerations

The requirements for subsurface concrete subject to a sulphate and chloride environment are presented in Canadian Standards Association specification, CSA A23.1-14 *"Concrete Materials and Methods of Concrete Construction, Tables 1-4"*. The results of chemical tests indicate that the soils have a high chloride concentration and generally a mild sulphate environment. It is recommended that subsurface concrete at the site have the following characteristics for a C-1 exposure class:

- minimum 56-day compressive strength = 35 MPa;
- maximum water to cementing materials ratio = 0.40;
- chloride ion penetrability requirement = < 1500 coulombs within 91 days)
- cementing materials; GU (general use hydraulic cement) or GUb (blended general use)
- air content; as per CSA A23.1-14 Table 4, air content category 1 (freeze-thaw environment)

The concrete should be placed without segregation and should be consolidated to achieve a uniform dense mass.

8.3 Methods for Specifying Concrete

Alternative methods of specifying concrete for a project are outlined in CSA A23.1-14 and allow for "*Performance*" or "*Prescription*" based methods. Each method attaches different levels of responsibility to the owner, the contractor, and the concrete supplier. The pros and cons of each method should be examined prior to completion of the specifications for the project.

9.0 EXCAVATION AND BACKFILL CONSIDERATIONS

9.1 General Excavation Considerations

All temporary excavations and unbraced side slopes in the soils should conform to standards set out in the Occupational Health and Safety Act, Ontario Regulation 213/91 "*Construction Projects*" (herein "*OHSA*"). The subsurface soils to be encountered during excavation at the site are expected, in general, to behave as Type "3" materials according to the OHSA classification in Part III. Type 3 materials are characteristic of the generally stiff native clayey to silty soils and fill material. The limestone bedrock has strength characteristics that exceed Type 1 soils.

Based on the findings of the investigation, it is considered that excavation of the overburden soils at the site can be carried out using a conventional backhoe excavator. Excavation into the dolostone bedrock will require the use of more unconventional, heavier excavation equipment such as a rock chisel/breaker or a rock-ripping (tiger teeth-fitted) excavator bucket, particularly as the competence of the limestone bedrock tends to improve very quickly with depth.

Excavation slopes in the overburden soils are expected to be stable for the construction period at slopes of about 45° degrees to the horizontal. The limestone bedrock is expected to remain relatively stable at near vertical slopes for short periods of time. It is recommended that the limestone bedrock excavation slopes be scaled of loose rock pieces and overhang, and that frequent inspections of exposed rock excavation faces are be carried out by a geotechnical engineer to ensure their stability is appropriately maintained.

Consideration should be given to existing service trenches and backfill that may be present directly behind cut slopes within the native soils that may appear to be stable on first excavation. In these circumstances, slopes can suddenly slough or collapse due to the effects of the adjacent backfill. Consequently, for excavation conditions that cannot satisfy the OHSA requirements for unbraced 1H:1V side slopes, a trench box system should be used, or temporary shoring should be installed to maintain safe working conditions. This may be more applicable to service trench excavations, though may also apply to basement excavations etc., particularly where in close proximity to new road pavements or associated infrastructure.

Details regarding the requirements of permits for dewatering during construction have been provided by Landtek under separate cover by the completion of a Hydrogeological Assessment at the site. The Hydrogeological Assessment report, reference 18210 and dated October 2, 2018, should be read in conjunction with this report.

It should be noted that the design of a temporary shoring system, should one be required, is the responsibility of the Contractor. Therefore, a specialist shoring contractor should be consulted to provide the most appropriate shoring type method and associated installation procedures. In any event, the shoring design should be based on the procedures outlined in the latest edition of the "*Canadian Foundation Engineering Manual*". It is also recommended that lateral and vertical movement of the shoring system be monitored during construction to ensure that movements are within the acceptable range.

Excavations for new foundations should satisfy the criteria given in Figure 2 to avoid overlapping stresses and minimize the risk of undermining existing adjacent foundations/utilities and/or triggering additional settlements of the existing foundations/utilities due to soil disturbance. Shoring may be required to avoid undermining existing adjacent foundations during construction or provide safe working conditions.

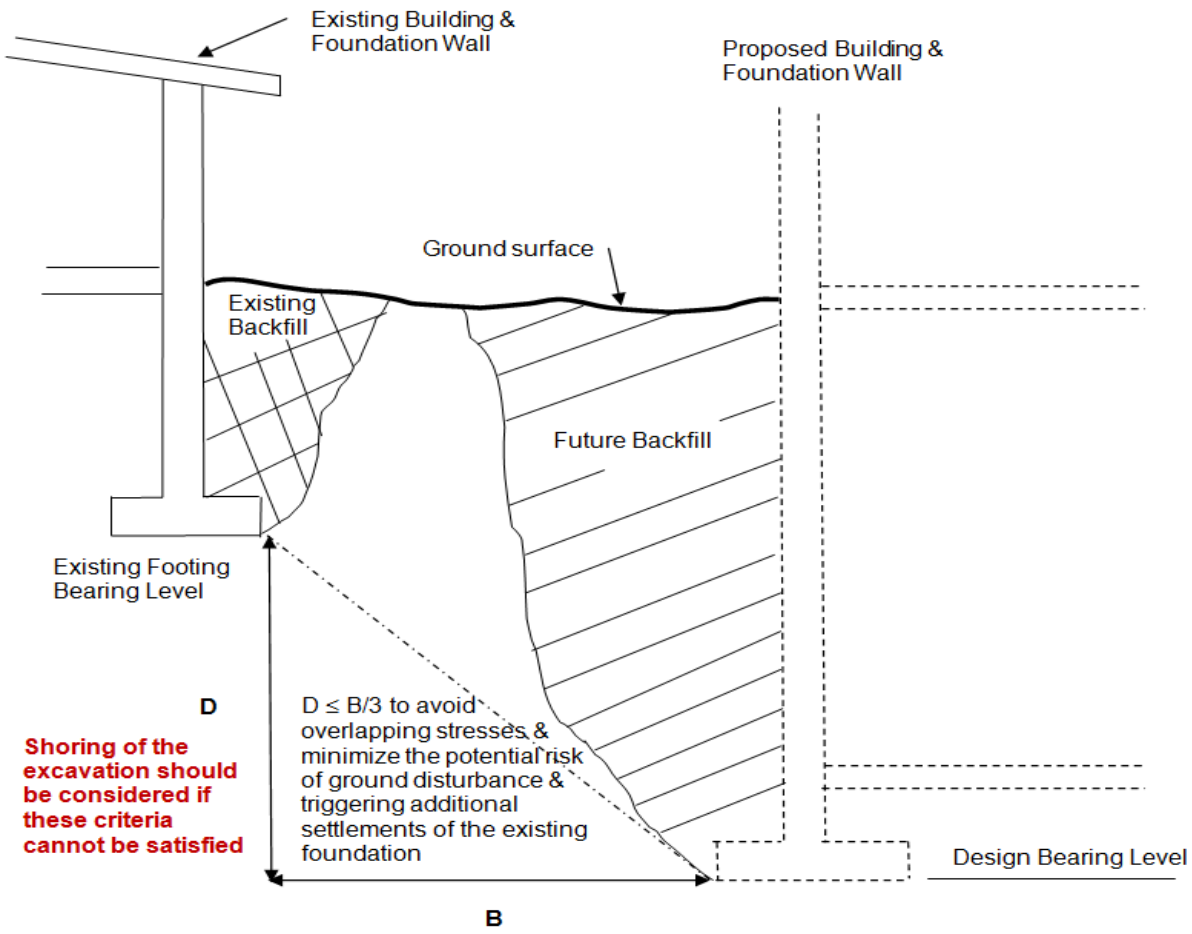


***Figure 2**: Criteria for Assessing Excavation Shoring Requirements (Not to Scale)*

9.2 General Backfill Considerations

Backfill next to foundation walls and in service trenches should be selected to be compactable in narrow trench conditions. The excavated limestone can be reused as backfill, provided that the limestone particle sizes are broken down to minus 200 mm material, the moisture content of the material is maintained at near optimum levels, the rock particles are mixed with the overburden clayey silt to silty clay soils to reduce void spaces, and the material is placed in maximum lift thicknesses of 300 mm.

Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. It is recommended that the target compaction specification for trench backfill be 97 % SPMDD with no individual test below 95 % SPMDD.

During inclement weather the native soils may become too wet to achieve satisfactory compaction. If construction is proposed for late in the year, a reduced level of trench compaction with a higher risk of future settlements is to be anticipated, and it is recommended that provisional contract quantities be established for the supply and placement of imported granular fill under such circumstances. The imported granular should meet the requirements of OPSS 1010 for Granular B Type I material as a minimum requirement.

10.0 SITE SERVICING CONSIDERATIONS

There is no indication that special pipe bedding materials or procedures are required for the installation of services. All bedding cover and backfill materials should be selected in accordance with OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material.

The pipes should be placed with a minimum bedding thickness in conformance of OPSD 802.010 series (typical 150 mm for flexible pipes, OPSD 802.010, 802.013 and 802.014). The use of normal Class B type bedding is applicable for the pipe.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 95 % of the SPMDD before a subsequent layer is placed. Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ by more than the 300 mm uncompacted layer. The remainder of the trench should be backfilled as per the requirements defined in Sections 9.0 and 11.0.

It is assumed all services installed within the overburden soil will have a minimum of 1.2 m of soil cover for frost protection. For services installed at shallower depths, suitable insulation for frost protection is recommended. Services installed within the limestone bedrock are generally deemed exempt from any frost protection requirements

11.0 ENVIRONMENTAL CONSIDERATIONS AND SOIL MANAGEMENT

11.1 Environmental Considerations

A total of eight selected, soil samples were submitted for chemical analyses to ALS Environmental to determine metals and inorganic properties *(Regulation 153/04).* The test results are provided in Appendix D and are summarized in Table 5 and Table 6.

Table 5: Summary of O. Reg. 153 Metals & Inorganics Test Data (Tables 1-MOECC Comparison)

Sample ID	Composite Sample Depth	Parameters Analyzed	MOECC Table Comparison	
			Table 1	Exceedance
BH3-SS1&SS2	0.0 m – 1.2 m	O. Reg. 153 Metals & Inorganics	✓	-
BH4-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH6-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH7-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH9-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH13-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✘	SAR*
BH14-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH16-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✘	SAR*, EC**

✓ Meets MOECC Tables 1 soil quality guidelines
× Does not meet MOECC Table 1 soil quality guidelines
* Sodium Adsorption Ratio
** Electrical Conductivity

Table 6: Summary of O. Reg. 153 Metals & Inorganics Test Data (Tables 2 &3-MOECC Comparison)

Sample ID	Composite Sample Depth	Parameters Analyzed	MOECC Table Comparison	
			Table 2&3	Exceedance
BH3-SS1&SS2	0.0 m – 1.2 m	O. Reg. 153 Metals & Inorganics	✓	-
BH4-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH6-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH7-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH9-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH13-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH14-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✓	-
BH16-SS1&SS2	0.7 m – 2.0 m	O. Reg. 153 Metals & Inorganics	✘	SAR*, EC**

✓ Meets MOECC Tables 2 and 3 soil quality guidelines
× Does not meet MOECC Tables 2 and 3 soil quality guidelines
* Sodium Adsorption Ratio
** Electrical Conductivity

In comparison to O. Reg. 153 Tables 2 and 3: Residential/Parkland/Institutional (herein *"RPI")* property use criteria of the Ministry of the Environment and Climate Change (herein *"MOECC")* document "*Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act*", dated April 15, 2011, all soils samples meet guidelines related to metals and inorganics, with the exception of soil samples obtained from borehole BH16 at

depths of between 0.7 m and 2.0 m below existing ground level which exceeded guideline limits for Sodium Adsorption Ratio (SAR) and Electrical Conductivity (EC).

The results were compared against the Table 1 standards for residential/parkland/institutional/ industrial/commercial/community (RPI/ICC) land use and soil samples obtained from borehole BH13 also exceeded guideline limits for SAR.

Elevated SAR and EC values are commonly associated with impacted conditions from long term use of road salt. Section 2 of Ontario Regulation 339 of the Revised Regulations of Ontario, 1990 (Classes of Contaminants – Exemptions) indicates that elevated residual road salt parameters in soils within the road allowance that result from winter salting that was placed to address safety issues are not regarded as an environmental problem that requires remediation. Given the close proximity of boreholes BH13 and BH16 to the existing roadway, the EC and SAR guideline exceedances are not regarded as an environmental problem.

If excavated subgrade soils are to be disposed of off-site it should be noted that there may be soil quality criteria established for the site that is to receive the excavated materials. As such, the owners or operators of potential receiving sites must be notified of the environmental quality of the material and must agree in writing to accept the material. Some variability in soil quality parameters may occur across the site, and the contractor may be required to complete testing to address a designated list of chemical parameters for the receiving site.

11.2 Soil Management

Construction for the proposed development may involve cut and fill operations. From a geotechnical perspective, and in order to optimize the use of the on-site soils, a Soil Management Plan should be established. The plan objective should be to achieve a self-sustainable development with respect to excavated materials, and control the placement of organic soils so that there is negligible impact on the settlement performance of the compacted fill material.

The soil management criteria should be as follows:

- Surface vegetation, topsoil and organic soils should not be placed within the proposed roadways, below finished subgrade level for pavement construction or building limits. These materials should be placed in landscaped areas where settlements are not critical;

- Excavated soils for structural fill in pavement areas and building floor slab areas, which does not have topsoil or organic matter and are compactable with moisture contents within 2 % to 3 % of the optimum value, should be placed and compacted to a target density of 97 % of the SPMDD with no individual test result below 95 % SPMDD; if engineered fill is required to support building foundations, the engineered fill should be placed and compacted in lifts to a target density of 100 % SPMDD with no individual tests below 98 % SPMDD; the soil should be placed in a loose lift thickness not exceeding 250 mm and should be compacted using a large (10 ton or larger) pad-foot type roller with vibratory capability; if engineered fill to support building foundations is being considered it is recommended that a pre-construction meeting be scheduled to review the proposed fill materials, fill placement and compaction procedures, and the testing and inspection requirements;

- Soils to be placed in landscaped areas where settlements are not critical should receive nominal compaction effort in order to achieve at least 90 % of the SPMDD; and,

12.0 STORM WATER POND CONSIDERATIONS

Stormwater management ponds are proposed in the sections of the site identified as blocks 3 and 5, as shown on the site layout plan attached in Appendix C, where the existing ground surface is at an approximate Geodetic elevation of 192 m. Though a detailed understanding of the proposed pond design is not available, the pond will most likely be a flood storage pond constructed by excavation.

Boreholes BH5, BH6, BH7, and BH8 were advanced in the vicinity of the proposed ponds and silty clay to clayey silt soils underlie this section of the site to depths of between 0.6 m (borehole BH5) and 5.2 m (boreholes BH7 and BH8) below existing ground level. Limestone bedrock was encountered underlying the overburden soils. Based on the tests conducted within the installed wells, the hydraulic conductivity in bedrock ranges from $1.09x10^{-5}$ cm/sec to $1.04x10^{-6}$ cm/sec.

It is to be anticipated that the the native soils in the area of the ponds will comprise primarily of clayey silt to silty clay, with a coefficient of permeability (k) expected to be in the order of $1x10^{-6}$ cm/sec to $1x10^{-7}$ cm/sec. Based on the assumption that the base of the proposed ponds will be within the limestone bedrock or silty clay to clayey silt, it is to be anticipated that the pond construction will require the inclusion of a clay liner.

The following general comments are provided for consideration for the pond liners' design:

- Where the base of the pond is to be constructed below the recorded, static groundwater table, dewatering will be required to allow the excavation of the pond and the construction of the clay liner;
- Prior to the placement of the pond liner, the liner subgrade should be prepared by scarifying the native clayey silt and remolding and compacting the soil to remove layering and/or silt seams;
- The liner subgrade soil should be initially compacted with an impact (sheep's-foot) roller to develop the required profile, and then sealed using smooth drum roller.
- A liner thickness of 0.5 m should be considered as a minimum.
- The liner materials required should be of high clay-containing soils of low permeability; in the order of 1×10^{-6} cm/s to 1×10^{-7} cm/s;
- Consideration could be given to the application of a geo-synthetic clay liner, such as Bentofix SNRWL, to maintain the design pond water levels. The product manufacturer should be consulted beforehand to confirm suitability for application;
- Ideally, pond side slopes should be no steeper that 4H:1V, however, steepening to approximately 1V:3H may be considered given the existing soil conditions and absence of groundwater provided that appropriate erosion protection measures are applied (i.e. vegetative cover or the provision of erosion blankets, granular mats, geo-synthetic liner etc.); and,
- Should slopes be steeper than 1V: 3H then an increased potential for erosion and higher maintenance requirements due to localized sloughing should be expected.

Any headwall or associated structures located in the vicinity of the SWM Pond can be founded in the native cohesive soils for a geotechnical reaction at the SLS of 150 kPa and a geotechnical resistance at the ULS of 225 kPa. Structures can be founded on the limestone bedrock for a geotechnical resistance 1 MPa at the ULS.

All excavations must comply with the definitions prescribed by the OHSA Ontario Regulation 213/91 "*Construction Projects*", as outlined in Section 10 of this report.

It should be noted that the City of Hamilton has conceded all SWM Pond overall design requirements to the provincial prerequisites, as has been acceptable to the Hamilton Conservation Authority (herein "*HCA*"). The location, function and design criteria of the proposed SWM Pond will require confirmation through consultation with the City of Hamilton and the HCA.

13.0 PAVEMENT CONSIDERATIONS

13.1 Pavement Design Considerations

The proposed development includes ground level asphalt pavement for car access and parking areas. The Corporation of the City of Hamilton (herein "*City of Hamilton*") standard section for the class of road to be constructed is presented in Table 7 and the standards have an expected design life of about 20 years.

Table 7: Pavement Structure Layer Thicknesses

Pavement Layer	Urban Local Residential Road City of Hamilton
Surface Course Asphalt OPSS HL 3	40 mm
Binder Course Asphalt OPSS HL 8	80 mm
OPSS Granular A Base	150 mm
Granular Subbase OPSS Granular B, Type II	300 mm[1]
Total Thickness	570 mm

Notes:
1. If construction proceeds late in the year (i.e. November and December), the design thickness of pavement granular materials may have to be increased to address potential problems with subgrade instability and facilitate construction vehicle and truck access.

13.2 Pavement Construction Considerations

The overall performance of the pavement structure will greatly depend upon the support provided by the developed subgrade. A number of factors should be considered at the construction stages to ensure that an acceptable subgrade condition is developed and maintained:

- Sub-drains should be installed and should be 100 mm diameter perforated plastic pipe, with outfalls to catch basins at a continuous and uniform grade. The sub-drains should conform to OPSD 216.01;
- Any soft areas of notable deflection to the subgrade should be sub-excavated and replaced with a suitable backfill material approved by a qualified geotechnical engineer and compacted to 98 % of its SPMDD;
- The subgrade should be properly shaped, crowned and then proof-rolled under the full time observation of a geotechnical representative of this office to delineate any soft areas which may require repair before placing the granular materials; and,
- Surface water should not be allowed to pond on the surface of or adjacent to the outside edges of any developed subgrade.

Should the pavements proposed for the development be constructed as a two-stage paving operation it will important to ensure that the following is undertaken to develop the surface of the binder course being used as a "*temporary*" surface during the construction phase:

- The surface is thoroughly cleaned and power washed to remove all residual contaminants;
- All deficiencies are corrected to meet the required design specifications; and,
- A suitable tack coat is appropriately applied immediately prior to the placement of the upper asphaltic concrete course(s).

Such preparatory works are to be completed in accordance with the appropriate OPSS, as required.

13.3 Pavement Materials

Granular Base Course and Subbase

The granular base course materials should meet OPSS Granular “A” specifications. Quarried 20 mm limestone crushed to Granular "A" gradation specifications is recommended. If the option with granular subbase material is used, the granular subbase should meet OPSS Granular B Type II requirements for 100 % crushed quarried bedrock (50 mm crusher-run limestone).

Hot Mix Asphalt

The binder course and surface course asphalt should meet current specifications for HL 8 and HL 3 respectively.

The standard asphalt binder grade for the climate conditions in Hamilton is PG 58-28. Given the observed low volume of commercial truck traffic it is considered that there is no requirement for a bump up to a higher PG grade of asphalt cement.

Compaction

Granular base course and subbase course fill material should be compacted to 100 % SPMDD. Hot mix asphalt should be compacted to the criteria set out by the City of Hamilton.

13.4 Subgrade Considerations

The subgrade conditions and bearing strength may be variable along the road section and some subgrade repairs should be anticipated.

It is recommended that, prior to the placement of pavement granular fill, the exposed subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck to traverse the exposed subgrade and provide for full coverage. The proof-rolling should be monitored by a geotechnical representative of this office to delineate any soft areas which may require repair. Repairs should be undertaken to avoid creating “*bathtub*” conditions in the subgrade within the pavement structure.

13.5 Sidewalk Considerations

The design and construction of the proposed concrete sidewalks should be completed to the satisfaction of the City of Hamilton’s Engineering Standards, and as detailed in Table 8. The concrete and aggregates should be produced and placed to meet those standards also stipulated by the City of Hamilton’s Engineering Standards.

It should be noted that the concrete sidewalk design specified in Table 8 addresses a use by

pedestrian traffic only and does not include for use by vehicular traffic.

Table 8: Minimum Concrete Sidewalk Specifications

Materials	Compaction Requirements	Layer Thickness
Normal Portland GU (30 MPa) (CAN3-CSA A23.1) - Class C-2	N/A	125 mm
Granular “A” Base	95 % SPMDD*	150 mm

* Standard Proctor Maximum Dry Density

Where finished sidewalks are on level ground, and to ensure that they remain free of ponding water, a final slope/gradient of the concrete sidewalk surface of at least 2 % should be maintained. In addition, construction joints in the sidewalk concrete should be properly sealed (e.g. bitumen filler) to minimize the water migration.

14.0 CLOSURE

The Limitations of Report, as stated in Appendix A, are an integral part of this report.

Soil samples will be retained and stored by Landtek for a period of three months after the report is issued. The samples will be disposed of at the end of the three month period unless a written request from the client to extend the storage period is received.

We trust this report will be of assistance with the design and construction of the proposed development. Should you have any questions, please do not hesitate to contact our office.

Yours sincerely,

LANDTEK LIMITED

Isaac Asonya, EIT
Author

James Dann, B.Eng. (Hons)
Manager-Geotechnical Projects



Ralph Di Cienzo, P. Eng.
Consulting Engineer

REFERENCES

[1] Freenstra, B.H. 1975: Quaternary Geology of the Grimsby Area, Southern Ontario; Ontario Div. Mines, Prelim. Map P.993. Geol. Ser., Scale 1:50 000. Geology 1974.

[2] Paleozoic geology of the Grimsby Area, southern Ontario; Ontario Division of Mines, Map M2343. scale 1: 50 000

APPENDIX A
LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the borehole locations. Subsurface and ground water conditions between and beyond the Boreholes may be different from those encountered at the borehole locations, and conditions may become apparent during construction that could not be detected or anticipated at the time of the geotechnical investigation. It is recommended practice that Landtek be retained during construction to confirm that the subsurface conditions throughout the site are consistent with the conditions encountered in the Boreholes.

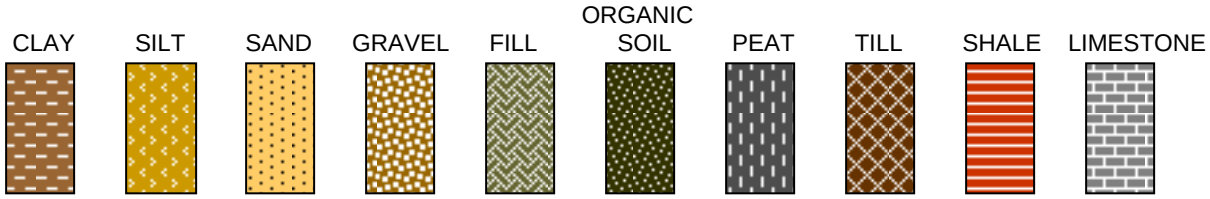
The comments made in this report on potential construction problems and possible remedial methods are intended only for the guidance of the designer. The number of Boreholes may not be sufficient to determine all the factors that may influence construction methods and costs. For example, the thickness and quality of surficial topsoil or fill layers may vary markedly and unpredictably. Additionally, bedrock contact depths throughout the site may vary significantly from what was encountered at the exact borehole locations. Contractors bidding on the project, or undertaking construction on the site should make their own interpretation of the factual borehole information, and establish their own conclusions as to how the subsurface conditions may affect their work.

The survey elevations in the report were obtained by Landtek Limited or others, and are strictly for use by Landtek in the preparation of the geotechnical report. The elevations should not be used by any other parties for any other purpose.

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. Landtek Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

This report does not reflect environmental issues or concerns related to the property unless otherwise stated in the report. The design recommendations given in the report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, it is recommended that Landtek Limited be retained during the final design stage to verify that the design is consistent with the report recommendations, and that the assumptions made in the report are still valid.

APPENDIX B
SYMBOLS AND TERMS USED IN THE REPORT



RELATIVE PROPORTIONS

Term	Range
Trace	0 - 5%
A Little	5 – 15%
Some	15 – 30%
With	30 – 50%

CLASSIFICATION BY PARTICLE SIZE

Boulder ---------- > 200 mm
Cobble ---------- 80 mm – 200 mm
Gravel -
Coarse ---------- 19 mm – 80 mm
Fine ---------- 4.75 mm – 19 mm
Sand -
Coarse ---------- 4.75 mm – 2 mm
Medium -------- 2 mm – 0.425 mm
Fine ---------- 0.425 mm – 0.75 mm
Silt ---------- 0.075 mm – 0.002 mm
Clay ---------- < 0.002 mm

DENSITY OF NON-COHESIVE SOILS

Descriptive Term	Relative Density	Standard Penetration Test
Very Loose	0 – 15%	0 – 4 Blows Per 300 mm Penetration
Loose	15 – 35%	4 – 10 Blows Per 300 mm Penetration
Compact	35 – 65%	10 – 30 Blows Per 300 mm Penetration
Dense	65 – 85%	30 – 50 Blows Per 300 mm Penetration
Very Dense	85 – 100%	Over 50 Blows Per 300 mm Penetration

CONSISTENCY OF COHESIVE SOILS

Descriptive Term	Undrained Shear Strength kPa (psf)	N Value Standard Penetration Test	Remarks
Very Soft	< 12 (< 250)	< 2	Can penetrate with fist
Soft	12 – 25 (250 – 500)	2 – 4	Can indent with fist
Firm	25 – 50 (500 –1000)	4 – 8	Can penetrate with thumb
Stiff	50 – 100 (1000 – 2000)	8 – 15	Can indent with thumb
Very Stiff	100 – 200 (2000 – 4000)	15 – 30	Can indent with thumb-nail
Hard	> 200 (> 4000)	> 30	Can indent with thumb-nail

Notes: 1. Relative density determined by standard laboratory tests.
2. N value – blows/300 mm penetration of a 623 N (140 Lb.) hammer falling 760 mm (30 in.) on a 50 mm O.D. split spoon soil sampler. The split spoon sampler is driven 450 mm (18 in.) or 610 mm (24 in.). The “N” value is the Standard Penetration Test (SPT) value and is normally taken as the number of blows to advance the sampler the last 300 mm.

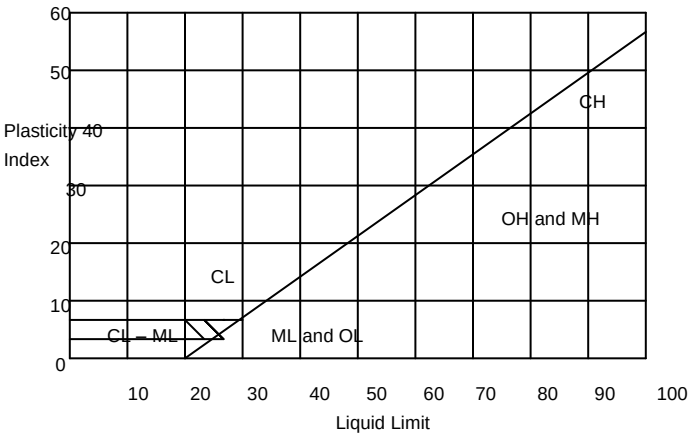
APPENDIX B CONTINUED
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM Designation: D 2487 - 69 AND D 2488 – 69
(Unified Soil Classification System)

<table>
<tr><th colspan="3">Major Divisions</th><th>Group Symbols</th><th>Typical Names</th><th colspan="3">Classification Criteria</th></tr>
<tr><td rowspan="8">Coarse-grained soils More than 50% retained on No. 200 sieve *</td><td rowspan="4">Gravels 50% or more of coarse fraction retained on No. 4 sieve</td><td rowspan="2">Clean gravels</td><td>GW</td><td>Well-graded gravels and gravel-sand mixtures, little or no fines</td><td rowspan="8">Classification on basis of percentage of fines
Less than 5% pass No. 200 sieve . . . . . GW, GP, SW, SP

More than 12% pass No. 200 sieve . . . . . GM, GC, SM, SC

5 to 12% pass No.200 sieve . . . . . Borderline classifications requiring use of dual symbols</td><td colspan="2">C_u=D60/D10 greater than 4;
$C_z = (D30)^2/(D10xD60)$ between 1 and 3</td></tr>
<tr><td>GP</td><td>Poorly graded gravels and gravel-sand mixtures, little or no fines</td><td colspan="2">Not meeting both criteria for GW</td></tr>
<tr><td rowspan="2">Gravels with fines</td><td>GM</td><td>Silty gravels, gravel-sand-silt mixtures</td><td>Atterberg limits below “A” line or P.I. less than 4</td><td rowspan="2">Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols</td></tr>
<tr><td>GC</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td>Atterberg limits above “A” line with P.I. greater than 7</td></tr>
<tr><td rowspan="4">Sands More than 50% of coarse fraction passes No. 4 sieve</td><td rowspan="2">Clean Sands</td><td>SW</td><td>Well-graded sands and gravelly sands, little or no fines</td><td colspan="2">C_u=D60/D10 greater than 6;
$C_z = (D30)^2/ (D10xD60)$ between 1 and 3</td></tr>
<tr><td>SP</td><td>Poorly graded sands and gravelly sands, little or no fines</td><td colspan="2">Not meeting both criteria for SW</td></tr>
<tr><td rowspan="2">Sands with fines</td><td>SM</td><td>Silty sands, sand-silt mixtures</td><td>Atterberg limits below “A” line or P.I. less than 4</td><td rowspan="2">Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols</td></tr>
<tr><td>SC</td><td>Clayey sands, sand-clay mixtures</td><td>Atterberg limits above “A” line with P.I. greater than 7</td></tr>
<tr><td rowspan="7">Fine-grained soils 50% or more passes No. 200 sieve *</td><td colspan="2" rowspan="3">Silts and clays Liquid limit 50% or less</td><td>ML</td><td>Inorganic silts, very fine sands, rock flour, silty or clayey fine sands</td><td colspan="3" rowspan="7">Plasticity Chart

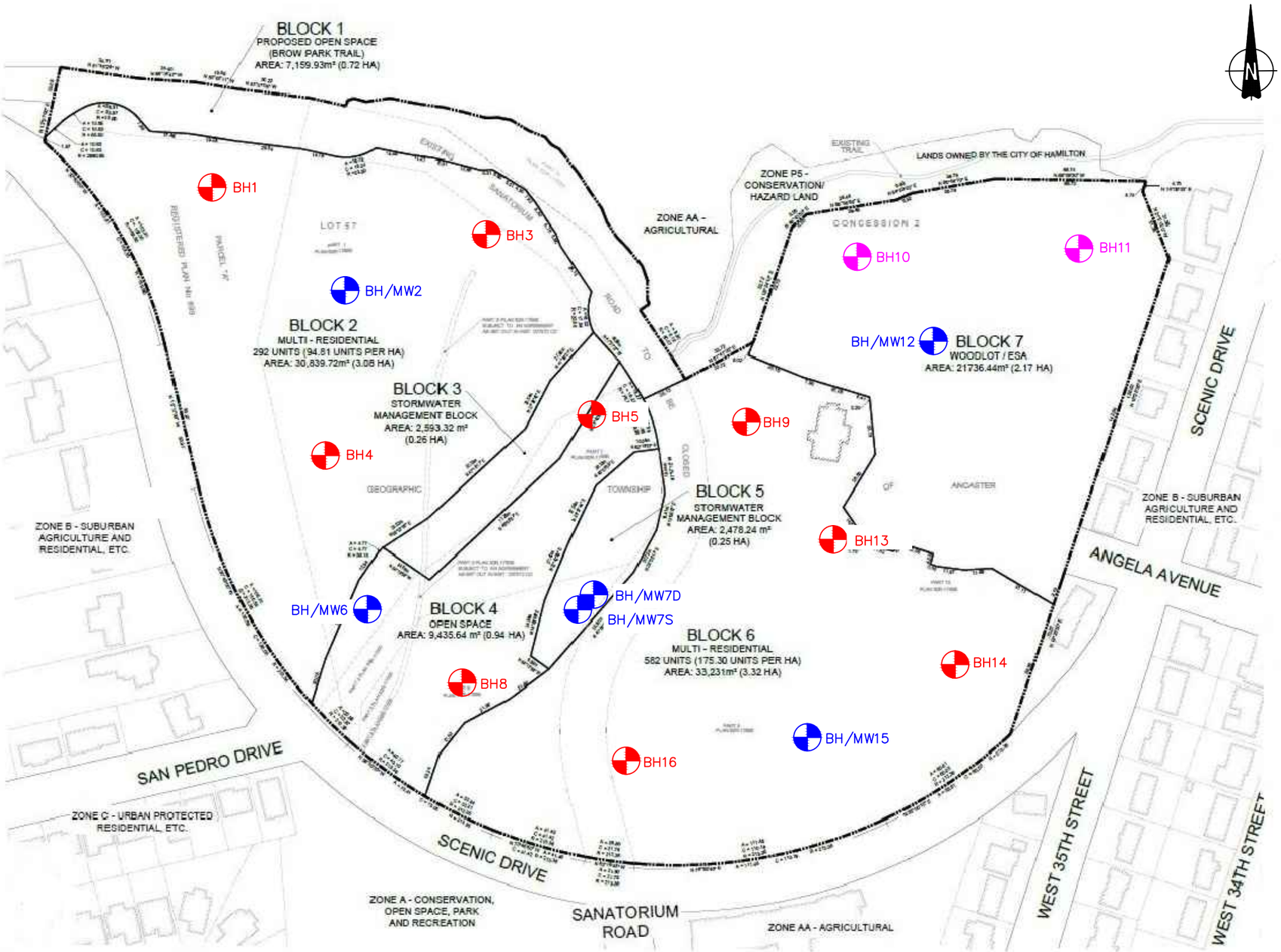
For classification of fine-grained soils and fine fraction of coarse-grained soils. Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.
Equation of A-line: PI=0.73 (LL-20)

 
</td></tr>
<tr><td>CL</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silts</td></tr>
<tr><td>OL</td><td>Organic silts and organic silts of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">Silts and clays Liquid limit greater than 50%</td><td>MH</td><td>Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts</td></tr>
<tr><td>CH</td><td>Inorganic clays of high plasticity, fat clays</td></tr>
<tr><td>OH</td><td>Organic clays of medium to high plasticity</td></tr>
<tr><td colspan="2">Highly organic soils</td><td>Pt</td><td>Peat, much and other highly organic soils</td><td colspan="3">* Based on the material passing the 3 in. (76mm) sieve.</td></tr>
</table>

APPENDIX C

DRAWING 1 - SITE PLAN SHOWING BOREHOLE LOCATIONS

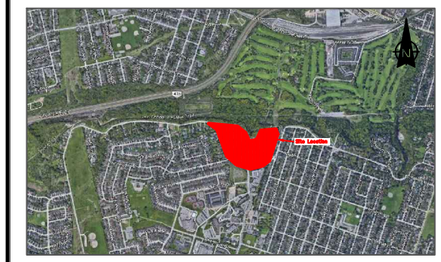
BOREHOLES LOGS



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project location



- Approximate location of boreholes drilled by Landtek Limited between June 25 and July 4, 2018.
- Approximate location of monitoring wells installed by Landtek Limited between June 25 and July 4, 2018.
- Boreholes BH10 and BH11 were not drilled due to access restrictions.

revisions/ submissions

#	date	description
1	2018-08-22	issued for report

client
Valery (Chedoke Browlands) Developments Inc.

municipality
City of Hamilton

project
Geotechnical Investigation
Scenic Drive, Hamilton

sheet
Borehole Location Plan

date: August 22, 2018
drawn: IA
checked: JD
project #: 18211
scale: NA

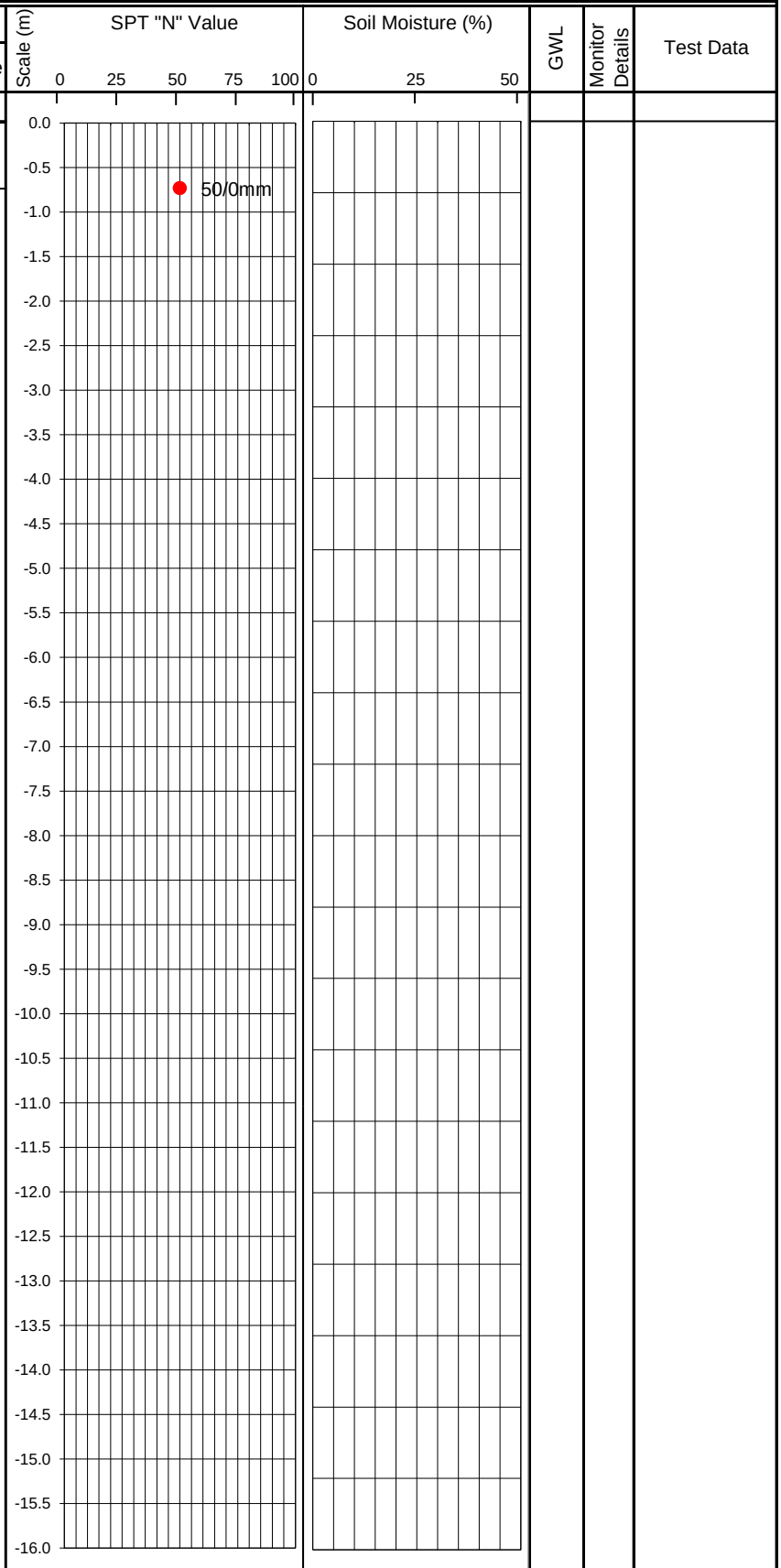
18211-01

LANDTEK LIMITED

BH 1

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		190.5			
±300 mm of topsoil		0.0			
CLAYEY SILT: brown, stiff, moist		189.8			
		0.7			
AUGER REFUSAL ON LIMESTONE BEDROCK					
BOREHOLE TERMINATED					



Notes:
1. Upon completion of drilling, borehole open to 0.7 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

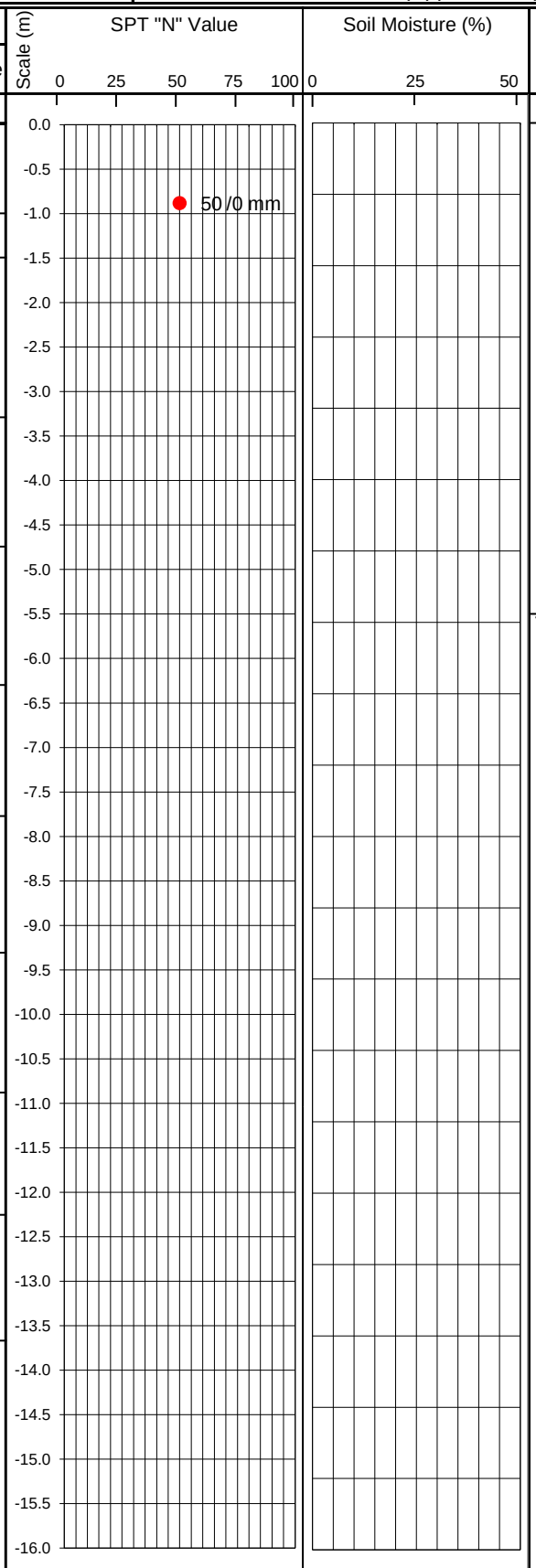
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LANDTEK LIMITED

BH/MW 2

Project No.:	18211	Drill Date:	June 28, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface	190.9			stick up=0.97m
±300 mm of topsoil	0.0			
CLAYEY SILT: brown/grey, stiff, moist	190.0			
LIMESTONE BEDROCK	0.9 / 189.4	RUN #1	63 mm	
highly fractured, slightly weathered to fresh, grey and dark grey, strong RQD = 11 % (Run 1) RQD = 26 % (Run 2)	1.5 / 187.6	CORE RUN #2	63 mm Dia. Core	bentonite backfill to ground level
slightly weathered to fresh, grey and dark grey, strong to very strong Recovery = 100 %; RQD = 46 % ∂ = 25 kN/m³; UCS strength = 145 MPa	3.3 / 186.1	CORE RUN #3	63 mm Dia. Core	
fresh, grey and dark grey, strong Recovery = 100 %; RQD = 70 % ∂ = 25 kN/m³; UCS strength = 74.8 MPa	4.8 / 184.6	CORE RUN #4	63 mm Dia. Core	WL = 5.5 m Aug. 31, 2018
fresh, grey and dark grey, very strong Recovery = 100 %; RQD = 94 % ∂ = 25 kN/m³; UCS strength = 135 MPa	6.3 / 183.1	CORE RUN #5	63 mm Dia. Core	
Recovery = 100 %; RQD = 90 % ∂ = 25 kN/m³ 8.5 m to 9.1 m: small vugs in bedrock	7.8 / 181.5	CORE RUN #6	63 mm Dia. Core	
Recovery = 100 %; RQD = 88 % ∂ = 25 kN/m³	9.4 / 180.0	CORE RUN #7	63 mm Dia. Core	sand backfill to 10.3 m
Recovery = 100 %; RQD = 100 % ∂ = 25 kN/m³	10.9 / 179.1	CORE RUN #8	63 mm Dia. Core	10.7 m
11.8 m to 12.4 m: fresh, SHALE	11.8 / 178.5			3.0 m of slotted 50 mm dia. PVC pipe
Recovery = 100 %; RQD = 100 % ∂ = 25 kN/m³	12.4 / 177.2	CORE RUN #9	63 mm Dia. Core	
BOREHOLE TERMINATED	13.7			13.7 m



Notes:
1. Upon completion of drilling, borehole open to 13.7 m
2. Water level reading: WL at 5.49 m depth on July 25, 2018
3. Water level reading: WL at 5.49 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

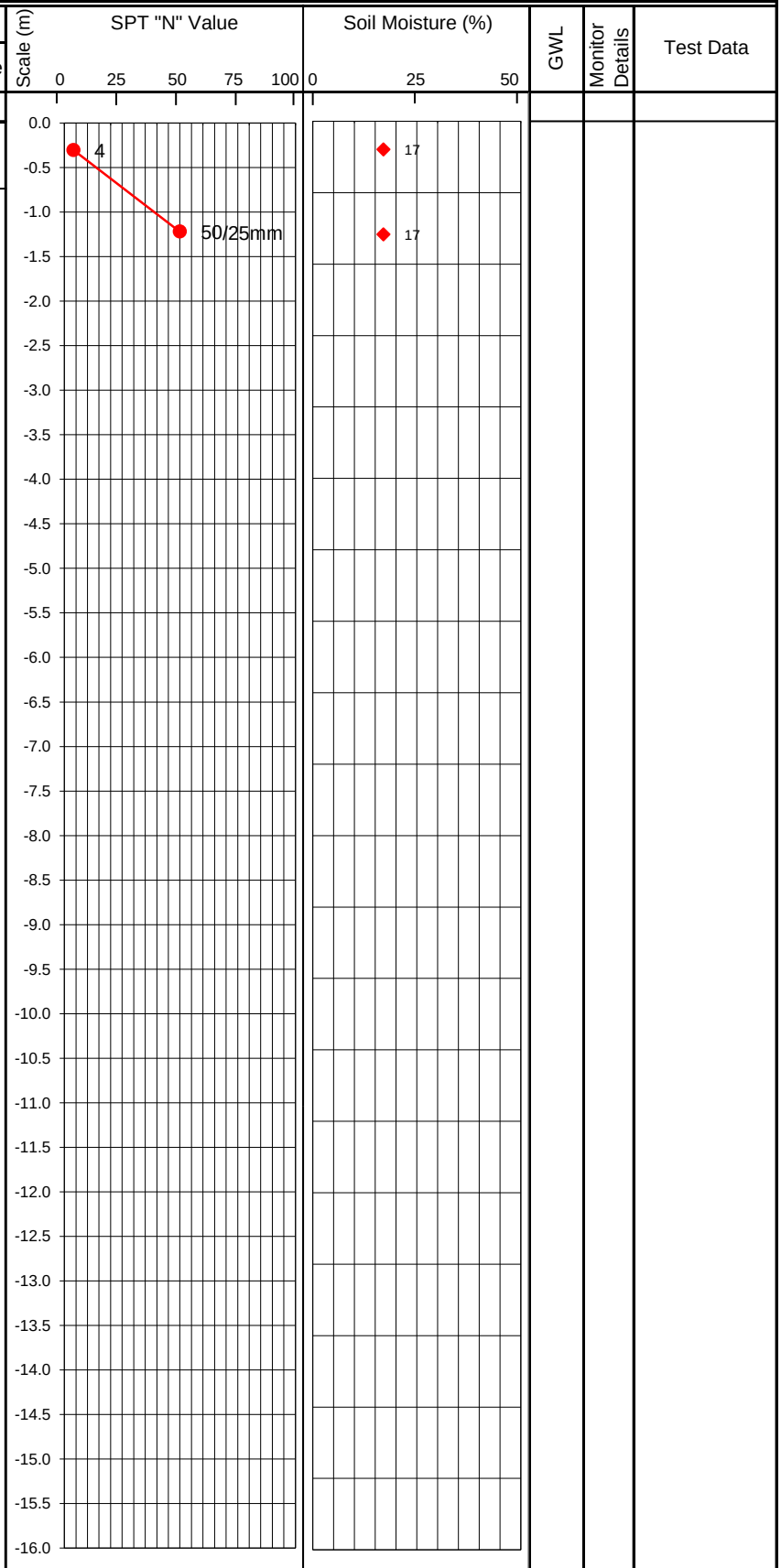
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BH 3

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		190.7			
FILL: clayey silt, brown, firm, moist		0.0 190.0	1	SS	
CLAYEY SILT: brown, stiff, moist		0.7 189.5			
AUGER REFUSAL ON LIMESTONE BEDROCK **BOREHOLE TERMINATED**		1.2			



Notes:
1. Upon completion of drilling, borehole open to 1.2 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

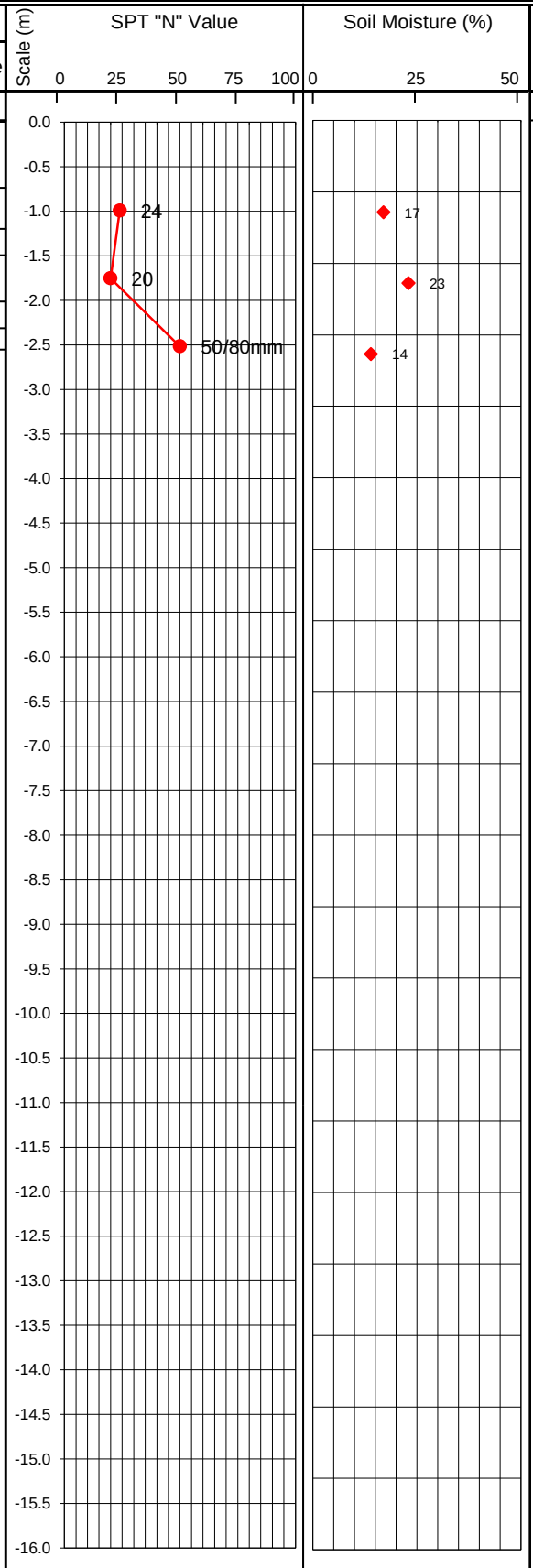
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BH 4

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		192.6			
±300 mm of topsoil		0.0			
SILTY CLAY: brown, very stiff, moist			1	SS	
		191.1			
CLAYEY SILT: brown, very stiff, trace iron staining, moist		1.5	2	SS	
		190.1	3	SS	
AUGER REFUSAL ON LIMESTONE BEDROCK BOREHOLE TERMINATED		2.5			



Notes:
1. Upon completion of drilling, borehole open to 2.5 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

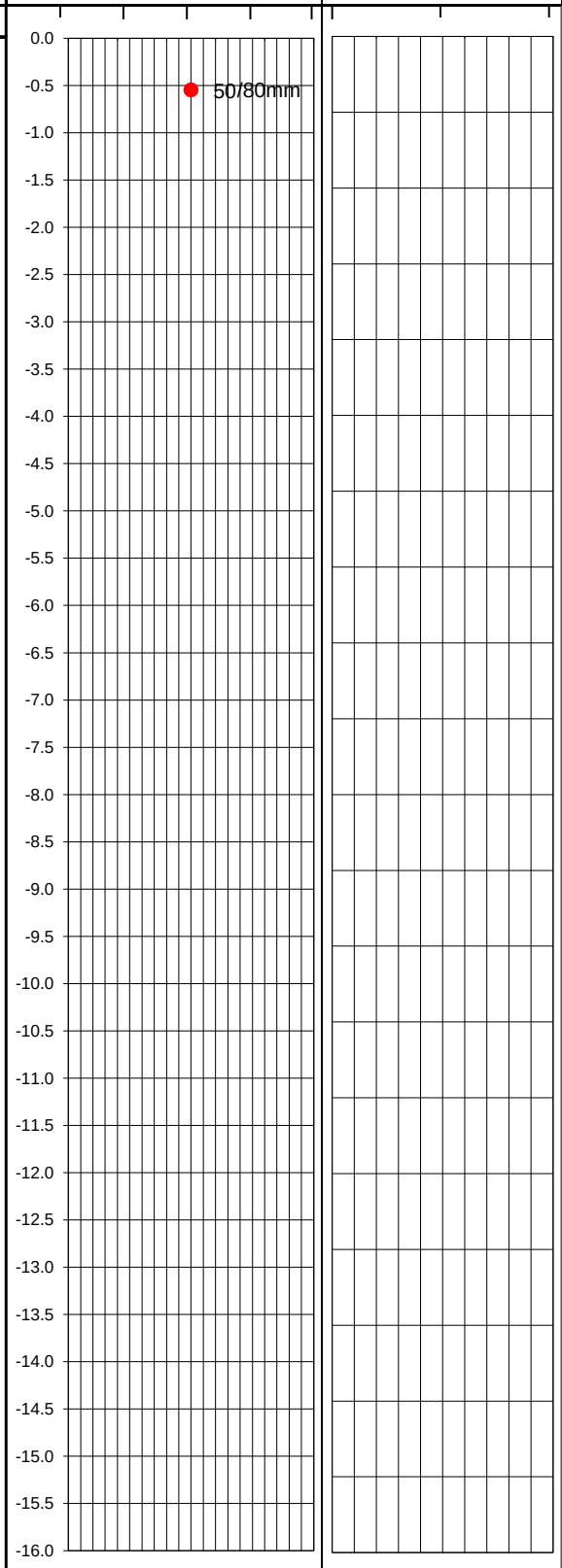
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BH 5

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Scale (m)	SPT "N" Value	Soil Moisture (%)	GWL	Monitor Details	Test Data
Ground Surface		191.5								
±250 mm of topsoil		0.0								
SILTY CLAY: brown, very stiff, moist		190.9								
AUGER REFUSAL ON LIMESTONE		0.6								
BEDROCK										
BOREHOLE TERMINATED										



Notes:
1. Upon completion of drilling, borehole open to 0.6 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

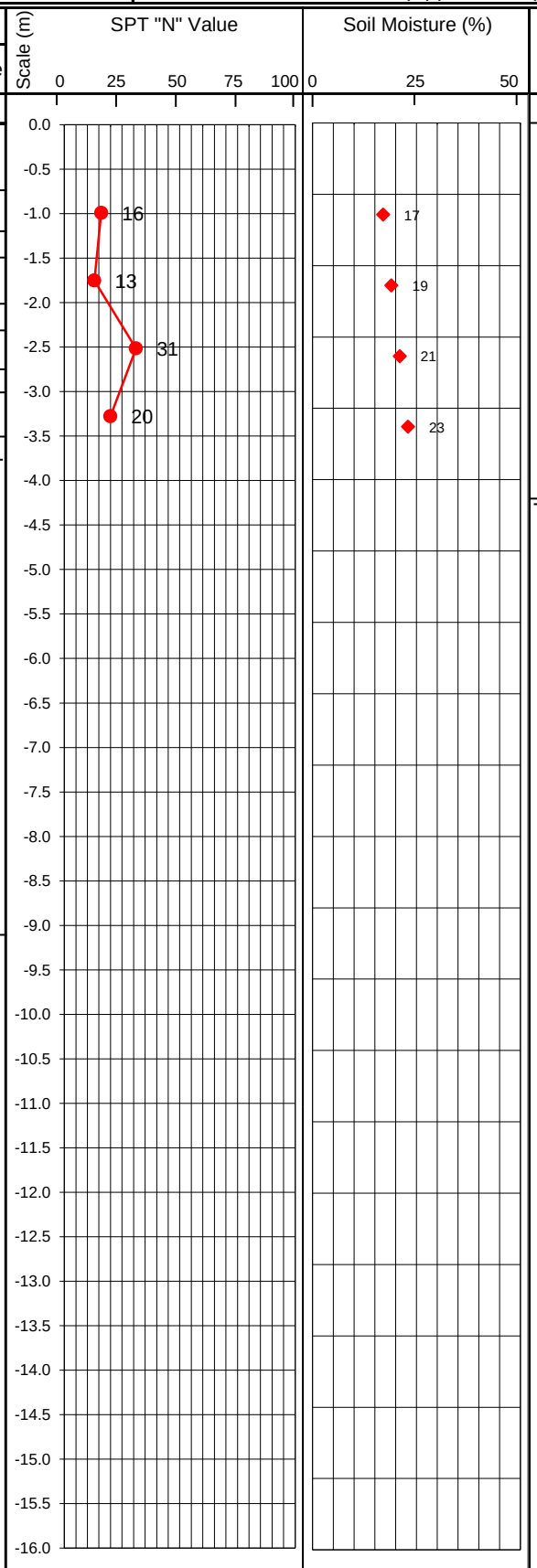
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BH/MW 6

Project No.:	18211	Drill Date:	July 4, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface	192.4			stick up=0.91 m
±300 mm of topsoil	0.0			
SILTY CLAY: trace gravel, brown, stiff to very stiff, moist		1	SS	
		2	SS	
	190.1			
CLAYEY SILT: brown, very stiff to hard, very moist, trace iron staining	2.3	3	SS	bentonite backfill to ground level
		4	SS	
	188.7			
LIMESTONE BEDROCK	3.7		TRICONE DRILLING	WL = 4.3 m Aug. 31, 2018
	183.3			sand backfill to 7.0 m
BOREHOLE TERMINATED	9.1			1.5 m of slotted 50 mm dia. PVC pipe; 9.1 m



Notes:
1. Upon completion of drilling, borehole open to 9.1 m
2. Water level reading: WL at 4.49 m depth on July 25, 2018
3. Water level reading: WL at 4.34 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

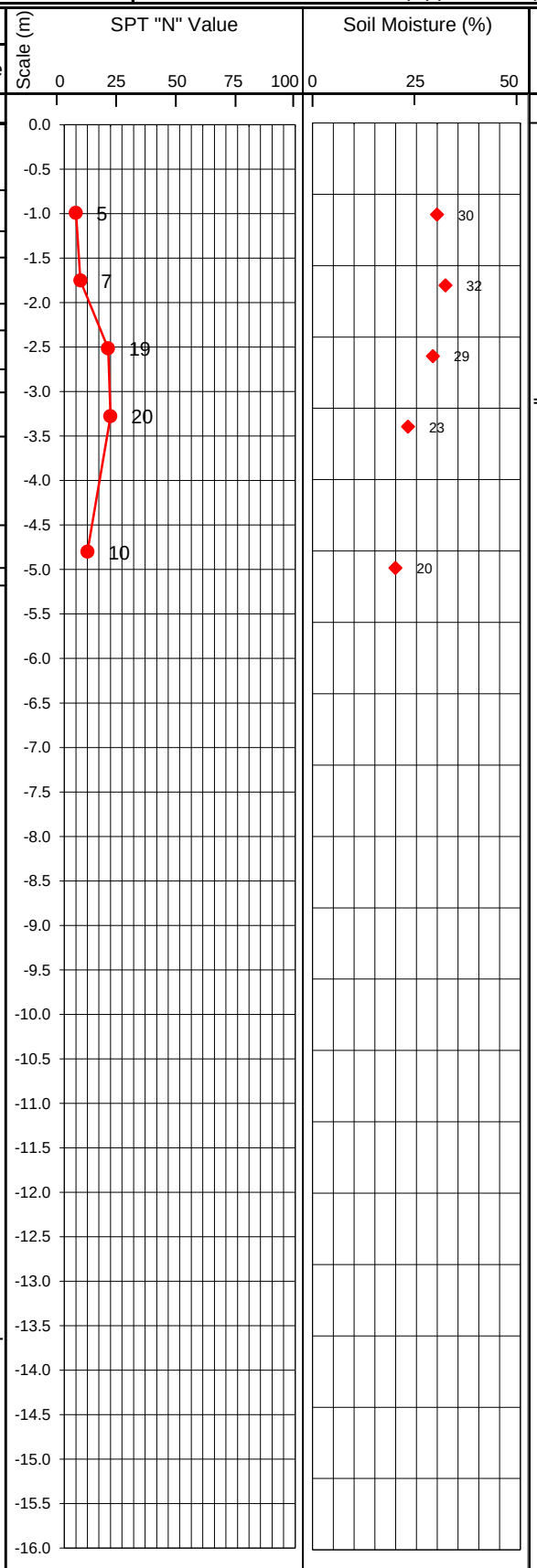
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BH/MW 7D

Project No.:	18211	Drill Date:	July 3, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. Depth	Samples No.	Type	Test Data
Ground Surface	193.5			stick up=0.91 m
±300 mm of topsoil	0.0			
FILL: silt, some clay, trace organics, brown, loose, very moist		1	SS	bentonite backfill to ground level
		2	SS	
	191.2			
SILTY CLAY: brown, very stiff, moist	2.3	3	SS	WL = 3.3 m Aug. 31, 2018
		4	SS	
	189.0			
CLAYEY SILT: brown, stiff, moist	4.5	5	SS	
	188.3			
LIMESTONE BEDROCK	5.2		TRICONE DRILLING	sand backfill to 11.5 m; 12.2 m; 1.5 m of slotted 50 mm dia. PVC pipe
	179.8			
BOREHOLE TERMINATED	13.7			13.7 m



Notes:
1. Upon completion of drilling, borehole open to 11.9 m
2. Water level reading: WL at 2.94 m depth on July 25, 2018
3. Water level reading: WL at 3.25 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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BH/MW 7S

Project No.:	18211	Drill Date:	June 29, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. Depth	Samples No.	Samples Type	Test Data
Ground Surface		193.5			stick up=0.90 m
±300 mm of topsoil		0.0			
FILL: silt, some clay, trace organics, brown, loose, very moist			1	SS	bentonite backfill to ground level
			2	SS	
		191.2			
SILTY CLAY: brown, very stiff, moist		2.3	3	SS	WL = 3.3 m Aug. 31, 2018
			4	SS	
		189.0			
CLAYEY SILT: brown, stiff, moist		4.5	5	SS	
		188.3			
LIMESTONE BEDROCK		5.2		TRICONE DRILLING	sand backfill to 9.6 m; 1.5 m of slotted 50 mm dia. PVC pipe
		181.6			
BOREHOLE TERMINATED		11.9			11.9 m

SPT "N" Value: 5, 7, 19, 20, 10

Soil Moisture (%): 30, 32, 29, 23, 20

Notes:
1. Upon completion of drilling, borehole open to 11.9 m
2. Water level reading: WL at 2.95 m depth on July 25, 2018
3. Water level reading: WL at 3.27 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

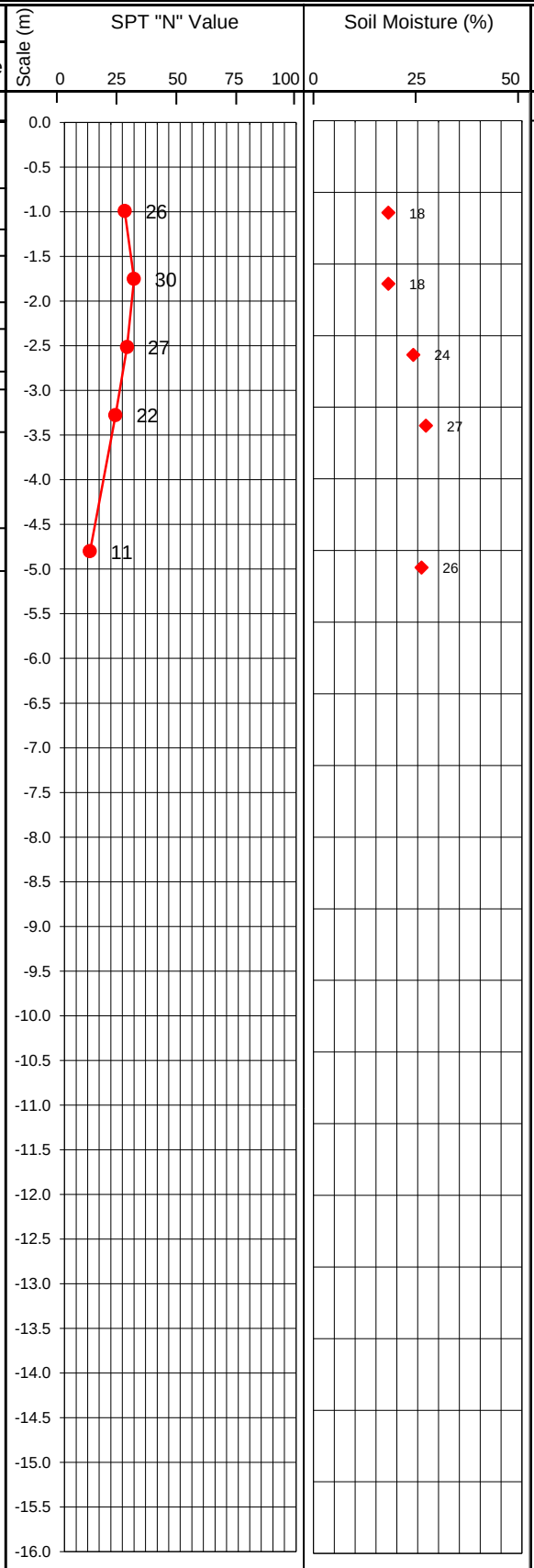
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BH 8

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		194.2			
±250 mm of topsoil		0.0			
CLAYEY SILT: brown and grey, very stiff, moist			1	SS	
		192.2	2	SS	
SILTY CLAY: brown and grey, very stiff, trace iron staining, moist		2.0	3	SS	
			4	SS	
4.5 m to 5.2 m: grey, stiff, very moist, low plastic			5	SS	
		189.0			
AUGER REFUSAL ON LIMESTONE BEDROCK BOREHOLE TERMINATED		5.2			



GWL

Monitor Details

Notes:
1. Upon completion of drilling, borehole open to 5.2 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

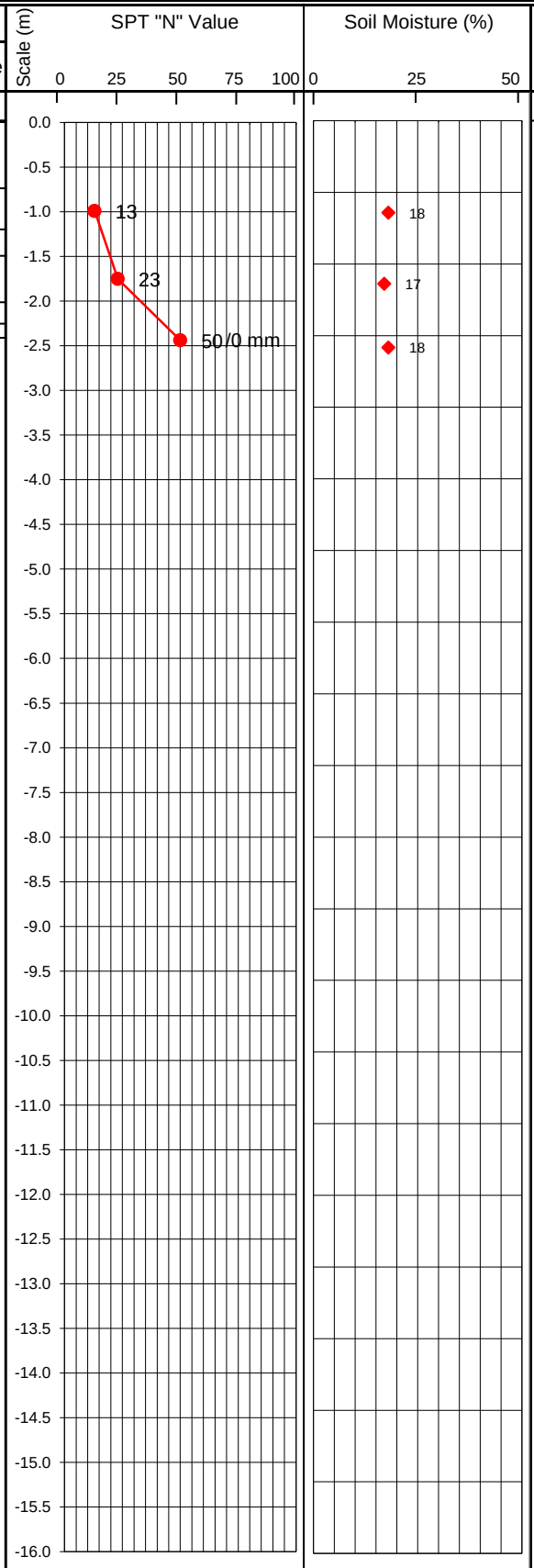
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BH 9

Project No.:	18211	Drill Date:	June 26, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		192.4			
±250 mm of topsoil		0.0			
CLAYEY SILT: brown and grey, stiff to very stiff, moist			1	SS	
			2	SS	
		190.0	3	SS	
AUGER REFUSAL ON LIMESTONE BEDROCK		2.4			
BOREHOLE TERMINATED					



GWL

Monitor Details

Notes:
1. Upon completion of drilling, borehole open to 2.4 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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BH/MW 12

Project No.:	18211	Drill Date:	June 27, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. Depth	Samples No.	Samples Type	Test Data
Ground Surface	192.5			stick up=0.88 m
±300 mm of topsoil	0.0			
CLAYEY SILT: brown/grey, stiff, moist		1	SS	
	190.5	2	SS	
LIMESTONE BEDROCK highly fractured, slightly weathered to fresh, dark grey, very strong: UCS Strength=154 Mpa; RQD=38 %	2.0 / 189.2	CORE RUN #1	63 mm Dia. Core	bentonite backfill to ground level
slightly weathered to fresh, grey and dark grey, strong Recovery = 100 %; RQD = 83 % ∂ = 24 kN/m³; UCS strength = 76.5 MPa	3.3 / 187.7	CORE RUN #2	63 mm Dia. Core	
fresh, grey and dark grey, very strong Recovery = 100 %; RQD = 93 % ∂ = 24 kN/m³; UCS strength = 177 MPa 5.0 m to 5.2 m: small vug in bedrock	4.8 / 186.1	CORE RUN #3	63 mm Dia. Core	WL = 4.5 m Aug. 31, 2018
Recovery = 100 %; RQD = 76 % 6.7 m to 6.8 m weathered to fresh, SHALE ∂ = 25 kN/m³	6.4 / 184.7	CORE RUN #4	63 mm Dia. Core	
Recovery = 100 %; RQD = 91 % ∂ = 25 kN/m³	7.8 / 183.2	CORE RUN #5	63 mm Dia. Core	
Recovery = 100 %; RQD = 83 % ∂ = 25 kN/m³ 10.2 m to 10.5 m: weathered to fresh, SHALE	9.3 / 181.6	CORE RUN #6	63 mm Dia. Core	sand backfill to 10.3 m
Recovery = 100 %; RQD = 91 % ∂ = 25 kN/m³	10.9 / 180.2	CORE RUN #7	63 mm Dia. Core	10.7 m; 3.0 m of slotted 50 mm dia. PVC pipe
Recovery = 100 %; RQD = 100 % ∂ = 25 kN/m³	12.3 / 178.8	CORE RUN #8	63 mm Dia. Core	
BOREHOLE TERMINATED	13.7			13.7 m

SPT "N" Value: 11, 50/130 mm

Soil Moisture (%): 24, 18

Notes:
1. Upon completion of drilling, borehole open to 13.7 m
2. Water level reading: WL at 4.53 m depth on July 25, 2018
3. Water level reading: WL at 4.56 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

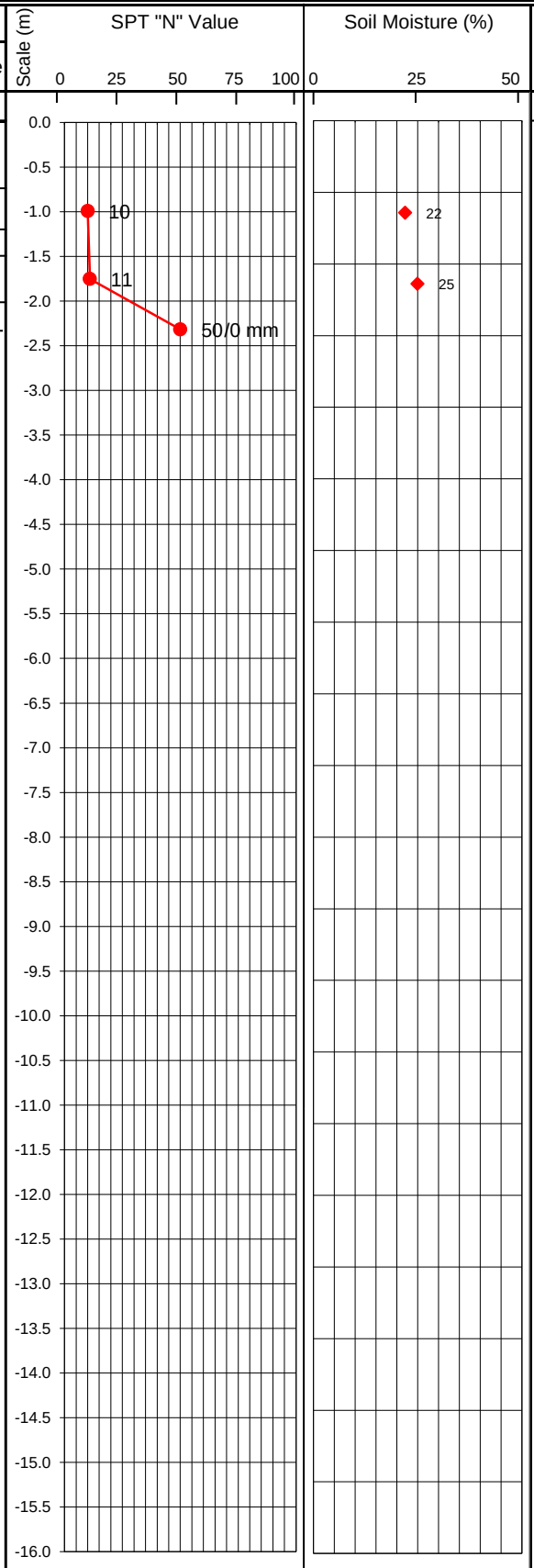
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BH 13

Project No.:	18211	Drill Date:	June 26, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		192.2			
±250 mm of topsoil		0.0			
CLAYEY SILT: brown and grey, stiff, moist			1	SS	
			2	SS	
		189.9			
AUGER REFUSAL ON LIMESTONE BEDROCK BOREHOLE TERMINATED		2.3			



Notes:
1. Upon completion of drilling, borehole open to 2.3 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

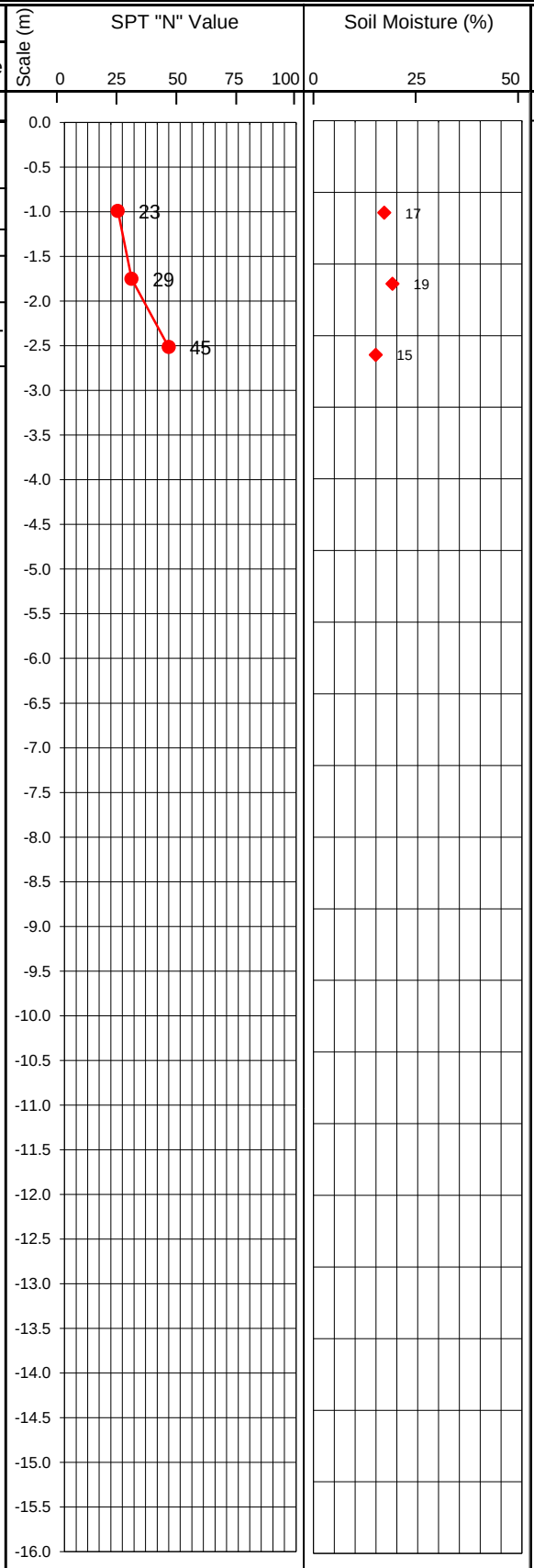
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BH 14

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		194.7			
±300 mm of topsoil		0.0			
CLAYEY SILT: brown and grey, stiff, moist			1	SS	
			2	SS	
			3	SS	
		191.8			
AUGER REFUSAL ON LIMESTONE BEDROCK		2.9			
BOREHOLE TERMINATED					



GWL

Monitor Details

Notes:
1. Upon completion of drilling, borehole open to 2.9 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

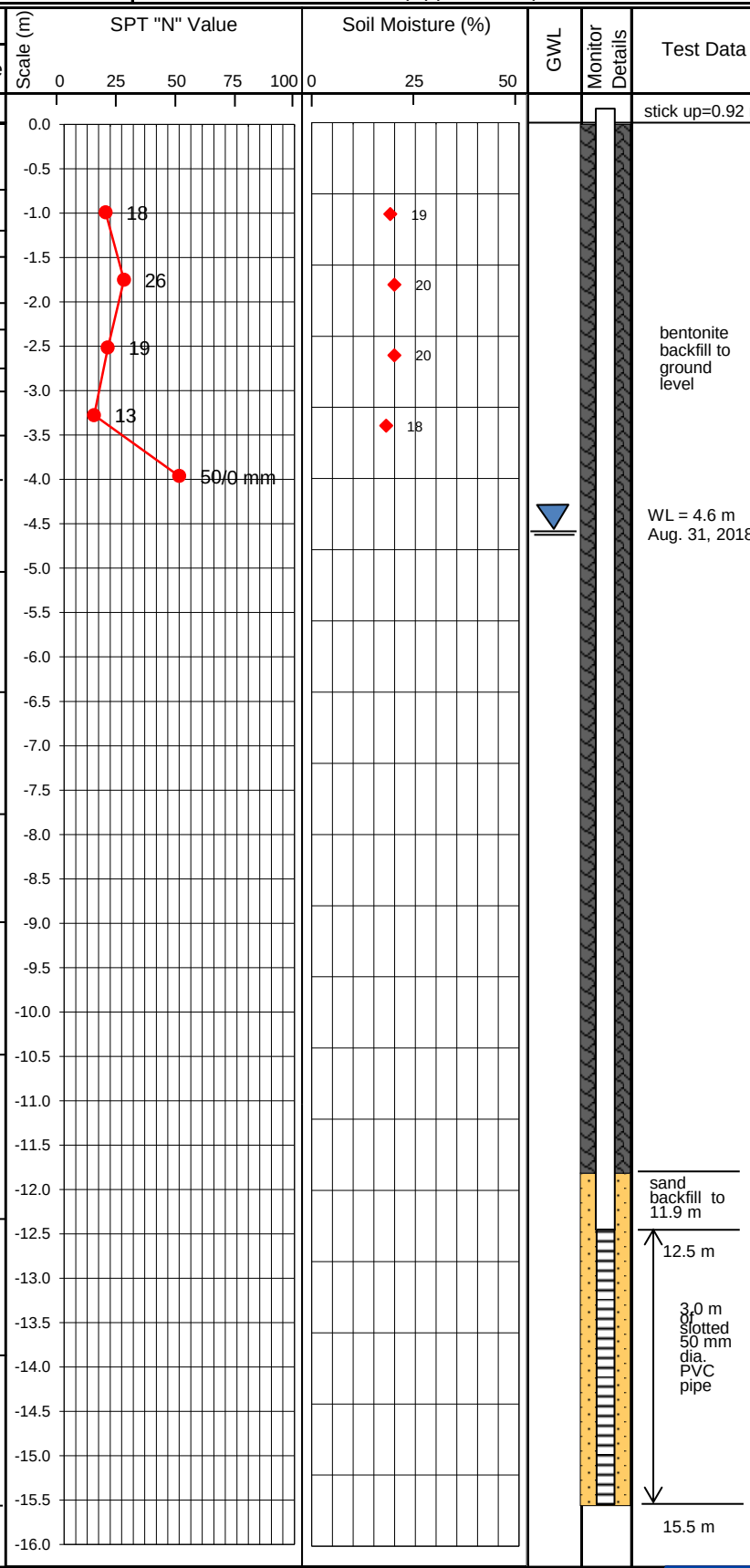
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BH/MW 15

Project No.:	18211	Drill Date:	June 26, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. Depth	Samples No.	Samples Type	Test Data
Ground Surface	194.2			stick up=0.92 m
±300 mm of topsoil	0.0			
FILL: clayey silt, brown/grey, stiff, moist	193.3	1	SS	
SILTY CLAY: brown/grey, very stiff, moist	0.9	2	SS	bentonite backfill to ground level
		3	SS	
	191.2			
CLAYEY SILT: trace gravel, brown/grey, stiff, moist	3.0	4	SS	
	190.2			
LIMESTONE BEDROCK slightly weathered to fresh, dark grey, strong: UCS Strength= 92 Mpa; RQD= 80 %	4.0	CORE RUN #1	63 mm Dia. Core	WL = 4.6 m Aug. 31, 2018
	189.3			
slightly weathered to fresh, grey and dark grey, very strong Recovery = 100 %; RQD = 91 % ∂ = 25 kN/m³; UCS strength = 173 MPa	4.9	CORE RUN #2	63 mm Dia. Core	
	187.8			
fresh, grey and dark grey, very strong Recovery = 100 %; RQD = 90 % ∂ = 25 kN/m³; UCS strength = 103 MPa	6.4	CORE RUN #3	63 mm Dia. Core	
	186.5			
Recovery = 100 %; RQD = 90 % ∂ = 25 kN/m³; UCS strength = 172 MPa	7.7	CORE RUN #4	63 mm Dia. Core	
8.6 m to 8.7 m: slightly weathered SHALE	185.2			
Recovery = 100 %; RQD = 96 % ∂ = 25 kN/m³	9.0	CORE RUN #5	63 mm Dia. Core	
	183.7			
Recovery = 100 %; RQD = 91 % ∂ = 25 kN/m³ 10.8 m to 11.8 m: small vugs in bedrock	10.5	CORE RUN #6	63 mm Dia. Core	
	181.8			
Recovery = 100 %; RQD = 100 % ∂ = 25 kN/m³	12.4	CORE RUN #7	63 mm Dia. Core	
13.7 m to 13.8 m: slightly weathered SHALE	180.3			
Recovery = 100 %; RQD = 90 % ∂ = 25 kN/m³	13.9	CORE RUN #8	63 mm Dia. Core	
	178.7			
BOREHOLE TERMINATED	15.5			12.5 m 3.0 m of slotted 50 mm dia. PVC pipe 15.5 m



Notes:
1. Upon completion of drilling, borehole open to 15.5 m
2. Water level reading: WL at 3.43 m depth on July 25, 2018
3. Water level reading: WL at 4.63 m depth on August 31, 2018

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

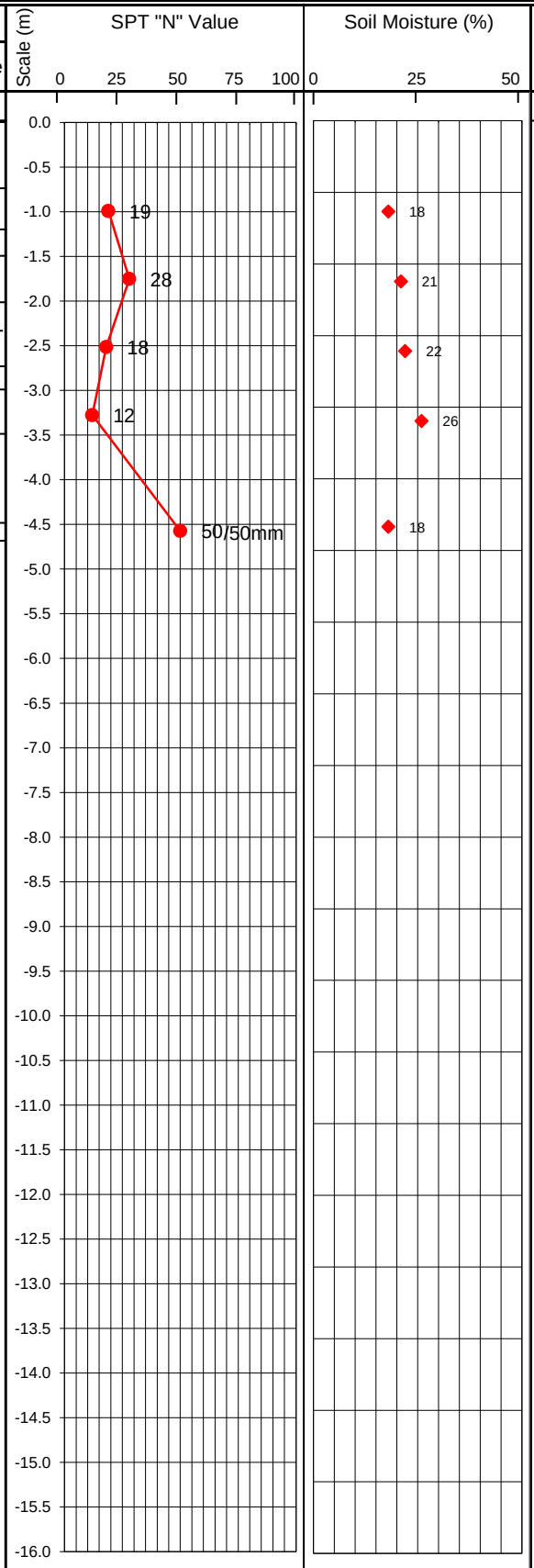
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BH 16

Project No.:	18211	Drill Date:	June 25, 2018
Project:	Geotechnical Investigation Report	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	Scenic Drive, Hamilton, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	Test Data
Ground Surface		194.0			
±300 mm of topsoil		0.0			
FILL: clayey silt, brown and grey, stiff, moist		193.1	1	SS	
SILTY CLAY: brown and grey, very stiff, moist		0.9	2	SS	
			3	SS	
		191			
CLAYEY SILT: brown and grey, stiff, moist		3.0	4	SS	
		189.4	5	SS	
AUGER REFUSAL ON LIMESTONE BEDROCK BOREHOLE TERMINATED		4.6			



GWL | Monitor Details

Notes:
1. Upon completion of drilling, borehole open to 4.6 m
2. Groundwater not encountered

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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APPENDIX D

ENVIRONMENTAL ANALYTICAL REPORT

ANALYTICAL REPORT

Summary of Guideline Exceedances

Guideline ALS ID	Client ID	Grouping	Analyte	Result	Guideline Limit	Unit
Ontario Regulation 153/04 - April 15, 2011 Standards - T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use						
L2126833-3	BH13-SS1 AND SS2	Saturated Paste Extractables	SAR	4.76	2.4	SAR
L2126833-4	BH16-SS1 AND SS2	Physical Tests	Conductivity	0.936	0.57	mS/cm
L2126833-11	BH16-SS2 AND SS3	Physical Tests	Conductivity	1.29	0.57	mS/cm
Ontario Regulation 153/04 - April 15, 2011 Standards - T3-Soil-Res/Park/Inst. Property Use (Fine)						
L2126833-4	BH16-SS1 AND SS2	Physical Tests	Conductivity	0.936	0.7	mS/cm
L2126833-11	BH16-SS2 AND SS3	Physical Tests	Conductivity	1.29	0.7	mS/cm

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Physical Tests - SOIL

		Lab ID	L2126833-1	L2126833-2	L2126833-3	L2126833-4	L2126833-5	L2126833-6	L2126833-7	L2126833-8	L2126833-9
		Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
		Sample ID	BH14-SS1 AND SS2	BH7-SS1 AND SS2	BH13-SS1 AND SS2	BH16-SS1 AND SS2	BH4-SS1 AND SS2	BH9-SS1 AND SS2	BH6-SS1 AND SS2	BH3-SS1 AND SS2	BH4-SS1 AND SS2
		Guide Limits									
Analyte	Unit	#1	#2								
Conductivity	mS/cm	0.57	0.7	0.144	0.245	0.153	0.936	0.138	0.136	0.125	0.256	0.133
% Moisture	%	-	-	16.1	17.3	19.0	15.4	18.3	15.0	15.5	15.1	19.0
pH	pH units	-	-	7.70	7.38	7.20	7.74	7.65	7.60	7.63	7.51	7.63
Redox Potential	mV	-	-									232
Resistivity	ohm*cm	-	-									7520

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Physical Tests - SOIL

		Guide Limits				
Lab ID				L2126833-10	L2126833-11	L2126833-12
Sample Date				04-JUL-18	04-JUL-18	04-JUL-18
Sample ID				BH6-SS3 AND SS4	BH16-SS2 AND SS3	BH9-SS1 AND SS2
Analyte	**Unit**	**#1**	**#2**			
Conductivity	mS/cm	0.57	0.7	0.130	1.29	0.127
% Moisture	%	-	-	21.9	21.0	15.4
pH	pH units	-	-	7.66	7.68	7.62
Redox Potential	mV	-	-	233	238	233
Resistivity	ohm*cm	-	-	7690	774	7870

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Leachable Anions & Nutrients - SOIL

		Guide Limits					
				Lab ID	L2126833-9	L2126833-10	L2126833-11	L2126833-12
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH4-SS1 AND SS2	BH6-SS3 AND SS4	BH16-SS2 AND SS3	BH9-SS1 AND SS2
Analyte	Unit	#1	#2					
Chloride	ug/g	-	-		<5.0	<5.0	515	<5.0

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Anions and Nutrients - SOIL

			Lab ID	L2126833-9	L2126833-10	L2126833-11	L2126833-12
			Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
			Sample ID	BH4-SS1 AND SS2	BH6-SS3 AND SS4	BH16-SS2 AND SS3	BH9-SS1 AND SS2
		Guide Limits					
Analyte	Unit	#1	#2				
Sulphate	mg/kg	-	-	21	22	488	<20

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Cyanides - SOIL

		Guide Limits								
				Lab ID	L2126833-1	L2126833-2	L2126833-3	L2126833-4	L2126833-5	L2126833-6	L2126833-7	L2126833-8
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH14-SS1 AND SS2	BH7-SS1 AND SS2	BH13-SS1 AND SS2	BH16-SS1 AND SS2	BH4-SS1 AND SS2	BH9-SS1 AND SS2	BH6-SS1 AND SS2	BH3-SS1 AND SS2
Analyte	Unit	#1	#2									
Cyanide, Weak Acid Diss	ug/g	0.051	0.051		<0.050	<0.050	<0.050	<0.050	<0.050	<0.050	<0.050	<0.050

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Inorganic Parameters - SOIL

		Guide Limits					
				Lab ID	L2126833-9	L2126833-10	L2126833-11	L2126833-12
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH4-SS1 AND SS2	BH6-SS3 AND SS4	BH16-SS2 AND SS3	BH9-SS1 AND SS2
Analyte	Unit	#1	#2					
Acid Volatile Sulphides	mg/kg	-	-		0.70	1.38	0.32	1.26

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

Saturated Paste Extractables - SOIL

Analyte	Unit	Guide Limits #1	Guide Limits #2	Lab ID	L2126833-1	L2126833-2	L2126833-3	L2126833-4	L2126833-5	L2126833-6	L2126833-7	L2126833-8
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH14-SS1 AND SS2	BH7-SS1 AND SS2	BH13-SS1 AND SS2	BH16-SS1 AND SS2	BH4-SS1 AND SS2	BH9-SS1 AND SS2	BH6-SS1 AND SS2	BH3-SS1 AND SS2
SAR	SAR	2.4	5		0.33 SAR:M	0.15	4.76	>32. SAR:L	0.24 SAR:M	0.25 SAR:M	0.15 SAR:M	0.42 SAR:M
Calcium (Ca)	mg/L	-	-		4.8	16.8	1.1	<1.0	5.2	4.8	5.1	14.7
Magnesium (Mg)	mg/L	-	-		<1.0	1.0	2.6	<1.0	<1.0	<1.0	<1.0	<1.0
Sodium (Na)	mg/L	-	-		2.7	2.3	40.2	190	2.0	2.0	1.2	5.8

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Metals - SOIL

Analyte	Unit	Guide Limits #1	Guide Limits #2	Lab ID	L2126833-1	L2126833-2	L2126833-3	L2126833-4	L2126833-5	L2126833-6	L2126833-7	L2126833-8
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH14-SS1 AND SS2	BH7-SS1 AND SS2	BH13-SS1 AND SS2	BH16-SS1 AND SS2	BH4-SS1 AND SS2	BH9-SS1 AND SS2	BH6-SS1 AND SS2	BH3-SS1 AND SS2
Antimony (Sb)	ug/g	1.3	7.5		<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Arsenic (As)	ug/g	18	18		5.1	5.3	5.9	4.6	5.5	5.4	5.0	4.5
Barium (Ba)	ug/g	220	390		90.4	80.8	92.6	110	82.3	81.3	99.0	44.2
Beryllium (Be)	ug/g	2.5	5		0.70	0.66	0.81	0.67	0.65	0.66	0.70	<0.50
Boron (B)	ug/g	36	120		9.0	7.3	6.5	9.6	7.0	7.7	9.5	5.0
Boron (B), Hot Water Ext.	ug/g	36	1.5		<0.10	0.31	0.16	<0.10	<0.10	<0.10	<0.10	0.13
Cadmium (Cd)	ug/g	1.2	1.2		<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50
Chromium (Cr)	ug/g	70	160		22.6	22.9	26.7	23.7	21.5	22.3	23.7	13.5
Cobalt (Co)	ug/g	21	22		11.7	10.7	13.8	13.2	12.5	12.7	12.2	7.4
Copper (Cu)	ug/g	92	180		26.8	31.4	35.0	25.2	26.8	28.1	27.0	27.1
Lead (Pb)	ug/g	120	120		12.0	27.5	14.5	12.5	11.6	11.3	12.5	11.7
Mercury (Hg)	ug/g	0.27	1.8		0.0173	0.0452	0.0277	0.0185	0.0180	0.0183	0.0165	0.0194
Molybdenum (Mo)	ug/g	2	6.9		<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Nickel (Ni)	ug/g	82	130		25.8	23.2	30.2	27.6	26.9	27.5	27.8	16.1
Selenium (Se)	ug/g	1.5	2.4		<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Silver (Ag)	ug/g	0.5	25		<0.20	<0.20	<0.20	<0.20	<0.20	<0.20	<0.20	<0.20
Thallium (Tl)	ug/g	1	1		<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50	<0.50
Uranium (U)	ug/g	2.5	23		<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Vanadium (V)	ug/g	86	86		32.1	32.5	38.1	33.6	30.2	31.9	32.7	20.1
Zinc (Zn)	ug/g	290	340		53.6	89.5	70.0	52.1	53.1	54.3	53.3	63.0

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

ANALYTICAL REPORT

Speciated Metals - SOIL

Analyte	Unit	Guide Limits #1	Guide Limits #2	Lab ID	L2126833-1	L2126833-2	L2126833-3	L2126833-4	L2126833-5	L2126833-6	L2126833-7	L2126833-8
				Sample Date	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18	04-JUL-18
				Sample ID	BH14-SS1 AND SS2	BH7-SS1 AND SS2	BH13-SS1 AND SS2	BH16-SS1 AND SS2	BH4-SS1 AND SS2	BH9-SS1 AND SS2	BH6-SS1 AND SS2	BH3-SS1 AND SS2
Chromium, Hexavalent	ug/g	0.66	10		<0.20	0.43	0.21	<0.20	<0.20	<0.20	<0.20	<0.20

Guide Limit #1: T1-Soil-Res/Park/Inst/Ind/Com/Commu Property Use
Guide Limit #2: T3-Soil-Res/Park/Inst. Property Use (Fine)

Detection Limit for result exceeds Guideline Limit. Assessment against Guideline Limit cannot be made.
Analytical result for this parameter exceeds Guide Limits listed. See Summary of Guideline Exceedances.

* Please refer to the Reference Information section for an explanation of any qualifiers noted.

Reference Information

Qualifiers for Individual Parameters Listed:

Qualifier	Description
SAR:L	SAR is incalculable due to Ca and Mg below DL. Lowest possible SAR is reported as minimum value.
SAR:M	Reported SAR represents a maximum value. Actual SAR may be lower if both Ca and Mg were detectable.

Methods Listed (if applicable):

ALS Test Code	Matrix	Test Description	Method Reference**
B-HWS-R511-WT	Soil	Boron-HWE-O.Reg 153/04 (July 2011)	HW EXTR, EPA 6010B

A dried solid sample is extracted with calcium chloride, the sample undergoes a heating process. After cooling the sample is filtered and analyzed by ICP/OES.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

CL-R511-WT	Soil	Chloride-O.Reg 153/04 (July 2011)	EPA 300.0

5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

CN-WAD-R511-WT	Soil	Cyanide (WAD)-O.Reg 153/04 (July 2011)	MOE 3015/APHA 4500CN I-WAD

The sample is extracted with a strong base for 16 hours, and then filtered. The filtrate is then distilled where the cyanide is converted to cyanogen chloride by reacting with chloramine-T, the cyanogen chloride then reacts with a combination of barbituric acid and isonicotinic acid to form a highly colored complex.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

CR-CR6-IC-WT	Soil	Hexavalent Chromium in Soil	SW846 3060A/7199

This analysis is carried out using procedures adapted from "Test Methods for Evaluating Solid Waste" SW-846, Method 7199, published by the United States Environmental Protection Agency (EPA). The procedure involves analysis for chromium (VI) by ion chromatography using diphenylcarbazide in a sulphuric acid solution.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

EC-WT	Soil	Conductivity (EC)	MOEE E3138

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

HG-200.2-CVAA-WT	Soil	Mercury in Soil by CVAAS	EPA 200.2/1631E (mod)

Soil samples are digested with nitric and hydrochloric acids, followed by analysis by CVAAS.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

MET-200.2-CCMS-WT	Soil	Metals in Soil by CRC ICPMS	EPA 200.2/6020A (mod)

This method uses a heated strong acid digestion with HNO3 and HCl and is intended to liberate metals that may be environmentally available. Silicate minerals are not solubilized. Dependent on sample matrix, some metals may be only partially recovered, including Al, Ba, Be, Cr, Sr, Ti, Tl, V, W, and Zr. Volatile forms of sulfur (including sulfide) may not be captured, as they may be lost during sampling, storage, or digestion. Analysis is by Collision/Reaction Cell ICPMS.

Reference Information

Methods Listed (if applicable):

ALS Test Code	Matrix	Test Description	Method Reference**

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011), unless a subset of the Analytical Test Group (ATG) has been requested (the Protocol states that all analytes in an ATG must be reported).

MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580

This analysis is carried out in accordance with the procedure described in the "APHA" method 2580 "Oxidation-Reduction Potential" 2012. Samples are extracted at a fixed ratio with DI water. Results are reported as observed oxidation-reduction potential of the platinum metal-reference electrode employed, in mV.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	MOECC E3138

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

SAR-R511-WT	Soil	SAR-O.Reg 153/04 (July 2011)	SW846 6010C

A dried, disaggregated solid sample is extracted with deionized water, the aqueous extract is separated from the solid, acidified and then analyzed using a ICP/OES. The concentrations of Na, Ca and Mg are reported as per CALA requirements for calculated parameters. These individual parameters are not for comparison to any guideline.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

SO4-WT	Soil	Sulphate	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

SULPHIDE-WT	Soil	Sulphide, Acid Volatile	APHA 4500S2J

This analysis is carried out in accordance with the method described in APHA 4500 S2-J. Hydrochloric acid is added to sediment samples within a purge and trap system. The evolved hydrogen sulphide (H2S) is carried into a basic solution by inert gas. The acid volatile sulfide is then determined colourimetrically.

**ALS test methods may incorporate modifications from specified reference methods to improve performance.

Chain of Custody Numbers:

17-619445

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Reference Information

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to fitness for a particular purpose, or non-infringement. ALS assumes no responsibility for errors or omissions in the information.

APPENDIX E

GEOTECHNICAL LABORATORY TESTING RESULTS

LANDTEK LIMITED

Atterberg Limits

Project:	Geotechnical Investigation Report	Sampled By:	R.F.
Location:	Scenic Drive, Hamilton	Tested By:	V.H.
Job No.:	18211	Sample Date:	June 25, 2018
Borehole No:	BH4-SS1	Depth (m):	0.3-1.5
Report Date:	August 3, 2018	Test Date:	July 28, 2018

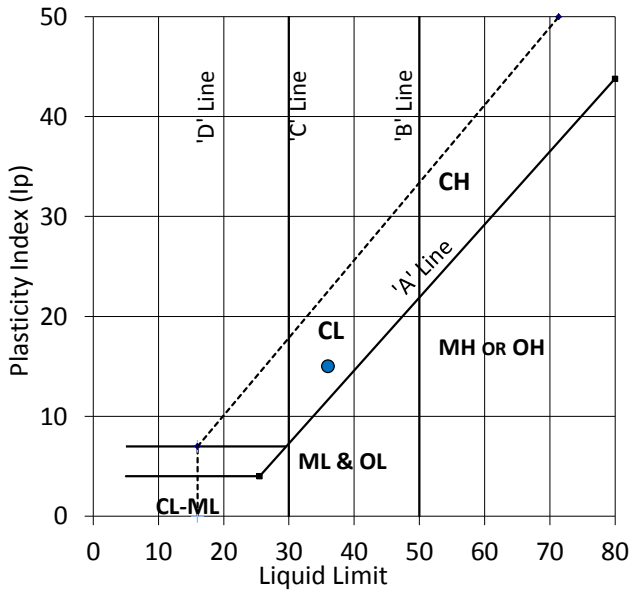
Liquid Limit Test

Trial	A	B	C
No. of Blows	16	24	34
Tare Number	NP18	NP32	NP26
Wt. of Tare, g	13.70	13.70	13.60
Wt. Wet Soil + Tare, g	34.80	31.30	29.40
Wt. Dry Soil + Tare, g	29.00	26.70	25.40
Wt. of Water, g	5.80	4.60	4.00
Wt. of Dry Soil, g	15.30	13.00	11.80
Moisture Content (%)	37.9	35.4	33.9

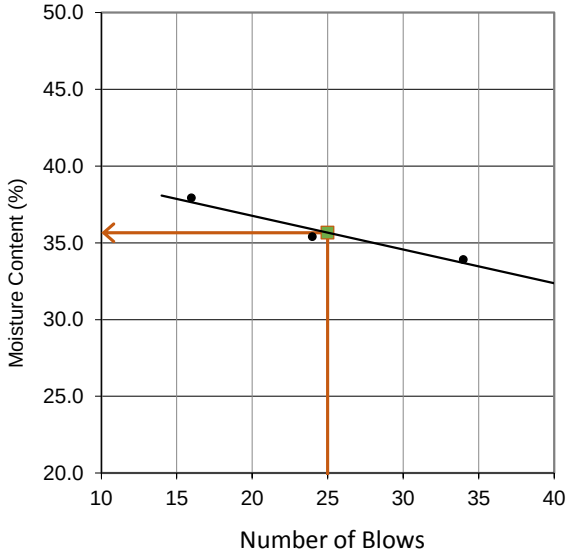
Plastic Limit Test

Trial	A	B
Tare Number	NP19	NP46
Wt. of Tare, g	13.50	13.60
Wt. Wet Soil + Tare, g	29.30	30.20
Wt. Dry Soil + Tare, g	27.10	26.80
Wt. of Water, g	2.20	3.40
Wt. of Dry Soil, g	13.60	13.20
Moisture Content (%)	16.2	25.8

Plasticity Chart



Liquid Limit



USCS Symbol: CL

Soil Description: Medium Plasticity

Liquid Limit (%): 36

Plastic Limit (%): 21

Plasticity Index (%): 15

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the results is provided only on written request.

LANDTEK LIMITED

Atterberg Limits

Project:	Geotechnical Investigation Report	Sampled By:	R.F.
Location:	Scenic Drive, Hamilton	Tested By:	V.H.
Job No.:	18211	Sample Date:	June 25, 2018
Borehole No:	BH6-SS2	Depth (m):	1.5-2.0
Report Date:	August 3, 2018	Test Date:	July 27, 2018

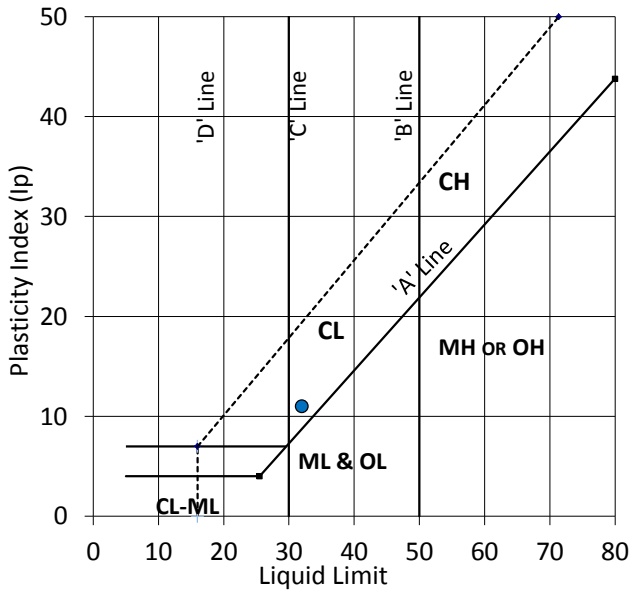
Liquid Limit Test

Trial	A	B	C
No. of Blows	16	23	35
Tare Number	NP21	NP43	NP35
Wt. of Tare, g	14.00	14.50	13.90
Wt. Wet Soil + Tare, g	31.30	34.20	34.70
Wt. Dry Soil + Tare, g	26.90	29.60	29.80
Wt. of Water, g	4.40	4.60	4.90
Wt. of Dry Soil, g	12.90	15.10	15.90
Moisture Content (%)	34.1	30.5	30.8

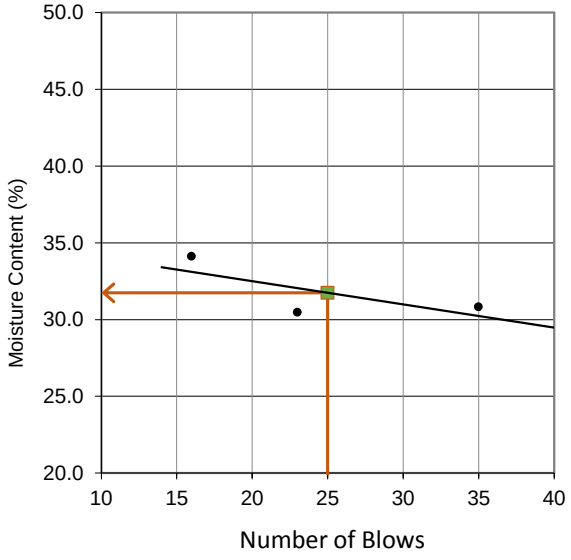
Plastic Limit Test

Trial	A	B
Tare Number	NP6	NP26
Wt. of Tare, g	13.30	14.10
Wt. Wet Soil + Tare, g	36.20	33.60
Wt. Dry Soil + Tare, g	31.50	30.80
Wt. of Water, g	4.70	2.80
Wt. of Dry Soil, g	18.20	16.70
Moisture Content (%)	25.8	16.8

Plasticity Chart



Liquid Limit



USCS Symbol: CL

Soil Description: Medium Plasticity

Liquid Limit (%): 32

Plastic Limit (%): 21

Plasticity Index (%): 11

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the results is provided only on written request.

LANDTEK LIMITED

Atterberg Limits

Project:	Geotechnical Investigation Report	Sampled By:	R.F.
Location:	Scenic Drive, Hamilton	Tested By:	V.H.
Job No.:	18211	Sample Date:	June 25, 2018
Borehole No:	BH8-SS5	Depth (m):	4.5-5.0
Report Date:	August 3, 2018	Test Date:	July 27, 2018

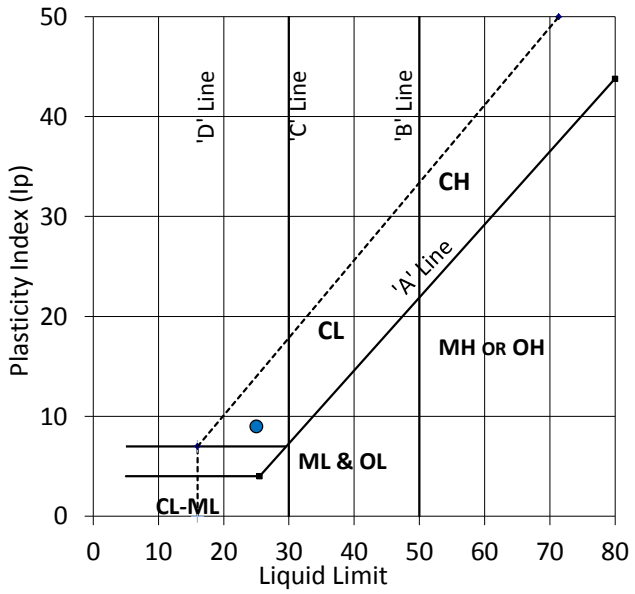
Liquid Limit Test

Trial	A	B	C
No. of Blows	18	21	33
Tare Number	NP43	NP35	NP32
Wt. of Tare, g	14.40	13.80	14.00
Wt. Wet Soil + Tare, g	33.00	32.70	37.70
Wt. Dry Soil + Tare, g	29.40	29.00	32.80
Wt. of Water, g	3.60	3.70	4.90
Wt. of Dry Soil, g	15.00	15.20	18.80
Moisture Content (%)	24.0	24.3	26.1

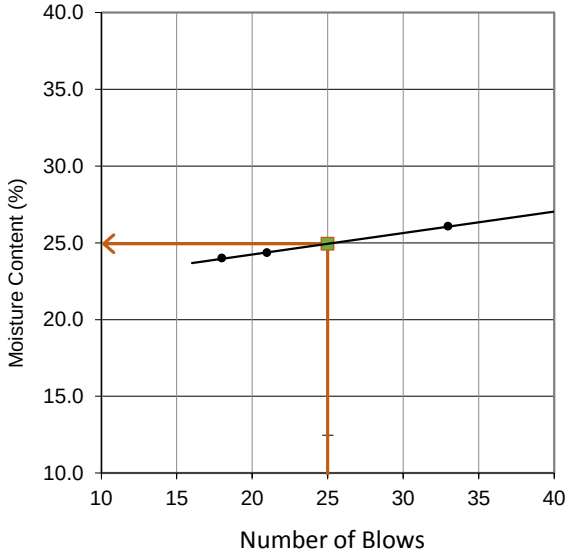
Plastic Limit Test

Trial	A	B
Tare Number	NP18	NP21
Wt. of Tare, g	13.70	14.50
Wt. Wet Soil + Tare, g	34.80	39.00
Wt. Dry Soil + Tare, g	31.90	35.60
Wt. of Water, g	2.90	3.40
Wt. of Dry Soil, g	18.20	21.10
Moisture Content (%)	15.9	16.1

Plasticity Chart



Liquid Limit



USCS Symbol: CL

Soil Description: Low Plasticity

Liquid Limit (%): 25

Plastic Limit (%): 16

Plasticity Index (%): 9

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the results is provided only on written request.

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1895782 (6000)	SAMPLE NUMBER	2
PROJECT NAME	Landtek Ltd/Lab Testing/Miss	SAMPLE DEPTH, m	6.25-6.39
BOREHOLE NUMBER	2	DATE:	2018/09/10

TEST CONDITIONS

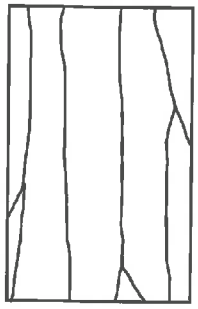
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.84

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.72	WATER CONTENT, (specimen) %	0.40
SAMPLE DIAMETER, cm	6.36	UNIT WEIGHT, kN/m^3	25.14
SAMPLE AREA, cm^2	31.81	DRY UNIT WT., kN/m^3	25.04
SAMPLE VOLUME, cm^3	372.77	SPECIFIC GRAVITY	-
WET WEIGHT, g	956.14	VOID RATIO	-
DRY WEIGHT, g	952.33		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	74.8

REMARKS:

L/D Ratio not in accordance with ASTM Standard

Checked By:

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date Sept. 11, 2018
Project 1895782(6000)

Golder Associates

Drawn Frank
Chkd.

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1895782 (6000)	SAMPLE NUMBER	5
PROJECT NAME	Landtek Ltd/Lab Testing/Miss	SAMPLE DEPTH, m	3.2-3.3
BOREHOLE NUMBER	12	DATE:	2018/09/10

TEST CONDITIONS

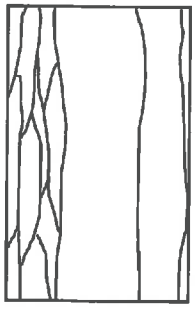
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.83

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.59	WATER CONTENT, (specimen) %	0.60
SAMPLE DIAMETER, cm	6.35	UNIT WEIGHT, kN/m^3	23.91
SAMPLE AREA, cm^2	31.68	DRY UNIT WT., kN/m^3	23.77
SAMPLE VOLUME, cm^3	367.26	SPECIFIC GRAVITY	-
WET WEIGHT, g	895.69	VOID RATIO	-
DRY WEIGHT, g	890.35		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	76.5

REMARKS:

L/D Ratio not in accordance with ASTM Standard

Checked By:

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date Sept. 11, 2018
Project 1895782(6000)

Golder Associates

Drawn Frank
Chkd.

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1895782 (6000)	SAMPLE NUMBER	7
PROJECT NAME	Landtek Ltd/Lab Testing/Miss	SAMPLE DEPTH, m	4.0-4.2
BOREHOLE NUMBER	15	DATE:	2018/09/10

TEST CONDITIONS

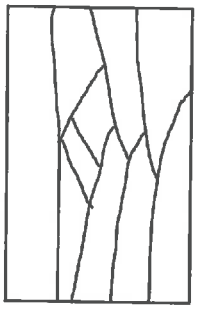
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.29

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.45	WATER CONTENT, (specimen) %	0.30
SAMPLE DIAMETER, cm	6.32	UNIT WEIGHT, kN/m^3	25.48
SAMPLE AREA, cm^2	31.36	DRY UNIT WT., kN/m^3	25.40
SAMPLE VOLUME, cm^3	453.13	SPECIFIC GRAVITY	-
WET WEIGHT, g	1177.78	VOID RATIO	-
DRY WEIGHT, g	1174.26		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	92.2

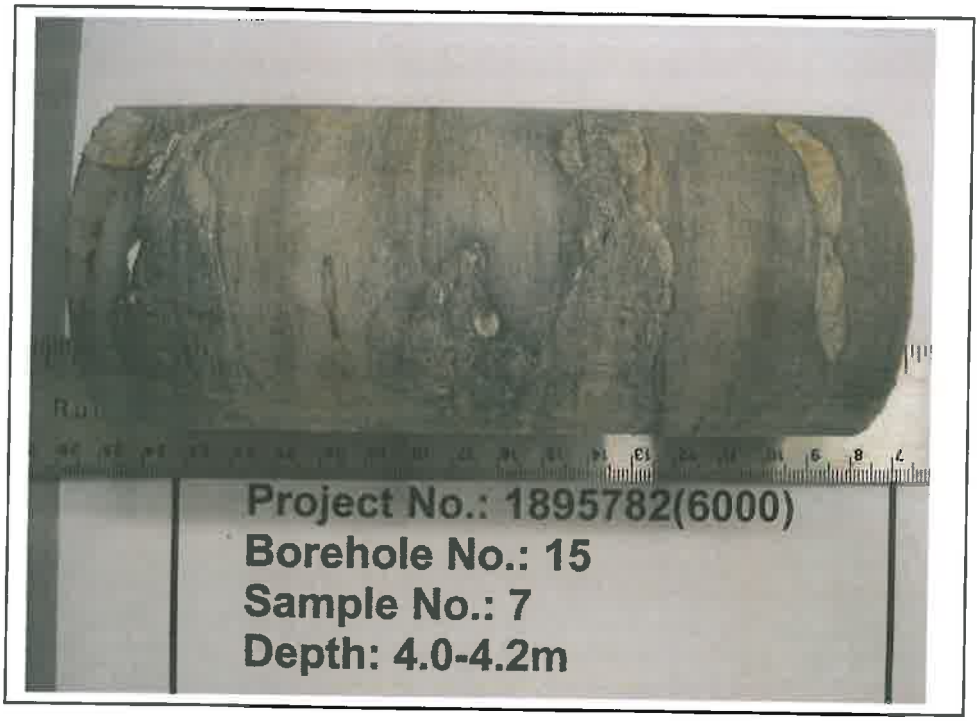
REMARKS:

Checked By:

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date Sept. 11, 2018
Project 1895782(6000)

Golder Associates

Drawn Frank
Chkd.

GOLDER

POINT LOAD TEST ON ROCK SAMPLES

PROJECT NO. 1895782 (6000)

TITLE Landtek Ltd/Lab Testing/Miss

DATE September, 2018

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm) (MPa)**	**Approx. [1] UCS (MPa)**
2	1	3.81-3.92	A	25.65	63.24	45.45	15,680.00	14.86	7.197	-	6.895	145
2	1	3.81-3.92	D	77.68	59.80	-	23,960.00	22.71	-	6.352	6.884	169
2	3	6.75-6.86	A	32.03	63.50	50.89	15,880.00	15.05	5.813	-	5.860	135
2	3	6.75-6.86	D	75.21	59.58	-	29,240.00	27.72	-	7.809	8.450	207
12	4	2.7-2.8	A	31.79	63.47	50.69	18,020.00	17.08	6.650	-	6.691	154
12	4	2.7-2.8	D	78.29	57.82	-	34,840.00	33.03	-	9.879	10.547	258
12	6	5.85-6.13	A	28.83	63.50	48.28	19,260.00	18.26	7.833	-	7.711	177
12	6	5.85-6.13	D	68.82	59.60	-	20,180.00	19.13	-	5.386	5.829	143
15	8	5.82-6.12	A	29.61	63.41	48.89	19,140.00	18.14	7.590	-	7.514	173
15	8	5.82-6.12	D	68.92	60.77	-	2,220.00	2.10	-	0.570	0.622	15
15	9	6.45-6.58	A	28.49	63.44	47.97	11,120.00	10.54	4.581	-	4.496	103
15	9	6.45-6.58	D	83.31	59.89	-	14,860.00	14.09	-	3.928	4.260	104
15	10	8.86-8.99	A	30.64	63.54	49.79	19,560.00	18.54	7.481	-	7.466	172
15	10	8.86-8.99	D	97.17	60.17	-	18,320.00	17.37	-	4.797	5.214	128

[1] Is_{50} x 21/23/24.5 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

[2] Actual distance between point load cones at time of failure.

Checked By:

Golder Associates

APPENDIX F

SITE PHOTOGRAPHS



Photograph #1
View of northwest section of site (close to intersection of Scenic Drive and Sanatorium Road)



Photograph #2
View of the Scenic Drive

LANDTEK LIMITED	
Project No. 18211	Date: June 2018
Geotechnical Investigation Report Scenic Drive, Hamilton, Ontario	
Title:	Photographs of Site Conditions



Photograph #3
General view of site



Photograph #4
View of existing culvert, located adjacent to BH/MW7

LANDTEK LIMITED	
Project No. 18211	Date: June 2018
Geotechnical Investigation Report Scenic Drive, Hamilton, Ontario	
Title:	Photographs of Site Conditions



Photograph #5
View of existing building located adjacent to the woodlot/ ESA Block



Photograph #6
View of raised cliff, located adjacent to the Bruce Side Trail

LANDTEK LIMITED	
Project No. 18211	Date: June 2018
Geotechnical Investigation Report Scenic Drive, Hamilton, Ontario	
Title:	Photographs of Site Conditions



Photograph #7
View of raised cliff edge, located adjacent to the Bruce Side Trail



Photograph #8
View of the existing Bruce Side Trail

LANDTEK LIMITED	
Project No. 18211	Date: June 2018
Geotechnical Investigation Report Scenic Drive, Hamilton, Ontario	
Title:	Photographs of Site Conditions



Photograph #9
View of the Woodlot



Photograph #10
View of the site, facing south towards Scenic Drive

LANDTEK LIMITED	
Project No. 18211	Date: June 2018
Geotechnical Investigation Report Scenic Drive, Hamilton, Ontario	
Title:	Photographs of Site Conditions

APPENDIX C:

Fluvial Geomorphology Assessment (GEO Morphix Ltd)

GEO Morphix Ltd.

Head Office
36 Main St. N., PO Box 205
Campbellville, ON, Canada L0P 1B0

T 416.920.0926

Ottawa Office
PO Box 336 Woodlawn PO
Dunrobin, Ontario, Canada K0A 1T0

T 613.979.7303

July 23, 2020

Valery (Chedoke Browlands) Developments Inc.
2140 King Street East
Hamilton, Ontario
L8K 1W6

Attention: Ted Valeri

Re: Meander Belt Width Delineation Update
Tributary of Chedoke Creek (Scenic Drive and Sanatorium Road)
City of Hamilton, Ontario
GEO Morphix Project No. 19110

This memo summarizes the meander belt width assessment completed for a section of the Chedoke Creek Tributary at Scenic Drive and Sanatorium Road in the City of Hamilton, Ontario. This work, in part, provides support in the definition of environmental constraints associated with future development on site.

To accommodate the development, a section of the Chedoke Creek Tributary is proposed for re-alignment between Scenic Drive and the Niagara Escarpment. Parish Geomorphic (2009) previously completed an assessment of the tributary and suggested a meander belt width of 42.9 m for the feature. GEO Morphix Ltd. was retained to review the Parish (2009) study and provide an update to the meander belt width in the context of available topographic survey, detailed field observations, and proposed future development on site.

In the case of realignment, the meander belt width is a product of the bankfull characteristics of a channel. If a channel is to be realigned, the meander belt width would need to be adjusted to account for the updated or proposed channel configuration. To refine the meander belt width for this section of the Chedoke Creek Tributary, we have reviewed various background data and reporting, completed site reconnaissance to document existing watercourse characteristics, and updated the meander belt width assessment at a reach scale based on existing information and newly collected field observations.

We have determined a range of meander belt widths for the post-restoration condition by defining a potential bankfull channel based on the following assumptions:

- Valley gradient of 0.9% based on the valley gradient determined through our detailed assessment completed November 29, 2019
- Sinuosity of 1.1 and width to depth ratio of 10:1, which is a stable configuration and representative of the system
- 2-year flow of 1.1 m^3/s and 2.6 m^3/s, determined through our detailed assessment completed November 29, 2019

Dillon Consulting summarized previously calculated 2-year flows for the channel in their report titled City of Hamilton Sanatorium Road Realignment Flood and Erosion Impact Assessment (June 2010). A significant range of 2-year flows were determined through various studies (values ranging from 1.7 m^3/s to 8.4 m^3/s). Dillon (2010) also simulated a maximum flow for the channel over a 16-year period, which resulted in a discharge of 10.6 m^3/s. Given that the maximum flow determined by Dillon is similar to the largest reported 2-year flow (8.4 m^3/s) in the Dillon report (2010), we would suggest that the 2-year flow is likely closer to the lower range of 1.7 m^3/s rather than 8.4 m^3/s.

To verify the 2-year flow information outlined by Dillon (2010), we completed a detailed channel survey to document existing bankfull dimensions. Based on our detailed geomorphological field assessment from November 29, 2020, we determined a bankfull discharge of 1.1 m^3/s for the channel. The bankfull channel indicators observed on site suggested a lower bankfull width and depth that resulted in the 1.1 m^3/s discharge. To be conservative, we also determined a secondary discharge by extending the bankfull channel width to the location where flow would spill into the floodplain. This resulted in a discharge of 2.6 m^3/s. Both discharge values still fall within the lower range of values outlined by Dillon (2010). Although the bankfull discharge of a channel is generally lower than the 2-year flow, it is what would normally be implemented for a channel design.

Based on the measured gradient from our detailed survey and the assigned sinuosity, width and depth for the restored channel were back-calculated using the 1.1 m^3/s and 2.6 m^3/s bankfull discharge determined from our detailed survey. The meander belt width was then determined using the modified Williams (1986) model and the back-calculated channel geometry measurements. The empirical relations from Williams (1986) were modified to include channel width and a 20% factor of safety. The empirical relationships are outlined below:

$$B_w = 18A^{0.65} + W_b \times (1.2) \qquad \text{[Eq. 1]}$$

$$B_w = 4.3{W_b}^{1.12} + W_b \times (1.2) \qquad \text{[Eq. 2]}$$

Where, B_w is meander belt width (m); A is bankfull cross-sectional area (m^2); and W_b is bankfull channel width (m).

The meander belt widths for each discharge scenario are outlined in **Table 1** below. The reported numbers include a 20 percent factor of safety.

Table 1. Meander Belt Widths for Realigned Tributary of Chedoke Creek

Discharge (m^3/s) Scenario	Meander Belt Width (m) Williams – Area Method (1986) [Eq. 1]	Meander Belt Width (m) Williams – Width Method (1986) [Eq. 2]
1.1	23.9	21.4
2.6	35.8	30.5

There are a range of meander belt widths provided as a result of different discharges and empirical models. Generally, most of the determined meander belt widths in **Table 1** fall within or around 30 m. Given that the channel can be designed to accommodate a range of discharges, we are confident that the meander belt width for the realigned channel would not exceed 30 m.

We trust this memo meets your current requirements. Should you have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,

Paul Villard Ph.D., P.Geo., CAN-CISEC, EP, CERP
Director, Principal Geomorphologist

Kat Woodrow, M.Sc.
Environmental Scientist

References

Dillon Consulting Limited, June 2010. City of Hamilton Sanatorium Road Realignment Flood and Erosion Impact Assessment. Final Report Submitted to City of Hamilton.

Parish Geomorphic, June 2009. Chedoke Creek Flooding and Erosion Control Impact Assessment, Stage 1: Analysis of Existing Conditions. Final Report Submitted to Dillon Consulting Ltd.

Williams, G.P., 1986. River meanders and channel size. Journal of Hydrology, 88 (1-2): 147-164.

Technical Design Brief: Tributary of Chedoke Creek Realignment

City of Hamilton, Ontario



Prepared for:
Valery Homes
2140 King Street East
Hamilton, ON
L8K 1W6

July 24, 2020
PN19110

GEO MORPHIX
Geomorphology
Earth Science
Observations

Report Prepared by:	GEO Morphix Ltd. 36 Main Street N. Campbellville, ON L0P 1B0
Report Title:	Technical Design Brief: Tributary of Chedoke Creek Realignment, City of Hamilton
Project Number:	PN19110
Status:	Final
Version:	1.0
First Submission Date:	July 24, 2020
Revision Date:	--
Prepared by:	Ben Miller, B.Sc., CAN-CISEC
Approved by:	Paul Villard, Ph.D., P.Geo., CAN-CISEC, EP, CERP
Approval Date:	July 24, 2020

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Appendices

Appendix A Site Map

Appendix B Photographic Record

Appendix C Field Observations

Appendix D Detailed Assessment Summary

Appendix E Meander Belt Width Delineation Memo

Appendix F Detailed Design Drawings

1 Introduction

This design brief provides design recommendations for a realignment and enhancement of a tributary of Chedoke Creek located to the northwest of the Scenic Drive and Sanitorium Road intersection within the City of Hamilton, Ontario. The existing channel will be realigned, which provides an opportunity to replace the existing degraded channel, with a naturalized watercourse that offers significant improvements to channel form and function. The existing morphologically-limited channel will be replaced with a riffle and pool system, with cross sectional dimensions closer to that of a natural watercourse conveying similar flows. The proposed realignment serves to improve channel form and function and enhance aquatic habitat. This technical design brief provides additional insight into the design and is to accompany the detailed design drawings.

Wetland elements will be incorporated in the design to provide a functionally diverse floodplain. The objective of these is to passively store and discharge subsurface flow over longer attenuated periods. These wetland features will also improve water quality through infiltration and retention processes. The channel design serves to improve channel form and function, aquatic habitat, and habitat variability, increase wetted width and low flow habitat, and provide greater substrate and morphological variability.

This report provides:

- Summary of existing channel conditions, including a detailed survey to estimate bankfull geometry
- Description of the natural channel design characteristics and geometry
- Hydraulic sizing of the channel materials
- Recommendations for design implementation including construction timing, and best management practices
- Description of a post-construction monitoring plan

This design brief is provided to facilitate review of the design. The following report outlines the current geomorphological condition of the Tributary of Chedoke Creek, design considerations, and technical details associated with channel sizing and restoration. It also provides recommendations for implementation and monitoring of the proposed design.

2 Existing Conditions

2.1 Field Observations

The upstream extent of the channel assessment began at Scenic Drive where the channel then flows in a north-easterly direction before being piped under Sanatorium Road towards the Niagara Escarpment. The assessed reach consisted of a single thread channel in a partially confined valley with low sinuosity and a moderate gradient. The reach was entrenched at the upstream extent and an old pedestrian crossing was noted at the downstream extent causing minor channel blockage. The riparian zone was mostly comprised of established trees and formed a continuous buffer. A moderate amount of woody debris was found within the channel. The average bankfull width and depth were

2.77 m and 0.32 m. Bank erosion was observed along approximately 75% of the banks with undercuts up to 0.15 m present. Leaning trees and exposed tree roots were also observed. The upstream portion of the reach was dominated by runs and riffle-pool sequences were only noted downstream of the pedestrian crossing. Bed substrate consisted of clay, silt, and sand except at the downstream extent where the bedrock was exposed, and gravel and cobbles were present.

Most of the study area is located within the Iroquois Plains physiographic region with the southern portion bordering the Haldimand Clay Plains region (OGS, 2010). The surficial geology consists of clay to silt-textured till derived from glaciolacustrine deposits or shale and the northern extent of the study site borders paleozoic bedrock (OGS, 2003).

Rapid geomorphological assessments of the Tributary of Chedoke Creek were completed on November 29, 2019. To provide context, a photographic record is provided in **Appendix B** and field notes are included in **Appendix C**. The rapid assessments including the following observations:

- Characterization of stream form, process, and evolution using the Rapid Geomorphological Assessment (RGA) (MOE, 2003, VANR, 2007)
- Assessment of the ecological function of the watercourse using the Rapid Stream Assessment Technique (RSAT) (Galli, 1996)
- Stream classification following a modified Downs (1995) and a modified Brierley and Fryirs (2005) River Styles Classification approach
- Reach-scale habitat sketch maps based on Newson and Newson (2000) outlining channel substrate, flow behaviour, geomorphological units, and riparian vegetation on the day
- Instream estimates of bankfull channel dimensions
- Bed and bank material composition and structure
- Georeferenced photographs to document the location of all observed erosion and infrastructure

The RGA evaluates systematic adjustments characterized as degradation, aggradation, widening, and planimetric form adjustment at the reach scale. The RGA method relies on the absence or presence of these indicators to evaluate the systematic adjustments in streams associated with natural causes or human activities. Systematic adjustments typically result in changes to the floodplain, channel or valley characteristics. The end result of the RGA is to produce a score, or stability index, which evaluates the degree to which a stream has departed from the equilibrium condition. A stream with a score of less than 0.20 is *in regime*, indicating minimal changes to its shape or processes over time. A score of 0.21 to 0.40 indicates that a stream is *in transition* or *stressed* and is experiencing major change to process and form outside the natural range of variability. A score of greater than 0.41 indicates that a stream is in extreme adjustment, likely exhibiting a new stream type and will continue to adjust to the point of returning to equilibrium, or is moving toward a new equilibrium (MOE, 2003).

The Downs (1995) model of channel evolution is a method used to evaluate the magnitude and potential for channel instability. This model uses physical indicators of systematic adjustment including channel, bank and bar morphology and stability to classify the type of channel evolution. By classifying channels using this model, the nature of fluvial and hillslope processes that are working to change the system can be inferred. Channels are classified as varying degrees of stable,

depositional, migrating laterally, enlarging, and experiencing various types of erosion (Downs, 1995; Simon and Downs, 1995).

The assessed tributary had an RGA score of 0.19, indicating that the channel is *in regime*. The dominant modes of adjustment were degradation and widening. These were characterized by exposed pipes, head cutting, exposed tree roots and occurrence of large organic debris. The Down's model classified this channel as 'E' – enlarging, indicating a consistent increase in channel width and depth.

The Rapid Stream Assessment Technique (RSAT) was also employed to provide a broader view of the system and consider the ecological functioning of the watercourse (Galli, 1996). Observations were made of channel stability, channel scouring or sediment deposition, instream and riparian habitats, and water quality. The RSAT score ranks the channel as maintaining a *poor* (<13), *fair* (13-24), *good* (25-34) or *excellent* (35-42) degree of stream health. The RSAT was completed and produced a score of 22, indicating the channel is in fair health. Details are provided in **Appendix C**.

2.2 Detailed Assessment

Following the rapid geomorphological assessment, a detailed assessment was completed for the Tributary of Chedoke Creek on November 29, 2019 to determine average bankfull channel characteristics, including cross-sectional geometry and hydraulics, for informing any potential restoration activities. The following activities were completed:

- Longitudinal profile along the channel bed to determine slope
- Eight representative cross-sectional surveys of the watercourse to determine average channel dimensions
- Detailed instream measurements at each cross-section including bankfull channel geometry, riparian conditions, bank material, bank height/angle, and bank root density
- Bed material / substrate sampling

Eight representative cross sections were surveyed, and channel measurements were then used to calculate bankfull flow characteristics such as discharge, average velocity, and erosion or sediment transport sensitivity. Measured and computed values are presented in **Table 1**. As part of the detailed assessment, a longitudinal survey of the bed was completed to determine slope and a composite sample was taken to characterize bed materials. Average bankfull width and depth were 2.79 m and 0.32 m, respectively. The gradient of the bankfull channel was documented to be 0.89%. Bankfull discharge was back-calculated to be 1.11 m^3/s. To be conservative, we also determined a secondary discharge by extending the bankfull channel width to the location where flow would spill into the floodplain. This resulted in a discharge of 2.57 m^3/s. A summary of the detailed survey results are provided in **Table 1** and a summary of the detailed assessment is provided in **Appendix D**.

Table 1: Measured and computed channel parameters

Channel parameter	Bankfull Indicator	Floodplain
Average bankfull channel width (m)	2.79	4.40
Average bankfull channel depth (m)	0.32	0.40
Bankfull channel gradient (%)	0.89	0.89
D_{50} (mm)	< 2.0	< 2.0
Manning's n roughness coefficient	0.040	0.040
Calculated		
Bankfull discharge (m^3/s) *	1.11	2.57
Average bankfull velocity (m/s)	1.26	1.46

* Based on Manning's equation

3 Natural Channel Design

3.1 Design Objectives

Given this section of the Tributary of Chedoke Creek is proposed to be restored and realigned, there is opportunity to replace the existing morphologically-limited channel with a dynamically stable channel containing a naturalized riffle and pool system, with cross sectional dimensions closer to that of a naturalized watercourse conveying similar flows. One goal of the natural channel design is to replace the existing degraded and previously disturbed channel with a watercourse that will offer significant improvements to channel form and function per unit length.

This section of the Tributary of Chedoke Creek was characterized with a perennial flow regime. To maintain and enhance these functions, the design serves to provide good communication with the floodplain, as well as diversity in channel and floodplain morphology. As such, online and offline wetland features will be constructed throughout the floodplain. These features enhance terrestrial habitat by increasing diversity and providing a more natural floodplain form. They also provide functional benefits by storing and discharging water over longer, attenuated periods.

The channel realignment and naturalization provide opportunities for improved riparian conditions and a well-developed bankfull channel with morphological variability. Improvement in morphology and function will provide additional benefits to the sediment balance and substrate variability, floodplain storage, vegetation communities, terrestrial habitat features, edge impacts, fish passage and water quality. From a habitat perspective, the important contributions of the watercourse include the provision of seasonal habitat, organic inputs to the system, provision of a more complex corridor system with elements that have a wide range of hydroperiods, and aquatic and terrestrial habitat elements.

The primary objectives of the design are to:

- Improve the physical form of the channel, including planform and instream characteristics
- Improve the function of the channel and promote interaction with the floodplain

- Improve water quality by extending detention of water through offline wetland features
- Enhance aquatic habitat through the provision of a morphologically diverse channel with spatially varied flows
- Improve riparian habitat by installing woody plantings

3.2 Channel Planform

The initial channel planform layout was created using the modelled radius of curvature value (*Rc*) as a guide. The radius of curvature (*Rc*) of meanders can be used to evaluate channel stability. For example, stable meanders typically exhibit larger *Rc* values as opposed to lower values that indicate increased channel bank erosion and avulsion. Bankfull width is often an appropriate indicator for this instability. Hickin and Nanson (1983) note that channel avulsions are common when meander *Rc* is approximately 1-2 times the channel bankfull width. For larger *Rc* (e.g., >5), the upstream limb of the meander will migrate more rapidly than the downstream limb (Hooke, 1975). Williams (1986) was used to derive values for the channel radius of curvature, using the following equation (Eq. 1):

$$Rc = 2.43 \times w \quad \text{[Eq. 1]}$$

where *Rc* is the radius of curvature and *w* is the average bankfull width.

Empirical models derived by Hey and Thorne (1986) were followed to determine riffle spacing. Hey and Thorne's (1986) modelled values are often applied in larger watercourses. As such, multiple methods (Eq. 2-4) were considered in order to provide a range of riffle spacing values. These are:

$$Z = 6.31 \times w \quad \text{[Eq. 2]}$$

$$Z = 9.1186 \times w^{0.8846} \quad \text{[Eq. 3]}$$

$$Z = 7.36 \times w^{0.896} \times S^{-0.03} \quad \text{[Eq. 4]}$$

where *Z* represents riffle spacing.

Stream power and unit stream power were calculated as a function of bankfull discharge and channel gradient (Eq. 5-6). Stream power values are important to determine the need for mitigating channel bank and bed erosion. Stream power is given by:

$$\Omega = \rho \times g \times d \times S \quad \text{[Eq. 5]}$$

where ρ is the density of water (kg/m^3), *g* is the acceleration due to gravity (m/s^2), and *Q* and *S* are discharge (m^3/s) and channel gradient, respectively.

Stream power per unit width (Eq. 6), is given by:

$$\omega = \frac{\Omega}{w} \quad \text{[Eq. 6]}$$

where as before, Ω and *w* are stream power and bankfull width, respectively.

The final channel planform was established through an iterative process. First, a cross section with defined bankfull geometry was developed to calculate parameters for the planform (i.e., radius of curvature). The cross section was then further refined, and riffle and pool lengths were determined based on channel gradient.

3.3 Bankfull Channel

The recommended restoration design focuses on a riffle-pool typology, which will provide significant improvements to not only the channel as it essentially replicates a natural system, but also to aquatic habitat. The proposed detailed design drawings are included in **Appendix E** and design elements are described in further detail below. When it is assessed to be an appropriate channel type, a riffle-pool system offers numerous benefits:

- Channel bed relief for flow variability
- Water aeration in riffle sections
- Relatively quiescent flows in pool sections to provide refuge for fish during high flows
- Increased depths in pools to provide relatively cool water
- In-channel energy dissipation

Channel design dimensions are determined by bankfull discharge, as this represents what is generally referred to as the "*channel-forming discharge*" or the "*dominant discharge*". Several methods can be applied to select an appropriate bankfull discharge. Back calculation of discharge from a reference reach along with support from hydrological modelling is usually the most appropriate. Based on our detailed assessment and channel survey from November, we determined a bankfull discharge of 1.11 m^3/s based on the bankfull channel indicators observed on site. To be conservative, a secondary discharge was determined by extending the bankfull channel width to the location where flow would spill into the floodplain. This resulted in a discharge of 2.57 m^3/s. As such, to be slightly more conservative while considering the observed bankfull channel indicators, a bankfull discharge of 2.0 m^3/s was defined to be implemented for the channel design.

A simple Manning's approach was used to iteratively back-calculate bankfull dimensions for the proposed channel. Since pools are designed to contain ineffective space, this model over-predicts the amount of discharge that they convey. As such, the modelled values for the riffles give a better prediction of the channel's capacity. Riffle and pool geometries, as well as anticipated bankfull conditions for the proposed channel, are provided in **Table 2**.

Table 2: Bankfull parameters of the proposed channel

Channel parameter	Riffle	Pool
Bankfull width (m)†	3.15	3.65
Average bankfull depth (m)†	0.32	0.38
Maximum bankfull depth (m)†	0.45	0.65
Bankfull width-to-depth ratio	7.00	5.61
Channel gradient (%)	3.60	0.89
Bankfull gradient (%)	0.89	0.89
Manning's roughness coefficient, *n*	0.04	0.03
Mean bankfull velocity (m/s) *	1.97	--
Bankfull discharge (m^3/s) *	2.01	--
Discharge to accommodate (m^3/s)	2.00	2.00
Tractive force at bankfull (N/m^2)††	158.86	54.82
Stream power (W/m)††	709.66	166.78
Unit stream power (W/m^2)††	270.35	69.28
Maximum grain size entrained (m)†† **	0.16	0.06
Mean grain size entrained (m)†† **	0.12	0.03

† Based on bankfull gradient
†† Based on riffle gradient
* Based on Manning's equation; as pools contain ineffective space, the velocity and discharge conveyed in them are not presented
** Based on Shields equation (Miller et al. (1977)), assuming Shields parameter equals 0.06 (gravel)

The sizing of proposed substrate materials was guided by a review of hydraulic conditions (i.e., tractive force, flow competency) in the typical cross sections. To provide for a stable bed and level of sorting, 60% 200 mm – 250 mm diameter Riverstone and 40% granular 'b' material is proposed for the riffles. Granular 'b' consists of a mix of stone where approximately 20 % - 50 % of the stone is greater than 0.005 m in diameter, but nothing larger than 0.15 m in diameter. These materials will always have a core of sediment that is not entrained under bankfull flow conditions. This material maintains the character of the native material, while providing slightly higher stability and opportunity for sediment sorting.

In the development of a natural channel design, the length of the watercourse proposed to be realigned is typically replicated or exceeded, to provide an overall gain in habitat. The existing length of the Tributary of Chedoke Creek proposed for realignment is approximately 205 m. The realigned corridor will provide a total linear distance of approximately 190 m. To produce a system similar to what would occur in nature, a sinuosity of approximately 1.2 was applied to the realigned channel, resulting in an increased channel length of 234 m. The proposed channel will therefore result in a significant increase in the area of restored and enhanced habitat.

The proposed realignment and naturalization of the Tributary of Chedoke Creek provides opportunities for improved riparian conditions and a well-developed bankfull channel with

morphological variability. Improvement in morphology and function would provide additional benefits to sediment balance, floodplain storage, vegetation communities and terrestrial habitat features, edge impacts, water balance, fish passage and water quality.

An online wetland/plunge pool will be installed at the upstream and downstream channel extents and will serve as added protection for the associated outlets. The plunge pool will have a stone core of hydraulically-sized rounded stone, which is the subsurface material used to ensure pool stability. The proposed stone core is expected to be stable under the predicted flow conditions in the wetlands. The substrate within the stone core is proposed to be a mix of 60% 250 mm – 300 mm diameter riverstone and 40% granular 'b'. A layer of topsoil will be installed on top of the stone core to improve vegetation establishment. The stone was hydraulically sized to limit entrainment. A range of techniques were utilized to determine the appropriate stone size, as summarized in the National Engineering Handbook (NRCS, 2007). These techniques are provided in **Table 3**. Based on our detailed assessment and channel survey, the calculated bankfull velocity of 2.28 m/s was used to determine the appropriate stone size for the online wetland/plunge pool. This velocity corresponds to the secondary discharge which was determined by extending the bankfull channel width to the location where flow would spill into the floodplain. The stone size includes a factor of safety to provide additional stability at the maximum outflow, while allowing for volumetric storage and infiltration at lower flows.

Table 3: Substrate sizes for the plunge pool based on a range of techniques

Model	Formula	Velocity (m/s)	Stone Size* (mm)
Plunge Pool Substrate			
Isbash Method (Isbash, 1936)	$D_{50} = \left(\frac{V_c}{C * \left(2 * g * \frac{\gamma_S - \gamma_w}{\gamma_w}\right)^{0.5}}\right)^2$	2.28	261
USBR Method (Peterka, 1958)	$D_{50} = 0.0122 * V^{2.06}$	2.28	282

*Includes 20% factor of safety

The Isbash method (Isbash, 1936) was developed for the construction of dams by placing rock into moving water. This model predicts the median stone size (D_{50}; ft) under the given flow conditions, given by:

$$D_{50} = \left(\frac{V_c}{C*(2*g*\frac{\gamma_S-\gamma_w}{\gamma_w})^{0.5}}\right)^2 \quad \text{[Eq.7]}$$

Where:

V_c = critical velocity (ft/s)
C = Isbash constant (dimensionless)
g = gravity (ft/s)
γ_s = stone density (lb/ft^3)
γ_w = water density (lb/ft^3)

The USBR Method (Peterka, 1958) was developed for sizing riprap below a stilling basin. This model predicts the median stone size (D_{50}; ft) under the given flow conditions, given by:

$$D_{50} = 0.0122 * V^{2.06} \quad \text{[Eq.8]}$$

Where:

V = average channel velocity (ft/s)

The values used for each variable in the Isbash method, and USBR method are provided in **Table 4**.

Table 4: Variables and values associated with the plunge pool substrate

Variable	Plunge Pool
Isbash Method	
Critical velocity (V_c) (ft/s)	7.48 (2.28 m/s)
Isbash constant (C) (unitless)	0.86
Gravity (g) (ft/s^2)	32.2 (9.81 m/s^2)
Stone density (γ_s) (lb/ft^3)	165.43 (2650 kg/m^3)
Water density (γ_w) (lb/ft^3)	62.43 (1000 kg/m^3)
USBR Method	
Velocity (V) (ft/s)	7.48 (2.28 m/s)

*note: Values used in modelling are in imperial units. Final values for stone size have been converted to SI units.

A vegetated rock buttress is proposed along the edges of the online wetlands/plunge pools to provide additional stability and reduce erosion. The vegetated rock buttress will consist of a constructed bank of 300 mm diameter stones with container grown plants staggered between the stones and spaced horizontally 1 m apart. The strength of the vegetated rock buttress will be augmented through vegetation establishment. Additionally, the plantings will provide additional thermal mitigation though shade, but will also provide a source of organic matter, to enhance semi-aquatic habitat.

3.4 Channel Corridor

Online and offline wetland features will be constructed in addition to the channel. These features enhance terrestrial habitat by increasing diversity and providing a more natural floodplain form. They also provide functional benefits such as short-term water retention and sediment banking. They will be irregularly shaped to maximize the perimeter for a given area, which increases the potential for edge effects. Submerged and dry mounds are proposed within the online wetlands to provide a topographically complex bottom to increase habitat heterogeneity. The short-term water retention function of these wetland types helps to polish water and moderate discharge of water into the channel.

The full channel corridor will be restored using native plant species. This includes appropriate species for the various seed mixes as well as woody vegetation. The plantings are intended to enhance the terrestrial habitat through the provision of species and habitat diversity, increase floodplain soil stability and floodplain roughness, and increase sedimentation.

To stabilize the banks, biodegradable erosion control blanket and live stakes are proposed. Prior to the application of the bank stabilization measures, the banks should be top soiled and graded to match the existing ground. The erosion control blanket is then applied in overlapping rows on top of the seed mix. The use of wooden stakes and live stakes will help keep the bank stabilization measures in place until vegetation has established.

Channel corridor sizing for the Tributary of Chedoke Creek was previously completed and included definition of a meander belt width. Meander belt width delineation is completed in support of defining requirements for a hydrological feature (i.e., the watercourse) within a proposed development. A refined meander belt width of 30 m has since been delineated using the bankfull channel discharges determined through our detailed assessment and outlined in **Table 1**. The accompanying memo provided in **Appendix E** provides additional details into the meander belt width delineation.

3.5 Habitat Restoration

The design incorporates several habitat elements within the channel corridor to improve riparian habitat and promote wildlife biodiversity. To maximize potential for wildlife passage, forage and residency, the habitat design incorporates varying topographies and woody debris. The habitat elements include brush mattresses and root wad bank treatments, which also serve as bank erosion control measures. The accompanying drawings provided in **Appendix F** provide design details and direction for the implementation of the proposed habitat features.

Brush mattress is proposed along the outside meander bend of certain meanders. This treatment consists of live brush cuttings installed parallel to the banks and tied in with coir twine and stakes. The brush mattress will provide bank stability and improve aquatic habitat through shading.

Root wad bank treatment is also proposed at specific locations within the meander pattern. The treatment is to extend the full length of the outside meander bend between riffles and consists of partially buried root wads on a bedding of riverstone. The rood wad bank treatment provides enhanced bank stability while improving aquatic habitat.

3.6 Natural Erosion Control

Newly constructed features can be vulnerable to erosion. This is particularly true before vegetation has established along the bioswale banks. While low-flow events should not intensify erosion, the concern for erosion occurs when there are high flows or precipitation events during construction.

For immediate erosion protection, mechanical stabilization in the form of biodegradable erosion control blankets (i.e., coir cloth, jute mat, etc.) should be used. As the blankets will biodegrade over time, this serves as a short-term stabilization measure.

For long-term stability, implementation of a planting plan is recommended. This includes deep rooting native grasses and other herbaceous species seeded along and within bioswale sections, prescription of flood tolerant native shrub and tree species, and use of seed banks within the local soil. Shrubs should be planted close to the bioswale margins to provided maximum benefit with respect to stabilization and bioswale cover.

Potential erosion locations (i.e., along the outside meander bends, immediately downstream of outlet features, etc.) should be anticipated, and should be reflected in the planting plan. Live staking and shrub stock should be used adjacent to the channel banks to provide immediate benefit as well as long-term infilling. If appropriate live staking methods are followed, this method should provide greater benefits than simple potted or bare root shrub plating because of the potential for higher densities with live staking.

4 Design Implementation

4.1 Construction Timing

Based on resident fish species and their respective life cycles, in-stream work will be restricted to July 1st to March 31st, unless otherwise directed by the Ministry of Natural Resources and Forestry (MNRF).

4.2 Best Management Practices

Site inspection should be performed by an inspector with experience overseeing channel works, as this type of work differs considerably from engineering projects. An experienced inspector will be able to provide quick and appropriate response to issues that may arise and ensure that construction proceeds in accordance with the approved design and contract.

The limits of construction will be delineated to prevent unanticipated impacts to natural surroundings, including trees and the watercourse. Most of the channel can be constructed without interference to the existing watercourse. When the proposed channel does cross the existing channel, cofferdams will be installed upstream and downstream of the work area and the water will be pumped around.

All isolated work areas will be dewatered to perform work under dry conditions. Water will be pumped to a sediment filtration system located at least 30 m from the receiving creek and be allowed to naturally flow over a well-vegetated surface and ultimately return to the channel downstream of the work area. This will allow particles to settle before reaching the watercourse.

All materials and equipment will be stored and operated in such a manner that prevents any deleterious substances from entering the water. Vehicle and equipment re-fuelling and/or maintenance will be conducted away from the watercourse and be free of fluid leaks and externally cleaned/degreased to prevent the release of deleterious substances.

4.3 Post-Construction Monitoring

A post-construction monitoring program is recommended to assess the performance of the implemented design. Monitoring observations can also be used to determine the need for remedial works. Monitoring is recommended for two full calendar years following the year of construction.

The following monitoring and reporting activities are proposed:

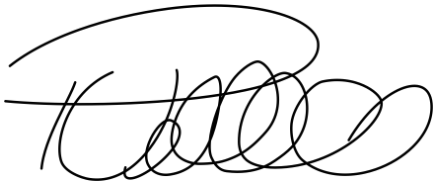
- General observations of the channel works should be documented after construction and after the first large flooding event to identify any potential areas of erosion concern
- Collection of a photographic record of site conditions

- Total station as-built survey of the channel planform, longitudinal profile and cross sections just after construction to obtain reference data for the following two years
- Installation of erosion pins at monumented cross sections after construction
- A general vegetation survey in the spring of each year
- Re-survey of the longitudinal profile and cross sections, as well as monitoring of erosion pins at monumented cross sections for three years following construction
- A yearly report for the first year, with a final report at the end of the two-year period

The monitoring would commence immediately after construction and sites would be reviewed annually to identify natural variability of the system. Reporting would be provided annually, with a summary report at the end of the monitoring period.

We trust this report meets your requirements. Should you have any questions, please contact the undersigned.

Respectfully submitted,





Paul Villard Ph.D., P.Geo., CAN-CISEC, EP, CERP
Director, Principal Geomorphologist

Ben Miller, B.Sc., CAN-CISEC
Restoration Technician

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Appendix A
Site Map



Legend

Extent of Reach Assessed

Watercourse

Chedoke Creek Realignment

Extent Assessed

Hamilton, Ontario

N

0 25 50 100

Metres

GEO MORPHIX™

Imagery: Google Earth Pro, June 2018.
Extent of Reach Assessed: GEO Morphix Ltd., 2020.
Watercourse: A.J. Clark and Associates Ltd., 2015 and GEO Morphix Ltd., 2020.
Print Date: July 2020. PN19110. Drawn By: W.B., B.M.

Appendix B
Photographic Record

Photo 1 Tributary of Chedoke Creek, Hamilton, Ontario	
	Looking upstream at the furthest upstream extent assessed. The single channel flows through predominantly residential lands and exits a culvert downstream of Scenic Drive. Yellow arrow denotes flow direction.
Photo 2 Tributary of Chedoke Creek, Hamilton, Ontario	
	Downstream view of the channel. Leaning and fallen trees, as well as exposed tree roots, were frequently noted through the extent of the reach and provided evidence of channel widening.

Photo 3 Tributary of Chedoke Creek, Hamilton, Ontario	
Upstream view of the channel. A rooted knickpoint was observed with apparent head-cutting due to knickpoint migration upstream.	
Photo 4 Tributary of Chedoke Creek, Hamilton, Ontario	
An exposed length of previously buried pipe was observed. Occurrence of large organic debris and substrate fouling was common.	

Photo 5 Tributary of Chedoke Creek, Hamilton, Ontario	
	Downstream view of the channel. Bank erosion was present through 60 to 100% of the subject reach and was observed through scouring of the channel banks (circled) and undercutting.
Photo 6 Tributary of Chedoke Creek, Hamilton, Ontario	
	The channel flows through a corrugated steel pipe (CSP) at a pedestrian crossing mid-reach. Upstream was dominated by runs indicating poor physical instream habitat.

Photo 7 Tributary of Chedoke Creek, Hamilton, Ontario	
	An additional length of exposed sanitary/storm sewer/pipeline was observed downstream of the pedestrian crossing providing further evidence of degradation.
Photo 8 Tributary of Chedoke Creek, Hamilton, Ontario	
	At the downstream extent of the reach, the valley was partially confined and valley wall contact was observed. The channel was worn into bedrock (pictured on right).

Photo 9 Tributary of Chedoke Creek, Hamilton, Ontario	
	At the downstream extent of the reach, the channel entered a culvert at Sanitorium Road. Evidence of widening and degradation were the key geomorphic indicators of channel stability through the reach.
Photo 10 Tributary of Chedoke Creek, Hamilton, Ontario	
	Downstream of Sanitorium Road, a rapid change in gradient was observed. The valley was confined beyond the study limits.

Appendix C
Field Observations

GEO MORPHIX
Geomorphology Earth Science Observations

General Site Characteristics

Project Code: PN1910

Date:	Nov 29, 2019	**Stream/Reach:**	CC1
Weather:	CLOUDY 1°C	**Location:**	Sanatorium, Hamilton
Field Staff:	CH + BB	**Watershed/Subwatershed:**	Chedoke Crk

Features

- Reach break
- Cross-section
- Flow direction
- Riffle
- Pool
- Medial bar
- Eroded bank
- Undercut bank
- Rip rap/stabilization/gabion
- Leaning tree
- Fence
- Culvert/outfall
- Swamp/wetland
- Grasses
- Tree
- Instream log/tree
- Woody debris
- Station location
- Vegetated island

Flow Type

H1	Standing water
H2	Scarcely perceptible flow
H3	Smooth surface flow
H4	Upwelling
H5	Rippled
H6	Unbroken standing wave
H7	Broken standing wave
H8	Chute
H9	Free fall

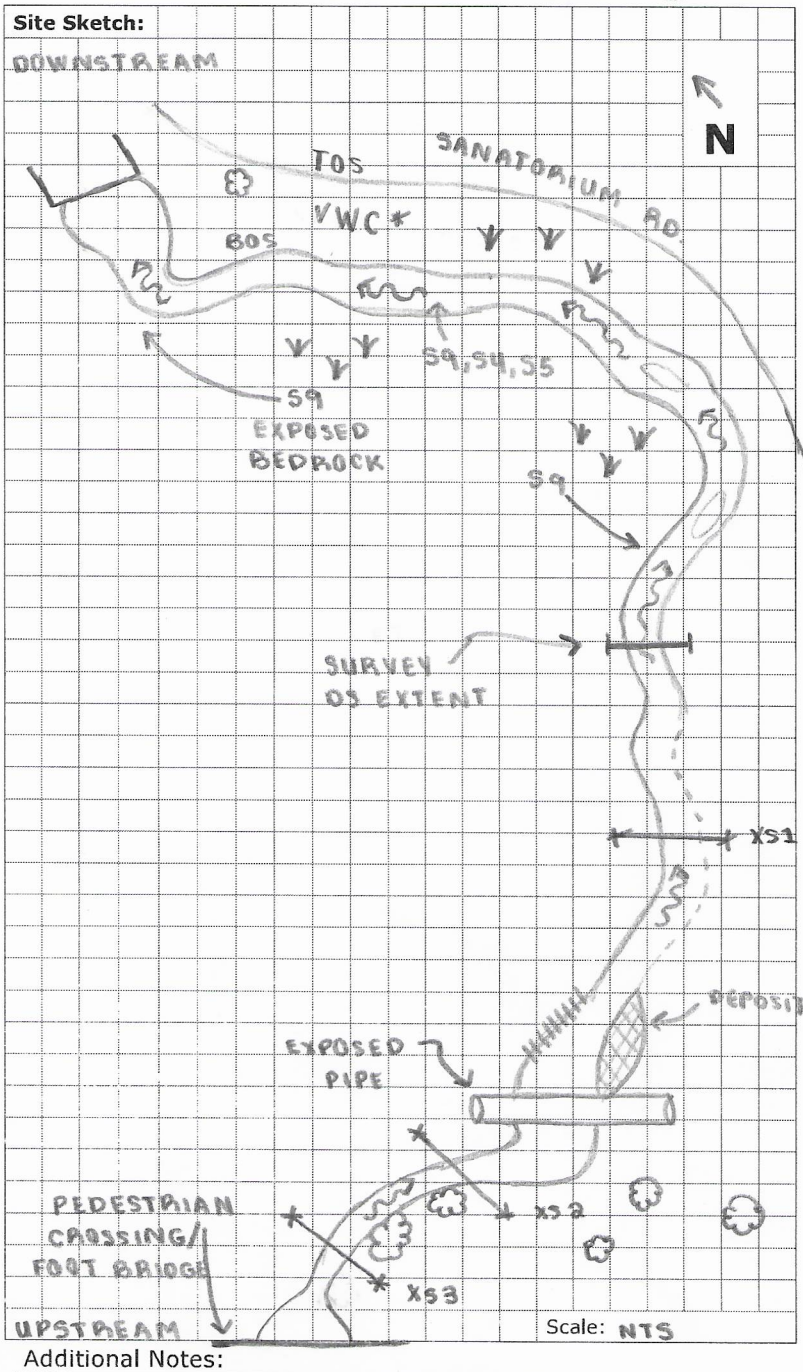
Substrate

S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Site Sketch:



Additional Notes:

* STEEP SLOPE TO ROAD @ VWC

Completed by: BB Checked by: _____

GEO MORPHIX
Geomorphology Earth Science Observations

General Site Characteristics

Project Code: PN19110

Date:	2019-11-29	**Stream/Reach:**	CC1
Weather:	CLOUDY 1°C	**Location:**	SANATORIUM RD, HAMILTON
Field Staff:	CH + BB	**Watershed/Subwatershed:**	CHEDOKE CREEK

Features

- Reach break
- Cross-section
- Flow direction
- Riffle
- Pool
- Medial bar
- Eroded bank
- Undercut bank
- Rip rap/stabilization/gabion
- Leaning tree
- Fence
- Culvert/outfall
- Swamp/wetland
- Grasses
- Tree
- Instream log/tree
- Woody debris
- Station location
- Vegetated island

Flow Type

H1	Standing water
H2	Scarcely perceptible flow
H3	Smooth surface flow
H4	Upwelling
H5	Rippled
H6	Unbroken standing wave
H7	Broken standing wave
H8	Chute
H9	Free fall

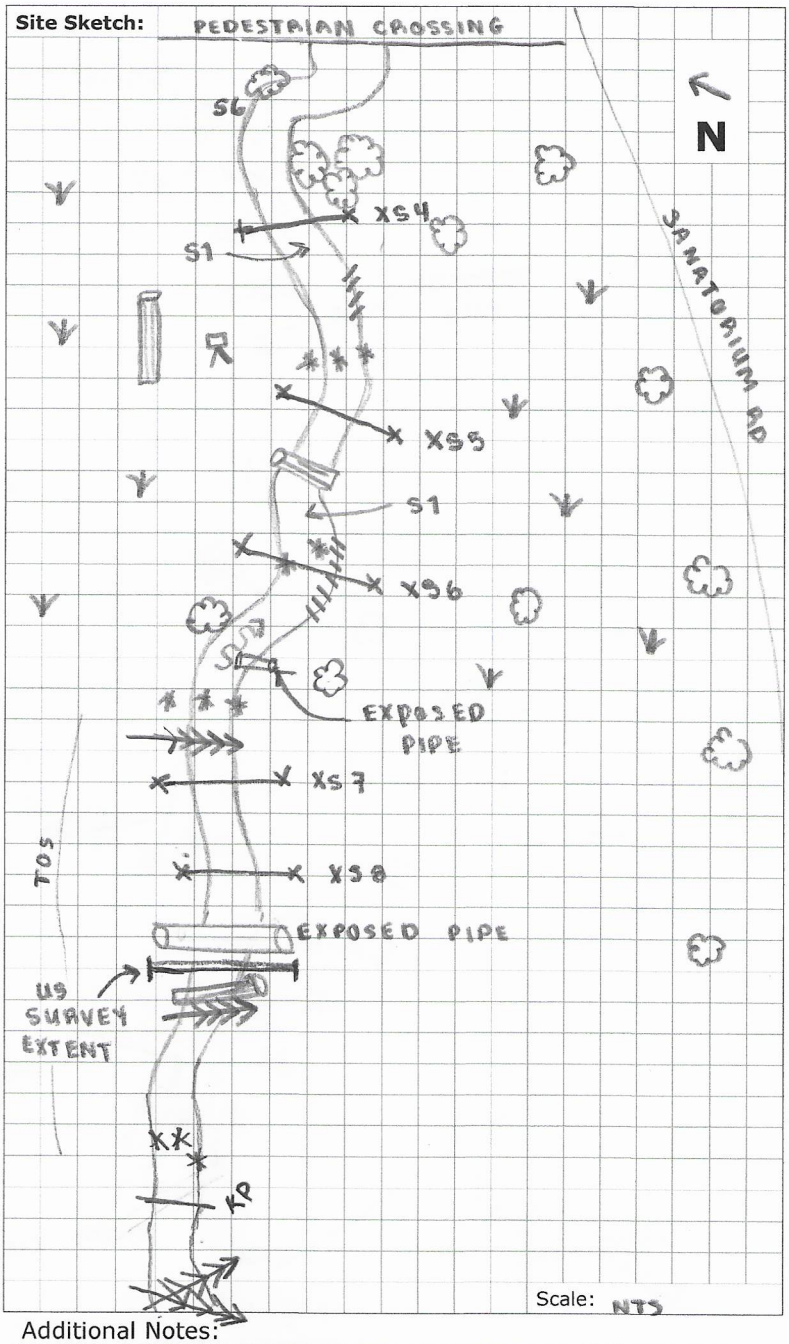
Substrate

S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

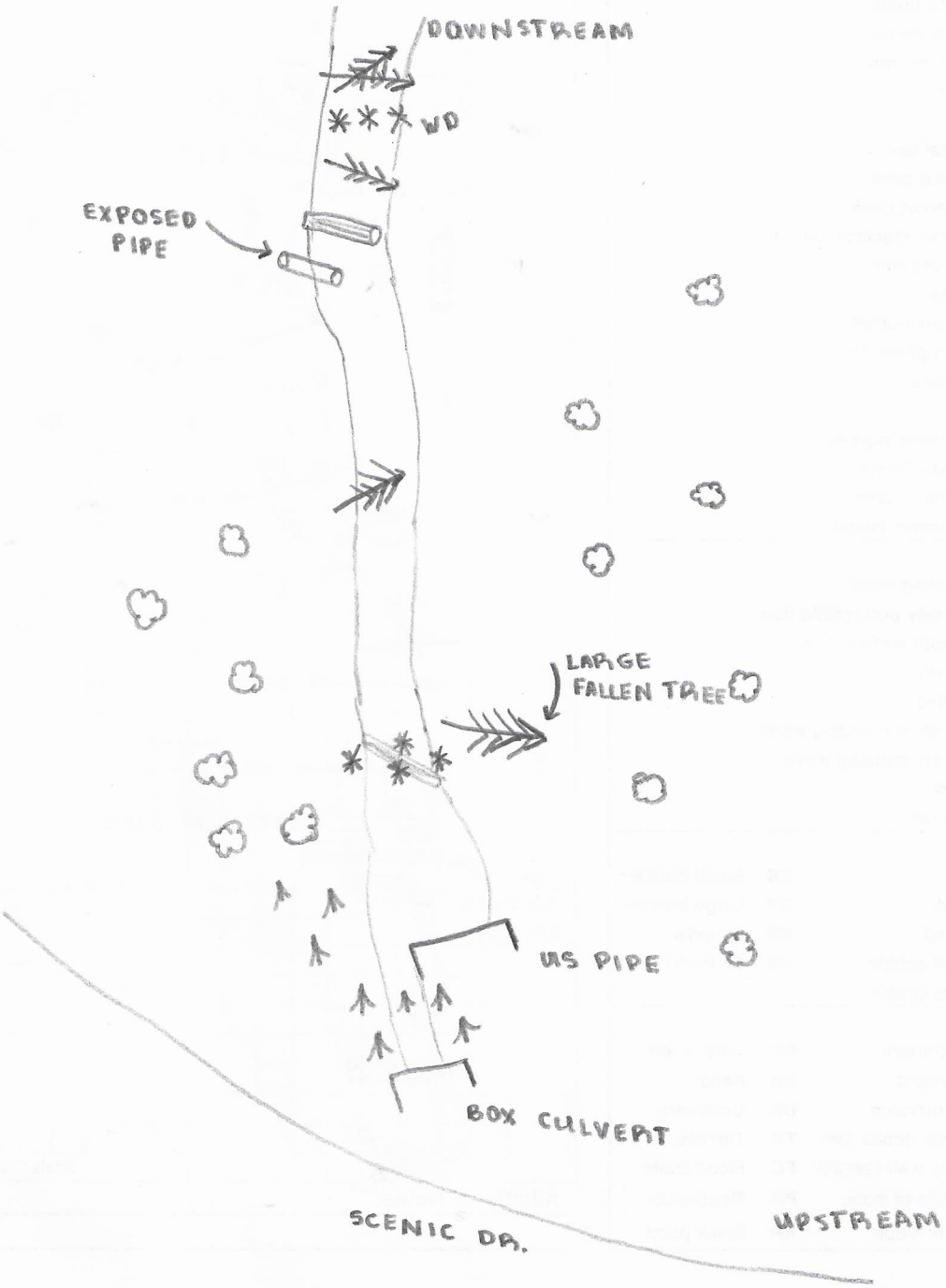
BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Site Sketch:



Additional Notes:

Completed by: BB Checked by: _____


DOWNSTREAM
WD
EXPOSED PIPE
LARGE FALLEN TREE
US PIPE
BOX CULVERT
SCENIC DR.
UPSTREAM

BB

Reach Characteristics

GEO MORPHIX — Geomorphology Earth Science Observations

Project Code/Phase: PN19110

Date:	Nov 29, 2019	**Stream/Reach:**	CC1
Weather:	CLOUDY 1°C	**Location:**	Sanatorium + Scenic, Hamilton
Field staff:	CH + BB	**Watershed/Subwatershed:**	Chedoke Crk
UTM (Upstream)		**UTM (Downstream)**	

Land Use (Table 1): 7 | **Valley Type** (Table 2): 3 | **Channel Type** (Table 3): 12 | **Channel Zone** (Table 4): 2 | **Flow Type** (Table 5): 1 | ☐ Groundwater | Evidence: _____

Riparian Vegetation

Dominant Type: (Table 6): 1
Species: _____

Coverage:
- ☐ None
- ☐ Fragmented
- ☑ Continuous

Channel widths
- ☐ 1-4
- ☑ 4-10
- ☐ > 10

Age Class (yrs) :
- ☐ Immature (<5)
- ☑ Established (5-30)
- ☐ Mature (>30)

Encroachment: (Table 7): 1

Aquatic/Instream Vegetation

Type (Table8): 6 | **Coverage of Reach (%):** 5

Woody Debris
- ☐ Present in Cutbank
- ☑ Present in Channel
- ☐ Not Present

Density of WD:
- ☐ Low
- ☑ Moderate
- ☐ High

WDJ/50m: 2

Water Quality

Odour (Table 16): 4

Turbidity (Table 17): 4

Channel Characteristics

Sinuosity (Type) (Table 9)	Sinuosity (Degree) (Table 10)	Gradient (Table 11)	Number of Channels (Table 12)
1	2	2	1

Entrenchment (Table 13)	Type of Bank Failure (Table 14)	Downs's Classification (Table 15)
2	1, 2	E

	Clay/Silt	Sand	Gravel	Cobble	Boulder	Parent	Rootlets
Riffle Substrate	☐	☐	☐	☐	☐	☑	☐
PLAIN ~~Pool~~ Substrate	☑	☐	☐	☐	☐	☐	☐
Bank Material	☑	☐	☐	☐	☐	☐	☑

Bankfull Width (m)	SEE DETAILED		**Wetted Width (m)**		
Bankfull Depth (m)	ASSESSMENT		**Wetted Depth (m)**		
Riffle/Pool Spacing (m)	N/A	**% Riffles:** 5	**% Pools:** 0	**Meander Amplitude:**	N/A
Pool Depth (m)	N/A	**Riffle Length (m)**	~3	**Undercuts (m)**	0.15
Velocity (m/s)				**Wiffle ball / ADV / Estimated**	

Bank Angle
- ☐ 0 – 30
- ☑ 30 – 60
- ☑ 60 – 90
- ☑ Undercut

Bank Erosion
- ☐ < 5%
- ☐ 5 – 30%
- ☐ 30 – 60%
- ☑ 60 – 100%

• APPROX. 75% ↙

Comments:

Notes:
- CROSSING IS BOX CULVERT BUT MAJORITY OF WATER COMING FROM ADJACENT PIPE
- 10 cm SOFT SEDIMENT

- 25 cm KNICK POINT
- EXPOSED BEDROCK @ DS EXTENT
- MOSTLY PLAIN BED HOWEVER RIFFLES @ DS EXTENT

Completed by: _____ Checked by: _____

GEO | MORPHIX

Rapid Geomorphic Assessment **Project Code:** PN19110

Date:	Nov 29, 2019	**Stream/Reach:**	CC1
Weather:	CLOUDY 1°C	**Watershed/Subwatershed:**	Chedoke Crk
Field Staff:	CH + BB	**Location:**	Sanatorium, Hamilton

Process	No.	Geomorphological Indicator: Description	Present? Yes	Present? No	Factor Value
Evidence of Aggradation (AI)	1	Lobate bar		✓	0/6
	2	Coarse materials in riffles embedded		✓	
	3	Siltation in pools	N/A		
	4	Medial bars		✓	
	5	Accretion on point bars		✓	
	6	Poor longitudinal sorting of bed materials		✓	
	7	Deposition in the overbank zone		✓	
		Sum of indices =	0	6	0.00
Evidence of Degradation (DI)	1	Exposed bridge footing(s)		N/A	3/8
	2	Exposed sanitary / storm sewer / pipeline / etc.	✓		
	3	Elevated storm sewer outfall(s)		✓	
	4	Undermined gabion baskets / concrete aprons / etc.		N/A	
	5	Scour pools downstream of culverts / storm sewer outlets		✓	
	6	Cut face on bar forms		✓	
	7	Head cutting due to knickpoint migration	✓		
	8	Terrace cut through older bar material		✓	
	9	Suspended armour layer visible in bank		✓	
	10	Channel worn into undisturbed overburden / bedrock	✓		
		Sum of indices =	3	5	0.38
Evidence of Widening (WI)	1	Fallen / leaning trees / fence posts / etc.	✓		3/8
	2	Occurrence of large organic debris	✓		
	3	Exposed tree roots	✓		
	4	Basal scour on inside meander bends		✓	
	5	Basal scour on both sides of channel through riffle		✓	
	6	Outflanked gabion baskets / concrete walls / etc.		N/A	
	7	Length of basal scour >50% through subject reach		✓	
	8	Exposed length of previously buried pipe / cable / etc.		✓	
	9	Fracture lines along top of bank		✓	
	10	Exposed building foundation		N/A	
		Sum of indices =	3	5	0.38
Evidence of Planimetric Form Adjustment (PI)	1	Formation of chute(s)		✓	0/7
	2	Single thread channel to multiple channel		✓	
	3	Evolution of pool-riffle form to low bed relief form		✓	
	4	Cut-off channel(s)		✓	
	5	Formation of island(s)		✓	
	6	Thalweg alignment out of phase with meander form		✓	
	7	Bar forms poorly formed / reworked / removed		✓	
		Sum of indices =	0	7	0.00

Additional notes:

Stability Index (SI) = (AI+DI+WI+PI)/4 = 0.19

Condition	In Regime	In Transition/Stress	In Adjustment
SI score =	☑ 0.00 - 0.20	☐ 0.21 - 0.40	☐ 0.41

Completed by: CH Checked by: _____

GEO | MORPHIX

Rapid Stream Assessment Technique

Project Code: PN19110

Date:	Nov 29, 2019	**Stream/Reach:**	EC1
Weather:	CLOUDY 1°C	**Location:**	Sanatorium, Hamilton
Field Staff:	CH + BB	**Watershed/Subwatershed:**	Chedoke Crk

Evaluation Category	Poor	Fair	Good	Excellent
Channel Stability	• < 50% of bank network stable • Recent bank sloughing, slumping or failure frequently observed	• 50-70% of bank network stable • Recent signs of bank sloughing, slumping or failure fairly common	• 71-80% of bank network stable • Infrequent signs of bank sloughing, slumping or failure	• > 80% of bank network stable • No evidence of bank sloughing, slumping or failure
	• Stream bend areas highly unstable • Outer bank height 1.2 m above stream bank (2.1 m above stream bank for large mainstem areas) • Bank overhang > 0.8-1.0 m	• Stream bend areas unstable • Outer bank height 0.9-1.2 m above stream bank (1.5-2.1 m above stream bank for large mainstem areas) • Bank overhang 0.8-0.9m	• Stream bend areas stable • Outer bank height 0.6-0.9 m above stream bank (1.2-1.5 m above stream bank for large mainstem areas) • Bank overhang 0.6-0.8 m	• Stream bend areas very stable • Height < 0.6 m above stream (< 1.2 m above stream bank for large mainstem areas) • Bank overhang < 0.6 m
	• Young exposed tree roots abundant • > 6 recent large tree falls per stream mile	• Young exposed tree roots common • 4-5 recent large tree falls per stream mile	• Exposed tree roots predominantly old and large, smaller young roots scarce • 2-3 recent large tree falls per stream mile	• Exposed tree roots old, large and woody • Generally 0-1 recent large tree falls per stream mile
	• Bottom 1/3 of bank is highly erodible material • Plant/soil matrix severely compromised	• Bottom 1/3 of bank is generally highly erodible material • Plant/soil matrix compromised	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material
	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally V- or U-shaped	• Channel cross-section is generally V- or U-shaped
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4 ☐ 5	☐ 6 ☑ 7 ☐ 8	☐ 9 ☐ 10 ☐ 11
Channel Scouring/ Sediment Deposition	• > 75% embedded (> 85% embedded for large mainstem areas)	• 50-75% embedded (60-85% embedded for large mainstem areas)	• 25-49% embedded (35-59% embedded for large mainstem areas)	• Riffle embeddedness < 25% sand-silt (< 35% embedded for large mainstem areas)
	• Few, if any, deep pools • Pool substrate composition >81% sand-silt	• Low to moderate number of deep pools • Pool substrate composition 60-80% sand-silt	• Moderate number of deep pools • Pool substrate composition 30-59% sand-silt	• High number of deep pools (> 61 cm deep) (> 122 cm deep for large mainstem areas) • Pool substrate composition <30% sand-silt
	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits uncommon	• Streambed streak marks and/or "banana"-shaped sediment deposits absent
	• Fresh, large sand deposits very common in channel • Moderate to heavy sand deposition along major portion of overbank area	• Fresh, large sand deposits common in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits uncommon in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits rare or absent from channel • No evidence of fresh sediment deposition on overbank
	• Point bars present at most stream bends, moderate to large and unstable with high amount of fresh sand	• Point bars common, moderate to large and unstable with high amount of fresh sand	• Point bars small and stable, well-vegetated and/or armoured with little or no fresh sand	• Point bars few, small and stable, well-vegetated and/or armoured with little or no fresh sand
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☑ 5 ☐ 6	☐ 7 ☐ 8

Date:	Nov 29, 2019	Reach:	CC1	Project Code:	PN1910

Evaluation Category	Poor	Fair	Good	Excellent
Physical Instream Habitat	• Wetted perimeter < 40% of bottom channel width (< 45% for large mainstem areas)	• Wetted perimeter 40-60% of bottom channel width (45-65% for large mainstem areas)	• Wetted perimeter 61-85% of bottom channel width (66-90% for large mainstem areas)	• Wetted perimeter > 85% of bottom channel width (> 90% for large mainstem areas)
	• Dominated by one habitat type (usually runs) and by one velocity and depth condition (slow and shallow) (for large mainstem areas, few riffles present, runs and pools dominant, velocity and depth diversity low)	• Few pools present, riffles and runs dominant. • Velocity and depth generally slow and shallow (for large mainstem areas, runs and pools dominant, velocity and depth diversity intermediate)	• Good mix between riffles, runs and pools • Relatively diverse velocity and depth of flow	• Riffles, runs and pool habitat present • Diverse velocity and depth of flow present (i.e., slow, fast, shallow and deep water)
	• Riffle substrate composition: predominantly gravel with high amount of sand • < 5% cobble	• Riffle substrate composition: predominantly small cobble, gravel and sand • 5-24% cobble	• Riffle substrate composition: good mix of gravel, cobble, and rubble material • 25-49% cobble	• Riffle substrate composition: cobble, gravel, rubble, boulder mix with little sand • > 50% cobble
	• Riffle depth < 10 cm for large mainstem areas	• Riffle depth 10-15 cm for large mainstem areas	• Riffle depth 15-20 cm for large mainstem areas	• Riffle depth > 20 cm for large mainstem areas
	• Large pools generally < 30 cm deep (< 61 cm for large mainstem areas) and devoid of overhead cover/structure	• Large pools generally 30-46 cm deep (61-91 cm for large mainstem areas) with little or no overhead cover/structure	• Large pools generally 46-61 cm deep (91-122 cm for large mainstem areas) with some overhead cover/structure	• Large pools generally > 61 cm deep (> 122 cm for large mainstem areas) with good overhead cover/structure
	• Extensive channel alteration and/or point bar formation/enlargement	• Moderate amount of channel alteration and/or moderate increase in point bar formation/enlargement	• Slight amount of channel alteration and/or slight increase in point bar formation/enlargement	• No channel alteration or significant point bar formation/enlargement
	• Riffle/Pool ratio 0.49:1 ; ≥1.51:1	• Riffle/Pool ratio 0.5-0.69:1 ; 1.31-1.5:1	• Riffle/Pool ratio 0.7-0.89:1 ; 1.11-1.3:1	• Riffle/Pool ratio 0.9-1.1:1
	• Summer afternoon water temperature > 27°C	• Summer afternoon water temperature 24-27°C	• Summer afternoon water temperature 20-24°C	• Summer afternoon water temperature < 20°C
Point range	☐ 0 ☐ 1 ☐ 2	☑ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☐ 8
Water Quality	• Substrate fouling level: High (> 50%)	• Substrate fouling level: Moderate (21-50%)	• Substrate fouling level: Very light (11-20%)	• Substrate fouling level: Rock underside (0-10%)
	• Brown colour • TDS: > 150 mg/L	• Grey colour • TDS: 101-150 mg/L	• Slightly grey colour • TDS: 50-100 mg/L	• Clear flow • TDS: < 50 mg/L
	• Objects visible to depth < 0.15m below surface	• Objects visible to depth 0.15-0.5m below surface	• Objects visible to depth 0.5-1.0m below surface	• Objects visible to depth > 1.0m below surface
	• Moderate to strong organic odour	• Slight to moderate organic odour	• Slight organic odour	• No odour
Point range	☐ 0 ☐ 1 ☐ 2	☑ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☐ 8
Riparian Habitat Conditions	• Narrow riparian area of mostly non-woody vegetation	• Riparian area predominantly wooded but with major localized gaps	• Forested buffer generally > 31 m wide along major portion of both banks	• Wide (> 60 m) mature forested buffer along both banks
	• Canopy coverage: <50% shading (30% for large mainstem areas)	• Canopy coverage: 50-60% shading (30-44% for large mainstem areas)	• Canopy coverage: 60-79% shading (45-59% for large mainstem areas)	• Canopy coverage: >80% shading (> 60% for large mainstem areas)
Point range	☐ 0 ☐ 1	☐ 2 ☐ 3	☑ 4 ☐ 5	☐ 6 ☐ 7

Total overall score (0–42) = 22	Poor (<13)	Fair (13-24)	Good (25-34)	Excellent (>35)

Completed by: CH Checked by: _____

Appendix D
Detailed Assessment Summary

Detailed Geomorphological Assessment Summary

Chedoke Creek - Bankfull Indicators

Project Number:	PN19110	**Date:**	November 29, 2019
Client:	Valery Homes	**Length Surveyed (m):**	107.0
Location:	Scenic Drive and Sanatorium Road	**# of Cross-Sections:**	8

Reach Characteristics

Drainage Area:	Not measured	**Dominant Riparian Vegetation Type:**	Trees
Geology/Soils:	Till (upstream) & bedrock (downstream)	**Extent of Riparian Cover:**	Continuous
Surrounding Land Use:	Residential	**Width of Riparian Cover:**	4-10 Channel widths
Valley Type:	Partially confined	**Age Class of Riparian Vegetation:**	Established (5-30 years)
Dominant Instream Vegetation Type:	Attached algae	**Extent of Encroachment into Channel:**	None
Portion of Reach with Vegetation:	<5%	**Density of Woody Debris:**	Moderate

Hydrology

Measured Discharge (m^3/s):	0.01	**Calculated Bankfull Discharge (m^3/s):**	1.11
Modelled 2-year Discharge (m^3/s):	Not modelled	**Calculated Bankfull Velocity (m/s):**	1.26
Modelled 2-year Velocity (m/s):	Not modelled		

Profile Characteristics

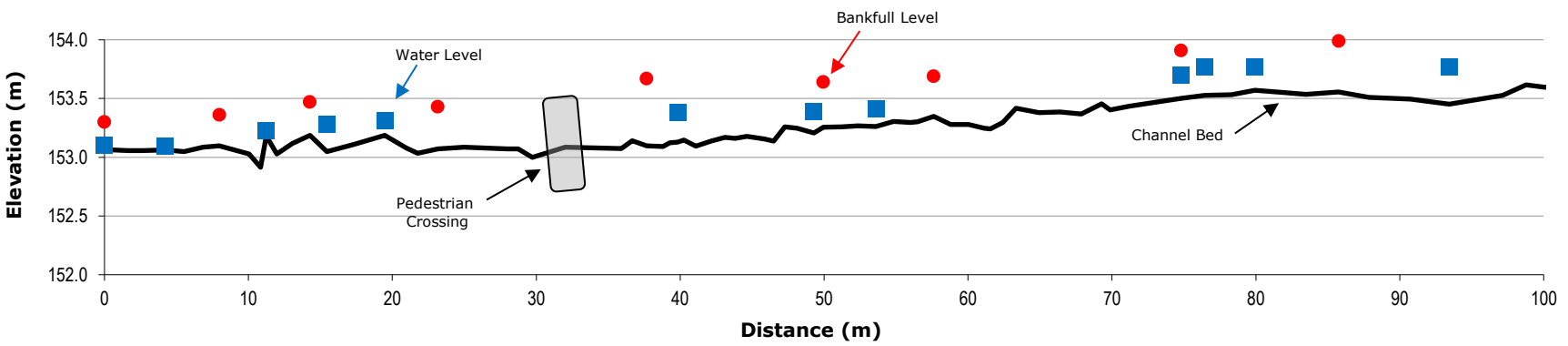
Bankfull Gradient (%):	0.89	
Channel Bed Gradient (%):	0.60	
Riffle Gradient (%):	0.76	*
Riffle Length (m):	5.28	*
Riffle-Pool Spacing (m):	5.19	*

Planform Characteristics

Sinuosity:	0.00
Meander Belt Width (m):	Not measured
Radius of Curvature (m):	Not measured
Meander Amplitude (m):	Not measured
Meander wavelength (m):	Not measured

*riffle-pool development only downstream of the pedestrian crossing

Longitudinal Profile



Bank Characteristics

	Minimum	Maximum	Average
Bank Height (m):	0.25	0.80	0.50
Bank Angle (deg):	30	90	50
Root Depth (m):	0.10	0.60	0.35
Root Density (%):	10	70	26
Bank Undercut (m):	0.00	0.29	0.03

	Minimum	Maximum	Average
Torvane Value (kg/cm^2):		Not measured	
Penetrometer Value (kg/cm^3):		Not measured	
Bank Material (range):		Clay and silt	

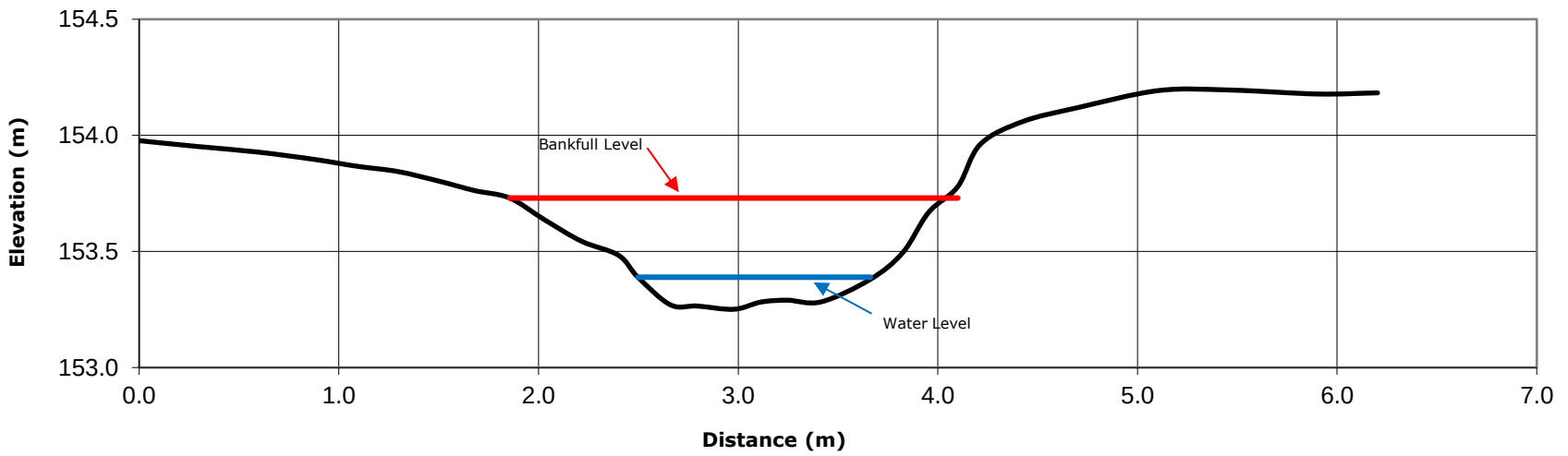
Cross-Sectional Characteristics

	Minimum	Maximum	Average
Bankfull Width (m):	2.05	4.90	2.79
Average Bankfull Depth (m):	0.22	0.42	0.32
Bankfull Width/Depth (m/m):	7	13	9
Wetted Width (m):	0.57	2.04	1.16
Average Water Depth (m):	0.04	0.37	0.19
Wetted Width/Depth (m/m):	2	14	8
Entrenchment (m):		Not measured	
Entrenchment Ratio (m/m):		Not measured	
Maximum Water Depth (m):	0.05	0.35	0.24
Manning's *n*:		0.040	



Photograph at cross section 4 (looking downstream)

Representative Cross-Section 4



Substrate Characteristics

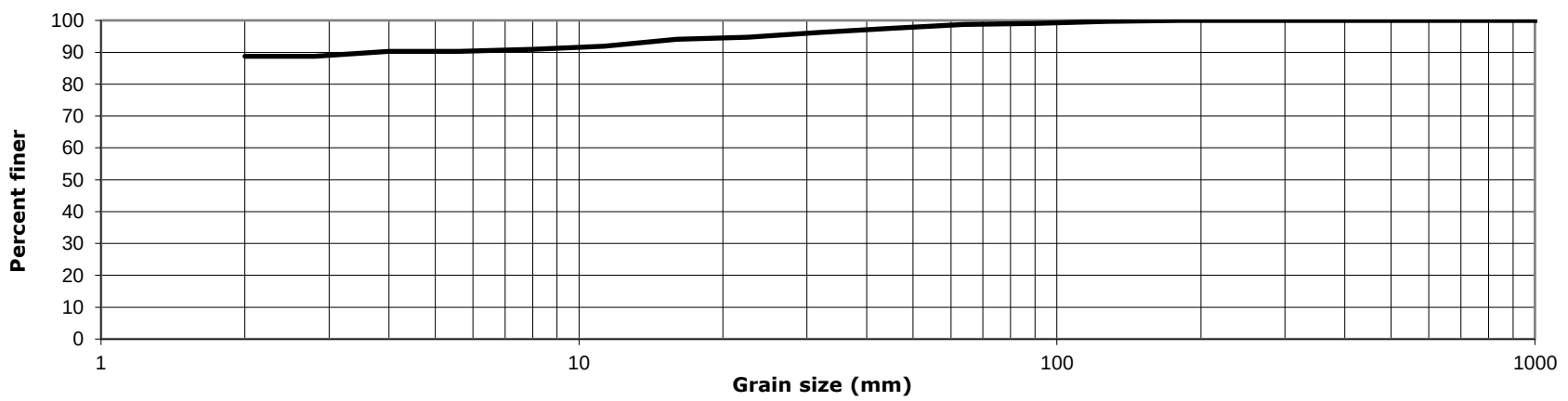
Particle Size (mm)

D_{10} :	<	2.0
D_{50} :	<	2.0
D_{84} :	<	2.0

Bed sample taken

Subpavement:	Till upstream and bedrock downstream
Particle shape:	Platy and angular (cross-section 1)
Embeddedness (%):	<5 (cross-section 1)
Particle range (riffle):	Coarse sand to small cobbles
Particle Range (pool):	Clay, silt, and sand

Cumulative Particle Size Distribution



Channel Thresholds

Flow Competency (m/s):		**Tractive Force at Bankfull (N/m^2):**	27.80
for D_{50}:	0.00	**Tractive Force at 2-year flow (N/m^2):**	Not modelled
for D_{84}:	0.00	**Critical Shear Stress (D_{50}) (N/m^2):**	0.00
Unit Stream Power at Bankfull (W/m^2):	34.93		

General Field Observations

Channel Description

Reach CC1 consisted of a single thread channel with a low sinuosity and a moderate gradient. The reach was more entrenched at the upstream extent, which is apparent at the upstream cross-sections, 7 and 8. An old pedestrian crossing was noted at the downstream extent of the survey, between cross-sections 3 and 4. The riparian vegetation was mostly comprised of trees and formed a continuous buffer. The average bankfull width and depth were 2.77 m and 0.32 m. Bank erosion was observed along approximately 75% of the banks. Leaning trees and exposed tree roots were also observed. Riffle-pool formation was only noted downstream of the pedestrian crossing. The reach consisted of a plain bed upstream of the pedestrian bridge to Scenic Drive. Substrate consisted of clay, silt, and sand except at cross-section 1, where the bedrock was exposed and gravel and cobbles were measured.

Cross Section 7 - Facing Upstream



Detailed Geomorphological Assessment Summary

Chedoke Creek - Floodplain

Project Number:	PN19110	**Date:**	November 29, 2019
Client:	Valery Homes	**Length Surveyed (m):**	107.0
Location:	Scenic Drive and Sanatorium Road	**# of Cross-Sections:**	8

Reach Characteristics

Drainage Area:	Not measured	**Dominant Riparian Vegetation Type:**	Trees
Geology/Soils:	Till (upstream) & bedrock (downstream)	**Extent of Riparian Cover:**	Continuous
Surrounding Land Use:	Residential	**Width of Riparian Cover:**	4-10 Channel widths
Valley Type:	Partially confined	**Age Class of Riparian Vegetation:**	Established (5-30 years)
Dominant Instream Vegetation Type:	Attached algae	**Extent of Encroachment into Channel:**	None
Portion of Reach with Vegetation:	<5%	**Density of Woody Debris:**	Moderate

Hydrology

Measured Discharge (m^3/s):	0.01	**Calculated Bankfull Discharge (m^3/s):**	2.57
Modelled 2-year Discharge (m^3/s):	Not modelled	**Calculated Bankfull Velocity (m/s):**	1.46
Modelled 2-year Velocity (m/s):	Not modelled		

Profile Characteristics

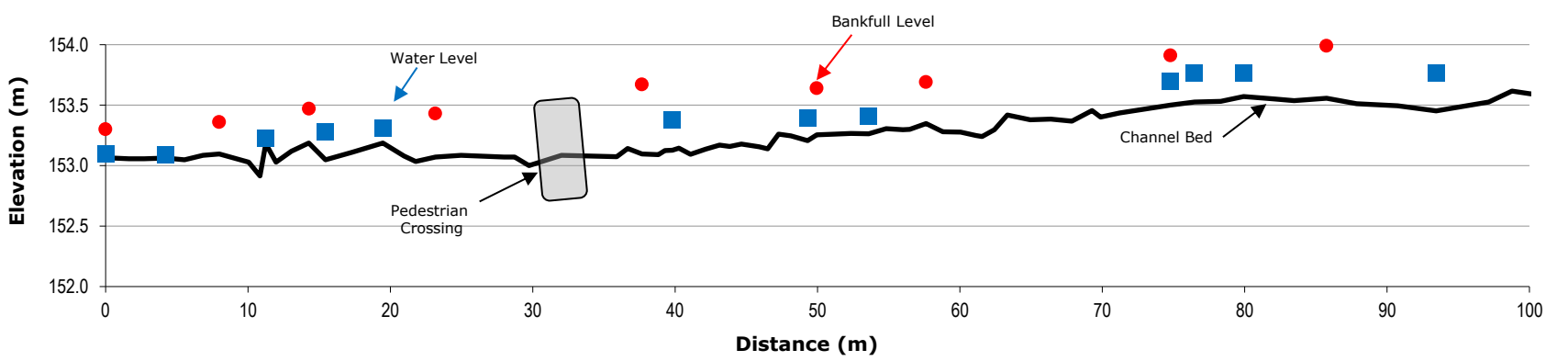
Bankfull Gradient (%):	0.89	
Channel Bed Gradient (%):	0.60	
Riffle Gradient (%):	0.76	*
Riffle Length (m):	5.28	*
Riffle-Pool Spacing (m):	5.19	*

Planform Characteristics

Sinuosity:	0.00
Meander Belt Width (m):	Not measured
Radius of Curvature (m):	Not measured
Meander Amplitude (m):	Not measured
Meander wavelength (m):	Not measured

*riffle-pool development only downstream of the pedestrian crossing

Longitudinal Profile



Bank Characteristics

	Minimum	Maximum	Average
Bank Height (m):	0.25	0.80	0.50
Bank Angle (deg):	30	90	50
Root Depth (m):	0.10	0.60	0.35
Root Density (%):	10	70	26
Bank Undercut (m):	0.00	0.29	0.03

	Minimum	Maximum	Average
Torvane Value (kg/cm^2):		Not measured	
Penetrometer Value (kg/cm^3):		Not measured	
Bank Material (range):		Clay and silt	

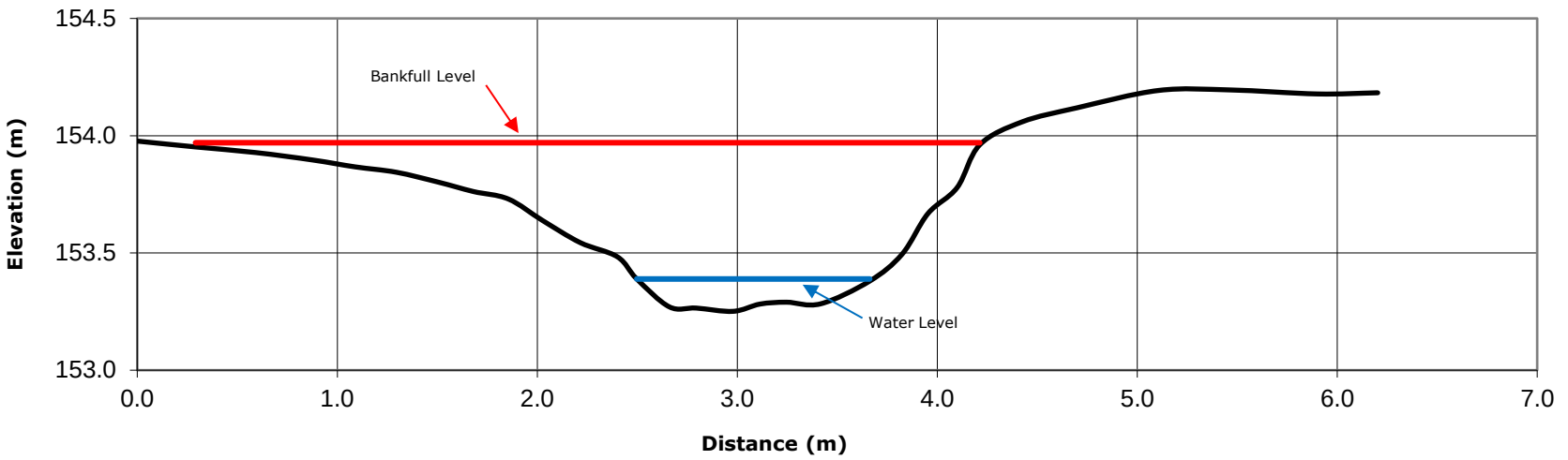
Cross-Sectional Characteristics

	Minimum	Maximum	Average
Bankfull Width (m):	3.03	6.58	4.40
Average Bankfull Depth (m):	0.27	0.58	0.40
Bankfull Width/Depth (m/m):	8	13	11
Wetted Width (m):	0.57	2.04	1.16
Average Water Depth (m):	0.04	0.37	0.19
Wetted Width/Depth (m/m):	1	14	8
Entrenchment (m):		Not measured	
Entrenchment Ratio (m/m):		Not measured	
Maximum Water Depth (m):	0.05	0.35	0.24
Manning's *n* :		0.040	



Photograph at cross section 4 (looking downstream)

Representative Cross-Section 4



Substrate Characteristics

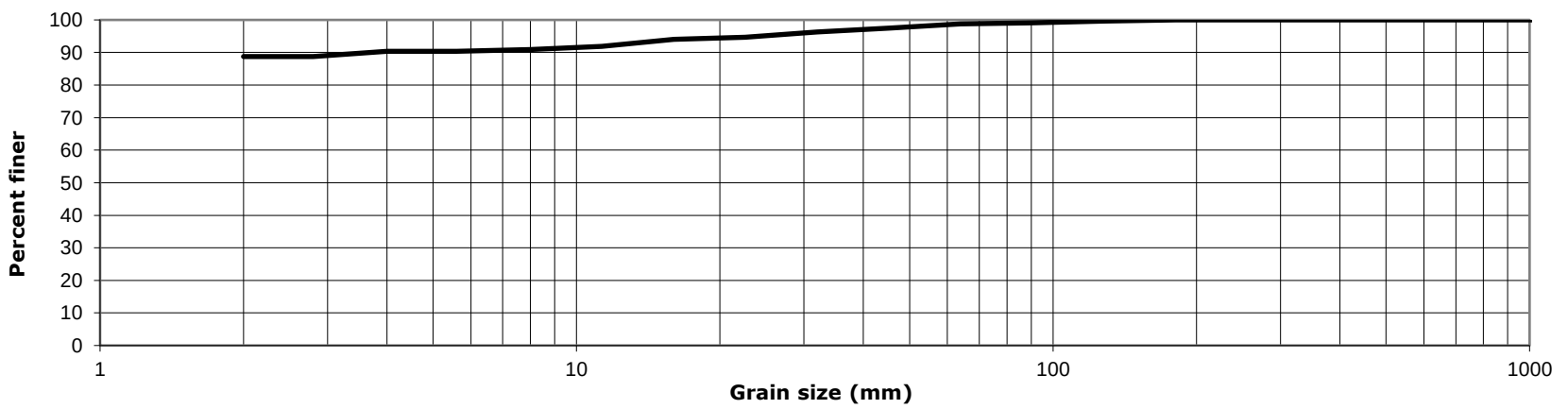
Particle Size (mm)

D_{10} :	<	2.0
D_{50} :	<	2.0
D_{84} :	<	2.0

Bed sample taken

Subpavement:	Till upstream and bedrock downstream
Particle shape:	Platy and angular (cross-section 1)
Embeddedness (%):	<5 (cross-section 1)
Particle range (riffle):	Coarse sand to small cobbles
Particle Range (pool):	Clay, silt, and sand

Cumulative Particle Size Distribution



Channel Thresholds

Flow Competency (m/s):		**Tractive Force at Bankfull (N/m^2):**	34.89
for D_{50}:	0.00	**Tractive Force at 2-year flow (N/m^2):**	Not modelled
for D_{84}:	0.00	**Critical Shear Stress (D_{50}) (N/m^2):**	0.00
Unit Stream Power at Bankfull (W/m^2):	51.00		

General Field Observations

Channel Description

Reach CC1 consisted of a single thread channel with a low sinuosity and a moderate gradient. The reach was more entrenched at the upstream extent, which is apparent at the upstream cross-sections, 7 and 8. An old pedestrian crossing was noted at the downstream extent of the survey, between cross-sections 3 and 4. The riparian vegetation was mostly comprised of trees and formed a continuous buffer. The average bankfull width and depth were 4.40 m and 0.40 m. Bank erosion was observed along approximately 75% of the banks. Leaning trees and exposed tree roots were also observed. Riffle-pool formation was only noted downstream of the pedestrian crossing. The reach consisted of a plain bed upstream of the pedestrian bridge to Scenic Drive. Substrate consisted of clay, silt, and sand except at cross-section 1, where the bedrock was exposed and gravel and cobbles were measured.

Cross Section 7 - Facing Upstream



Appendix E
Meander Belt Width Delineation Memo

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July 23, 2020

Valery (Chedoke Browlands) Developments Inc.
2140 King Street East
Hamilton, Ontario
L8K 1W6

Attention: Ted Valeri

Re: Meander Belt Width Delineation Update
Tributary of Chedoke Creek (Scenic Drive and Sanatorium Road)
City of Hamilton, Ontario
GEO Morphix Project No. 19110

This memo summarizes the meander belt width assessment completed for a section of the Chedoke Creek Tributary at Scenic Drive and Sanatorium Road in the City of Hamilton, Ontario. This work, in part, provides support in the definition of environmental constraints associated with future development on site.

To accommodate the development, a section of the Chedoke Creek Tributary is proposed for re-alignment between Scenic Drive and the Niagara Escarpment. Parish Geomorphic (2009) previously completed an assessment of the tributary and suggested a meander belt width of 42.9 m for the feature. GEO Morphix Ltd. was retained to review the Parish (2009) study and provide an update to the meander belt width in the context of available topographic survey, detailed field observations, and proposed future development on site.

In the case of realignment, the meander belt width is a product of the bankfull characteristics of a channel. If a channel is to be realigned, the meander belt width would need to be adjusted to account for the updated or proposed channel configuration. To refine the meander belt width for this section of the Chedoke Creek Tributary, we have reviewed various background data and reporting, completed site reconnaissance to document existing watercourse characteristics, and updated the meander belt width assessment at a reach scale based on existing information and newly collected field observations.

We have determined a range of meander belt widths for the post-restoration condition by defining a potential bankfull channel based on the following assumptions:

- Valley gradient of 0.9% based on the valley gradient determined through our detailed assessment completed November 29, 2019
- Sinuosity of 1.1 and width to depth ratio of 10:1, which is a stable configuration and representative of the system
- 2-year flow of 1.1 m^3/s and 2.6 m^3/s, determined through our detailed assessment completed November 29, 2019

Dillon Consulting summarized previously calculated 2-year flows for the channel in their report titled City of Hamilton Sanatorium Road Realignment Flood and Erosion Impact Assessment (June 2010). A significant range of 2-year flows were determined through various studies (values ranging from 1.7 m^3/s to 8.4 m^3/s). Dillon (2010) also simulated a maximum flow for the channel over a 16-year period, which resulted in a discharge of 10.6 m^3/s. Given that the maximum flow determined by Dillon is similar to the largest reported 2-year flow (8.4 m^3/s) in the Dillon report (2010), we would suggest that the 2-year flow is likely closer to the lower range of 1.7 m^3/s rather than 8.4 m^3/s.

To verify the 2-year flow information outlined by Dillon (2010), we completed a detailed channel survey to document existing bankfull dimensions. Based on our detailed geomorphological field assessment from November 29, 2020, we determined a bankfull discharge of 1.1 m^3/s for the channel. The bankfull channel indicators observed on site suggested a lower bankfull width and depth that resulted in the 1.1 m^3/s discharge. To be conservative, we also determined a secondary discharge by extending the bankfull channel width to the location where flow would spill into the floodplain. This resulted in a discharge of 2.6 m^3/s. Both discharge values still fall within the lower range of values outlined by Dillon (2010). Although the bankfull discharge of a channel is generally lower than the 2-year flow, it is what would normally be implemented for a channel design.

Based on the measured gradient from our detailed survey and the assigned sinuosity, width and depth for the restored channel were back-calculated using the 1.1 m^3/s and 2.6 m^3/s bankfull discharge determined from our detailed survey. The meander belt width was then determined using the modified Williams (1986) model and the back-calculated channel geometry measurements. The empirical relations from Williams (1986) were modified to include channel width and a 20% factor of safety. The empirical relationships are outlined below:

$$B_w = 18A^{0.65} + W_b \times (1.2) \qquad \text{[Eq. 1]}$$

$$B_w = 4.3{W_b}^{1.12} + W_b \times (1.2) \qquad \text{[Eq. 2]}$$

Where, B_w is meander belt width (m); A is bankfull cross-sectional area (m^2); and W_b is bankfull channel width (m).

The meander belt widths for each discharge scenario are outlined in **Table 1** below. The reported numbers include a 20 percent factor of safety.

Table 1. Meander Belt Widths for Realigned Tributary of Chedoke Creek

Discharge (m^3/s) Scenario	Meander Belt Width (m) Williams – Area Method (1986) [Eq. 1]	Meander Belt Width (m) Williams – Width Method (1986) [Eq. 2]
1.1	23.9	21.4
2.6	35.8	30.5

There are a range of meander belt widths provided as a result of different discharges and empirical models. Generally, most of the determined meander belt widths in **Table 1** fall within or around 30 m. Given that the channel can be designed to accommodate a range of discharges, we are confident that the meander belt width for the realigned channel would not exceed 30 m.

We trust this memo meets your current requirements. Should you have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,

Paul Villard Ph.D., P.Geo., CAN-CISEC, EP, CERP
Director, Principal Geomorphologist

Kat Woodrow, M.Sc.
Environmental Scientist

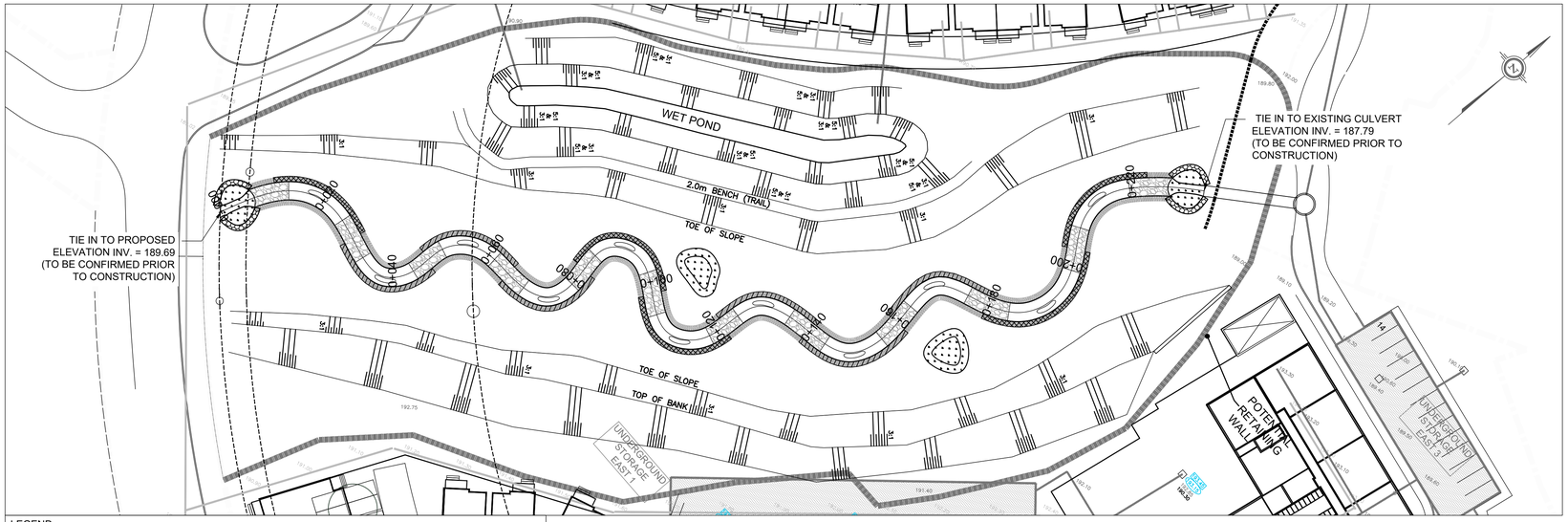
References

Dillon Consulting Limited, June 2010. City of Hamilton Sanatorium Road Realignment Flood and Erosion Impact Assessment. Final Report Submitted to City of Hamilton.

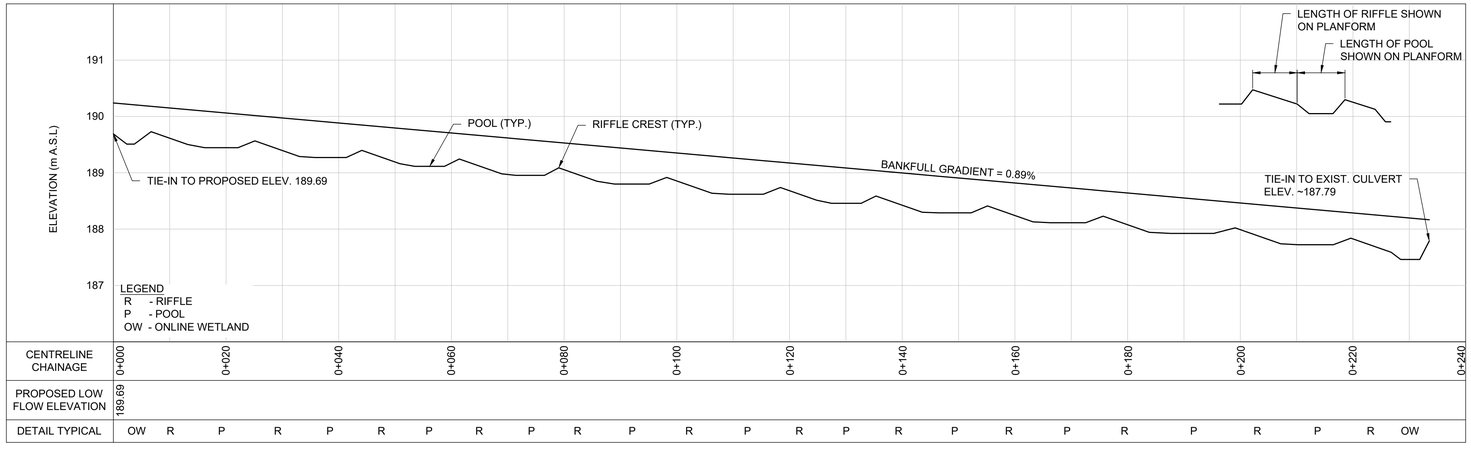
Parish Geomorphic, June 2009. Chedoke Creek Flooding and Erosion Control Impact Assessment, Stage 1: Analysis of Existing Conditions. Final Report Submitted to Dillon Consulting Ltd.

Williams, G.P., 1986. River meanders and channel size. Journal of Hydrology, 88 (1-2): 147-164.

Appendix F
Detailed Design Drawings

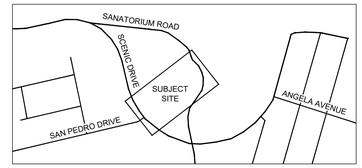


PLANFORM
1:500



PROFILE
H = 1:500; V=1:50

SCALED FOR PLOT ON 'ARCH D'



N
N.T.S.

KEY MAP
N.T.S.

GENERAL NOTES

1. ALL CONTRACT DRAWINGS, SPECIFICATIONS AND APPLICABLE PERMITS MUST BE KEPT ON SITE DURING CONSTRUCTION FOR REFERENCE.
2. THE CONTRACTOR MUST NOTIFY THE CONTRACT ADMINISTRATOR AND CONSERVATION AUTHORITY OF THE INTENT TO COMMENCE WORK AT LEAST 48 HOURS IN ADVANCE.
3. THE CONTRACTOR IS RESPONSIBLE FOR ALL UTILITY LOCATES
4. LAYOUT MUST BE REVIEWED AND APPROVED BY THE CONTRACT ADMINISTRATOR.

TIMING OF WORKS

1. WORKS SHALL BE COMPLETED BETWEEN JULY 1ST TO MARCH 31ST.
2. TREE CLEARING SHOULD BE COMPLETED OUTSIDE THE BIRD NESTING SEASON TO COMPLY WITH THE FEDERAL MIGRATORY BIRDS CONVENTION ACT. ANY TREES THAT REQUIRE REMOVAL OUTSIDE OF THIS TIMING WINDOW MUST FIRST BE INSPECTED BY A QUALIFIED BIOLOGIST TO DETERMINE THE PRESENCE OF NESTING BIRDS.
3. THE WEATHER FORECAST SHOULD BE CONTINUALLY MONITORED TO ENSURE THAT WORKS ARE UNDERTAKEN ONLY DURING FAVOURABLE WEATHER CONDITIONS.
4. COMPLETE THE WORKS WITH MINIMAL AVOIDABLE INTERRUPTIONS ONCE THEY COMMENCE.

SITE AND MATERIAL MANAGEMENT

1. ALL CONSTRUCTION EQUIPMENT AND MATERIALS (IMPORTED OR EXCAVATED) MUST BE STORED AT LEAST 30 m AWAY FROM ANY WATERBODY IN A STABLE AREA ABOVE THE ACTIVE FLOODPLAIN, OR IN A DESIGNATED STAGING/STORAGE AREA.
2. IN THE EVENT OF AN UNEXPECTED STORM, ALL UNFIXED ITEMS THAT HAVE THE POTENTIAL TO CAUSE A SPILL OR AN OBSTRUCTION TO FLOW MUST BE MOVED A STABLE AREA ABOVE ACTIVE FLOODPLAIN.
3. STOCKPILES MUST BE LOCATED OUTSIDE THE ISOLATED WORK AREAS.
4. STABILIZE STOCKPILED SOILS THAT ARE STORED FOR PROLONGED PERIODS WITH THE APPLICATION OF A NURSE CROP AT A RATE OF 60 kg/ha.
5. STABILIZE, TEMPORARILY OR PERMANENTLY, ANY DISTURBED AREAS AS WORK PROGRESSES, OR SOON AS CONDITIONS ALLOW. ON SOILS THAT WILL BE EXPOSED FOR PROLONG PERIODS, TEMPORARILY INSTALL A BIODEGRADABLE EROSION CONTROL BLANKET ON EXPOSED SOILS, OR APPLY A NURSE CROP AT A RATE OF 60 KG/HA.
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1.	2020-07-24	BWM	FIRST SUBMISSION
	DATE	BY	REVISIONS

DESIGNED BY: P.V.	CHECKED BY: P.V.
DRAWN BY: B.W.M.	DATE: JULY 2020

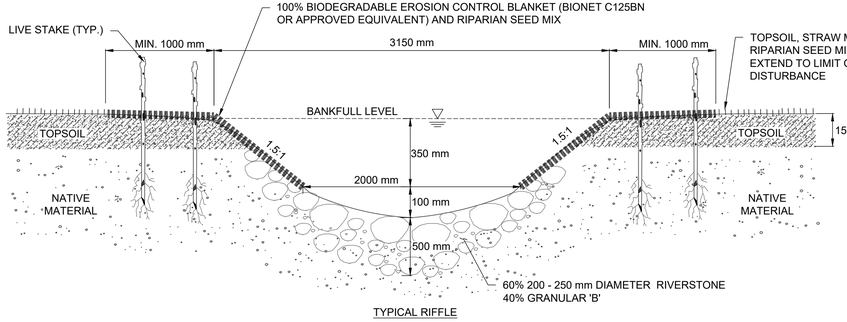
PROFESSIONAL GEOSCIENTIST
PAUL V. VILLARD
PRACTISING MEMBER
0957
ONTARIO
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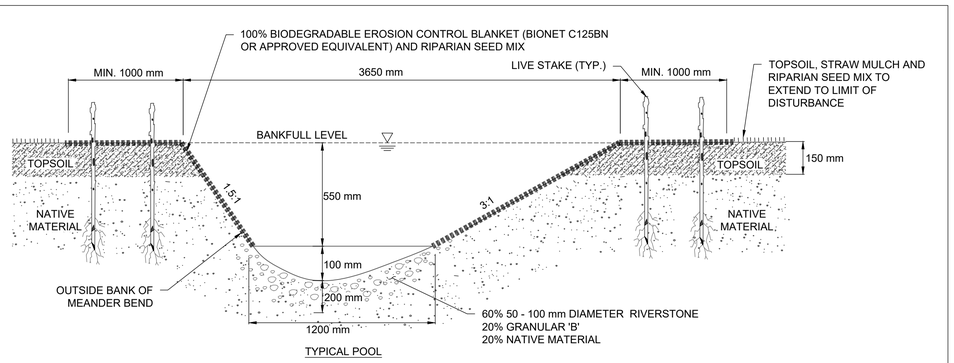
SCENIC DRIVE AND SANATORIUM ROAD VALERY HOMES

CHEDOKE CREEK PLANFORM AND PROFILE

PROJECT No.: 19010	DRAWING No.: GEO-1
SCALE: AS NOTED	SHEET 1 OF 3

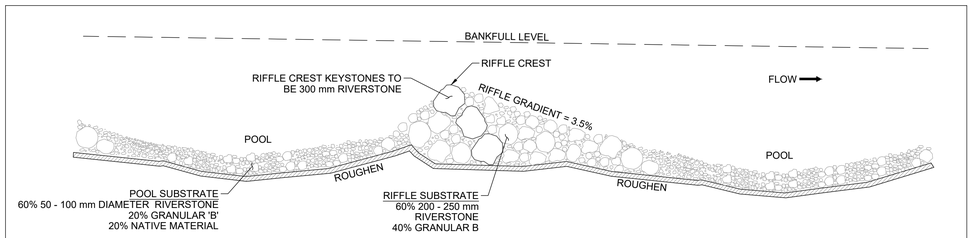


TYPICAL RIFFLE

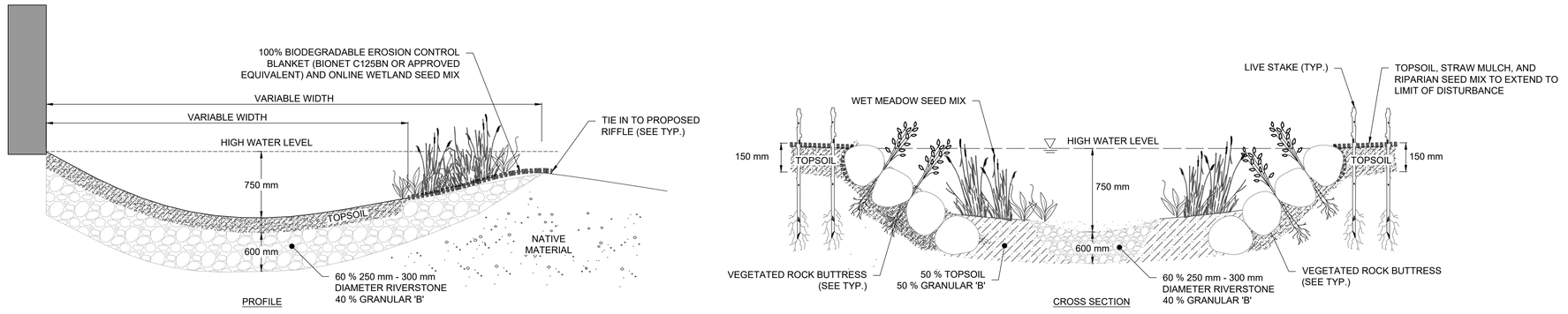


TYPICAL POOL

CHANNEL CROSS SECTIONS
N.T.S.



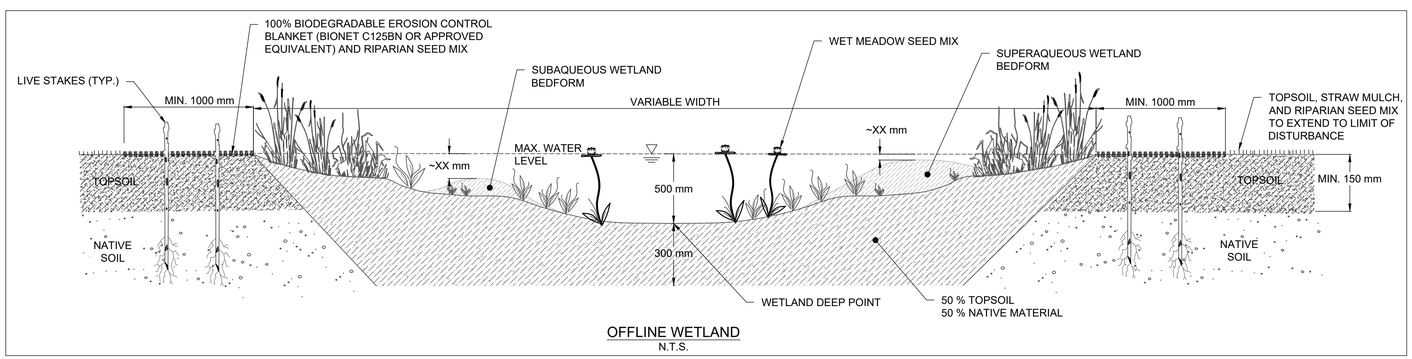
TYPICAL RIFFLE-POOL SEQUENCE
N.T.S.



PROFILE

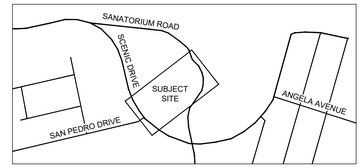
CROSS SECTION

ONLINE WETLAND / PLUNGE POOL
N.T.S.



OFFLINE WETLAND
N.T.S.

SCALED FOR PLOT ON 'ARCH D'



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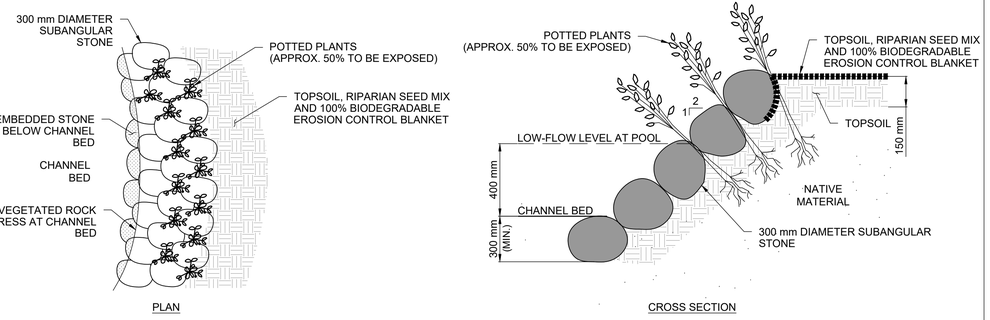
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SCENIC DRIVE AND SANATORIUM ROAD VALERY HOMES

CHEDOKE CREEK DESIGN DETAILS

PROJECT No.: 19010	DRAWING No.: DET-1
SCALE: AS NOTED	SHEET 2 OF 3



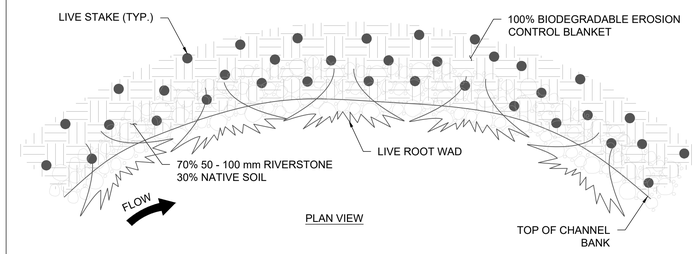
CONSTRUCTION NOTES

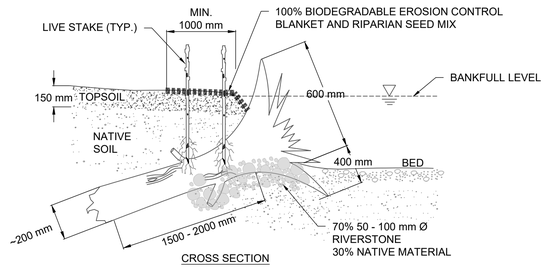
1. VEGETATED ROCK BUTTRESS TO BE INSTALLED IN LIFTS.
2. TOE STONES TO BE EMBEDDED INTO CHANNEL BED A MINIMUM OF 300 mm.
3. INSTALL PLANTS 1 m O/C IN EACH LAYER.
4. LOWEST LAYER OF PLANTS TO BE AT LOW-FLOW LEVEL.
5. LATERALLY STAGGER EACH SUCCESSIVE LAYER OF PLANTS TO AVOID VERTICAL STACKING.
6. WILLOW SPECIES TO BE CONCENTRATED IN LOWER LIFTS.

PLANTINGS SPECIES AND QUANTITIES

COMMON NAME	SCIENTIFIC NAME	QUANTITY	CONDITION
RED OSIER DOGWOOD	*Salix discolor*	40	1 m ht. POTTED
SANDBAR WILLOW	*Salix exigua*	40	1 m ht. POTTED
SHINING WILLOW	*Salix lucida*	40	1 m ht. POTTED

VEGETATED ROCK BUTTRESS
N.T.S.

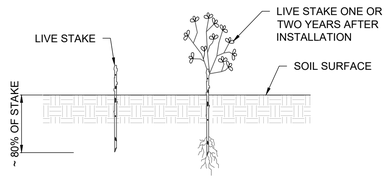




NOTES

1. SALVAGE NATIVE TREES ON SITE, IF POSSIBLE, AND CUT TO DIMENSIONS SHOWN.
2. ROOT WAD TREATMENT TO EXTEND FULL LENGTH OF OUTSIDE BANK OF MEANDER BEND BETWEEN RIFFLES WHERE SHOWN ON PLANFORM.

ROOT WAD BANK ENHANCEMENT WITH STONE BEDDING
N.T.S.



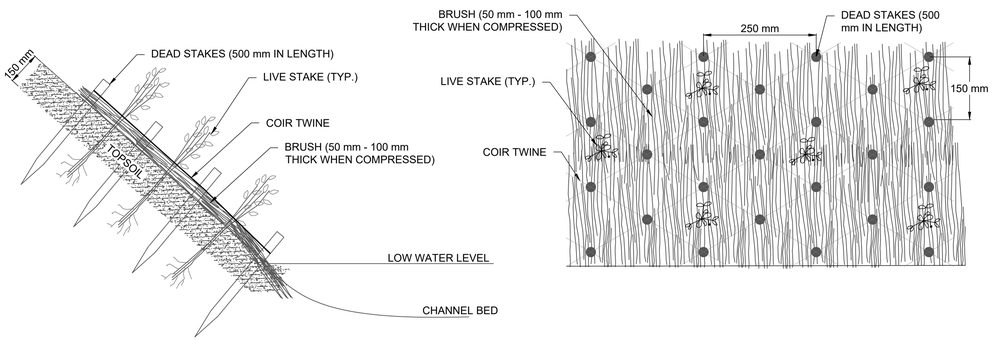
NOTES

1. QUANTITY TO BE DETERMINED BASED ON AREA OF DISTURBANCE TO BE RESTORED
2. LIVE STAKES SHOULD BE FROM AT MINIMUM 2-YEAR OLD STOCK.
3. LIVE STAKES ARE TO BE INSTALLED AT A DENSITY OF 3 STAKES PER SQUARE METRE.
4. LIVE STAKES SHOULD BE PRE-SOAKED (SUBMERGED IN WATER) FOR AT LEAST 24 HOURS AFTER HARVESTING AND IMMEDIATELY BEFORE INSTALLATION.
5. LIVE STAKES SHOULD NOT BE STORED FOR A PERIOD LONGER THAN 2 DAYS, UNLESS THEY ARE BEING SOAKED.
6. THE CONTRACTOR SHALL PROTECT PLANT MATERIALS FROM DRYING FROM THE TIME OF HARVEST UNTIL INSTALLED.
7. LIVE STAKES ARE TO BE A MINIMUM OF 25 mm IN DIAMETER AND CUT TO A LENGTH OF 1000 mm.
8. CUT ANGLE AT THE BOTTOM OF THE STAKE AND FLAT ON THE TOP.
9. TRIM ALL SIDE BRANCHES WHILE TAKING CARE NOT TO DAMAGE THE BARK.
10. INSTALL STAKES WITH BUDS POINTING UPWARDS AND THICKER STEM IN THE BED.
11. LIVE STAKES SHOULD BE INSTALLED USING A LARGE RUBBER MALLET.
12. 80% OF THE STAKE IS TO BE BELOW SURFACE.
13. TAMP THE LIVE STAKE INTO THE GROUND AT RIGHT ANGLE TO THE SURFACE.
14. IN COMPACT SOIL A PILOT HOLE SHOULD BE USED TO LIMIT DAMAGE TO THE STAKES.
15. IF USING A PILOT HOLE REPACK SOIL AROUND THE LIVE STAKE.
16. LIVE STAKES SHOULD STAND FIRM FROM THE SOIL FOLLOWING INSTALLATION.
17. ALL STAKES NOT PLANTED TO THE SPECIFICATIONS ABOVE WILL BE REPLACED AT THE CONTRACTOR'S EXPENSE.

SPECIES AND QUANTITIES

COMMON NAME	SCIENTIFIC NAME	QTY	CONDITION
RED OSIER DOGWOOD	*Cornus stolonifera*	725	1 m, BARE ROOT
PUSSY WILLOW	*Salix discolor*	725	1 m, BARE ROOT
SANDBAR WILLOW	*Salix exigua*	725	1 m, BARE ROOT

LIVE STAKE
N.T.S.



NOTES

1. LIVE BRANCHES TO CONSIST OF WILLOW AND DOGWOOD SPECIES, APPROXIMATELY 1 m IN LENGTH AND 50 mm - 100 mm IN WIDTH.
2. BRANCHES TO BE KEPT IN MOIST AND COLD UNTIL INSTALLATION.
3. BRUSH MATTRESS TO BE INSTALLED WHILE BRANCHES ARE DORMANT.
4. BRANCHES TO BE PLACED ON SLOPE WITH BUTT END TOWARDS VALLEY FLOOR AND PUSHED INTO SOIL.
5. BRANCHES MUST BE FLEXIBLE ENOUGH TO CONFORM TO THE SLOPE SURFACE IRREGULARITIES.
6. POUND DEAD STAKES TO HALF THEIR LENGTH INTO SOIL BETWEEN BRANCHES. TIE COIR TWINE AROUND DEAD STAKES AND TIGHTLY OVER BRANCHES. USE A CLOVE HITCH TO SECURE STAKES. POUND STAKES INTO SLOPE TO COMPRESS BRANCHES AGAINST GROUND.
7. TAMP LIVE STAKES BETWEEN DEAD STAKES.
8. FILL VOIDS BETWEEN BRANCHES OF THE BRUSH MATTRESS WITH SOIL TO PROMOTE ROOTING.

BRUSH MATTRESS
N.T.S.

RIPARIAN SEED MIX

COMMON NAME	SPECIES	PERCENTAGE
BIG BLUESTEM	*Andropogon gerardii*	25
VIRGINIA WILD RYE	*Elymus virginicus*	5
FOWL BLUEGRASS	*Poa palustris*	25
FOWL MEADOW GRASS	*Glyceria striata*	2
FOX SEDGE	*Carex vulpinoidea*	25
NEW ENGLAND ASTER	*Symphyotrichum novae-angliae*	2
ALLEGHENY MONKEYFLOWER	*Mimulus ringens*	5
AWL FRUITED SEDGE	*Carex stipata*	2
BEBB'S SEDGE	*Carex bebbii*	2
SPOTTED JOE PYE WEED	*Eupatorium maculatum*	1
BONESET	*Eupatorium perfoliatum*	2
BLADDER SEDGE	*Carex intumescens*	1
SWAMP ASTER	*Symphyotrichum puniceum*	2
CANADA ANEMONE	*Anemone canadensis*	1

NOTES

1. APPLY SEED MIX AT A RATE OF 30 kg PER HECTARE.
2. SEEDING SHALL OVERLAP ADJACENT GROUND COVER BY 300 mm.
3. APPLY A NURSE CROP OF ANNUAL OATS (*Avena Sativa*) AT A RATE OF 60 kg PER HECTARE.
4. WATER SOIL AFTER SEED APPLICATION.

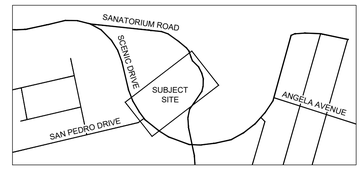
WET MEADOW SEED MIX

COMMON NAME	SPECIES	PERCENTAGE
AWL FRUITED SEDGE	*Carex stipata*	5
BEBB'S SEDGE	*Carex bebbii*	5
FOX SEDGE	*Carex vulpinoidea*	25
BLACK EYED SUSAN	*Rudbeckia hirta*	5
BLADDER SEDGE	*Carex intumescens*	5
BIG BLUESTEM	*Andropogon gerardii*	2
INDIAN GRASS	*Sorghastrum nutans*	6
SPOTTED JOE PYE WEED	*Eupatorium maculatum*	1
FOWL MEADOW GRASS	*Poa palustris*	1
TUSSOCK SEDGE	*Carex stricta*	5
VIRGINIA WILD RYE	*Elymus virginicus*	25
COMMON MILKWEED	*Asclepias syriaca*	12
BLACK EYED SUSAN	*Rudbeckia hirta*	2
NEW ENGLAND ASTER	*Symphyotrichum novae-angliae*	1

NOTES

1. APPLY SEED MIX AT A RATE OF 30 kg PER HECTARE.
2. SEEDING SHALL OVERLAP ADJACENT GROUND COVER BY 300 mm.
3. SIMULTANEOUSLY APPLY A NURSE CROP OF ANNUAL OATS (*Avena sativa*) AT A RATE OF 60 kg PER HECTARE.
4. WATER SOIL AFTER SEED APPLICATION.

SCALED FOR PLOT ON 'ARCH D'



KEY MAP
N.T.S.

GENERAL NOTES

1. ALL CONTRACT DRAWINGS, SPECIFICATIONS AND APPLICABLE PERMITS MUST BE KEPT ON SITE DURING CONSTRUCTION FOR REFERENCE.
2. THE CONTRACTOR MUST NOTIFY THE CONTRACT ADMINISTRATOR AND CONSERVATION AUTHORITY OF THE INTENT TO COMMENCE WORK AT LEAST 48 HOURS IN ADVANCE.
3. THE CONTRACTOR IS RESPONSIBLE FOR ALL UTILITY LOCATES.
4. LAYOUT MUST BE REVIEWED AND APPROVED BY THE CONTRACT ADMINISTRATOR.

TIMING OF WORKS

1. WORKS SHALL BE COMPLETED BETWEEN JULY 1ST TO MARCH 31ST.
2. TREE CLEARING SHOULD BE COMPLETED OUTSIDE THE BIRD NESTING SEASON TO COMPLY WITH THE FEDERAL MIGRATORY BIRDS CONVENTION ACT. ANY TREES THAT REQUIRE REMOVAL OUTSIDE OF THIS TIMING WINDOW MUST FIRST BE INSPECTED BY A QUALIFIED BIOLOGIST TO DETERMINE THE PRESENCE OF NESTING BIRDS.
3. THE WEATHER FORECAST SHOULD BE CONTINUALLY MONITORED TO ENSURE THAT WORKS ARE UNDERTAKEN ONLY DURING FAVOURABLE WEATHER CONDITIONS.
4. COMPLETE THE WORKS WITH MINIMAL AVOIDABLE INTERRUPTIONS ONCE THEY COMMENCE.

SITE AND MATERIAL MANAGEMENT

1. ALL CONSTRUCTION EQUIPMENT AND MATERIALS (IMPORTED OR EXCAVATED) MUST BE STORED AT LEAST 30 m AWAY FROM ANY WATERBODY IN A STABLE AREA ABOVE THE ACTIVE FLOODPLAIN, OR IN A DESIGNATED STAGING/STORAGE AREA.
2. IN THE EVENT OF AN UNEXPECTED STORM, ALL UNFIXED ITEMS THAT HAVE THE POTENTIAL TO CAUSE A SPILL OR AN OBSTRUCTION TO FLOW MUST BE MOVED A STABLE AREA ABOVE ACTIVE FLOODPLAIN.
3. STOCKPILES MUST BE LOCATED OUTSIDE THE ISOLATED WORK AREAS.
4. STABILIZE STOCKPILED SOILS THAT ARE STORED FOR PROLONGED PERIODS WITH THE APPLICATION OF A NURSE CROP AT A RATE OF 60 kg/ha.
5. STABILIZE, TEMPORARILY OR PERMANENTLY, ANY DISTURBED AREAS AS WORK PROGRESSES, OR SOON AS CONDITIONS ALLOW. ON SOILS THAT WILL BE EXPOSED FOR PROLONG PERIODS, TEMPORARILY INSTALL A BIODEGRADABLE EROSION CONTROL BLANKET ON EXPOSED SOILS, OR APPLY A NURSE CROP AT A RATE OF 60 KG/HA.
6. MINIMIZE THE AREA OF DISTURBANCE TO THE EXTENT POSSIBLE.
7. ALL VEGETATION, ADJACENT TO THE WORK AREA, MUST BE PROTECTED AND DELINEATED WITH CONSTRUCTION FENCING OR TREE PROTECTION BARRIERS.
8. ALL GRADES IN THE AREA REGULATED BY THE CONSERVATION AUTHORITY MUST BE MAINTAINED OR MATCHED, UNLESS OTHERWISE AUTHORIZED IN THE APPLICABLE PERMIT.

EROSION AND SEDIMENT CONTROL

1. ALL TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES MUST BE INSTALLED PRIOR TO START OF WORKS.
2. SEDIMENT CONTROLS MUST BE INSPECTED DAILY TO ENSURE THAT THEY ARE IN GOOD REPAIR AND FUNCTIONING AS INTENDED.
3. EROSION AND SEDIMENT CONTROLS MUST BE MAINTAINED DURING CONSTRUCTION, AND ANY REQUIRED REPAIRS OR REPLACEMENTS MUST BE COMPLETED WITHIN 24 HOURS AFTER THEY HAVE BEEN IDENTIFIED DURING THE MONITORING.
4. EROSION AND SEDIMENT CONTROLS MAY REQUIRE PERIODIC ADJUSTMENTS TO REFLECT CHANGING SITE CONDITIONS. THE CONTRACTOR WILL BE RESPONSIBLE FOR THESE ADJUSTMENTS TO ENSURE PROPER FUNCTION.
5. ANY CHANGES TO THE EROSION AND SEDIMENT CONTROL PLAN BEYOND MINOR ADJUSTMENTS MUST BE APPROVED BY THE CONTRACT ADMINISTRATOR.
6. ADDITIONAL EROSION AND SEDIMENT CONTROL SUPPLIES MUST BE KEPT ON SITE IN ORDER TO FACILITATE IMMEDIATE REPAIRS AND/OR UPGRADES AS NEEDED.
7. ALL TEMPORARY SEDIMENT CONTROLS MUST BE REMOVED AFTER THE CONTRACT ADMINISTRATOR DEEMS THE SITE TO BE STABLE.

DELETERIOUS SUBSTANCE CONTROL/SPILL MANAGEMENT

1. PREVENT THE RELEASE OF SEDIMENT, SEDIMENT-LADEN WATER, RAW CONCRETE, CONCRETE LEACHATE OR ANY OTHER DELETERIOUS SUBSTANCES INTO ANY WATERBODY, RAVINE OR STORM SEWER SYSTEM.
2. ENSURE EQUIPMENT AND MACHINERY ARE IN GOOD OPERATING CONDITION (POWER WASHED), FREE OF LEAKS, EXCESS OIL, AND GREASE.
3. NO EQUIPMENT REFUELLING OR SERVICING SHOULD BE UNDERTAKEN WITHIN 30 m OF ANY WATERCOURSE OR SURFACE WATER DRAINAGE.
4. A SPILL CONTAINMENT KIT MUST BE READILY ACCESSIBLE ON SITE IN THE EVENT OF A RELEASE OF A DELETERIOUS SUBSTANCE TO THE ENVIRONMENT. ONSITE STAFF MUST BE TRAINED IN ITS USE.
5. THE CONTRACT ADMINISTRATOR MUST BE NOTIFIED IMMEDIATELY IN THE EVENT OF A SPILL OF DELETERIOUS SUBSTANCE.

WORK AREA ISOLATION

1. ALL WORK IN ISOLATED WORK AREAS MUST BE COMPLETED IN THE DRY. AN ADEQUATE NUMBER OF PUMPS MUST BE USED FOR UNWATERING.
2. THE UNWATERING DISCHARGE LOCATION MUST BE LOCATED AT LEAST 30 M FROM ANY WATERCOURSE OR WETLAND IN AN AREA WITH DENSE VEGETATIVE GROUNDCOVER, AND WHERE THE DISCHARGE CAN RETURN TO THE WATERBODY DOWNSTREAM OF THE WORK AREA OVER THE GROUNDCOVER.
3. FISH MUST BE REMOVED FROM THE WORK AREA ONCE ISOLATED. FISH SALVAGE MUST BE COMPLETED BY A QUALIFIED TECHNICIAN WITH A LICENSE FROM THE ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY.

1.	2020-07-24	BWM	FIRST SUBMISSION
	DATE	BY	REVISIONS

DESIGNED BY: P.V.	CHECKED BY: P.V.
DRAWN BY: B.W.M.	DATE: JULY 2020

PROFESSIONAL GEOSCIENTIST ONTARIO
PAUL V. VILLARD
PRACTISING MEMBER
0957
2020-07-24

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SCENIC DRIVE AND SANATORIUM ROAD VALERY HOMES

CHEDOKE CREEK DESIGN DETAILS

PROJECT No.: 19010	DRAWING No.: DET-2
SCALE: AS NOTED	SHEET 3 OF 3

APPENDIX D:

Sanitary Sewer Calculations

Estimated Sanitary Demand per Ontario Building Code (Part 8)

Project: Chedoke Mountain Brow -West Site- 204 Apartment Units and 56 Townhome Units
Job No.: TPB188083
Date: August 2020
Prepared by: J.W. Enos

Sanitary Demand for Apartment Building

Residential Occupancy (OBC Table 8.2.1.3.A. Residential Occupancy)
Type of Development = Apartments (275 litres per person per day)

Number of Units = 204 — 102 - 1 bedroom units; 102- 2 bedroom units; 0 - 3 bedroom units

Per OBC Section 3.1.17.1. Occupant Load Determination

2 persons per sleeping room

For number of people

102 - 1 bedrooms x 2 people =		204	people
102 - 2 bedrooms x 4 people =		408	people
0 - 3 bedrooms x 6 people =		0	people
Total	=	612	people

For amenity area of $0m^2$

$0m^2$ / $9.3m^2$ per person	=	0	people

For commercial area of $0m^2$

$0m^2$ / 3.7 m^2 per person =	=	0	people
Apartment Building Total	**=**	**612**	people

Flow = 275 Litres / day / per person

Total Daily Sanitary Flow = 612 people x 275 L/d/ca = 168,300 Litres / day
= **1.95 Litres / sec** **Sanitary Demand**

Water Usage for Apartment Buildings

Based on sanitary flow = 168,300 Litres / day
= **1.95 Litres / sec**

Sanitary Demand for Townhomes

Residential Occupancy (OBC Table 8.2.1.3.A. Residential Occupancy)
Type of Development = Townhomes (1600 litres per 3 bedroom unit)

Per OBC Table 8.2.1.3.A.

` Number of 3 Bedroom Townhomes = 56

Total Daily Sanitary Flows for 56 Townhomes= 56x1600= 89,600 Litres / day
1.04 Litres / sec **Sanitary Flow**

Water Usage for Townhomes

Based on sanitary flow = 89,600 Litres / day
= **1.04 Litres / sec**

Total Daily Sanitary Flow for Apartment Buildings and Townhomes Combined = 257,900 Litres / day
2.98 Litres / sec **Sanitary Flow**

Total Daily Water Usage Flow for Apartment Buildings and Townhomes Combined = 257,900 Litres / day
2.98 Litres / sec

This is only preliminary and subject to change based on Architectural and Mechanical design.

Estimated Sanitary Demand per Ontario Building Code (Part 8)

Project: Chedoke Mountain Brow -East Site- 370 Apartment Units
Job No.: TPB188083
Date: August 2020
Prepared by: J.W. Enos

Sanitary Demand for Apartment Buildings (Phase 1)

Residential Occupancy (OBC Table 8.2.1.3.A. Residential Occupancy)
Type of Development = Apartments (275 litres per person per day)

Number of Units = 254
- 127 - 1 bedroom units
- 127 - 2 bedroom units
- 0 - 3 bedroom units

Per OBC Section 3.1.17.1. Occupant Load Determination

2 persons per sleeping room

For number of people

127 - 1 bedrooms x 2 people	=	254	people
127 - 2 bedrooms x 4 people	=	508	people
0 - 3 bedrooms x 6 people	=	0	people
Total	=	762	people

For office area (Heritage Building) of $600m^2$

$600m^2$ / $9.3m^2$ per person	=	65	people

For commercial area of $0m^2$

$0m^2$ / 3.7 m^2 per person =	=	0	people
Grand Total	**=**	**827**	people

Flow = 275 Litres / day / per person

Total Daily Flow = 827 people x 275 L/d/ca = 227,425 Litres / day
= **2.63 Litres / sec** **Sanitary Demand**

Water Usage for Apartment Buildings (Phase 1)

Based on sanitary demand = 227,425 Litres / day
= **2.63 Litres / sec**

This is only preliminary and subject to change based on Architectural and Mechanical design.

Estimated Sanitary Demand per Ontario Building Code (Part 8)

Project: Chedoke Mountain Brow -East Site- 370 Apartment Units
Job No.: TPB188083
Date: August 2020
Prepared by: J.W. Enos

Sanitary Demand for Apartment Buildings (Phase 2)

Residential Occupancy (OBC Table 8.2.1.3.A. Residential Occupancy)
Type of Development = Apartments (275 litres per person per day)

Number of Units = 116
- 58 - 1 bedroom units
- 58 - 2 bedroom units
- 0 - 3 bedroom units

Per OBC Section 3.1.17.1. Occupant Load Determination

2 persons per sleeping room

For number of people

58 - 1 bedrooms x 2 people	=	116	people
58 - 2 bedrooms x 4 people	=	232	people
0 - 3 bedrooms x 6 people	=	0	people
Total	=	348	people

For amenity area of 0 m^2

0m^2 / 9.3m^2 per person	=	0	people

For commercial area of 0m^2

0m^2 / 3.7 m^2 per person =	=	0	people
Grand Total	**=**	**348**	people

Flow = 275 Litres / day / per person

Total Daily Flow = 348 people x 275 L/d/ca = 95,700 Litres / day
= **1.11 Litres / sec** **Sanitary Demand**

Water Usage for Apartment Buildings (Phase 2)

Based on sanitary demand = 95,700 Litres / day
= **1.11 Litres / sec**

This is only preliminary and subject to change based on Architectural and Mechanical design.

CITY OF HAMILTON

Sanitary Sewer Design

Location: CHEDOKE MOUNTAIN BROW CONDOS - WEST SITE
Checked By: J.W. Enos
Date: 27-Aug-20

Flow Factor: 360 l/day/cap
Peaking Factor: 5*(P^-0.2)
Infiltration: 0.4 l/s/ha for Deep Sorm
0.6 0.6 l/s/ha for Shallow Storm

Area No.	Street Name	From MH	To MH	Pop. Density (ppl/ha)	Area (ha)	Cumm. Area (ha)	Equiv. Pop.	Cumm. Equiv. Pop.	Sanitary Flow			Infilt (l/s)	Total Flow (l/s)	SEWER DESIGN								
									Peak Factor	Q Avg (l/s)	Q Peak (l/s)			Dia. (mm)	Type	Grade (%)	Mann. n	Vel. (m/s)	Q Full (l/s)	% Flow	% Vel.	Actual Vel. (m/s)
A1	SITE ROAD	A6	A5	-	0.18	0.18	22	22	5.00	0.09	0.5	0.1	0.6	200		1.00	0.015	0.91	29.7	2%		
A2	SITE ROAD	A5	A4	-	0.74	0.92	269	291	5.00	1.21	6.1	0.6	6.6	200		0.50	0.015	0.65	21.0	32%		
A3	SITE ROAD	A4	A3	-	0.72	1.64	247	538	5.00	2.24	11.2	1.0	12.2	200		0.50	0.015	0.65	21.0	58%		
A4	SITE ROAD	A7	A3	-	0.17	0.17	31	31	5.00	0.13	0.6	0.1	0.7	200		1.00	0.015	0.91	29.7	3%		
A5	SITE ROAD	A3	A2	-	0.17	1.98	22	591	5.00	2.46	12.3	1.2	13.5	200		0.50	0.015	0.65	21.0	64%		
	SITE ROAD	A2	A1	-	0	1.98	0	591	5.00	2.46	12.3	1.2	13.5	200		3.30	0.015	1.66	53.9	25%		
	SITE ROAD	A1	MAIN	-	0	1.98	0	591	5.00	2.46	12.3	1.2	13.5	200		1.00	0.015	0.91	29.7	46%		

CITY OF HAMILTON

Sanitary Sewer Design

Location: CHEDOKE MOUNTAIN BROW CONDOS - WEST SITE
Checked By: J.W. Enos
Date: 27-Aug-20

Flow Factor: 360 l/day/cap
Peaking Factor: 5*(P^-0.2)
Infiltration: 0.4 l/s/ha for Deep Sorm
0.6 0.6 l/s/ha for Shallow Storm

Area No.	Street Name	From MH	To MH	Pop. Density (ppl/ha)	Area (ha)	Cumm. Area (ha)	Equiv. Pop.	Cumm. Equiv. Pop.	Sanitary Flow			Infilt (l/s)	Total Flow (l/s)	SEWER DESIGN								
									Peak Factor	Q Avg (l/s)	Q Peak (l/s)			Dia. (mm)	Type	Grade (%)	Mann. n	Vel. (m/s)	Q Full (l/s)	% Flow	% Vel.	Actual Vel. (m/s)
A6	SITE ROAD	A14	A13	-	0.15	0.15	18	18	5.00	0.08	0.4	0.1	0.5	200		1.00	0.015	0.91	29.7	2%		
A7	SITE ROAD	A13	A12	-	0.10	0.25	13	31	5.00	0.13	0.6	0.2	0.8	200		0.50	0.015	0.65	21.0	4%		
A8	SITE ROAD	A12	A11	-	0.21	0.46	31	62	5.00	0.26	1.3	0.3	1.6	200		0.50	0.015	0.65	21.0	7%		
A9	SITE ROAD	A11	A9	-	0.12	0.58	9	71	5.00	0.30	1.5	0.3	1.8	200		0.50	0.015	0.65	21.0	9%		
A10	SITE ROAD	A7	A10	-	0.20	0.20	35	35	5.00	0.15	0.7	0.1	0.8	200		1.00	0.015	0.91	29.7	3%		
A11	SITE ROAD	A10	A9	-	0.13	0.33	18	53	5.00	0.22	1.1	0.2	1.3	200		0.50	0.015	0.65	21.0	6%		
A12	SITE ROAD	A9	A8	-	0.07	0.98	0	124	5.00	0.52	2.6	0.6	3.2	200		0.50	0.015	0.65	21.0	15%		
	SITE ROAD	A8	EX MH	-	0.00	0.98	0	124	5.00	0.52	2.6	0.6	3.2	200		0.50	0.015	0.65	21.0	15%		

CITY OF HAMILTON

Sanitary Sewer Design

Location: CHEDOKE MOUNTAIN BROW CONDOS - EAST SITE
Checked By: J.W. Enos
Date: 18-Aug-20

Flow Factor: 360 l/day/cap
Peaking Factor: 5*(P^-0.2)
Infiltration: 0.4 l/s/ha for Deep Sorm
0.6 0.6 l/s/ha for Shallow Storm

Area No.	Street Name	From MH	To MH	Pop. Density (ppl/ha)	Area (ha)	Cumm. Area (ha)	Equiv. Pop.	Cumm. Equiv. Pop.	Sanitary Flow			Infilt (l/s)	Total Flow (l/s)	SEWER DESIGN								
									Peak Factor	Q Avg (l/s)	Q Peak (l/s)			Dia. (mm)	Type	Grade (%)	Mann. n	Vel. (m/s)	Q Full (l/s)	% Flow	% Vel.	Actual Vel. (m/s)
A1	SITE (PHASE 2)	A28	A26	-	0.64	0.64	266	266	5.00	1.11	5.5	0.4	5.9	200		1.00	0.015	0.91	29.7	20%		
	SCENIC DRIVE	A26	A25	-	0	0.64	0	266	5.00	1.11	5.5	0.4	5.9	250		0.50	0.015	0.75	38.0	16%		
	SCENIC DRIVE	A25	A24	-	0	0.64	0	266	5.00	1.11	5.5	0.4	5.9	250		0.50	0.015	0.75	38.0	16%		
	SCENIC DRIVE	A24	A23	-	0	0.64	0	266	5.00	1.11	5.5	0.4	5.9	250		0.50	0.015	0.75	38.0	16%		
A2	SITE (PHASE 1)	A27	A23	-	1.4	1.40	632	632	5.00	2.64	13.2	0.8	14.0	200		1.00	0.015	0.91	29.7	47%		
	SCENIC DRIVE	A24	A23	-	0	2.04	0	898	5.00	3.74	18.7	1.2	19.9	250		0.50	0.015	0.75	38.0	52%		
	SCENIC DRIVE	A23	EX A21	-	0	2.04	0	898	5.00	3.74	18.7	1.2	19.9	250		0.50	0.015	0.75	38.0	52%		

CITY OF HAMILTON

Sanitary Sewer Design

Location: EXTERNAL AREAS - EXISTING CONDITIONS
Checked By: MJS/JWE
Date: 27-Aug-20

Flow Factor: 360 l/day/cap
Peaking Factor: 5*(P^-0.2)
Infiltration: 0.4 l/s/ha for Deep Sorm
0.6 **0.6 l/s/ha for Shallow Storm**

Area No.	Street Name	From MH	To MH	Pop. Density (ppl/ha)	Area (ha)	Cumm. Area (ha)	Equiv. Pop.	Cumm. Equiv. Pop.	Sanitary Flow			Infilt (l/s)	Total Flow (l/s)	SEWER DESIGN								
									Peak Factor	Q Avg (l/s)	Q Peak (l/s)			Dia. (mm)	Type	Grade (%)	Mann. n	Vel. (m/s)	Q Full (l/s)	% Flow	% Vel.	Actual Vel. (m/s)
EXT	PRIVATE EXT	EXT	HD14A039	106	20.9		2215															
	SCENIC	HD14A039	HD14A041			20.90		2215	4.26	9.24	39.4	12.5	51.9	375		0.50	0.015	0.98	112.1	46%		
EXT 1	SAN PEDRO	HD14A043	HD14A041	60	0.55		33															
EXT 2	SCENIC		HD14A041	60	1.72		103															
	SCENIC	HD14A041	HD14A069			23.17		2352	4.21	9.81	41.3	13.9	55.2	375		0.54	0.015	1.02	116.2	48%		
	SCENIC	HD14A069	HD14A067			23.17		2352	4.21	9.81	41.3	13.9	55.2	375		0.51	0.015	0.99	113.1	49%		
	SCENIC	HD14A067	HD14A065			23.17		2352	4.21	9.81	41.3	13.9	55.2	375		0.51	0.015	0.99	113.1	49%		
	SCENIC	HD14A065	HD14A063			23.17		2352	4.21	9.81	41.3	13.9	55.2	375		0.48	0.015	0.96	109.3	51%		
EXT 3A	SAN PAULO	HD14A058	HD14A063	60	13.6		816															
EXT 3B	SAN PAULO	HD14A058	HD14A063	**113.5**	5.84		663															
	SCENIC	HD14A063	HD14A060			42.61		3830	3.82	15.97	61.1	25.6	86.6	375		0.18	0.015	0.58	66.5	130%		
	SCENIC	HD14A060	HD14A023			42.61		3830	3.82	15.97	61.1	25.6	86.6	375		0.21	0.015	0.64	72.5	120%		

CITY OF HAMILTON

Sanitary Sewer Design

Location: EXTERNAL AREAS WITH BROWLANDS
Checked By: MJS/JWE
Date: 27-Aug-20

Flow Factor: 360 l/day/cap
Peaking Factor: 5*(P^-0.2)
Infiltration: 0.4 l/s/ha for Deep Sorm
0.6 **0.6 l/s/ha for Shallow Storm**

Area No.	Street Name	From MH	To MH	Pop. Density (ppl/ha)	Area (ha)	Cumm. Area (ha)	Equiv. Pop.	Cumm. Equiv. Pop.	Sanitary Flow			Infilt (l/s)	Total Flow (l/s)	SEWER DESIGN								
									Peak Factor	Q Avg (l/s)	Q Peak (l/s)			Dia. (mm)	Type	Grade (%)	Mann. n	Vel. (m/s)	Q Full (l/s)	% Flow	% Vel.	Actual Vel. (m/s)
EAST	SITE ROAD	A21	HD14A039		2.04		898															
EXT	PRIVATE EXT	EXT	HD14A039	106	20.9		2215															
	SCENIC	HD14A039	HD14A041			22.94		3113	3.98	12.98	51.7	13.8	65.5	375		0.50	0.015	0.98	112.1	58%		
WEST	SITE ROAD	A8	HD14A041		0.98		124															
EXT 1	SAN PEDRO	HD14A043	HD14A041	60	0.55		33															
EXT 2	SCENIC		HD14A041	60	1.72		103															
	SCENIC	HD14A041	HD14A069			26.19		3374	3.92	14.07	55.2	15.7	70.9	375		0.54	0.015	1.02	116.2	61%		
	SCENIC	HD14A069	HD14A067			26.19		3374	3.92	14.07	55.2	15.7	70.9	375		0.51	0.015	0.99	113.1	63%		
	SCENIC	HD14A067	HD14A065			26.19		3374	3.92	14.07	55.2	15.7	70.9	375		0.51	0.015	0.99	113.1	63%		
	SCENIC	HD14A065	HD14A063			26.19		3374	3.92	14.07	55.2	15.7	70.9	375		0.48	0.015	0.96	109.3	65%		
WEST	SITE ROAD	A1	MAIN		1.98		591															
EXT 3A	SAN PAULO	HD14A058	HD14A063	60	13.6		816															
EXT 3B	SAN PAULO	HD14A058	HD14A063	**113.5**	5.84		663															
	SCENIC	HD14A063	HD14A060			47.61		5443	3.56	22.70	80.9	28.6	109.4	375		0.18	0.015	0.58	66.5	165%		
	SCENIC	HD14A060	HD14A023			47.61		5443	3.56	22.70	80.9	28.6	109.4	375		0.21	0.015	0.64	72.5	151%		

APPENDIX E:

Storm Sewer Calculations

Date: Augst 22, 2020

Project: Chedoke Mountain Brow - West Site

STORM SEWER DESIGN CALCULATIONS

Design Frequency: 100 Year

Job No.: TPB188083

i=A / (B + Tc)^C A= 2317.4 B= 11.00 C= 0.836 Design n 0.013

AREA			DRAINAGE AREA						RUNOFF			PIPE SELECTION						% Full	REMARKS
	FROM MH No.	TO MH No.	A ha	C	A	C	AC	Cumul. AC	Cumul. Tc.	i mm/h	Q m3/sec	Dia. mm	Pipe Slope %	Actual Capacity (full) m3/sec	Velocity (full) m/s	Pipe Length m	Time of Flow min.		
									10.00										
A1	CBMH 7	CBMH 6	0.260	0.87			0.226	0.226	10.14	180.82	0.114	300	2.00	0.143	1.96	16.2	0.14	79.5	NO SURCHARGE
A2	CBMH 6	CBMH 5	0.040	0.51			0.020	0.247	10.28	179.84	0.123	375	0.50	0.129	1.13	40.6	0.60	95.2	NO SURCHARGE
									10.00										
A4	CBMH 11	CBMH 5	0.110	0.84			0.092	0.092	10.49	178.34	0.046	300	0.50	0.071	0.98	28.8	0.49	64.1	NO SURCHARGE
A3	CBMH 5	MH 4	0.070	0.63			0.044	0.383	10.87	175.73	0.187	450	0.50	0.210	1.28	35.9	0.47	89.0	NO SURCHARGE
									10.00										
A6	CBMH 10	MH 4	0.210	0.51			0.107	0.107	10.17	180.56	0.054	250	5.00	0.139	2.74	28.7	0.17	38.7	NO SURCHARGE
									10.00										
A5	RYCB 1	MH 4	0.060	0.61			0.037	0.037	10.04	181.56	0.018	250	1.00	0.062	1.22	2.6	0.04	29.8	NO SURCHARGE
	MH 4	CBMH 3	0.000	0.00			0.000	0.527	11.34	172.65	0.253	525	0.50	0.317	1.42	36.4	0.43	79.8	NO SURCHARGE
A8	CBMH 9	CBMH 3	0.150	0.64			0.096	0.096	10.00	181.81	0.048	250	5.00	0.139	2.74	32.8	0.20	34.9	NO SURCHARGE
A7	CBMH 3	CBMH 2	0.070	0.51			0.036	0.659	11.77	169.94	0.311	525	0.50	0.317	1.42	30.2	0.35	98.2	NO SURCHARGE
A9	CBMH 2	MH 1	0.040	0.45			0.018	0.677	12.12	167.76	0.315	525	0.50	0.317	1.42	27.1	0.32	99.6	NO SURCHARGE
									10.00										
A10	CBMH8	MH 1	0.080	0.84			0.067	0.067	10.07	181.29	0.034	250	10.00	0.196	3.87	16.7	0.07	17.2	NO SURCHARGE
	MH 1	SWM TANK	0.000	0.00			0.000	0.744	12.44	165.85	0.343	525	0.70	0.375	1.68	3.5	0.03	91.5	NO SURCHARGE

Date: August 22, 2020

Project: Chedoke Mountain Brow - West Site

STORM SEWER DESIGN CALCULATIONS

Design Frequency: 100 Year

Job No.: TPB188083

i=A / (B + Tc)^C A= 2317.4 B= 11.00 C= 0.836 Design n 0.013

Job No.: TPB188083

AREA			DRAINAGE AREA						RUNOFF			PIPE SELECTION								
	FROM MH No.	TO MH No.	A ha	C	A	C	AC	Cumul. AC	Cumul. Tc.	i mm/h	Q m3/sec	Dia. mm	Pipe Slope %	Actual Capacity (full) m3/sec	Velocity (full) m/s	Pipe Length m	Time of Flow min.	% Full	REMARKS
									10.00										
A13	CBMH 15	MAIN	0.110	0.85			0.094	0.094	10.07	181.32	0.047	250	1.00	0.062	1.22	5.0	0.07	75.9	NO SURCHARGE
									10.00										
A12	MH 14	MH 13	0.900	0.80			0.720	0.814	10.26	179.98	0.407	525	1.00	0.448	2.01	30.8	0.26	90.8	NO SURCHARGE
	MH 13	HW1	0.000	0.00			0.000	0.814	10.46	178.55	0.403	525	1.00	0.448	2.01	24.6	0.20	90.1	NO SURCHARGE
									10.00										
									10.00										
A11	MH 12	HW 2	0.360	0.84			0.302	0.302	10.36	179.22	0.151	375	1.00	0.183	1.60	35.0	0.36	82.3	NO SURCHARGE

Date: August 10, 2020

Project: Chedoke Mountain Brow - East Site

STORM SEWER DESIGN CALCULATIONS

Design Frequency: 100 Year

Job No.: TPB188083

i=A / (B + Tc)^C A= 2317.4 B= 11.00 C= 0.836 Design n 0.013

AREA			DRAINAGE AREA						RUNOFF			PIPE SELECTION							
	FROM MH No.	TO MH No.	A ha	C	A	C	AC	Cumul. AC	Cumul. Tc.	i mm/h	Q m3/sec	Dia. mm	Pipe Slope %	Actual Capacity (full) m3/sec	Velocity (full) m/s	Pipe Length m	Time of Flow min.	% Full	REMARKS
									10.00										
A1	MH24	MH23	0.420	0.86			0.361	0.361	10.66	177.17	0.178	450	0.50	0.210	1.28	50.7	0.66	84.6	NO SURCHARGE
A2	MH23	MH22	0.220	0.77			0.169	0.531	11.29	172.99	0.255	525	0.50	0.317	1.42	53.5	0.63	80.5	NO SURCHARGE
-	MH22	OGS 21	0.000	0.00			0.000	0.531	12.10	167.91	0.247	525	0.50	0.317	1.42	68.8	0.81	78.2	NO SURCHARGE
-	OGS 21	560m^3 TANK	0.000	0.00			0.000	0.531	12.26	166.93	0.246	525	0.50	0.317	1.42	13.9	0.16	77.7	NO SURCHARGE
									10.00										
A3	25	560m^3 TANK	0.150	0.84			0.126	0.126	10.05	181.44	0.064	300	0.50	0.071	0.98	3.0	0.05	88.9	NO SURCHARGE
									10.00										
A4	26	560m^3 TANK	0.230	0.90			0.207	0.207	10.06	181.35	0.104	375	0.50	0.129	1.13	4.4	0.06	80.6	NO SURCHARGE
	560m^3 TANK	MH35	0.000	0.00			0.000	0.864	12.26	166.93	0.400	525	1.00	0.448	2.01	2.5	0.02	89.4	NO SURCHARGE
									10.00										
A6	28	70m^3 TANK	0.180	0.90			0.162	0.162	10.05	181.44	0.082	375	0.50	0.129	1.13	3.5	0.05	63.1	NO SURCHARGE

APPENDIX F:

Stormwater Management Calculations

Chedoke Browlands Development – Valery Homes
Site Reconnaissance (January 8, 2019)



Photo 1: Overview of Headwall Upstream of Sanatorium



Photo 2: Headwall at Sanatorium



Photo 3: Looking Downstream through CSP Arch

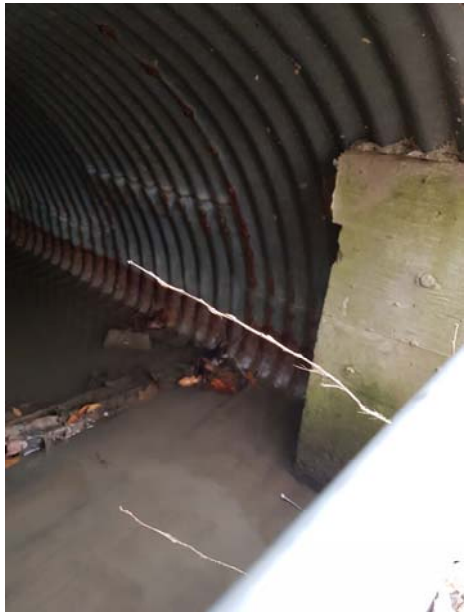


Photo 4: Connection between Concrete and CSP Arch

Chedoke Browlands Development – Valery Homes
Site Reconnaissance (January 8, 2019)



Photo 5: CSP Arch Outfall at Escarpment



Photo 6: Low Flow through plugged CSP pipe



Photo 7: CSP Arch Above and Low Flow Outlet Below



Photo 8: Looking Downstream along Escarpment



Photo 9: Looking West at Outfall



Photo 10: Corrosion and Erosion at CSP Arch Outfall



Photo 11: Close-up of CSP Arch Outfall Erosion



Photo 12: Scour/Erosion Beneath CSP Arch Outfall

Using MTO Design Chart 5.39 - inlet control of a box culvert (use scale 1 since outlet has wingwalls to direct flow)
Typical transition to orifice flow is at 1.5 D
Existing control structure rating curve

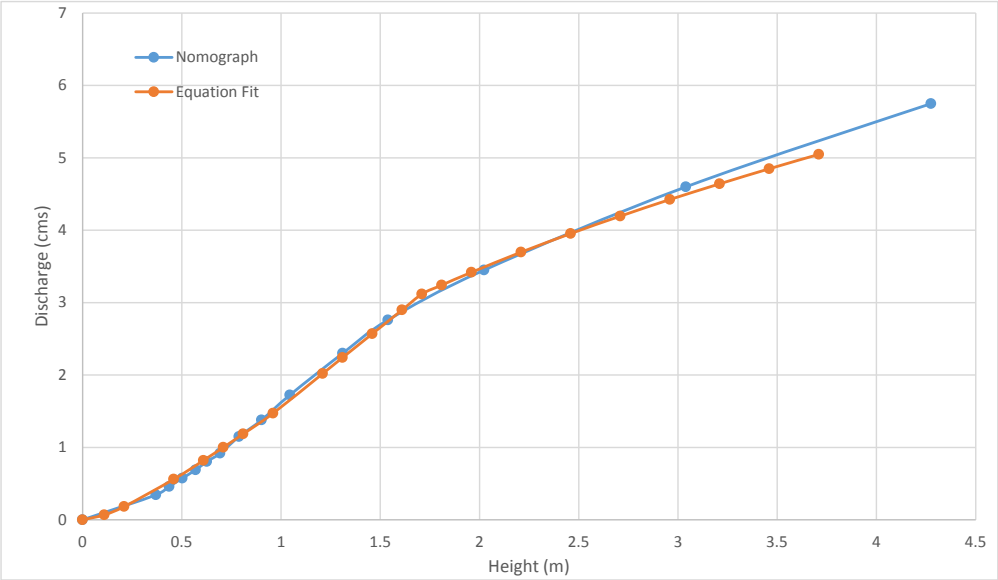
n =	0.024	dimensionl
S =	0.004	m/m
C =	0.58	

	Height (D)	Width (B)
	0.95	1.15

HW/D	Q/NB	HW	Q
		0	0
0.39	0.3	0.3705	0.35
0.46	0.4	0.437	0.46
0.53	0.5	0.5035	0.58
0.6	0.6	0.57	0.69
0.66	0.7	0.627	0.81
0.73	0.8	0.6935	0.92
0.83	1	0.7885	1.15
0.95	1.2	0.9025	1.38
1.1	1.5	1.045	1.73
1.38	2	1.311	2.30
1.62	2.4	1.539	2.76
2.13	3	2.0235	3.45
3.2	4	3.04	4.60
4.5	5	4.275	5.75

USING COMBINATION MANNING'S AND ORIFICE EQUATIONS

H	h	A (m2)	WP (m)	Q (cms)
187.79	0	0	1.15	0.00
187.9	0.11	0.1265	1.37	0.07
188	0.21	0.2415	1.57	0.18
188.25	0.46	0.529	2.07	0.56
188.4	0.61	0.7015	2.37	0.82
188.5	0.71	0.8165	2.57	1.00
188.6	0.81	0.9315	2.77	1.19
188.75	0.96	1.104	3.07	1.47
189	1.21			2.02
189.1	1.31			2.24
189.25	1.46			2.57
189.4	1.61			2.90
189.5	1.71	1.0925		3.12
189.6	1.81	1.0925		3.24
189.75	1.96	1.0925		3.42
190	2.21	1.0925		3.70
190.25	2.46	1.0925		3.95
190.5	2.71	1.0925		4.20
190.75	2.96	1.0925		4.42
191	3.21	1.0925		4.64
191.25	3.46	1.0925		4.85
191.5	3.71	1.0925		5.05

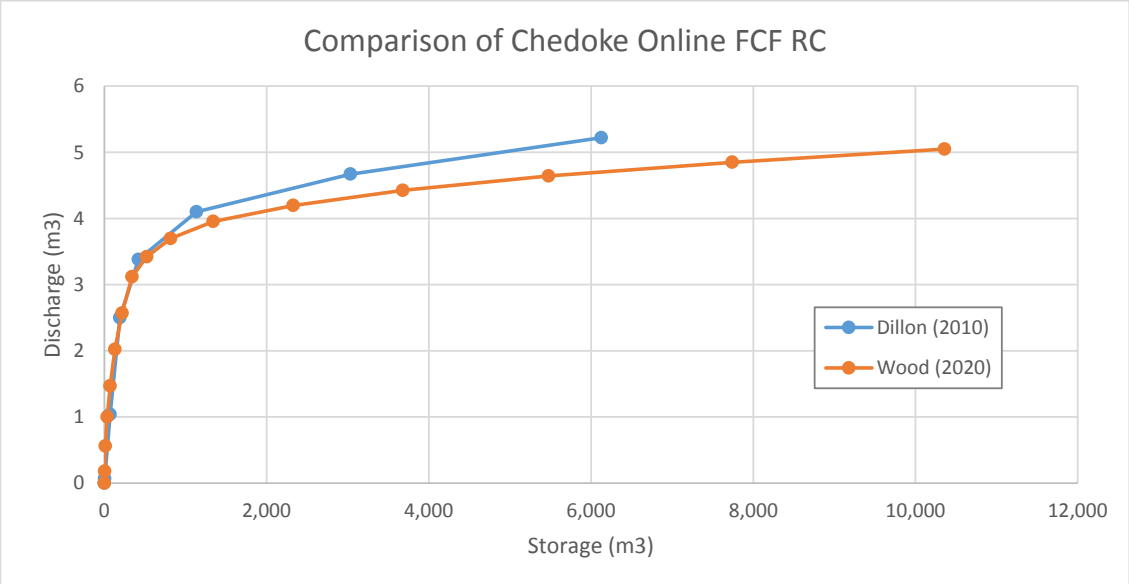


UPDATED EXISTING CONDITIONS RC (WOOD, 2020)

Elev (m)	Storage (1000 m3)	Storage (m3)	Discharge (m3/s)
187.79		0	0.00
188.00	0.0026	3	0.18
188.25	0.0116	12	0.56
188.50	0.0343	34	1.00
188.75	0.0711	71	1.47
189.00	0.1301	130	2.02
189.25	0.2174	217	2.57
189.50	0.3402	340	3.12
189.75	0.5226	523	3.42
190.00	0.8162	816	3.70
190.25	1.3413	1,341	3.95
190.50	2.3279	2,328	4.20
190.75	3.6777	3,678	4.42
191.00	5.476	5,476	4.64
191.25	7.7395	7,740	4.85
191.50	10.3587	10,359	5.05

PREVIOUS EXISTING CONDITIONS RC (DILLON, 2010)

Elev (m)	Height (m)	Discharge (m3/s)	Storage (m3)
187.9	0	0	0
188	0.1	0.07	4
188.5	0.6	1.04	70
189	1.1	2.5	190
189.5	1.6	3.38	420
190	2.1	4.1	1136
190.5	2.6	4.67	3034
191	3.1	5.22	6126

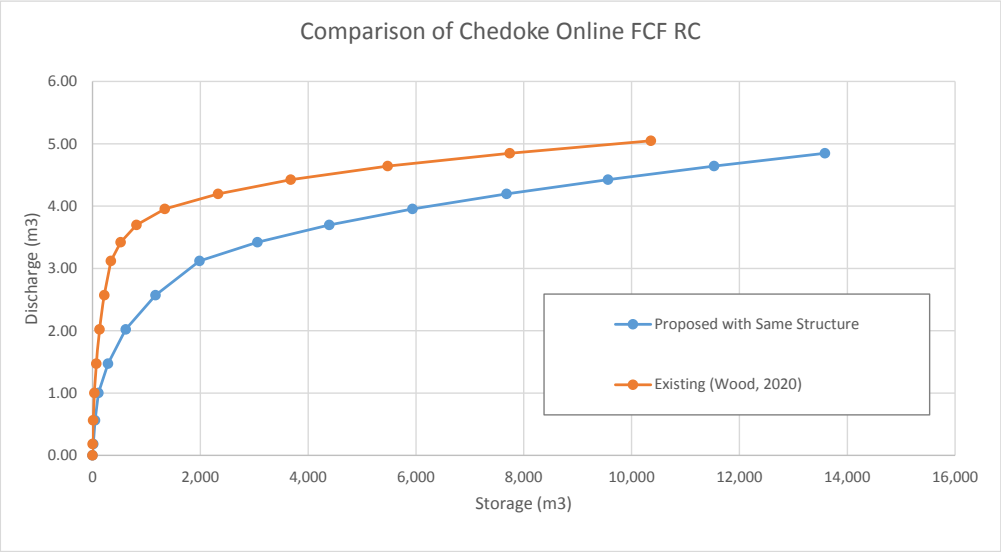


UPDATED EXISTING CONDITIONS RC (WOOD, 2020)

Elev (m)	Storage (1000 m3)	Storage (m3)	Discharge (m3/s)
187.79		0	0.00
188.00	0.0026	3	0.18
188.25	0.0116	12	0.56
188.50	0.0343	34	1.00
188.75	0.0711	71	1.47
189.00	0.1301	130	2.02
189.25	0.2174	217	2.57
189.50	0.3402	340	3.12
189.75	0.5226	523	3.42
190.00	0.8162	816	3.70
190.25	1.3413	1,341	3.95
190.50	2.3279	2,328	4.20
190.75	3.6777	3,678	4.42
191.00	5.476	5,476	4.64
191.25	7.7395	7,740	4.85
191.50	10.3587	10,359	5.05

PROPOSED CONDITIONS RC WITH SAME CONTROL STRUCTURE

Elev (m)	Storage (1000 m3)	Storage (m3)	Discharge (m3/s)
187.79		0	0.00
188	0.015	15	0.18
188.25	0.0461	46	0.56
188.5	0.1099	110	1.00
188.75	0.287	287	1.47
189	0.6149	615	2.02
189.25	1.1679	1,168	2.57
189.5	1.9855	1,986	3.12
189.75	3.0589	3,059	3.42
190	4.3916	4,392	3.70
190.25	5.9348	5,935	3.95
190.5	7.6796	7,680	4.20
190.75	9.5608	9,561	4.42
191	11.5314	11,531	4.64
191.25	13.5858	13,586	4.85



Project: Chedoke Browlands

TABLE F1: FORMER SANATORIUM RD CULVERT CAPACITY - PROPOSED CONDITIONS

Design Frequency: Regional Storm

Job No.: TPB188083

Date: September 2020

LOCATION		DRAINAGE AREA				RUNOFF			PIPE SELECTION									PROFILE					
FROM	TO	A	C	AC	Cumul. AC	Cumul. Tc.	i	Q	Pipe Height	Pipe Width	Approx Equivalent Diameter	Manning's Roughness	Pipe Slope	Actual Capacity (full)	Velocity (full)	Pipe Length	Time of Flow	Percent Full	Fall in Sewer	UPSTREAM		DOWNSTREAM		REMARKS
																				Surface Elev.	Inv. Elev.	Surface Elev.	Inv. Elev.	
MH No.	MH No.	ha					mm/h	m^3/sec	mm	mm	mm	n	%	m^3/sec	m/s	m	min.	%	m	m	m	m	m	
								4.28	1500			0.013	0.50%	5.00	2.83	19.0	0.11	85.6%						Attenuated flow
								5.66	1500			0.013	0.50%	5.00	2.83	19.0	0.11	113.2%						Unattenuated flow
								4.28	1130	1630	1350	0.024	2.20%	4.29	3.00	16.4	0.09	99.8%						Attenuated flow, assumed slope
								5.66	1130	1630	1350	0.024	2.20%	4.29	3.00	16.4	0.09	132.0%						Unattenuated flow, assumed slope

SWM FACILITY 1 (WET POND)

Return Period (Years)	Depth (m)	Storage (m^3)	Orifice 1 Discharge - 100 mm diameter (m^3/s)	Orifice 2 Discharge (m^3/s)	Total Discharge (m^3/s)
2	0.35	396	0.012		0.012
5	0.53	622	0.015		0.015
10	0.64	781	0.017		0.017
25	0.77	991	0.018		0.018
50	0.86	1135	0.019		0.019
100	0.96	1299	0.021		0.021

SWM FACILITY 2 (NORTH-WEST OUTLET TANK)

Return Period (Years)	Depth (m)	Storage (m^3)	Orifice 1 Discharge - 200 mm diameter (m^3/s)	Orifice 2 Discharge - 450 mm diameter at 1.2 m height (m^3/s)	Total Discharge (m^3/s)
2	0.75	74	0.069	0.000	0.069
5	1.18	118	0.090	0.000	0.090
10	1.39	139	0.098	0.056	0.154
25	1.51	151	0.102	0.117	0.220
50	1.64	164	0.107	0.196	0.302
100	1.75	175	0.111	0.247	0.358

SWM FACILITY 3 (SOUTH-EAST OUTLET TANK)

Return Period (Years)	Depth (m)	Storage (m^3)	Orifice 1 Discharge - 100 mm diameter (m^3/s)	Orifice 2 Discharge - 250 mm diameter at 1.15 m height (m^3/s)	Total Discharge (m^3/s)
2	0.74	228	0.018	0.000	0.018
5	1.11	345	0.022	0.000	0.022
10	1.30	402	0.024	0.021	0.045
25	1.47	456	0.026	0.060	0.086
50	1.61	500	0.027	0.078	0.105
100	1.78	552	0.028	0.096	0.124

SWM FACILITY 4 (NORTH-EAST OUTLET TANK)

Return Period (Years)	Depth (m)	Storage (m^3)	Orifice 1 Discharge - 100 mm diameter (m^3/s)	Orifice 2 Discharge (m^3/s)	Total Discharge (m^3/s)
2	0.53	21	0.015		0.015
5	0.82	33	0.019		0.019
10	1.02	41	0.021		0.021
25	1.30	52	0.024		0.024
50	1.50	60	0.026		0.026
100	1.72	69	0.028		0.028

STORMCEPTOR®
ESTIMATED NET ANNUAL SEDIMENT (TSS) LOAD REDUCTION

08/27/2020

Province:	Ontario
City:	Hamilton
Nearest Rainfall Station:	HAMILTON AP
NCDC Rainfall Station Id:	3195
Years of Rainfall Data:	34

Site Name:	Tank 2

Drainage Area (ha):	1.09
% Imperviousness:	65.80

Runoff Coefficient 'c': 0.69

Particle Size Distribution:	CA ETV
Target TSS Removal (%):	60.0

Required Water Quality Runoff Volume Capture (%):	90.00
Estimated Water Quality Flow Rate (L/s):	28.83
Oil / Fuel Spill Risk Site?	No
Upstream Flow Control?	No
Peak Conveyance (maximum) Flow Rate (L/s):	
Site Sediment Transport Rate (kg/ha/yr):	

Project Name:	Browlands
Project Number:	32844
Designer Name:	Matthew Senior
Designer Company:	Amec Foster Wheeler
Designer Email:	matt.senior@amecfw.com
Designer Phone:	905-335-2353
EOR Name:	
EOR Company:	
EOR Email:	
EOR Phone:	

Net Annual Sediment (TSS) Load Reduction Sizing Summary

Stormceptor Model	TSS Removal Provided (%)
EF4	54
EF6	61
EF8	64
EF10	65
EF12	67

Recommended Stormceptor EF Model: EF6

Estimated Net Annual Sediment (TSS) Load Reduction (%): 61

Water Quality Runoff Volume Capture (%): > 90

imbrium

THIRD-PARTY TESTING AND VERIFICATION

►**Stormceptor® EF and Stormceptor® EFO** are the latest evolutions in the Stormceptor® oil-grit separator (OGS) technology series, and are designed to remove a wide variety of pollutants from stormwater and snowmelt runoff. These technologies have been third-party tested in accordance with the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**and performance has been third-party verified in accordance with the **ISO 14034 Environmental Technology Verification (ETV)** protocol.

PERFORMANCE

►**Stormceptor® EF and EFO** remove stormwater pollutants through gravity separation and floatation, and feature a patent-pending design that generates positive removal of total suspended solids (TSS) throughout each storm event, including high-intensity storms. Captured pollutants include sediment, free oils, and sediment-bound pollutants such as nutrients, heavy metals, and petroleum hydrocarbons. Stormceptor is sized to remove a high level of TSS from the frequent rainfall events that contribute the vast majority of annual runoff volume and pollutant load. The technology incorporates an internal bypass to convey excessive stormwater flows from high-intensity storms through the device without resuspension and washout (scour) of previously captured pollutants. Proper routine maintenance ensures high pollutant removal performance and protection of downstream waterwavs.

PARTICLE SIZE DISTRIBUTION (PSD)

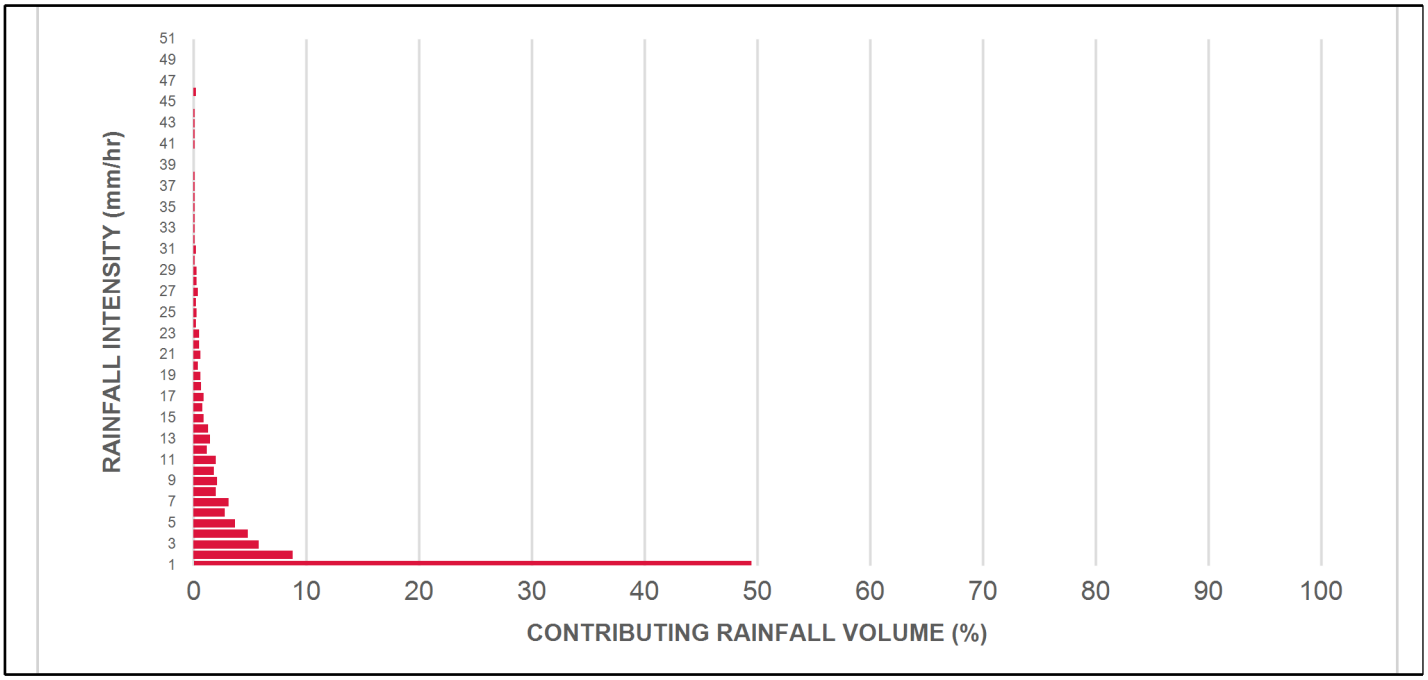
►The **Canadian ETV PSD** shown in the table below was used, or in part, for this sizing. This is the identical PSD that is referenced in the Canadian ETV ***Procedure for Laboratory Testing of Oil-Grit Separators*** for both sediment removal testing and scour testing. The Canadian ETV PSD contains a wide range of particle sizes in the sand and silt fractions, and is considered reasonably representative of the particle size fractions found in typical urban stormwater runoff.

Particle Size (µm)	Percent Less Than	Particle Size Fraction (µm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

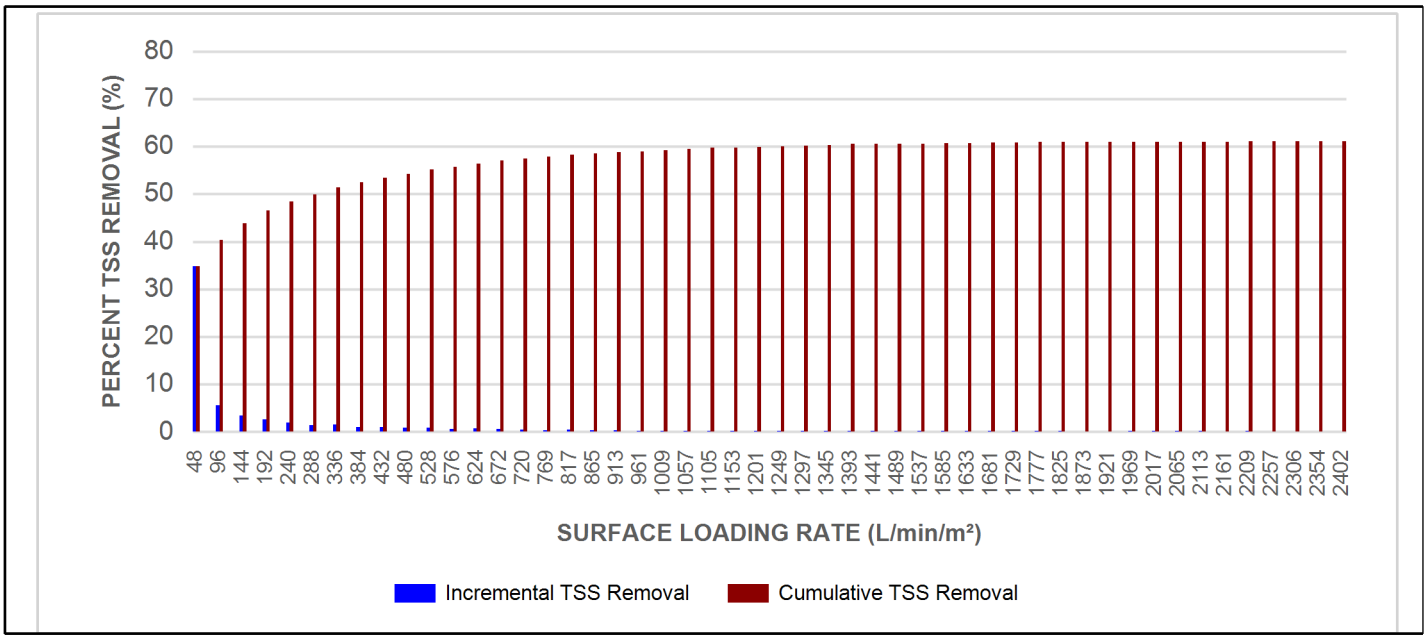
Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
1	49.5	49.5	2.11	126.0	48.0	70	34.8	34.8
2	8.8	58.3	4.21	253.0	96.0	63	5.6	40.4
3	5.8	64.1	6.32	379.0	144.0	59	3.4	43.8
4	4.8	68.9	8.42	505.0	192.0	55	2.6	46.5
5	3.7	72.6	10.53	632.0	240.0	53	2.0	48.4
6	2.8	75.4	12.63	758.0	288.0	52	1.4	49.9
7	3.1	78.5	14.74	884.0	336.0	50	1.6	51.4
8	2.0	80.5	16.84	1011.0	384.0	49	1.0	52.4
9	2.1	82.6	18.95	1137.0	432.0	48	1.0	53.4
10	1.8	84.4	21.05	1263.0	480.0	47	0.9	54.2
11	2.0	86.4	23.16	1390.0	528.0	47	0.9	55.2
12	1.2	87.6	25.26	1516.0	576.0	46	0.6	55.7
13	1.5	89.1	27.37	1642.0	624.0	46	0.7	56.4
14	1.3	90.4	29.48	1769.0	672.0	46	0.6	57.0
15	0.9	91.3	31.58	1895.0	720.0	45	0.4	57.4
16	0.8	92.1	33.69	2021.0	769.0	45	0.4	57.8
17	0.9	93.0	35.79	2147.0	817.0	45	0.4	58.2
18	0.7	93.7	37.90	2274.0	865.0	45	0.3	58.5
19	0.6	94.3	40.00	2400.0	913.0	44	0.3	58.8
20	0.4	94.7	42.11	2526.0	961.0	44	0.2	58.9
21	0.6	95.3	44.21	2653.0	1009.0	44	0.3	59.2
22	0.5	95.8	46.32	2779.0	1057.0	45	0.2	59.4
23	0.5	96.3	48.42	2905.0	1105.0	45	0.2	59.7
24	0.2	96.5	50.53	3032.0	1153.0	46	0.1	59.7
25	0.3	96.8	52.63	3158.0	1201.0	47	0.1	59.9

Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
26	0.2	97.0	54.74	3284.0	1249.0	47	0.1	60.0
27	0.4	97.4	56.85	3411.0	1297.0	48	0.2	60.2
28	0.3	97.7	58.95	3537.0	1345.0	48	0.1	60.3
29	0.3	98.0	61.06	3663.0	1393.0	49	0.1	60.5
30	0.1	98.1	63.16	3790.0	1441.0	48	0.0	60.5
31	0.2	98.3	65.27	3916.0	1489.0	46	0.1	60.6
32	0.1	98.4	67.37	4042.0	1537.0	45	0.0	60.6
33	0.1	98.5	69.48	4169.0	1585.0	43	0.0	60.7
34	0.1	98.6	71.58	4295.0	1633.0	42	0.0	60.7
35	0.1	98.7	73.69	4421.0	1681.0	41	0.0	60.8
36	0.1	98.8	75.79	4548.0	1729.0	40	0.0	60.8
37	0.1	98.9	77.90	4674.0	1777.0	39	0.0	60.9
38	0.1	99.0	80.00	4800.0	1825.0	38	0.0	60.9
39	0.0	99.0	82.11	4927.0	1873.0	37	0.0	60.9
40	0.0	99.0	84.22	5053.0	1921.0	36	0.0	60.9
41	0.1	99.1	86.32	5179.0	1969.0	35	0.0	60.9
42	0.1	99.2	88.43	5306.0	2017.0	34	0.0	61.0
43	0.1	99.3	90.53	5432.0	2065.0	33	0.0	61.0
44	0.1	99.4	92.64	5558.0	2113.0	33	0.0	61.0
45	0.0	99.4	94.74	5685.0	2161.0	32	0.0	61.0
46	0.2	99.6	96.85	5811.0	2209.0	31	0.1	61.1
47	0.0	99.6	98.95	5937.0	2257.0	30	0.0	61.1
48	0.0	99.6	101.06	6064.0	2306.0	30	0.0	61.1
49	0.0	99.6	103.16	6190.0	2354.0	29	0.0	61.1
50	0.0	99.6	105.27	6316.0	2402.0	29	0.0	61.1
Estimated Net Annual Sediment (TSS) Load Reduction =								**61 %**

RAINFALL DATA FROM HAMILTON AP RAINFALL STATION



INCREMENTAL AND CUMULATIVE TSS REMOVAL FOR THE RECOMMENDED STORMCEPTOR® MODEL



Maximum Pipe Diameter / Peak Conveyance

Stormceptor EF / EFO	Model Diameter		Min Angle Inlet / Outlet Pipes	Max Inlet Pipe Diameter		Max Outlet Pipe Diameter		Peak Conveyance Flow Rate	
	(m)	(ft)		(mm)	(in)	(mm)	(in)	(L/s)	(cfs)
EF4 / EFO4	1.2	4	90	609	24	609	24	425	15
EF6 / EFO6	1.8	6	90	914	36	914	36	990	35
EF8 / EFO8	2.4	8	90	1219	48	1219	48	1700	60
EF10 / EFO10	3.0	10	90	1828	72	1828	72	2830	100
EF12 / EFO12	3.6	12	90	1828	72	1828	72	2830	100

SCOUR PREVENTION AND ONLINE CONFIGURATION

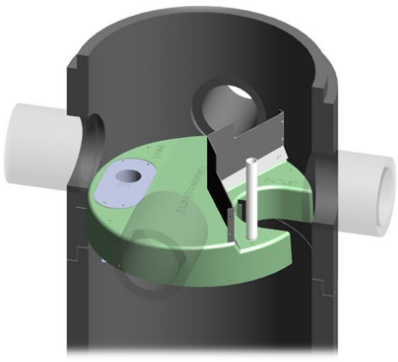
►**Stormceptor® EF and EFO** feature an internal bypass and superior scour prevention technology that have been demonstrated in third-party testing according to the scour testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**, and the exceptional scour test performance has been third-party verified in accordance with the ISO 14034 ETV protocol. As a result, Stormceptor EF and EFO are approved for online installation, eliminating the need for costly additional bypass structures, piping, and installation expense.

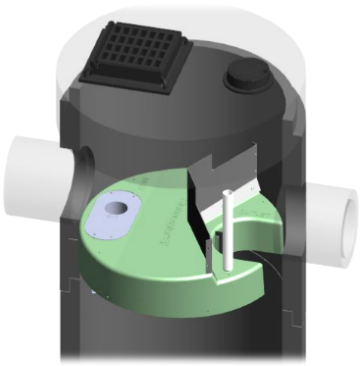
DESIGN FLEXIBILITY

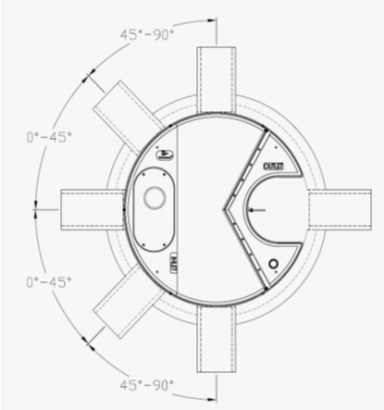
►**Stormceptor® EF** and **EFO** offers design flexibility in one simplified platform, accepting stormwater flow from a single inlet pipe or multiple inlet pipes, and/or surface runoff through an inlet grate. The device can also serve as a junction structure, accommodate a 90-degree inlet-to-outlet bend angle, and can be modified to ensure performance in submerged conditions.

OIL CAPTURE AND RETENTION

►While Stormceptor® EF will capture and retain oil from dry weather spills and low intensity runoff,**Stormceptor® EFO** has demonstrated superior oil capture and greater than 99% oil retention in third-party testing according to the light liquid re-entrainment testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators** Stormceptor EFO is recommended for sites where oil capture and retention is a requirement.







INLET-TO-OUTLET DROP

Elevation differential between inlet and outlet pipe inverts is dictated by the angle at which the inlet pipe(s) enters the unit.
0° - 45° : The inlet pipe is 1-inch (25mm) higher than the outlet pipe.
45° - 90° : The inlet pipe is 2-inches (50mm) higher than the outlet pipe.

HEAD LOSS

The head loss through Stormceptor EF is similar to that of a 60-degree bend structure. The applicable K value for calculating minor losses through the unit is 1.1. For submerged conditions the applicable K value is 3.0.

Pollutant Capacity

Stormceptor EF / EFO	Model Diameter		Depth (Outlet Pipe Invert to Sump Floor)		Oil Volume		Recommended Sediment Maintenance Depth *		Maximum Sediment Volume *		Maximum Sediment Mass **	
	(m)	(ft)	(m)	(ft)	(L)	(Gal)	(mm)	(in)	(L)	(ft^3)	(kg)	(lb)
EF4 / EFO4	1.2	4	1.52	5.0	265	70	203	8	1190	42	1904	5250
EF6 / EFO6	1.8	6	1.93	6.3	610	160	305	12	3470	123	5552	15375
EF8 / EFO8	2.4	8	2.59	8.5	1070	280	610	24	8780	310	14048	38750
EF10 / EFO10	3.0	10	3.25	10.7	1670	440	610	24	17790	628	28464	78500
EF12 / EFO12	3.6	12	3.89	12.8	2475	655	610	24	31220	1103	49952	137875

*Increased sump depth may be added to increase sediment storage capacity
** Average density of wet packed sediment in sump = 1.6 kg/L (100 lb/ft^3)

Feature	Benefit	Feature Appeals To
Patent-pending enhanced flow treatment and scour prevention technology	Superior, verified third-party performance	Regulator, Specifying & Design Engineer
Third-party verified light liquid capture and retention for EFO version	Proven performance for fuel/oil hotspot locations	Regulator, Specifying & Design Engineer, Site Owner
Functions as bend, junction or inlet structure	Design flexibility	Specifying & Design Engineer
Minimal drop between inlet and outlet	Site installation ease	Contractor
Large diameter outlet riser for inspection and maintenance	Easy maintenance access from grade	Maintenance Contractor & Site Owner

STANDARD STORMCEPTOR EF/EFO DRAWINGS

For standard details, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

STANDARD STORMCEPTOR EF/EFO SPECIFICATION

For specifications, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

Table of TSS Removal vs Surface Loading Rate Based on Third-Party Test Results
Stormceptor® EF

SLR ($L/min/m^2$)	TSS % REMOVAL	SLR ($L/min/m^2$)	TSS % REMOVAL	SLR ($L/min/m^2$)	TSS % REMOVAL	SLR ($L/min/m^2$)	TSS % REMOVAL
1	70	660	46	1320	48	1980	35
30	70	690	46	1350	48	2010	34

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60	67	720	45	1380	49	2040	34
90	63	750	45	1410	49	2070	33
120	61	780	45	1440	48	2100	33
150	58	810	45	1470	47	2130	32
180	56	840	45	1500	46	2160	32
210	54	870	45	1530	45	2190	31
240	53	900	45	1560	44	2220	31
270	52	930	44	1590	43	2250	30
300	51	960	44	1620	42	2280	30
330	50	990	44	1650	42	2310	30
360	49	1020	44	1680	41	2340	29
390	48	1050	45	1710	40	2370	29
420	48	1080	45	1740	39	2400	29
450	48	1110	45	1770	39	2430	28
480	47	1140	46	1800	38	2460	28
510	47	1170	46	1830	37	2490	28
540	47	1200	47	1860	37	2520	27
570	46	1230	47	1890	36	2550	27
600	46	1260	47	1920	36	2580	27
630	46	1290	48	1950	35		

STANDARD PERFORMANCE SPECIFICATION FOR "OIL GRIT SEPARATOR" (OGS) STORMWATER QUALITY TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground Oil Grit Separator (OGS) device for stormwater quality treatment, with third-party testing results and a Statement of Verification in accordance with ISO 14034 Environmental Management – Environmental Technology Verification (ETV).

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental management – Environmental technology verification (ETV)

Canadian Environmental Technology Verification (ETV) Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, OGS stormwater quality treatment product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 OGS POLLUTANT STORAGE

The OGS device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants. The **minimum** sediment & petroleum hydrocarbon storage capacity shall be as follows:

2.1.1

4 ft (1219 mm) Diameter OGS Units:	1.19 m^3 sediment / 265 L oil
6 ft (1829 mm) Diameter OGS Units:	3.48 m^3 sediment / 609 L oil
8 ft (2438 mm) Diameter OGS Units:	8.78 m^3 sediment / 1,071 L oil
10 ft (3048 mm) Diameter OGS Units:	17.78 m^3 sediment / 1,673 L oil
12 ft (3657 mm) Diameter OGS Units:	31.23 m^3 sediment / 2,476 L oil

PART 3 – PERFORMANCE & DESIGN

3.1 GENERAL

The OGS stormwater quality treatment device shall be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV). The OGS stormwater quality treatment device shall remove oil, sediment and gross pollutants from stormwater runoff during frequent wet weather events, and retain these pollutants during less frequent high flow wet weather events below the insert within the OGS for later removal during maintenance. The Manufacturer shall have at least ten (10) years of local experience, history and success in engineering design, manufacturing and production and supply of OGS stormwater quality treatment device systems, acceptable to the Engineer of Record.

3.2 SIZING METHODOLOGY

The OGS device shall be engineered, designed and sized to provide stormwater quality treatment based on treating a minimum of 90 percent of the average annual runoff volume and a minimum removal of an annual average 60% of the sediment (TSS) load based on the Particle Size Distribution (PSD) specified in the sizing report for the specified device. Sizing shall be determined using historical rainfall data and a sediment removal performance curve derived from the actual third-party verified laboratory testing data. The OGS device shall also have sufficient annual sediment storage capacity as specified and calculated in Section 2.1.

3.3 CANADIAN ETV or ISO 14034 ETV VERIFICATION OF SCOUR TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of third-party scour testing conducted in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

3.3.1 To be acceptable for on-line installation, the OGS device must demonstrate an average scour test effluent concentration less than 10 mg/L at each surface loading rate tested, up to and including 2600 L/min/m^2.

STORMCEPTOR®
ESTIMATED NET ANNUAL SEDIMENT (TSS) LOAD REDUCTION

08/27/2020

Province:	Ontario
City:	Hamilton
Nearest Rainfall Station:	HAMILTON AP
NCDC Rainfall Station Id:	3195
Years of Rainfall Data:	34

Site Name:	Tank 3

Drainage Area (ha):	0.65
% Imperviousness:	88.00

Runoff Coefficient 'c': 0.82

Particle Size Distribution:	CA ETV
Target TSS Removal (%):	60.0

Required Water Quality Runoff Volume Capture (%):	90.00
Estimated Water Quality Flow Rate (L/s):	20.49
Oil / Fuel Spill Risk Site?	Yes
Upstream Flow Control?	No
Peak Conveyance (maximum) Flow Rate (L/s):	
Site Sediment Transport Rate (kg/ha/yr):	

Project Name:	Browlands
Project Number:	32844
Designer Name:	Matthew Senior
Designer Company:	Amec Foster Wheeler
Designer Email:	matt.senior@amecfw.com
Designer Phone:	905-335-2353
EOR Name:	
EOR Company:	
EOR Email:	
EOR Phone:	

Net Annual Sediment (TSS) Load Reduction Sizing Summary

Stormceptor Model	TSS Removal Provided (%)
EFO4	55
EFO6	62
EFO8	65
EFO10	67
EFO12	68

Recommended Stormceptor EFO Model: EFO6

Estimated Net Annual Sediment (TSS) Load Reduction (%): 62

Water Quality Runoff Volume Capture (%): > 90

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THIRD-PARTY TESTING AND VERIFICATION

►**Stormceptor® EF and Stormceptor® EFO** are the latest evolutions in the Stormceptor® oil-grit separator (OGS) technology series, and are designed to remove a wide variety of pollutants from stormwater and snowmelt runoff. These technologies have been third-party tested in accordance with the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**and performance has been third-party verified in accordance with the **ISO 14034 Environmental Technology Verification (ETV)** protocol.

PERFORMANCE

►**Stormceptor® EF and EFO** remove stormwater pollutants through gravity separation and floatation, and feature a patent-pending design that generates positive removal of total suspended solids (TSS) throughout each storm event, including high-intensity storms. Captured pollutants include sediment, free oils, and sediment-bound pollutants such as nutrients, heavy metals, and petroleum hydrocarbons. Stormceptor is sized to remove a high level of TSS from the frequent rainfall events that contribute the vast majority of annual runoff volume and pollutant load. The technology incorporates an internal bypass to convey excessive stormwater flows from high-intensity storms through the device without resuspension and washout (scour) of previously captured pollutants. Proper routine maintenance ensures high pollutant removal performance and protection of downstream waterwavs.

PARTICLE SIZE DISTRIBUTION (PSD)

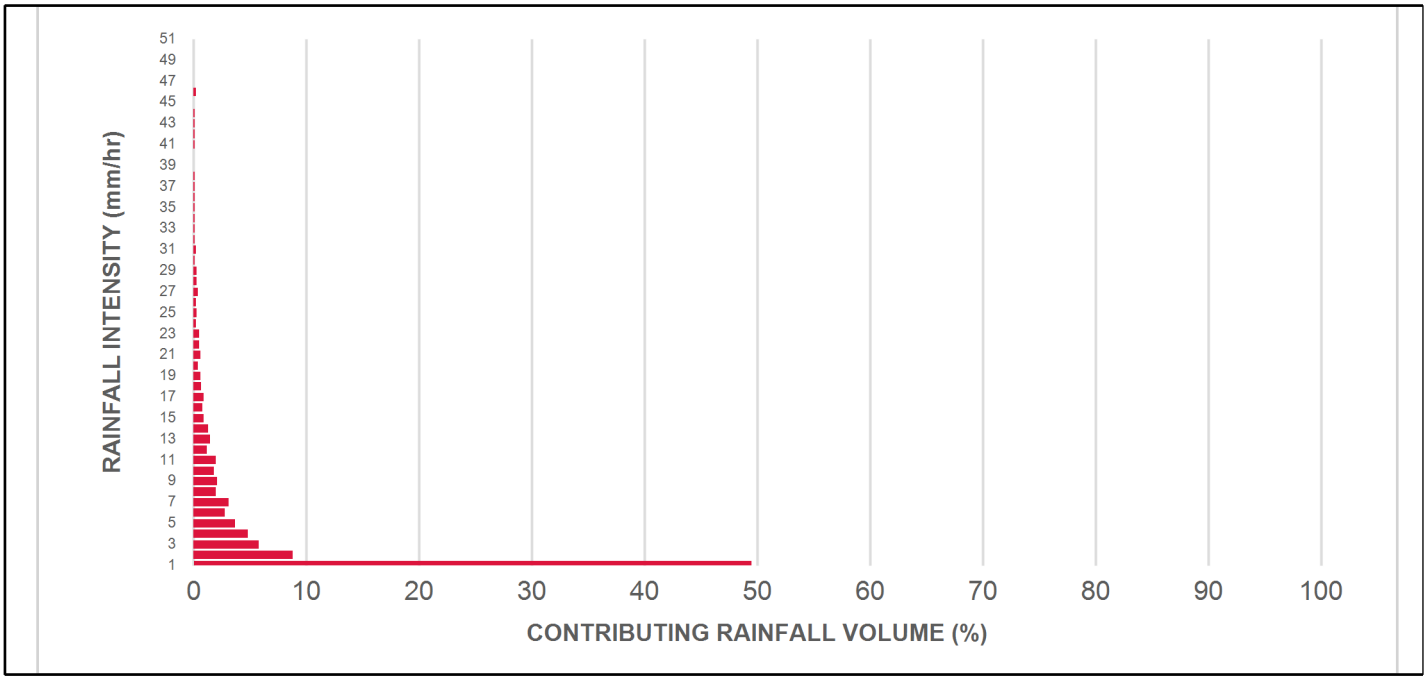
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Particle Size (µm)	Percent Less Than	Particle Size Fraction (µm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

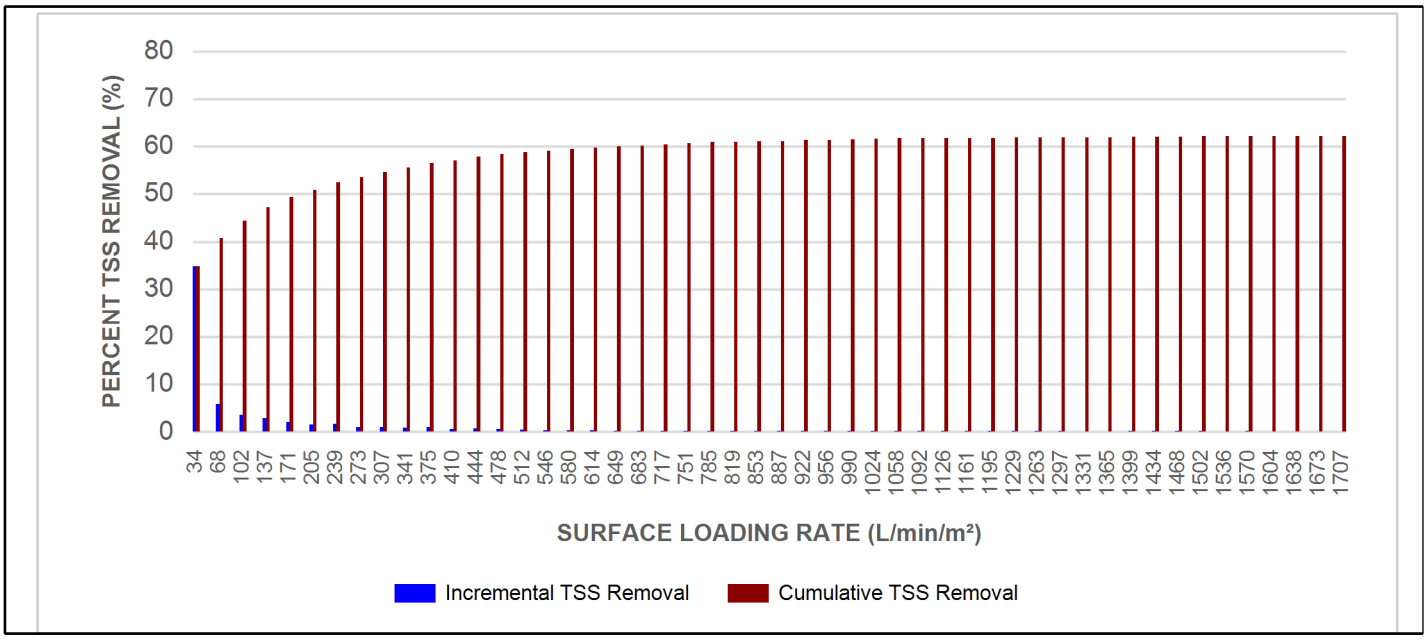
Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
1	49.5	49.5	1.50	90.0	34.0	70	34.8	34.8
2	8.8	58.3	2.99	180.0	68.0	67	5.9	40.8
3	5.8	64.1	4.49	269.0	102.0	62	3.6	44.4
4	4.8	68.9	5.98	359.0	137.0	60	2.9	47.2
5	3.7	72.6	7.48	449.0	171.0	57	2.1	49.3
6	2.8	75.4	8.98	539.0	205.0	54	1.5	50.9
7	3.1	78.5	10.47	628.0	239.0	53	1.6	52.5
8	2.0	80.5	11.97	718.0	273.0	52	1.0	53.5
9	2.1	82.6	13.47	808.0	307.0	51	1.1	54.6
10	1.8	84.4	14.96	898.0	341.0	50	0.9	55.5
11	2.0	86.4	16.46	987.0	375.0	49	1.0	56.5
12	1.2	87.6	17.95	1077.0	410.0	48	0.6	57.1
13	1.5	89.1	19.45	1167.0	444.0	47	0.7	57.8
14	1.3	90.4	20.95	1257.0	478.0	46	0.6	58.4
15	0.9	91.3	22.44	1347.0	512.0	45	0.4	58.8
16	0.8	92.1	23.94	1436.0	546.0	44	0.4	59.1
17	0.9	93.0	25.44	1526.0	580.0	43	0.4	59.5
18	0.7	93.7	26.93	1616.0	614.0	42	0.3	59.8
19	0.6	94.3	28.43	1706.0	649.0	42	0.3	60.0
20	0.4	94.7	29.92	1795.0	683.0	42	0.2	60.2
21	0.6	95.3	31.42	1885.0	717.0	41	0.2	60.4
22	0.5	95.8	32.92	1975.0	751.0	41	0.2	60.7
23	0.5	96.3	34.41	2065.0	785.0	41	0.2	60.9
24	0.2	96.5	35.91	2155.0	819.0	41	0.1	60.9
25	0.3	96.8	37.40	2244.0	853.0	41	0.1	61.1

Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
26	0.2	97.0	38.90	2334.0	887.0	41	0.1	61.1
27	0.4	97.4	40.40	2424.0	922.0	40	0.2	61.3
28	0.3	97.7	41.89	2514.0	956.0	40	0.1	61.4
29	0.3	98.0	43.39	2603.0	990.0	40	0.1	61.5
30	0.1	98.1	44.89	2693.0	1024.0	40	0.0	61.6
31	0.2	98.3	46.38	2783.0	1058.0	39	0.1	61.7
32	0.1	98.4	47.88	2873.0	1092.0	39	0.0	61.7
33	0.1	98.5	49.37	2962.0	1126.0	38	0.0	61.7
34	0.1	98.6	50.87	3052.0	1161.0	38	0.0	61.8
35	0.1	98.7	52.37	3142.0	1195.0	37	0.0	61.8
36	0.1	98.8	53.86	3232.0	1229.0	37	0.0	61.9
37	0.1	98.9	55.36	3322.0	1263.0	36	0.0	61.9
38	0.1	99.0	56.86	3411.0	1297.0	36	0.0	61.9
39	0.0	99.0	58.35	3501.0	1331.0	35	0.0	61.9
40	0.0	99.0	59.85	3591.0	1365.0	35	0.0	61.9
41	0.1	99.1	61.34	3681.0	1399.0	34	0.0	62.0
42	0.1	99.2	62.84	3770.0	1434.0	33	0.0	62.0
43	0.1	99.3	64.34	3860.0	1468.0	33	0.0	62.0
44	0.1	99.4	65.83	3950.0	1502.0	32	0.0	62.1
45	0.0	99.4	67.33	4040.0	1536.0	31	0.0	62.1
46	0.2	99.6	68.83	4130.0	1570.0	30	0.1	62.1
47	0.0	99.6	70.32	4219.0	1604.0	30	0.0	62.1
48	0.0	99.6	71.82	4309.0	1638.0	29	0.0	62.1
49	0.0	99.6	73.31	4399.0	1673.0	29	0.0	62.1
50	0.0	99.6	74.81	4489.0	1707.0	28	0.0	62.1
						Estimated Net Annual Sediment (TSS) Load Reduction =		62 %

RAINFALL DATA FROM HAMILTON AP RAINFALL STATION



INCREMENTAL AND CUMULATIVE TSS REMOVAL FOR THE RECOMMENDED STORMCEPTOR® MODEL



Maximum Pipe Diameter / Peak Conveyance

Stormceptor EF / EFO	Model Diameter		Min Angle Inlet / Outlet Pipes	Max Inlet Pipe Diameter		Max Outlet Pipe Diameter		Peak Conveyance Flow Rate	
	(m)	(ft)		(mm)	(in)	(mm)	(in)	(L/s)	(cfs)
EF4 / EFO4	1.2	4	90	609	24	609	24	425	15
EF6 / EFO6	1.8	6	90	914	36	914	36	990	35
EF8 / EFO8	2.4	8	90	1219	48	1219	48	1700	60
EF10 / EFO10	3.0	10	90	1828	72	1828	72	2830	100
EF12 / EFO12	3.6	12	90	1828	72	1828	72	2830	100

SCOUR PREVENTION AND ONLINE CONFIGURATION

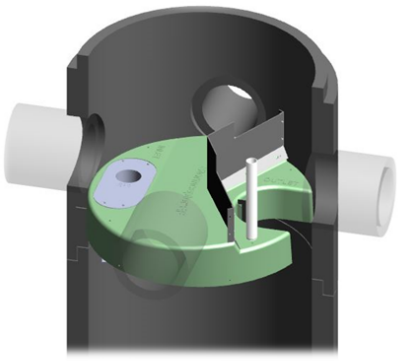
►**Stormceptor® EF and EFO** feature an internal bypass and superior scour prevention technology that have been demonstrated in third-party testing according to the scour testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**, and the exceptional scour test performance has been third-party verified in accordance with the ISO 14034 ETV protocol. As a result, Stormceptor EF and EFO are approved for online installation, eliminating the need for costly additional bypass structures, piping, and installation expense.

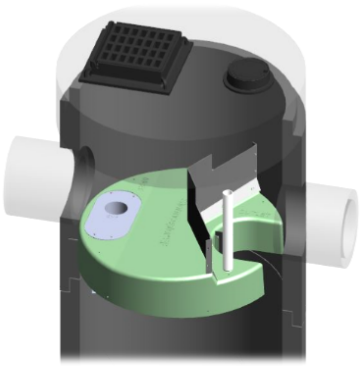
DESIGN FLEXIBILITY

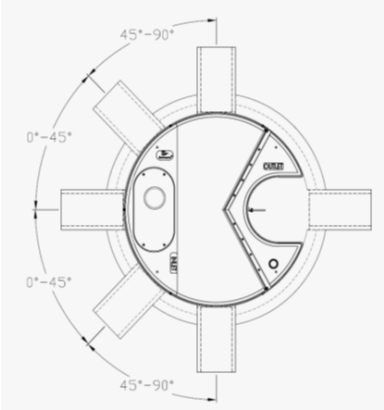
►**Stormceptor® EF** and **EFO** offers design flexibility in one simplified platform, accepting stormwater flow from a single inlet pipe or multiple inlet pipes, and/or surface runoff through an inlet grate. The device can also serve as a junction structure, accommodate a 90-degree inlet-to-outlet bend angle, and can be modified to ensure performance in submerged conditions.

OIL CAPTURE AND RETENTION

►While Stormceptor® EF will capture and retain oil from dry weather spills and low intensity runoff,**Stormceptor® EFO** has demonstrated superior oil capture and greater than 99% oil retention in third-party testing according to the light liquid re-entrainment testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators** Stormceptor EFO is recommended for sites where oil capture and retention is a requirement.







INLET-TO-OUTLET DROP

Elevation differential between inlet and outlet pipe inverts is dictated by the angle at which the inlet pipe(s) enters the unit.

0° - 45° : The inlet pipe is 1-inch (25mm) higher than the outlet pipe.

45° - 90° : The inlet pipe is 2-inches (50mm) higher than the outlet pipe.

HEAD LOSS

The head loss through Stormceptor EF is similar to that of a 60-degree bend structure. The applicable K value for calculating minor losses through the unit is 1.1. For submerged conditions the applicable K value is 3.0.

Pollutant Capacity

Stormceptor EF / EFO	Model Diameter		Depth (Outlet Pipe Invert to Sump Floor)		Oil Volume		Recommended Sediment Maintenance Depth *		Maximum Sediment Volume *		Maximum Sediment Mass **	
	(m)	(ft)	(m)	(ft)	(L)	(Gal)	(mm)	(in)	(L)	(ft^3)	(kg)	(lb)
EF4 / EFO4	1.2	4	1.52	5.0	265	70	203	8	1190	42	1904	5250
EF6 / EFO6	1.8	6	1.93	6.3	610	160	305	12	3470	123	5552	15375
EF8 / EFO8	2.4	8	2.59	8.5	1070	280	610	24	8780	310	14048	38750
EF10 / EFO10	3.0	10	3.25	10.7	1670	440	610	24	17790	628	28464	78500
EF12 / EFO12	3.6	12	3.89	12.8	2475	655	610	24	31220	1103	49952	137875

*Increased sump depth may be added to increase sediment storage capacity

** Average density of wet packed sediment in sump = 1.6 kg/L (100 lb/ft^3)

Feature	Benefit	Feature Appeals To
Patent-pending enhanced flow treatment and scour prevention technology	Superior, verified third-party performance	Regulator, Specifying & Design Engineer
Third-party verified light liquid capture and retention for EFO version	Proven performance for fuel/oil hotspot locations	Regulator, Specifying & Design Engineer, Site Owner
Functions as bend, junction or inlet structure	Design flexibility	Specifying & Design Engineer
Minimal drop between inlet and outlet	Site installation ease	Contractor
Large diameter outlet riser for inspection and maintenance	Easy maintenance access from grade	Maintenance Contractor & Site Owner

STANDARD STORMCEPTOR EF/EFO DRAWINGS

For standard details, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

STANDARD STORMCEPTOR EF/EFO SPECIFICATION

For specifications, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

Table of TSS Removal vs Surface Loading Rate Based on Third-Party Test Results
Stormceptor® EFO

SLR (L/min/m^2)	TSS % REMOVAL	SLR (L/min/m^2)	TSS % REMOVAL	SLR (L/min/m^2)	TSS % REMOVAL	SLR (L/min/m^2)	TSS % REMOVAL
1	70	660	46	1320	48	1980	35
30	70	690	46	1350	48	2010	34

imbrium

60	67	720	45	1380	49	2040	34
90	63	750	45	1410	49	2070	33
120	61	780	45	1440	48	2100	33
150	58	810	45	1470	47	2130	32
180	56	840	45	1500	46	2160	32
210	54	870	45	1530	45	2190	31
240	53	900	45	1560	44	2220	31
270	52	930	44	1590	43	2250	30
300	51	960	44	1620	42	2280	30
330	50	990	44	1650	42	2310	30
360	49	1020	44	1680	41	2340	29
390	48	1050	45	1710	40	2370	29
420	48	1080	45	1740	39	2400	29
450	48	1110	45	1770	39	2430	28
480	47	1140	46	1800	38	2460	28
510	47	1170	46	1830	37	2490	28
540	47	1200	47	1860	37	2520	27
570	46	1230	47	1890	36	2550	27
600	46	1260	47	1920	36	2580	27
630	46	1290	48	1950	35		

STANDARD PERFORMANCE SPECIFICATION FOR "OIL GRIT SEPARATOR" (OGS) STORMWATER QUALITY TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground Oil Grit Separator (OGS) device for stormwater quality treatment, with third-party testing results and a Statement of Verification in accordance with ISO 14034 Environmental Management – Environmental Technology Verification (ETV).

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental management – Environmental technology verification (ETV)

Canadian Environmental Technology Verification (ETV) Program's **Procedure for Laboratory Testing of Oil-Grit Separators**

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, OGS stormwater quality treatment product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 OGS POLLUTANT STORAGE

The OGS device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants. The minimum sediment & petroleum hydrocarbon storage capacity shall be as follows:

2.1.1	4 ft (1219 mm) Diameter OGS Units:	1.19 m^3 sediment / 265 L oil
	6 ft (1829 mm) Diameter OGS Units:	3.48 m^3 sediment / 609 L oil
	8 ft (2438 mm) Diameter OGS Units:	8.78 m^3 sediment / 1,071 L oil
	10 ft (3048 mm) Diameter OGS Units:	17.78 m^3 sediment / 1,673 L oil
	12 ft (3657 mm) Diameter OGS Units:	31.23 m^3 sediment / 2,476 L oil

PART 3 – PERFORMANCE & DESIGN

3.1 GENERAL

The OGS stormwater quality treatment device shall be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV). The OGS stormwater quality treatment device shall

remove oil, sediment and gross pollutants from stormwater runoff during frequent wet weather events, and retain these pollutants during less frequent high flow wet weather events below the insert within the OGS for later removal during maintenance. The Manufacturer shall have at least ten (10) years of local experience, history and success in engineering design, manufacturing and production and supply of OGS stormwater quality treatment device systems, acceptable to the Engineer of Record.

3.2 SIZING METHODOLOGY

The OGS device shall be engineered, designed and sized to provide stormwater quality treatment based on treating a minimum of 90 percent of the average annual runoff volume and a minimum removal of an annual average 60% of the sediment (TSS) load based on the Particle Size Distribution (PSD) specified in the sizing report for the specified device. Sizing shall be determined using historical rainfall data and a sediment removal performance curve derived from the actual third-party verified laboratory testing data. The OGS device shall also have sufficient annual sediment storage capacity as specified and calculated in Section 2.1.

3.3 CANADIAN ETV or ISO 14034 ETV VERIFICATION OF SCOUR TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of third-party scour testing conducted in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

3.3.1 To be acceptable for on-line installation, the OGS device must demonstrate an average scour test effluent concentration less than 10 mg/L at each surface loading rate tested, up to and including 2600 L/min/m^2.

3.4 LIGHT LIQUID RE-ENTRAINMENT SIMULATION TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of completed third-party Light Liquid Re-entrainment Simulation Testing in accordance with the Canadian ETV **Program's Procedure for Laboratory Testing of Oil-Grit Separators,** with results reported within the Canadian ETV or ISO 14034 ETV verification. This re-entrainment testing is conducted with the device pre-loaded with low density polyethylene (LDPE) plastic beads as a surrogate for light liquids such as oil and fuel. Testing is conducted on the same OGS unit tested for sediment removal to assess whether light liquids captured after a spill are effectively retained at high flow rates.

3.4.1 For an OGS device to be an acceptable stormwater treatment device on a site where vehicular traffic occurs and the potential for an oil or fuel spill exists, the OGS device must have reported verified performance results of greater than 99% cumulative retention of LDPE plastic beads for the five specified surface loading rates (ranging 200 L/min/m2 to 2600 L/min/m2) in accordance with the Light Liquid Re-entrainment Simulation Testing within the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**However, an OGS device shall not be allowed if the Light Liquid Re-entrainment Simulation Testing was performed with screening components within the OGS device that are effective at retaining the LDPE plastic beads, but would not be expected to retain light liquids such as oil and fuel.

APPENDIX G:

Watermain Hydraulic Analysis

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited
3450 Harvester Road, Suite 100
Burlington, ON L7N 3W5, Canada
T: 905-335-2353
www.woodplc.com

September 18, 2020
TPB188083

Mr. Paul Valeri, Vice President
Valery Homes
2140 King Street East
Hamilton, ON L8K 1W6

RE: Browlands Development (801, 820, 828, 855, 865 and 870 Scenic Drive)
Watermain Hydraulic Analysis Report – City of Hamilton

Dear Sir,

Please see attached the Watermain Assessment Hydraulic Report for the proposed development at 865 Sanatorium Road in Hamilton, Ontario. This report provides the hydraulic assessment of the proposed development as part of the Functional Servicing Report (FSR) documentation for this Site. Wood has conducted a hydraulic assessment for the proposed development and has considered City of Hamilton and Ministry of Environment, Climate and Parks (MECP) criteria in its assessment.

The analysis documented in this report includes individual hydraulic examination of the Average Day demand, Maximum day plus fire flow and the Maximum (Peak) Hour demand of the development for the 2011 (present) and 2031 demand conditions. Fire flow demands have been determined through OBC methodology as per City of Hamilton memorandum. Two alternative scenarios have been assessed (with and without upgrades on Scenic Drive in place). The City of Hamilton's approved WaterCAD model (as supplied by the City of Hamilton) has been applied for the analysis of Pressure District 5 (PD5).

The completed modelling indicates that the development can achieve the hydraulic requirements of the City of Hamilton, the MECP, and OBC with the proposed watermain upgrades along Scenic Drive in place, as outlined in the attached report.

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Ali Aamir, B.Eng, EIT
Water Resources Engineer in Training

Reviewed by:

Rajan Sawhney, M.E. (Env), P.Eng.
Senior Water/Wastewater Engineer

'Wood' is a trading name for John Wood Group PLC and its subsidiaries

Watermain Hydraulic Report

Chedoke Browlands Development
Hamilton, Ontario
Project TPB188083; Client name: Valery Homes

Prepared for:

Valery Homes
2140 King Street East, Hamilton, ON L8K 1W6

9/18/2020

Watermain Hydraulic Report

Chedoke Browlands Development
City of Hamilton
Project TPB188083; Client name: Valery Homes

Prepared for:

Valery Homes
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9/18/2020

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Table 4.2.2 Available Fire Flow Range (Liters per Second) for Existing (2018) and Ultimate (2031) Conditions (Upgrades on Scenic Drive)

1.0 Introduction

The proposed development is located in Hamilton, Ontario, north of Scenic Drive, west of Upper Paradise Road, in the vicinity of 865 Sanatorium Road. It is bisected by Sanatorium Road (thereby being split into east and west portions).

This proposed development consists of residential housing units consisting of fourteen (14) townhomes, and two (2) high rise buildings on the west side, totaling 56 townhome units, and 408 high rise units, respectively. The east side of the site consists of five (5) high rise buildings, totaling 370 units.

The proposed development will be located within Pressure District 5 (PD5) of the City of Hamilton water network. It will connect to the City's existing network at two (2) locations as shown in Figure 1; northwest of San Pedro Drive, and northeast of the Scenic Drive/Sanatorium Road intersection.

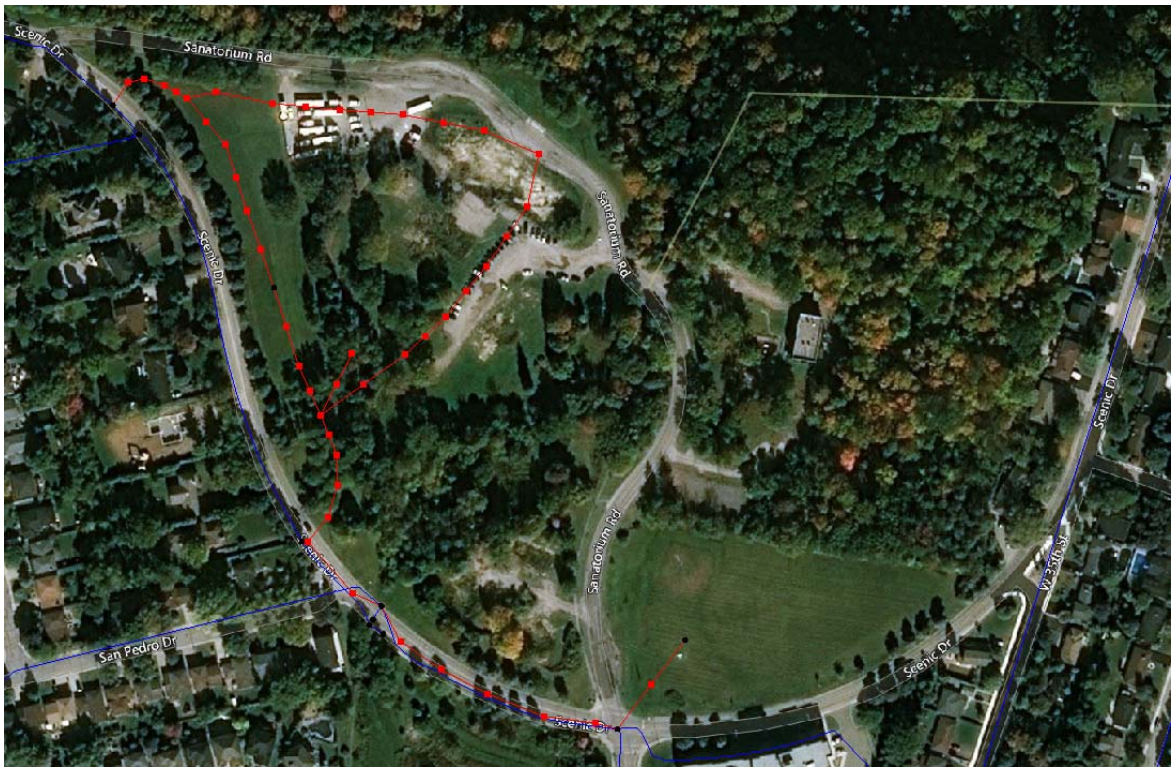


Figure 1 – Proposed Site Layout

The purpose of this report is to provide an overview of water servicing needs for the proposed development giving due consideration to the City of Hamilton and Ministry of the Environment, Conservation and Parks (MECP) standards.

2.0 Design Criteria

2.1 Relevant Guidelines

The following documents have been reviewed to develop demands and criteria for the proposed development:

- City of Hamilton Water and Wastewater Master Plan (WWWMP), KMK Consultants, November 2006;
- City of Hamilton Development Charge Background Study, Watson and Associates Economists Ltd., June 2014;
- City of Hamilton Comprehensive Development Guidelines and Financial Policies Manual, 2019;
- Ontario Building Code (OBC), 2019;
- City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy (PW19096) Memorandum, November 18, 2019;
- Ministry of the Environment, Climate and Parks (MECP) Watermain Design Criteria.

2.2 Domestic Water Demands

Domestic water demands for the proposed development have been calculated using the City of Hamilton Water and Wastewater Master Plan (WWWMP), November 2006 criteria (KMK, 2006). The WWWMP has been used to determine overall average demands, as well as the Max Day Demand (MDD) and Peak Hour Demand (PHD) peaking factors.

Additionally, the population within the proposed development has been calculated based upon the average persons per unit (PPU) as described in the City of Hamilton's Development Charge Background Study (Watson and Associates Economists Ltd., 2014). The development is proposed to consist of low-density townhomes, and high-density high-rise buildings by way of condominiums.

The above criterion is summarized in Table 2.1, with calculated demands presented in Appendix B.

Table 2.1 – City of Hamilton Demand Criteria and Persons per Unit

Building Type	Persons per Unit (PPU)	Average Day Demand (L/capita/day)	Peaking Factors	
			Max Day Demand (MDD)	Peak Hour Demand (PHD)
Low Density Residential	3.39	360	1.9	3
High Density Residential	1.76			

The City of Hamilton criteria (KMK, 2006) indicates that 360 liters per capita per day (L/capita/day) should be used in demand generation for residential developments. This document further states that an MDD peaking factor of 1.9, and a peak hour demand peaking factor of 3.0 should be applied to determine demands under max day and peak hour flow scenarios.

The townhomes at the proposed development are considered low density and are therefore assumed to have 3.39 persons per unit (Watson and Associates Economists Ltd., 2014). As each townhome structure is

a two (2) storey residential building with 4 units, the total population per townhome building is 13.56 (~14) persons, and 196 persons in total for all townhomes.

The condominiums at the proposed development are considered high density and are therefore assumed to have 1.76 persons per unit (Watson and Associates Economists Ltd., 2014). As each condominium structure has multiple stories, the total unit count stands at 408 units for the western portion of the site and 370 units for the eastern portion of the site. As such, the total expected population is 718.1 (~719) persons on the western portion, and 651.2 (~652) persons on the eastern portion of the proposed development. The total proposed population based upon the foregoing is estimated to be 1,567 persons.

2.3 Fire Flow Demands

The required fire flow used in the model has been calculated using the Ontario Building Code (OBC) standard. The City has previously used the "Water Supply for Public Fire Protection" by the Fire Underwriters Survey (FUS, 1999).

However, the required fire flow used in the model has been calculated using the Ontario Building Code (OBC) standard, as per the City of Hamilton interpretation memo regarding the City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy (PW19096), and appended in Appendix A.

As per this memorandum, fire flow requirements will be based on the greater of the required fire flow calculated using the OBC method and a City Target Available Fire Flow (AFF) based upon land use. System available fire flow should meet or exceed the greater of the OBC required fire flow, or the AFF for the land use being proposed. Wood has calculated the relevant fire flows as per the OBC methodology and compared them as per Table 1 within the City memorandum (ref. Table 1, City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy Memorandum, Appendix A).

The higher of these values have been used to determine the fire flows for each given unit. The fire flow demands calculated from OBC, AFF, and the required target fire flows selected for the modelling for each building are shown in Table 2.2. Table 2.2 shows the site being bisected into western and eastern portions. The OBC fire flow calculations are included in Appendix C.

Table 2.2 – Fire Flow Demands per Structure for Proposed Development (OBC vs. AFF)

Proposed Development - West Side			
Building Type	**OBC Flow Rate (LPS)**	**Target Available Fire Flow - Residential (LPS)**	**Required FF (LPS)**
2 Storey Residential (x14), 4 Units each	45	150	150
8 Storey Residential A, 204 Units	150	150	150
8 Storey Residential B, 204 Units	150	150	150
Proposed Development - East Side			
Building Type	**OBC Flow Rate (LPS)**	**Target AFF - Residential (LPS)**	**Required FF (LPS)**
8 Storey Condominium, 75 units	105	150	150
8 Storey Condominium, 141 units	150	150	150
5 Storey Condominium (x2), 58 units each	150	150	150
4 Storey Condominium, 38 units	90	150	150

The fire flow demands for the eastern portion have been capped at 150 LPS (liters per second), as the stub connection to the eastern portion of the site will provide service to the solitary hydrant there, with the building sprinkler systems expected to provide the majority of fire-fighting capabilities. The same principle has been applied to the two high rise structures on the western portion of the proposed development.

Per communication with the architects (ref. email Senior-Harrison, August 5 2020), the 8-storey high rise structures will be non-combustible and sprinklered, with a 2-hour fire rated floor construction as per the OBC. The 5/6 storey high rise structures will be non-combustible and sprinklered, with a 1-hour fire rated floor construction as per the OBC. The townhome structures will be of combustible construction as per the OBC, with unrated floors and 1-hour fire separations between each unit.

As per Table 2.2, the fire flow rate as calculated by OBC is typically lower than, or equivalent to, that of the target available fire flow based upon a residential land use. For the target available fire flow (AFF), all structures in the proposed development are “Residential Multi” type, consisting of greater than 3 units. As such, the target AFF is 150 LPS which has been carried forth for fire flow analysis.

2.4 Pressures

As outlined in the City of Hamilton’s WWWMP (KMK, 2006), the acceptable pressures under normal conditions range between 275 kPa (40 psi) and 690 kPa (100 psi).

Under fire flow conditions, the pressures in the proposed development and at all points in the system must not be lower than 140 kPa (20 psi) under a fire flow event

3.0 Hydraulic Model

3.1 Model Source

The existing conditions hydraulic model, built upon WaterCAD software, was received by Wood from City of Hamilton staff on May 21, 2020, and consisted of 2018, 2021, and 2031 scenarios with demands attributed for average day demand (ADD), maximum day demand (MDD), and peak hour demand (PHD) for the City network.

Based upon a review by Wood, it has been noted that nodes in the City's 2021 and 2031 scenarios do not have demands allocated at several nodes, wherein they are populated in the 2018 scenario. While this is a discrepancy, the total system demands are increasing per scenario, which is to be expected. As this report outlines the water demands of a localized proposed development, this discrepancy in node allocations in future conditions scenarios is acceptable.

The proposed development has been added to this model to simulate the availability of flow and maintenance of pressures while connected to the larger City of Hamilton water network. Junction elevations have been based upon local ground elevation grades.

Model updates included adding a 250 mm diameter PVC watermain, and a section of 300 mm diameter PVC watermain on the western portion of the proposed development with a 123-meter-long, 300 mm diameter PVC watermain twinned from the connection point at the intersection of Scenic Drive/Sanatorium Road, to the intersection of Scenic Drive/San Pedro Drive. A 300 mm diameter PVC watermain supplying a stub connection on the eastern portion, will service the proposed high-rise structures there.

As further detailed in Section 4.0, additional upgrades to existing City infrastructure on Scenic Drive will have to be implemented for this proposed development to meet City criteria for fire flow demands.

The Hazen-Williams friction factors used for the proposed pipe network are based upon nominal pipe diameters as obtained from Section 3.0 from the MOECC Watermain Design Criteria and have been listed in Table 3.1.

Table 3.1 – Hazen Williams Friction Factors (MOECC Watermain Design Criteria)

Nominal Diameter (mm)	Hazen Williams Friction (C) Factor
150	100
200 to 250	110
300 to 600	120
Over 600	130

Proposed pipe diameters fall within the 200 mm to 250 mm diameter, and 300 mm to 600 mm diameter ranges, of PVC material. As such, the proposed pipe network has a Hazen Williams factor of 110 for pipes that are between 200 mm to 250 mm in diameter, with 300 mm diameter pipes having a Hazen Williams factor of 120.

3.2 Boundary Conditions

Two modelling alternatives have been modelled as required by the City of Hamilton, characterized by the initial water levels in tank HDR05. Alternative 1 consisted of the tank being 50% full, at 232.32 m. Alternative 2 consisted of the tank being 70% full, at 233.90 m. The operating conditions at the Woodward Pump station have not been changed from the provided City model. The operating conditions are identical and apply to both alternatives (50% full reservoir, 70% full reservoir) and both scenarios for 2018 (existing) and 2031 (proposed/ultimate). The pump status is presented in Table 3.2.

Table 3.2 – Pump Status for Woodward Station (both alternatives and scenarios)

System Component (Pumps)	Average Day Demand (ADD)	Maximum Day Demand (MDD)	Maximum Day Demand + Fire Flow (MDD + FF)	Peak Hour Demand (PHD)
HWHLP-PMP-1	ON	ON	ON	ON
HWHLP-PMP-2	OFF	OFF	OFF	OFF
HWHLP-PMP-3	OFF	OFF	OFF	OFF
HWHLP-PMP-4	ON	ON	ON	ON
HWHLP-PMP-5	OFF	OFF	OFF	OFF
HWHLP-PMP-6	ON	ON	ON	ON

3.3 Model Validation

The hydraulic model has been validated against hydrant testing information as provided by the City of Hamilton. This reservoir scenario was performed under Alternative 1, with the initial reservoir capacity at 50% (232.32 m). One set of the hydrant testing results has been provided to Wood, dated February 12, 2020 (ref. Appendix A) and is based upon the output from the City of Hamilton GIS database. The hydraulic model has been validated based upon the 2018 ADD conditions to compare hydrant pressures. As the City hydrant database is more discrete in comparison to the provided modelling, a select number of nodes within the model in the vicinity of the proposed site have been used to extract pressures to compare against. Table 3.3 outlines the pressure results from hydrant testing and compares these to the nodal points which were found from the hydraulic model.

Table 3.3 – Hydrant Testing vs. Hydraulic Model Validation

Address (from City Hydrant Flow Test Results)	Pressure District	Date Field Data Observed	Field Tested Static Pressure (psi)	Model Junction	Model Pressure (psi)	Pressure Difference (psi)
675 SCENIC DR HAMILTON	5	2017/9/14 1:10 PM	59	HC20T003	61	2
SCENIC DR HAMILTON	5	2017/9/14 1:41 PM	60	HC20T010	54	-6
SANATORIUM RD HAMILTON	5	2017/9/14 1:39 PM	54	HC20L001	52	-2
SANATORIUM RD HAMILTON	5	2017/9/14 1:37 PM	54	HC20T011	51	-3
901 SCENIC DR HAMILTON	5	2017/9/14 1:42 PM	64	HC21T009	60	-4

Comparing the pressures obtained from the hydraulic model to the City of Hamilton fire hydrant pressures in Table 3.3, it can be seen that there is minimal pressure difference between the hydraulic model and the pressures obtained from the City data, with the difference ranging from 2 psi to 6 psi. It should be noted that as the hydraulic model does not model City hydrants as elements, proximate junctions in the hydraulic model were selected, with pressures extracted therein. These junctions may not be in the physical location of the City hydrants but have been selected based on their relative proximity, which justifies the differences in pressures.

4.0 Hydraulic Analysis

The proposed watermain layout has been modelled for Average Day (ADD), Maximum Day (MDD), Maximum Day plus Fire Flow (MDD + FF), and Peak Hour (PHD) under existing (2018) and ultimate (2031) conditions using the WaterCAD Connect Edition 2 software (V10i) of the PD5 network as detailed in Section 3.0. The aforementioned has been modelled for both alternatives with tank HDR05 being 50% full for Alternative 1, and 70% full for Alternative 2.

It should be noted that the existing City system will not have the capability to meet the fire flow demands of the proposed development. Wood has determined that the fire flow demands of the proposed development can only be met via the upgrade of the existing watermain along Scenic Drive. As such, the hydraulic analysis is divided into Section 4.1 (consisting of the proposed development without upgrading existing watermains on Scenic Drive), and Section 4.2 (consisting of the proposed development with upgrading some existing watermains on Scenic Drive to be able to meet the fire flow demands required by the proposed development).

The most conservative scenario of Alternative 1 (tank HDR05 50% initial capacity), with max day demand plus fire flow conditions under the ultimate 2031 build out conditions has been used to determine pipe sizing for the proposed development.

Five (5) fire hydrants have been proposed on the western portion of the site, with one (1) on the eastern portion. The high-rise structures are expected to have sprinklers installed, which will provide the bulk of fire-fighting potential. Hydrant spacing has been determined through the City of Hamilton Comprehensive Development Guidelines and Financial Policies Manual (2019). As per the guidelines, fire hydrants must be placed at 110 m spacing on collector streets, high density residential streets, commercial, and industrial streets.

4.1 Hydraulic Assessment (Existing Conditions)

The following system pressures and available fire flows are based upon “as-is” existing conditions, with no additions to City infrastructure.

4.1.1 System Pressures

Table 4.1.1 summarizes the predicted pressure ranges for the proposed development under ADD, MDD, and PHD demand regimes while considering Alternatives 1 and 2 for both existing and ultimate conditions.

Table 4.1.1 – Average Day, Maximum Day, and Peak Hour Pressures for Existing (2018) and Ultimate (2031) Conditions (No Upgrades on Scenic Drive)

Alternative 1 (Reservoir 50% Full)	ADD (kPa)	MDD (kPa)	PHD (kPa)
2018 (Existing)	350 - 411	338 - 399	318 - 378
2031 (Ultimate)	363 - 423	323 - 383	277 - 337
Alternative 2 (Reservoir 70% Full)	**ADD (kPa)**	**MDD (kPa)**	**PHD (kPa)**
2018 (Existing)	350 - 411	338 - 399	318 - 378
2031 (Ultimate)	363 - 423	338 - 398	292 - 353

Based upon Table 4.1.1, under both alternatives and flow regimes, the proposed development is able to remain within the acceptable pressure range of 275 kPa to 690 kPa.

4.1.2 Available Fire Flow

The minimum allowable pressure under the max day demand plus fire flow (MDD+FF) scenario is 140 kPa (20 psi), at the location of the proposed development. The OBC methodology and the AFF have been used (ref. Section 2.3) to determine required fire flows for the proposed development. Table 4.1.2 outlines the ranges of available fire flow at the proposed development.

Table 4.1.2 – Available Fire Flow Range (Liters per Second) for Existing (2018) and Ultimate (2031) Conditions (No Upgrades on Scenic Drive)

Alternative 1 (Reservoir 50% Full)	MDD + FF (LPS)
2018 (Existing)	143.8 - 185.8*
2031 (Ultimate)	124.6 - 155.5*
Alternative 2 (Reservoir 70% Full)	**MDD + FF (LPS)**
2018 (Existing)	143.8 - 185.8*
2031 (Ultimate)	130.6 - 164.1*

**Upper limit only applies to eastern portion of the site*

Based upon the fire flow results in Table 4.1.2, the proposed development will not be able to meet the fire flow demands for the western portion of the proposed development under current conditions (without upgrading, or adding to, existing City infrastructure on Scenic Road). The eastern portion of the proposed development will be able to meet fire flow criteria (see note for Table 4.1.2). Regardless, a minimum pressure of 140 kPa (20 psi) will be maintained on the proposed development.

Based upon Section 2.3, at least 150 LPS of available fire flow must be provided to the proposed development to meet City criteria. From Wood's hydraulic fire flow analysis, the eastern portion of the proposed development will be able to meet this fire flow criteria without any upgrades or changes to existing City infrastructure.

However, the western portion of the proposed development will not be able to meet fire flow demands. Under the worst-case scenario (Alternative 1, 2031 MDD + FF), the western portion of the site will be supplied by only 124.6 LPS of flow, 25.4 LPS lower than the required 150 LPS. Therefore, the proposed development will not be able to meet the fire flow demands without implementing additional upgrades to existing City infrastructure on Scenic Drive.

4.2 Hydraulic Assessment (Proposed Conditions – Twin Pipes Infrastructure)

Under this proposed configuration, approximately 123 m of 300 mm diameter PVC piping will be twinned along the northern portion of Scenic Drive, in addition to the aforementioned proposed development configuration. This upgrade will span from the intersection of Scenic Drive and Sanatorium Road, to the proposed development northwest of the intersection of Scenic Drive and San Pedro Drive. These upgrades will help achieve the fire flow demand criteria required by the proposed development. The full results from the following analyses are presented in Appendix D.

4.2.1 System Pressures

Table 4.2.1 summarizes the predicted pressure ranges for the proposed development under ADD, MDD, and PHD demand regimes while considering Alternatives 1 and 2 for both existing and ultimate conditions.

Table 4.2.1 – Average Day, Maximum Day, and Peak Hour Pressures for Existing (2018) and Ultimate (2031) Conditions (Upgrades on Scenic Drive)

Alternative 1 (Reservoir 50% Full)	ADD (kPa)	MDD (kPa)	PHD (kPa)
2018 (Existing)	350 - 411	338 - 399	317 - 378
2031 (Ultimate)	363 - 423	323 - 383	277 - 338
Alternative 2 (Reservoir 70% Full)	**ADD (kPa)**	**MDD (kPa)**	**PHD (kPa)**
2018 (Existing)	350 - 411	338 - 399	318 - 378
2031 (Ultimate)	363 - 423	338 - 398	292 - 353

Based upon Table 4.2.1, under both alternatives and flow regimes, the proposed development is able to remain within the acceptable pressure range of 275 kPa to 690 kPa.

4.2.2 Available Fire Flow

The OBC methodology and the AFF have been used (ref. Section 2.3) to determine required fire flows for the proposed development. Table 4.2.2 outlines the ranges of available fire flow at the proposed development, while considering upgrading existing City infrastructure.

Table 4.2.2 – Available Fire Flow Range (Liters per Second) for Existing (2018) and Ultimate (2031) Conditions (Upgrades on Scenic Drive)

Alternative 1 (Reservoir 50% Full)	MDD + FF (LPS)
2018 (Existing)	177.3 - 180.8
2031 (Ultimate)	155.5 - 157.9
Alternative 2 (Reservoir 70% Full)	**MDD + FF (LPS)**
2018 (Existing)	185.7 - 189.5
2031 (Ultimate)	155.5 - 157.9

Based upon the fire flow results in Table 4.2.2, the proposed development will be able to meet the fire flow demands of an available 150 LPS, also attaining the minimum required pressure of 140 kPa (20 psi) in the proposed development.

The fire flow results for the proposed development are provided in Appendix D.

4.3 Transient Pressures

Based upon the MECP Watermain Design Criteria, all watermains shall be designed such that pipes and joints are able to withstand the maximum operating pressure, plus the surge pressure that would be created by stopping a water column moving at 0.6 m/s. Appropriate pipe classification, thrust blocks, and pipe restraints are to be added during the detailed design phase for the proposed development while considering MECP Watermain Design Criteria. These considerations should be further evaluated during the detailed design stage.

4.4 System Flushing

A modelled flushing test has been performed through WaterCAD for the proposed development's water distribution network, under existing (2018) ADD conditions. The MECP Watermain Design criteria requires a minimum flushing velocity of 0.8 m/s. WaterCAD allows for flushing tests by representing a modeled junction as a flow emitter, via a flow emitter coefficient. This flow emitter coefficient is equivalent to the components of the hydrant, consisting of the lateral, valve, bends, and outlet, to represent the effects of a hydrant collectively. The emitter coefficient for this model has been taken as 150 gpm/psi$^{0.5}$, which is within the typical range of emitter coefficients for hydrants across North America.

Based on the proposed distribution network, the flushing criteria is met by half of the distribution network consisting of ten (10) pipes out of a total of twenty (20) modelled pipes. An additional six (6) pipes had flushing velocities ranging from 0.76 m/s to 0.79 m/s, which approach the MECP criteria. The remaining four (4) pipes were not able to meet, or approach, this criterion, with results ranging from 0.46 m/s to 0.68 m/s. Not all pipes are able to meet the MECP flushing criteria due to the use of larger pipe diameters in the watermain servicing for the proposed development. Wood has optimized the system to meet fire flow demands, while still accounting for flushing where possible. While smaller pipes may be used to improve flushing, they will not be able to achieve the fire flow criteria necessitated by the proposed development. The data from the flushing test has been included in Appendix F.

5.0 Conclusions and Recommendations

5.1 Conclusions

Based on the assessments outlined within this report, the proposed development at 865 Sanatorium Road will be able to meet the required hydraulic criteria as prescribed by the MECP and the City of Hamilton demand criteria for ADD, MDD, and PHD scenarios under Wood's recommended watermain servicing solution for both 2018 and 2031 scenarios (i.e. twinning along Scenic Drive). Fire flow demands as determined using OBC methodology will also be achieved under the proposed servicing.

5.2 Recommendations

Wood recommends the installation of a 300 mm diameter PVC watermain from the intersection of Scenic Drive/Sanatorium Road, to the intersection of Scenic Drive/San Pedro Drive. This addition will, in effect, twin the existing 300 mm diameter ductile iron piping, thereby allowing sufficient flow to reach the western portion of the proposed development.

The proposed development will consist of a 300 mm diameter stub connection for the eastern portion of the site, with the western portion of the site being serviced by a 300 mm diameter PVC watermain connected at the intersection of Scenic Drive/San Pedro Drive to the connection point at the proposed high rise units. The remaining pipe network on the western portion is proposed to be of 250 mm diameter PVC piping, which has been optimized to provide flushing for as much of the site as possible, while still maintaining fire flow demand criteria.

Respectfully submitted,

Wood Environment & Infrastructure Solutions,
a division of Wood Canada Limited

Per: Ali Aamir, E.I.T.
Engineer in Training

Per: Rajan Sawhney, P.Eng.
Senior Water/Wastewater Engineer

AA/RS

APPENDIX A:

Background Information

CITY OF HAMILTON
PUBLIC WORKS DEPARTMENT
Hamilton Water Division

TO:	Chair and Members Public Works Committee
COMMITTEE DATE:	November 18, 2019
SUBJECT/REPORT NO:	City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy (PW19096) (City Wide) **(Outstanding Business List Item)**
WARD(S) AFFECTED:	City Wide
PREPARED BY:	Bert Posedowski, (905) 546-2424 Ext. 3199
SUBMITTED BY: **SIGNATURE:**	Mark Bainbridge Director, Water & Wastewater Planning & Capital Public Works Department

RECOMMENDATION(S)

(a) That the Watermain Fire Flow Requirement Design Guidelines Policy attached as Appendix A to Report PW19096 be approved;

(b) That the appropriate staff be authorized and directed to revise and update the Comprehensive Development Guidelines and Financial Policies Manual as required, and to bring forward for Council's consideration any necessary amendments to the Adequate Services By-law or any other City of Hamilton By-laws;

(c) That delegated authority be granted to the General Manager of Planning and Economic Development or a designate to make adjustments and changes as may be required to implement the Watermain Fire Flow Requirement Design Guidelines Policy, as described as a method of transition in this report, to any approved Draft Plan of Subdivision or Registered Plans of Subdivision; and

(d) That item AAL - Correspondence from Suzanne Mammel, Hamilton-Halton Home Builders' Association, respecting the Implementation of the Proposed New Hamilton Fire Flow Policy be identified as completed and be removed from the Public Works Committee Outstanding Business List.

EXECUTIVE SUMMARY

Several stakeholders in the development industry have expressed concern regarding the implementation of the City of Hamilton's (City's) current watermain fire flow requirement design guidelines approach that is based on the Fire Underwriters Survey (FUS) Insurance Underwriters Guidelines. In order to address these concerns, staff initiated a study to review the City's current guidelines for development application approvals and to provide recommendations for a revised approach.

After an extensive review and evaluation process that included collaboration with stakeholders; staff from Planning and Economic Development, Legal Services, Public Works, Hamilton Fire and members of the Hamilton Halton Home Builders Association (HHHBA), a preferred approach based on two levels of policy has been developed. It includes a Master Plan Approach based on the Ministry of the Environment, Conservation and Parks (MECP) guidelines; and a Development Application Approach approval process based on the Ontario Building Code (OBC) and a Target Available Fire Flow. The proposed City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy is attached as Appendix "A" to Report PW19096.

The proposed two-level approach will ensure that a robust and reliable trunk water distribution system is retained while also offering a reasonable level of service, at the local street level, by following an OBC and land-use targets strategy. The recommended policy will provide clarity for stakeholders and will assist staff in reviewing and processing applications. This aligns with the City's Open for Business mandate to create consistent, predictable, and customer-focused services that encourage investment.

Alternatives for Consideration - Not Applicable

FINANCIAL – STAFFING – LEGAL IMPLICATIONS

Financial: N/A.

Staffing: N/A

Legal: N/A

HISTORICAL BACKGROUND

In 2005, Council adopted Report No.(PW05050) that endorsed the City's Water and Wastewater Master Plan Policy Paper with policy statements and implementation strategies for the 2006 Master Plan. Policy No.W.06 of the policy paper stated that: "The City of Hamilton shall consider the Ministry of the Environment Guidelines and the Insurance Underwriters (FUS) Guidelines for establishing the acceptable level of fire flow".

In 2016, Council adopted the Comprehensive Development Guidelines and Financial Policies Manual (PED12165) that included watermain fire flow requirement design guidelines based on Policy No. W.06. The City's current guidelines require that the applicant calculate fire flow needs for their development based on FUS. The applicant may also be required to conduct hydrant tests to determine the available fire flow at their proposed development. If the identified FUS based fire flow needs exceed the system fire flows available at the hydrant, the applicant is required to either alter their development plan so that FUS based fire flow needs are met or undertake capacity upgrades to City infrastructure.

The City's development community and stakeholders have for some time expressed concern and raised a number of issues regarding the implementation of the current approach including:

- Long approvals times and delays resulting in the need to make multiple submissions which is time consuming and costly.
- Potential higher construction, maintenance and replacement costs resulting from building bigger pipes that are required to meet FUS fire flow needs.
- Lack of clarity and consistency when using the approach that often leads to interpretation conflicts (FUS guideline for required fire flow calculation can be confusing and open to differing interpretations).
- The FUS approach can be conservative leading to oversizing of local watermains that can potentially cause water quality issues.

As a matter of best practice, staff are dedicated to continuously reviewing current practices, processes and policies to identify improvements and ensure that the delivery of client services responds to the needs of the City's customers and stakeholders. Therefore, to address the City's customers and stakeholder's concerns, staff initiated a study that would review the City's current watermain fire flow requirement level of service and design guidelines approach for purposes of development application approvals. The study would further compare the City's approach with industry best practices.

In July 2017, a consultant (GM BluePlan Engineering) was retained to carry out this study as part of the on-going City-Wide Water/Wastewater and Stormwater Master Plans update project. The study was summarized in a Technical Memo that will form part of the City-Wide Water/Wastewater and Stormwater Master Plans report.

POLICY IMPLICATIONS AND LEGISLATED REQUIREMENTS

Policies and by-laws that may require updating as a result of the recommendations of this report include:

- City's Comprehensive Development Guidelines and Financial Policies, 2018
- City's Water and Wastewater Master Plan Policy, 2006
- City's Adequate Services By-Law

Legislation requirements to which the recommendations of this report align include:

- *Safe Drinking Water Act, 2002*
- *Ontario Building Code and Building Code Act, 1992*
- *Fire Protection and Prevention Act, 1997*

RELEVANT CONSULTATION

Staff conducted several meetings and workshops which included staff from Planning and Economic Development, Legal Services, Public Works, Hamilton Fire and representatives of HHHBA and the Development Industry Liaison Group (DILG). In addition, the consultant conducted a review of approaches for neighbouring municipalities.

ANALYSIS AND RATIONALE FOR RECOMMENDATION(S)

The study evaluated several fire flow requirement options with varying levels of service. The variations ranged from minimal fire flows based on OBC, to conservatively high fire flows based on FUS.

After an extensive review process that included collaboration with stakeholders including staff from Planning and Economic Development, Public Works, Hamilton Fire and members of the HHHBA, an intermediate fire flow service level was selected for the Development Application Approach. This approach utilizes fire flow requirements based on the greater of the required fire flow calculated using the OBC method and a City Target Available Fire Flow based on land use. Under this approach, developers shall be responsible for providing the system available fire flow appropriate for the

development being proposed. The requirement to provide system available fire flow is a departure from the FUS guideline where required fire flow for a building could be altered through changes in building construction and/or building separation (often referred to as changes to built form).

The change being proposed in this report primarily deals with a change to the Development Application Approach. The proposed approach is generally consistent with what other municipalities are using and will provide a level of service that is adequate and appropriate for the City. It is a reasonable approach that establishes land-use-based targets for the water distribution system and available fire flows to ensure that the water system is robust and reliable.

The approach may represent a reduction in watermain fire flow requirements from the current City guidelines which are based on FUS. It will not however, materially impact the City's insurance grading because that grading is not predominantly based on watermain fire flow design guidelines. As stated on the Fire Underwriters Survey (Canada) website, FUS grading criteria assesses the water supply system's ability to provide reliable water supplies along with an assessment of storage, pumping and duplication of various parts of the system. The City's water distribution system is well looped and is backed up with adequate storage and pumping capacity and will continue to provide reliability and redundancy as required by the FUS grading criteria.

A preliminary, high level review of the distribution system has identified that approximately 86% of commercial lands, 83% of institutional lands and 77% of residential lands have systems that currently meet the Target Available Fire Flow; 61% of industrial lands have systems that currently meet the Target Available Fire Flow. The City will continue to upgrade watermains to achieve Target Available Fire Flows, where practically feasible, through its ongoing state of good repair program.

The rationale for recommending this approach was based on satisfying the following key policy objectives:

1. Providing a safe, high quality and consistent supply of drinking water.

2. Building and maintaining a robust water distribution system that meets set targets of pressure, quality, reliability and fire flow.
3. Ensuring that adequate fire safety and building protection is provided.

4. Provide a consistent and transparent methodology for establishing fire flow requirements.

In addition to the foregoing analysis of fire flow requirements for Development Applications, the study reviewed fire flow requirement for the master Planning level

system-wide sizing of storage and trunk infrastructure. The City's current policy and design criteria for water storage and trunk watermains is as follows:

- Storage Volume - the system is to provide storage volume for each Pressure District of Max Day Pressure District Demand plus MECP Guidelines Fire Flow Storage. This calculation generates a storage volume that provides water for equalization, emergency and fire protection within the zone.
- Transmission - Trunk watermains are sized in order to convey Maximum Day Demands + MECP fire flow. The fire flow is based on service area population, typically delineated by Pressure District.

The study recommended that at a master plan level, the City continues to use the current policies.

The recommended Watermain Fire Flow Requirement Design Guidelines Policy will be based on two policy levels:

1. Master Plan Approach

   "The City of Hamilton will establish acceptable trunk infrastructure levels of service for fire flow and storage through consideration of land use and the Ministry of Environment, Conservation and Parks Design Guidelines."

2. Development Application Approach

   "The City of Hamilton endeavours, through this policy, to provide a local water distribution network with a system Available Fire Flow (AFF – water available for fighting a fire) that meets the greater of the Required Fire Flow calculated using the Ontario Building Code water supply flow rate method or the City's Target Available Fire Flow based on land use. Developers shall be responsible for providing the system AFF appropriate for the development being proposed."

The recommended policy provides the overall guiding principles for Watermain Fire Flow Design Guideline Policy. Key policy principles include:

1. Developers are required to meet OBC standards for building construction. No credits will be considered for reducing required fire flow outside of any provisions contained within the *Ontario Building Code Act* or regulations under the *Act.*

2. OBC required fire flow calculations will be required as part of any development application submission. The required fire flow will be determined using the OBC water supply flow rate method (OBC section A-3.2.5.7). This methodology will be

applied to all buildings falling under Part 3 and Part 9 of the Building Code (OBC sections 1.1.2.2 and 1.1.2.4).

3. System available fire flow calculations will be required as part of a development application submission and will be based on field testing and/or hydraulic modelling (as directed by the City). System available fire flow shall meet or exceed the greater of OBC required fire flow or the target available fire flow for the land use being proposed. For mixed use developments the target available fire flow will be based on the proposed land-use with the highest target available fire flow. The target available fire flow will be as defined in Table 1.

Table 1: Target Available Fire Flow

Land Use	Target AFF (L/s)
Commercial	150
Small ICI (<1,800 m^3)[1]	100
Industrial	250
Institutional	150
Residential Multi (greater than 3 units)	150
Residential Medium (3 or less units)	125
Residential Single	75
Residential Single (Dead End)	50

[1] 1800m^3 represents a maximum building volume that qualifies as “Small ICI”

4. System upgrades required to achieve the greater of the OBC required fire flow or the target available fire flow (Table 1) will be the responsibility of the developer subject to local servicing policy and subject to the City’s state of good repair program.

5. The City’s Master Plan process will continue to establish system level of service for fire flow (trunk system and facilities).

6. The City’s Master Plan process, which will be based on Growth Related Integrated Development Strategy (GRIDS2) and the City’s Official Plan, will proactively develop intensification programs that will identify development related upgrades that can address both growth and fire flow deficiencies.

7. The City will be setting minimum available fire flow targets based on the recommendations of this study. The City will upgrade watermains to achieve target available fire flows, where practically feasible, through its ongoing state of good repair program.

Method of Transition

A transition plan has been developed through consultation with the Planning and Economic Development Department to address the treatment of development files at various stages of approval.

Based on where the application is in the review process, one of the following options will be applied:

A. Change to Municipal Infrastructure Pipe Size (Form 1) where Engineering Approvals have been granted and watermain infrastructure has not been constructed.

    - Submit revised hydraulic assessment (indicating the greater of OBC fire flow method or Target Available Fire Flow (Table 1). A change in pipe size after the approval of engineering design drawings and Form 1 will require a resubmission of engineering design drawings and Form 1 submission, along with the applicable fees.

B. Change to Municipal Infrastructure Pipe Size (Form 1) where Engineering Approvals has not been granted and watermain infrastructure has not been constructed.

    - Submit hydraulic assessment (indicating the greater of OBC Fire Flow method or Target Available Fire Flow (Table 1) and Form 1 along with the applicable fees.

C. Change to the Approved Method of Construction (for the purpose of this transition policy, reference to the term “method of construction” relates to building material (i.e. brick finish vs siding) or additional firewalls or sprinkler systems etc. that were required to reduce the calculative area and exposure impacts to meet the existing FUS guidelines) and the watermain infrastructure is in place and building permit not issued.

    - Complete required fire flow calculations using the OBC method and confirm AFF from existing infrastructure meets or exceeds the required fire flows based on OBC method. Submit calculations along with the applicable fees.

D. Change in Pipe Size where Site Servicing Approval has been granted and private water infrastructure has not been constructed

    - Will require submission of a revised Site Servicing Plan along with applicable fees.

ALTERNATIVES FOR CONSIDERATION

Not Applicable

ALIGNMENT TO THE 2016 – 2025 STRATEGIC PLAN

Community Engagement and Participation
Hamilton has an open, transparent and accessible approach to City government that engages with and empowers all citizens to be involved in their community.

Economic Prosperity and Growth
Hamilton has a prosperous and diverse local economy where people have opportunities to grow and develop.

Healthy and Safe Communities
Hamilton is a safe and supportive City where people are active, healthy, and have a high quality of life.

Our People and Performance
Hamiltonians have a high level of trust and confidence in their City government.

APPENDICES AND SCHEDULES ATTACHED

Appendix A - Watermain Fire Flow Requirement Design Guidelines Policy Summary Table

City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy

Table 1 - City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy Table

Policy No.	Policy Area	Policy Statement	Best Practices and Criteria
2019-FF-1	Development Application Approach	"The City of Hamilton endeavours, through this policy, to provide a water distribution network with a system Available Fire Flow (AFF – water available for fighting a fire) that meets the greater of the Required Fire Flow calculated using the Ontario Building Code (OBC) water supply flow rate method or the City's Target AFF based on land use. Developers shall be responsible for providing the system AFF appropriate for the development being proposed."	• Shorter approvals times with fewer submissions • Potential reduced construction, maintenance and replacement costs • Clarity and consistency in the calculations approach • Reasonable sizing of local watermains • Aligns with established Ontario Building Code-OBC practice
2019-FF-1a	Development Application Approach	"Developers are required to meet OBC standards for building construction. No credits will be considered for reducing required fire flow outside of any provisions contained within the *Ontario Building Code Act* or regulations under the *Act."*	
2019-FF-1b	Development Application Approach	"OBC required fire flow calculations will be required as part of any development application submission. The required fire flow will be determined using the OBC water supply flow rate method (OBC section A-3.2.5.7). This methodology will be applied to all buildings falling under Part 3 and Part 9 of the Building Code (OBC sections 1.1.2.2 and 1.1.2.4). "	
2019-FF-1c	Development Application Approach	"System available fire flow calculations will be required as part of a development application submission and will be based on field testing and/or hydraulic modelling (as directed by the City). System available fire flow shall meet or exceed the greater of OBC required fire flow or the target AFF for the land use being proposed. For mixed use developments the target available fire flow	

City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy

Policy No.	Policy Area	Policy Statement	Best Practices and Criteria
		will be based on the proposed land-use with the highest target available fire flow. The target available fire flow will be as defined in Table 1: Target AFF" Table 1: Target AFF Land Use — Target AFF (L/s) Commercial — 150 Small ICI (<1,800 m3)[1] — 100 Industrial — 250 Institutional — 150 Residential Multi[2] — 150 Residential Medium (3 or less units)[3] — 125 Residential Single — 75 Residential Single (Dead End) — 50 1 1800m3 represents a maximum building volume that qualifies as "Small ICI" 2Residential Multi is defined as a residential dwelling with > 3 units 3Residential Medium is defined as a residential dwelling with ≤ 3 units	
2019-FF-1d	Development Application Approach	"System upgrades required to achieve the greater of the OBC required fire flow or the target available fire flow (Table 1) will be the responsibility of the developer subject	

City of Hamilton Watermain Fire Flow Requirement Design Guidelines Policy

Policy No.	Policy Area	Policy Statement	Best Practices and Criteria
		to local servicing policy and subject to the City's state of good repair program."	
2019-FF-2	Master Plan Approach	"The City of Hamilton will establish acceptable trunk infrastructure levels of service for fire flow and storage through consideration of land use and the Ministry of Environment, Conservation and Parks Design Guidelines".	• Robust and reliable trunk network and infrastructure from which local sub-networks are serviced • Offers flexibility in growth options and GRIDS2 growth strategies
2019-FF-2a	Master Plan Approach	"The City's Master Plan process will continue to establish system level of service for fire flow (trunk system and facilities)".	
2019-FF-2b	Master Plan Approach	"The City's Master Plan process, which will be based on Growth Related Integrated Development Strategy (GRIDS2) and the City's Official Plan, will proactively develop intensification programs that will identify development related upgrades that can address both growth and fire flow deficiencies".	
2019-FF-3	State of Good Repair Approach	"The City will be setting minimum available fire flow targets based on the recommendations of this study. The City will upgrade watermains to achieve target available fire flows, where practically feasible, through its ongoing state of good repair program".	
2019-FF-4	Conformity with Legislation	As required this policy will be reviewed and amended to align with changes in related legislation.	

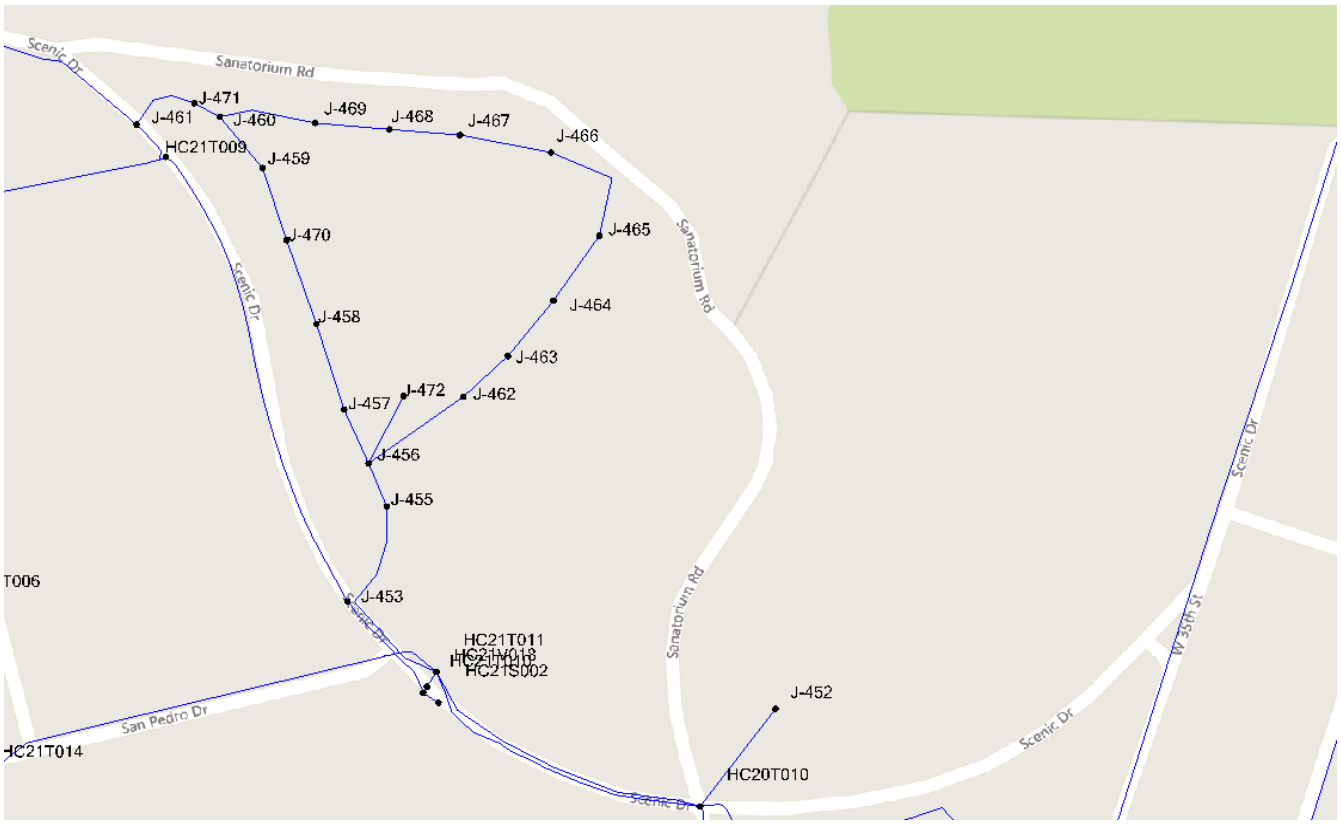
City of Hamilton Hydrant Pressures

Hydrant ID	**Address**	**Pressure Zone**	**Date of Most Recent HFI2 Test (Field work)** (mm/dd/yy)	**Static Pressure** [psi]	**Residual Pressure** [psi]	**Test Flow** [Imp Gal/ min]	**DSR** [psi]	**DSR2** [psi]	**FAR20** [Imp Gal/ min]
HC20H027	77 WEST 35TH ST HAMILTON	5	2017/9/14 1:16 PM	60	58	810	2	40	4084
HC20H028	749 SCENIC DR HAMILTON	5	2017/9/14 1:12 PM	57	55	1050	2	37	5075
HC20H029	717 SCENIC DR HAMILTON	5	2017/9/14 1:11 PM	62	53	890	9	42	2045
HC20H033	675 SCENIC DR HAMILTON	5	2017/9/14 1:10 PM	59	47	930	12	39	1758
HC20H034	REDFERN AVE HAMILTON	5	2017/7/30 12:34 PM	58	55	950	3	38	3742
HC20H035	SCENIC DR HAMILTON	5	2017/9/14 1:41 PM	60	57	1160	3	40	4698
HC20H036	SANATORIUM RD HAMILTON	5	2017/9/14 1:39 PM	54	52	1010	2	34	4664
HC20H039	SANATORIUM RD HAMILTON	5	2017/9/14 1:37 PM	54	52	1010	2	34	4664
HC21H002	901 SCENIC DR HAMILTON	5	2017/9/14 1:42 PM	64	58	870	6	44	2551
HC21H003	869 SCENIC DR HAMILTON	5	2017/9/14 1:40 PM	58	56	940	2	38	4610
HC21H004	931 SCENIC DR HAMILTON	5	2017/9/14 1:44 PM	62	57	990	5	42	3124
HC21H005	957 SCENIC DR HAMILTON	5	2017/9/14 1:45 PM	63	56	960	7	43	2559
HC21H006	985 SCENIC DR HAMILTON	5	2017/9/14 1:46 PM	66	58	1010	8	46	2597
HC21H012	65 SAN PEDRO DR HAMILTON	5	2017/7/30 1:28 PM	64	60	1100	4	44	4016
HC21H013	27 SAN PEDRO DR HAMILTON	5	2017/7/30 1:29 PM	60	56	1120	4	40	3883

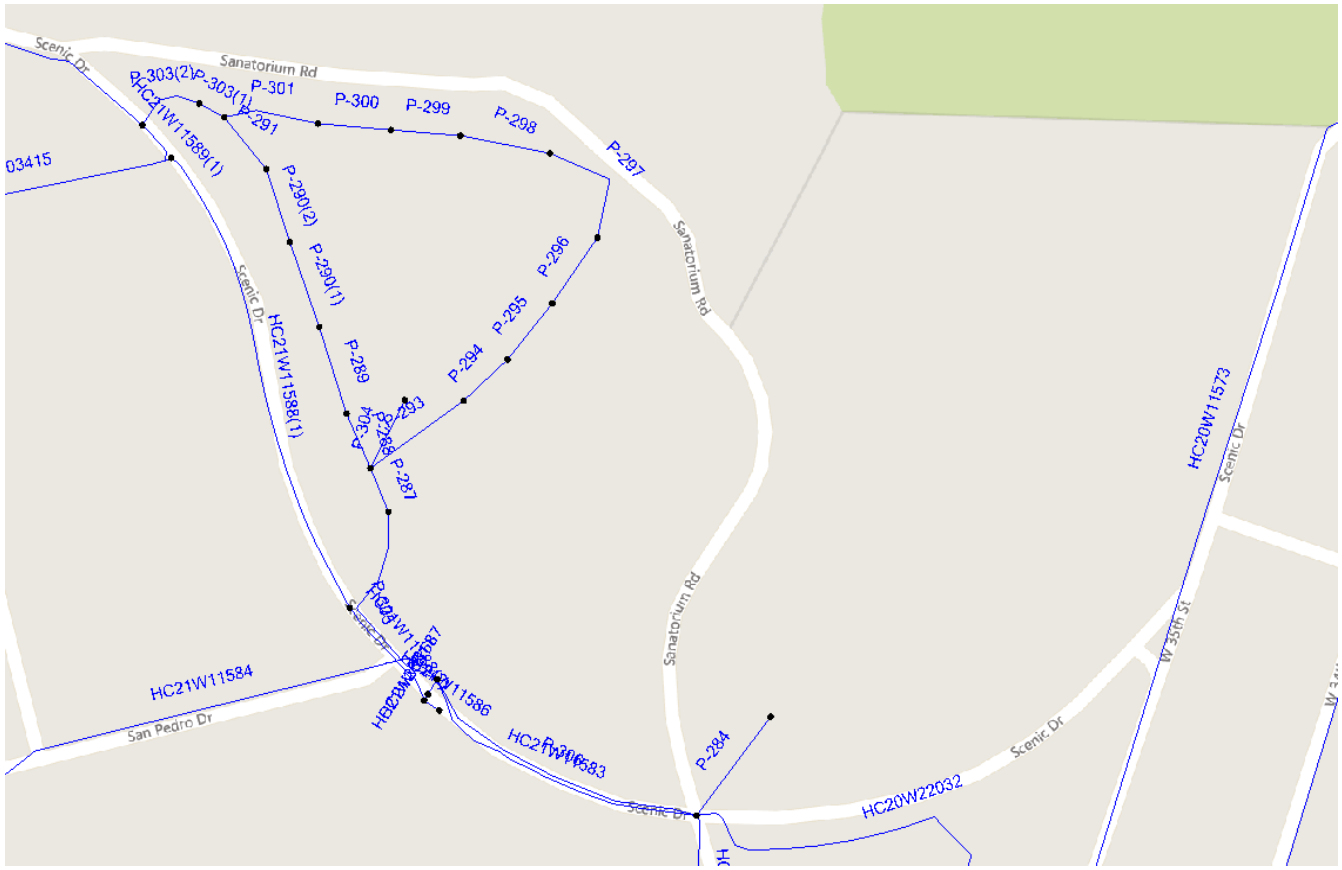
APPENDIX B:

System Layout and Demand Calculations

Proposed Development Layout (Junction IDs)



Proposed Development Layout (Pipe IDs)



City of Hamilton Demand Criteria				
Building Type	**PPU**	**ADD**	**MDD Peaking Factor**	**PHD Peaking Factor**
Townhomes	3.39	360 L/cap/day	1.9	3
Apartment/Condo	1.76	360 L/cap/day	1.9	3

Western Portion of Proposed Development							
Building Type	**Number of Buildings**	**Number of Units**	**Population**	**ADD (LPS)**	**MDD (LPS)**	**PHD (LPS)**	**MDD + FF (from OBC)**
2 Storey Residential, 4 Units each	14	4	13.56	0.06	0.11	0.17	150.11
8 Storey Residential, 204 Units each	2	204	359.04	2.99	5.68	8.98	155.68
Totals			372.6	3.05	5.79	9.15	305.79

Eastern Portion of Proposed Development							
Building Type	**Number of Buildings**	**Number of Units**	**Population**	**ADD (LPS)**	**MDD (LPS)**	**PHD (LPS)**	**MDD + FF (from OBC)**
8 Storey Condominium, 75 units	1	75	132	0.55	1.05	1.65	151.05
8 Storey Condominium, 141 units	1	141	248.16	1.03	1.96	3.10	151.96
5 Storey Condominium, 58 units each	2	58	102.08	0.85	1.62	2.55	151.62
4 Storey Condominium, 38 units	1	38	66.88	0.28	0.53	0.84	150.53
Totals			549.12	2.71	5.16	8.14	605.16

Notes: Peaking factors from City of Hamilton Water and Wastewate Master Plan (KMK, 2006)
PPU (persons per unit) from City of Hamilton Development Charge Background Study (Watson and Associates, 2014)
Flow demands for the townhomes in the western portion of the proposed development, are *per* townhome
Fire flows for the eastern portion of the proposed development have been taken as an average of all high rise units at *151.36 LPS*

APPENDIX C:

Fire Flow Calculations

Western Portion of Proposed Development										
Building Type	**Number**	**Occupancy Type (OBC 3.1.2.1)**	**K (Table 1)**	**Building Volume V (m3)**	**Length from Building Face to PL (m)**	**S-total**	**Demand Q (L)**	**OBC Flow Rate (LPS)**	**Target AFF (LPS)**	**Required FF (LPS)**
2 Storey Residential, 4 Units each	14	C - Residential	23	3093.546	8	1.4	99,612.18	45	150	150
8 Storey Residential A, 204 Units	1	C - Residential	10	33264	8	1.4	465,696.00	150	150	150
8 Storey Residential B, 204 Units	1	C - Residential	10	33264	8	1.4	465,696.00	150	150	150

Eastern Portion of Proposed Development										
Building Type	**Number**	**Occupancy Type (OBC 3.1.2.1)**	**K (Table 1)**	**Building Volume V (m3)**	**Length from Building Face to PL (m)**	**S-total**	**Demand Q (L)**	**OBC Flow Rate (LPS)**	**Target AFF (LPS)**	**Required FF (LPS)**
8 Storey Condominium, 75 units	1	C - Residential	10	17616	8	1.2	211,392.00	105	150	150
8 Storey Condominium, 141 units	1	C - Residential	10	42912	8	1.2	514,944.00	150	150	150
5 Storey Condominium, 58 units each	2	C - Residential	16	16080	10+	1.2	308,736.00	150	150	150
4 Storey Condominium, 38 units	1	C - Residential	16	10560	8	1	168,960.00	90	150	150

OBC vs Target AFF			
West Side			
Building Type	**OBC Flow Rate (LPS)**	**Target AFF - Residential (LPS)**	**Required FF (LPS)**
2 Storey Residential, 4 Units each,	45	150	150
8 Storey Residential A, 204 Units	150	150	150
8 Storey Residential B, 204 Units	150	150	150
East Side			
Building Type	**OBC Flow Rate (LPS)**	**Target AFF (LPS)**	**Required FF (LPS)**
8 Storey Condominium, 75 units	105	150	150
8 Storey Condominium, 141 units	150	150	150
5 Storey Condominium, 58 units each	150	150	150
4 Storey Condominium, 38 units	90	150	150

6.3 Buildings Requiring On-Site Fire Protection Water Supply

(a) Except for sprinklered buildings and as required by Sections 6.3 (c) and 6.3 (d), new buildings shall be provided with a supply of water available for fire fighting purposes not less than the quantity derived from the following formula:

$$Q = KVS_{Tot}$$

where

Q = minimum supply of water in litres (L)
K = water supply coefficient from Table 1
V = total building volume in cubic metres
S_{Tot} = total of spatial coefficient values from property line exposures on all sides, as obtained from the formula:

$$S_{Tot} = 1.0 + [(S_{Side1}) + (S_{Side2}) + (S_{Side3}) + \text{... etc.}]$$

where

S_{Side} values are obtained from Figure 1, as modified by Sections 6.3 (e) and 6.3 (f) of this guideline, and
S_{Tot} need not exceed 2.0
(see also Section 7.0 of this guideline)

(b) Except as provided in Section 6.3 (d), water supply flow rates shall not be less than that specified in Table 2. Where the water supply is from a municipal or industrial water supply system, then the required flow rate shall be available at a minimum pressure of 140 kPa.

(c) Except as provided in Section 6.3 (d), the minimum fire protection water supply "Q" required in Section 6.3 (a) shall not be less than what is needed to provide the minimum flow rate specified in Table 2 for a duration of 30 minutes.

(d) In elementary and secondary schools, the water supply determined in accordance with Sections 6.3 (a) and 6.3 (b) may be reduced. The level of reduction to be applied should be at the discretion of the local jurisdictional authority and should not exceed 30%. Factors to consider should include fire department response time, fire department resources and the size and complexity of the school building
(see Section 9.10 of this guideline for additional information).

Table 1: Water Supply Coefficient - K					
TYPE OF CONSTRUCTION	**Classification by Group or Division in Accordance with Table 3.1.2.1 of the Ontario Building Code**				
	A-2 B-1 B-2 B-3 C D	**A-4 F-3**	**A-1 A-3<**	**E F-2**	**F-1**
Building is of noncombustible construction with fire separations and fire-resistance ratings provided in accordance with Subsection 3.2.2. of the OBC, including loadbearing walls, columns and arches.	10	12	14	17	23
Building is of noncombustible construction or of heavy timber construction conforming to Article 3.1.4.6. of the OBC. Floor assemblies are fire separations but with no fire-resistance rating. Roof assemblies, mezzanines, loadbearing walls, columns and arches do not have a fire-resistance rating.	16	19	22	27	37
Building is of combustible construction with fire separations and fire-resistance ratings provided in accordance with Subsection 3.2.2. of the OBC, including loadbearing walls, columns and arches. Noncombustible construction may be used in lieu of fire-resistance rating where permitted in Subsection 3.2.2. of the OBC.	18	22	25	31	41
Building is of combustible construction. Floor assemblies are fire separations but with no fire-resistance rating. Roof assemblies, mezzanines, loadbearing walls, columns and arches do not have a fire-resistance rating.	23	28	32	39	53
Column 1	2	3	4	5	6

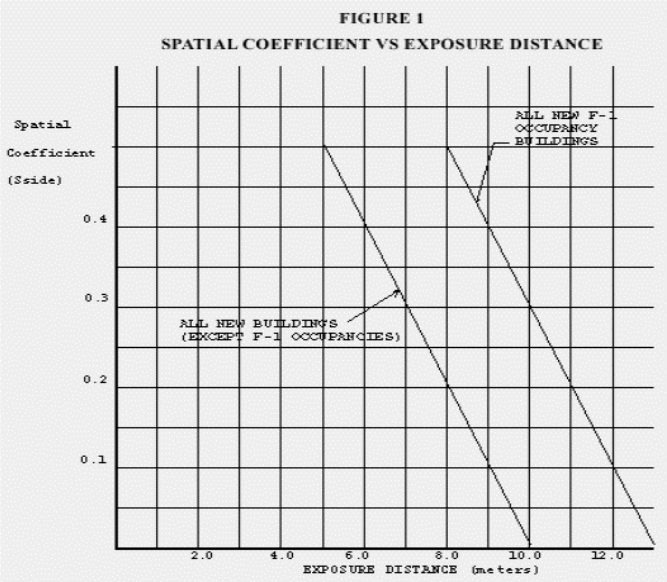


Table 2: Minimum Water Supply Flow Rates	
Building Code, Part 3 Buildings	**Required Minimum Water Supply Flow Rate (L/min.)**
One-storey building with building area not exceeding 600m² (excluding F-1 occupancies)	1800
All other buildings	2700 (If **Q** ≤ 108,000L)[(1)] 3600 (If **Q** > 108,000L **and** ≤ 135,000L)[(1)] 4500 (If **Q** > 135,000L **and** ≤ 162,000L)[(1)] 5400 (If **Q** > 162,000L **and** ≤ 190,000L)[(1)] 6300 (If **Q** > 190,000L **and** ≤ 270,000L)[(1)] 9000 (If **Q** > 270,000L)[(1)]

Note: [(1)] $\mathbf{Q=KVS_{Tot}}$ as referenced in Section 3(a)

3. System available fire flow calculations will be required as part of a development application submission and will be based on field testing and/or hydraulic modelling (as directed by the City). System available fire flow shall meet or exceed the greater of OBC required fire flow or the target available fire flow for the land use being proposed. For mixed use developments the target available fire flow will be based on the proposed land-use with the highest target available fire flow. The target available fire flow will be as defined in Table 1.

Table 1: Target Available Fire Flow

Land Use	Target AFF (L/s)
Commercial	150
Small ICI (<1,800 m^3)[1]	100
Industrial	250
Institutional	150
Residential Multi (greater than 3 units)	150
Residential Medium (3 or less units)	125
Residential Single	75
Residential Single (Dead End)	50

[1] 1800m^3 represents a maximum building volume that qualifies as “Small ICI”

APPENDIX D:

Model Results

Alternative 1 Pipe Tables

2018 ADD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	2.71	0.04	0
33345	P-287	J-455	J-456	13	300	PVC	120	4.89	0.07	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.04	0.01	0
33349	P-289	J-457	J-458	38	250	PVC	120	0.98	0.02	0
33353	P-291	J-459	J-460	14	250	PVC	120	0.8	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	0.86	0.01	0
33361	P-294	J-462	J-463	35	300	PVC	120	0.8	0.01	0
33363	P-295	J-463	J-464	38	250	PVC	120	0.74	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	0.68	0.01	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.62	0.01	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.56	0.01	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.5	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.44	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.38	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	0.92	0.02	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	0.86	0.02	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.18	0.02	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.12	0.02	0
33386	P-304	J-456	J-472	20	300	PVC	120	2.99	0.04	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	4.95	0.07	0

2018 MDD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	5.16	0.07	0
33345	P-287	J-455	J-456	13	300	PVC	120	8.7	0.12	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.63	0.02	0
33349	P-289	J-457	J-458	38	250	PVC	120	1.52	0.03	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.19	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	1.39	0.02	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.28	0.02	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.17	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.06	0.02	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.95	0.02	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.84	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.73	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.62	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.51	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	1.41	0.03	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.3	0.03	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.7	0.03	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.59	0.03	0
33386	P-304	J-456	J-472	20	300	PVC	120	5.68	0.08	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	8.81	0.12	0

2018 PHD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	8.14	0.12	0
33345	P-287	J-455	J-456	13	300	PVC	120	13.35	0.19	0
33347	P-288	J-456	J-457	21	300	PVC	120	2.35	0.03	0
33349	P-289	J-457	J-458	38	250	PVC	120	2.18	0.04	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.67	0.03	0
33359	P-293	J-456	J-462	33	300	PVC	120	2.02	0.03	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.85	0.03	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.68	0.03	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.51	0.03	0
33367	P-297	J-465	J-466	59	250	PVC	120	1.34	0.03	0
33369	P-298	J-466	J-467	37	250	PVC	120	1.17	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	1	0.02	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.83	0.02	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.66	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	2.01	0.04	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.84	0.04	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	2.33	0.05	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	2.16	0.04	0
33386	P-304	J-456	J-472	20	300	PVC	120	8.98	0.13	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	13.52	0.19	0

2031 ADD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	2.71	0.04	0
33345	P-287	J-455	J-456	13	300	PVC	120	4.44	0.06	0
33347	P-288	J-456	J-457	21	300	PVC	120	0.77	0.01	0
33349	P-289	J-457	J-458	38	250	PVC	120	0.71	0.01	0
33353	P-291	J-459	J-460	14	250	PVC	120	0.53	0.01	0
33359	P-293	J-456	J-462	33	300	PVC	120	0.68	0.01	0
33361	P-294	J-462	J-463	35	300	PVC	120	0.62	0.01	0
33363	P-295	J-463	J-464	38	250	PVC	120	0.56	0.01	0
33365	P-296	J-464	J-465	33	250	PVC	120	0.5	0.01	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.44	0.01	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.38	0.01	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.32	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.26	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.2	0	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	0.65	0.01	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	0.59	0.01	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	0.73	0.01	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	0.67	0.01	0
33386	P-304	J-456	J-472	20	300	PVC	120	2.99	0.04	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	4.5	0.06	0

2031 MDD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	5.16	0.07	0
33345	P-287	J-455	J-456	13	300	PVC	120	8.76	0.12	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.6	0.02	0
33349	P-289	J-457	J-458	38	250	PVC	120	1.49	0.03	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.16	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	1.37	0.02	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.26	0.02	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.15	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.04	0.02	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.93	0.02	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.82	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.71	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.6	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.49	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	1.38	0.03	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.27	0.03	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.65	0.03	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.54	0.03	0
33386	P-304	J-456	J-472	20	300	PVC	120	5.79	0.08	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	8.87	0.13	0

2031 PHD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	8.14	0.12	0
33345	P-287	J-455	J-456	13	300	PVC	120	13.34	0.19	0
33347	P-288	J-456	J-457	21	300	PVC	120	2.4	0.03	0
33349	P-289	J-457	J-458	38	250	PVC	120	2.23	0.05	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.72	0.04	0
33359	P-293	J-456	J-462	33	300	PVC	120	1.97	0.03	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.8	0.04	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.63	0.03	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.46	0.03	0
33367	P-297	J-465	J-466	59	250	PVC	120	1.29	0.03	0
33369	P-298	J-466	J-467	37	250	PVC	120	1.12	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.95	0.02	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.78	0.02	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.61	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	2.06	0.04	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.89	0.04	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	2.32	0.05	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	2.15	0.04	0
33386	P-304	J-456	J-472	20	300	PVC	120	8.98	0.13	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	13.51	0.19	0

Alternative 2 Pipe Tables

2018 ADD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	2.71	0.04	0
33345	P-287	J-455	J-456	13	300	PVC	120	4.89	0.07	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.04	0.01	0
33349	P-289	J-457	J-458	38	250	PVC	120	0.98	0.02	0
33353	P-291	J-459	J-460	14	250	PVC	120	0.8	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	0.86	0.01	0
33361	P-294	J-462	J-463	35	300	PVC	120	0.8	0.01	0
33363	P-295	J-463	J-464	38	250	PVC	120	0.74	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	0.68	0.01	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.62	0.01	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.56	0.01	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.5	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.44	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.38	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	0.92	0.02	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	0.86	0.02	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.18	0.02	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.12	0.02	0
33386	P-304	J-456	J-472	20	300	PVC	120	2.99	0.04	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	4.95	0.07	0

2018 MDD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	5.16	0.07	0
33345	P-287	J-455	J-456	13	300	PVC	120	8.7	0.12	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.63	0.02	0
33349	P-289	J-457	J-458	38	250	PVC	120	1.52	0.03	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.19	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	1.39	0.02	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.28	0.02	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.17	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.06	0.02	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.95	0.02	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.84	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.73	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.62	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.51	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	1.41	0.03	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.3	0.03	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.7	0.03	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.59	0.03	0
33386	P-304	J-456	J-472	20	300	PVC	120	5.68	0.08	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	8.81	0.12	0

2018 PHD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	8.14	0.12	0
33345	P-287	J-455	J-456	13	300	PVC	120	13.35	0.19	0
33347	P-288	J-456	J-457	21	300	PVC	120	2.35	0.03	0
33349	P-289	J-457	J-458	38	250	PVC	120	2.18	0.04	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.67	0.03	0
33359	P-293	J-456	J-462	33	300	PVC	120	2.02	0.03	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.85	0.03	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.68	0.03	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.51	0.03	0
33367	P-297	J-465	J-466	59	250	PVC	120	1.34	0.03	0
33369	P-298	J-466	J-467	37	250	PVC	120	1.17	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	1	0.02	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.83	0.02	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.66	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	2.01	0.04	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.84	0.04	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	2.33	0.05	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	2.16	0.04	0
33386	P-304	J-456	J-472	20	300	PVC	120	8.98	0.13	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	13.52	0.19	0

2031 ADD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	2.71	0.04	0
33345	P-287	J-455	J-456	13	300	PVC	120	4.44	0.06	0
33347	P-288	J-456	J-457	21	300	PVC	120	0.77	0.01	0
33349	P-289	J-457	J-458	38	250	PVC	120	0.71	0.01	0
33353	P-291	J-459	J-460	14	250	PVC	120	0.53	0.01	0
33359	P-293	J-456	J-462	33	300	PVC	120	0.68	0.01	0
33361	P-294	J-462	J-463	35	300	PVC	120	0.62	0.01	0
33363	P-295	J-463	J-464	38	250	PVC	120	0.56	0.01	0
33365	P-296	J-464	J-465	33	250	PVC	120	0.5	0.01	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.44	0.01	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.38	0.01	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.32	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.26	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.2	0	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	0.65	0.01	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	0.59	0.01	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	0.73	0.01	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	0.67	0.01	0
33386	P-304	J-456	J-472	20	300	PVC	120	2.99	0.04	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	4.5	0.06	0

2031 MDD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	5.16	0.07	0
33345	P-287	J-455	J-456	13	300	PVC	120	8.76	0.12	0
33347	P-288	J-456	J-457	21	300	PVC	120	1.6	0.02	0
33349	P-289	J-457	J-458	38	250	PVC	120	1.49	0.03	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.16	0.02	0
33359	P-293	J-456	J-462	33	300	PVC	120	1.37	0.02	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.26	0.02	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.15	0.02	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.04	0.02	0
33367	P-297	J-465	J-466	59	250	PVC	120	0.93	0.02	0
33369	P-298	J-466	J-467	37	250	PVC	120	0.82	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	0.71	0.01	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.6	0.01	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.49	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	1.38	0.03	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.27	0.03	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	1.65	0.03	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	1.54	0.03	0
33386	P-304	J-456	J-472	20	300	PVC	120	5.79	0.08	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	8.87	0.13	0

2031 PHD										
ID	Label	Start Node	Stop Node	Length (User Defined) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
33336	P-284	HC20T010	J-452	18	300	PVC	120	8.14	0.12	0
33345	P-287	J-455	J-456	13	300	PVC	120	13.35	0.19	0
33347	P-288	J-456	J-457	21	300	PVC	120	2.35	0.03	0
33349	P-289	J-457	J-458	38	250	PVC	120	2.18	0.04	0
33353	P-291	J-459	J-460	14	250	PVC	120	1.67	0.03	0
33359	P-293	J-456	J-462	33	300	PVC	120	2.02	0.03	0
33361	P-294	J-462	J-463	35	300	PVC	120	1.85	0.03	0
33363	P-295	J-463	J-464	38	250	PVC	120	1.68	0.03	0
33365	P-296	J-464	J-465	33	250	PVC	120	1.51	0.03	0
33367	P-297	J-465	J-466	59	250	PVC	120	1.34	0.03	0
33369	P-298	J-466	J-467	37	250	PVC	120	1.17	0.02	0
33371	P-299	J-467	J-468	37	250	PVC	120	1	0.02	0
33373	P-300	J-468	J-469	36	250	PVC	120	0.83	0.02	0
33374	P-301	J-469	J-460	23	250	PVC	120	0.66	0.01	0
33378	P-290(1)	J-458	J-470	37	250	PVC	120	2.01	0.04	0
33379	P-290(2)	J-470	J-459	38	250	PVC	120	1.84	0.04	0
33382	P-303(1)	J-460	J-471	26	250	PVC	120	2.33	0.05	0
33383	P-303(2)	J-471	J-461	62	250	PVC	120	2.16	0.04	0
33386	P-304	J-456	J-472	20	300	PVC	120	8.98	0.13	0
33395	P-305	HC21T011	J-455	108	300	PVC	120	13.52	0.19	0

Alternative 1 Junctions

2018 ADD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	2.71	231.63	350
33342	J-455	189.7	5	0.06	231.62	410
33344	J-456	189.9	5	0	231.62	408
33346	J-457	190.1	5	0.06	231.62	406
33348	J-458	190.4	5	0.06	231.62	403
33350	J-459	190.05	5	0.06	231.62	407
33352	J-460	189.85	5	0	231.62	409
33358	J-462	193.6	5	0.06	231.62	372
33360	J-463	193.72	5	0.06	231.62	371
33362	J-464	193.4	5	0.06	231.62	374
33364	J-465	193.45	5	0.06	231.62	374
33366	J-466	193.2	5	0.06	231.62	376
33368	J-467	193.3	5	0.06	231.62	375
33370	J-468	193.35	5	0.06	231.62	375
33372	J-469	193.55	5	0.06	231.62	373
33377	J-470	190.35	5	0.06	231.62	404
33381	J-471	189.65	5	0.06	231.62	411
33385	J-472	193.05	5	2.99	231.62	378

Pressures (kPa)	
MAX	411
MIN	350

2018 MDD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	5.16	230.44	338
33342	J-455	189.7	5	0.11	230.42	399
33344	J-456	189.9	5	0	230.42	397
33346	J-457	190.1	5	0.11	230.42	395
33348	J-458	190.4	5	0.11	230.42	392
33350	J-459	190.05	5	0.11	230.42	395
33352	J-460	189.85	5	0	230.42	397
33358	J-462	193.6	5	0.11	230.42	360
33360	J-463	193.72	5	0.11	230.42	359
33362	J-464	193.4	5	0.11	230.42	362
33364	J-465	193.45	5	0.11	230.42	362
33366	J-466	193.2	5	0.11	230.42	364
33368	J-467	193.3	5	0.11	230.42	363
33370	J-468	193.35	5	0.11	230.42	363
33372	J-469	193.55	5	0.11	230.42	361
33377	J-470	190.35	5	0.11	230.42	392
33381	J-471	189.65	5	0.11	230.42	399
33385	J-472	193.05	5	5.68	230.42	366

Pressures (kPa)	
MAX	399
MIN	338

2018 PHD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	8.14	228.3	318
33342	J-455	189.7	5	0.17	228.26	377
33344	J-456	189.9	5	0	228.26	375
33346	J-457	190.1	5	0.17	228.26	373
33348	J-458	190.4	5	0.17	228.26	370
33350	J-459	190.05	5	0.17	228.26	374
33352	J-460	189.85	5	0	228.26	376
33358	J-462	193.6	5	0.17	228.26	339
33360	J-463	193.72	5	0.17	228.26	338
33362	J-464	193.4	5	0.17	228.26	341
33364	J-465	193.45	5	0.17	228.26	341
33366	J-466	193.2	5	0.17	228.26	343
33368	J-467	193.3	5	0.17	228.26	342
33370	J-468	193.35	5	0.17	228.26	342
33372	J-469	193.55	5	0.17	228.26	340
33377	J-470	190.35	5	0.17	228.26	371
33381	J-471	189.65	5	0.17	228.26	378
33385	J-472	193.05	5	8.98	228.26	345

Pressures (kPa)	
MAX	378
MIN	318

2031 ADD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	2.71	232.92	363
33342	J-455	189.7	5	0.06	232.91	423
33344	J-456	189.9	5	0	232.91	421
33346	J-457	190.1	5	0.06	232.91	419
33348	J-458	190.4	5	0.06	232.91	416
33350	J-459	190.05	5	0.06	232.91	419
33352	J-460	189.85	5	0	232.91	421
33358	J-462	193.6	5	0.06	232.91	385
33360	J-463	193.72	5	0.06	232.91	384
33362	J-464	193.4	5	0.06	232.91	387
33364	J-465	193.45	5	0.06	232.91	386
33366	J-466	193.2	5	0.06	232.91	389
33368	J-467	193.3	5	0.06	232.91	388
33370	J-468	193.35	5	0.06	232.91	387
33372	J-469	193.55	5	0.06	232.91	385
33377	J-470	190.35	5	0.06	232.91	417
33381	J-471	189.65	5	0.06	232.91	423
33385	J-472	193.05	5	2.99	232.91	390

Pressures (kPa)	
MAX	423
MIN	363

2031 MDD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	5.16	228.83	323
33342	J-455	189.7	5	0.11	228.81	383
33344	J-456	189.9	5	0	228.81	381
33346	J-457	190.1	5	0.11	228.81	379
33348	J-458	190.4	5	0.11	228.81	376
33350	J-459	190.05	5	0.11	228.81	379
33352	J-460	189.85	5	0	228.81	381
33358	J-462	193.6	5	0.11	228.81	345
33360	J-463	193.72	5	0.11	228.81	343
33362	J-464	193.4	5	0.11	228.81	347
33364	J-465	193.45	5	0.11	228.81	346
33366	J-466	193.2	5	0.11	228.81	348
33368	J-467	193.3	5	0.11	228.81	347
33370	J-468	193.35	5	0.11	228.81	347
33372	J-469	193.55	5	0.11	228.81	345
33377	J-470	190.35	5	0.11	228.81	376
33381	J-471	189.65	5	0.11	228.81	383
33385	J-472	193.05	5	5.79	228.81	350

Pressures (kPa)	
MAX	383
MIN	323

2031 PHD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	8.14	224.2	277
33342	J-455	189.7	5	0.17	224.15	337
33344	J-456	189.9	5	0	224.15	335
33346	J-457	190.1	5	0.17	224.15	333
33348	J-458	190.4	5	0.17	224.14	330
33350	J-459	190.05	5	0.17	224.14	334
33352	J-460	189.85	5	0	224.14	336
33358	J-462	193.6	5	0.17	224.15	299
33360	J-463	193.72	5	0.17	224.15	298
33362	J-464	193.4	5	0.17	224.14	301
33364	J-465	193.45	5	0.17	224.14	300
33366	J-466	193.2	5	0.17	224.14	303
33368	J-467	193.3	5	0.17	224.14	302
33370	J-468	193.35	5	0.17	224.14	301
33372	J-469	193.55	5	0.17	224.14	299
33377	J-470	190.35	5	0.17	224.14	331
33381	J-471	189.65	5	0.17	224.14	338
33385	J-472	193.05	5	8.98	224.14	304

Pressures (kPa)	
MAX	338
MIN	277

Alternative 2 Junctions

2018 ADD						
ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	2.71	231.63	350
33342	J-455	189.7	5	0.06	231.62	410
33344	J-456	189.9	5	0	231.62	408
33346	J-457	190.1	5	0.06	231.62	406
33348	J-458	190.4	5	0.06	231.62	403
33350	J-459	190.05	5	0.06	231.62	407
33352	J-460	189.85	5	0	231.62	409
33358	J-462	193.6	5	0.06	231.62	372
33360	J-463	193.72	5	0.06	231.62	371
33362	J-464	193.4	5	0.06	231.62	374
33364	J-465	193.45	5	0.06	231.62	374
33366	J-466	193.2	5	0.06	231.62	376
33368	J-467	193.3	5	0.06	231.62	375
33370	J-468	193.35	5	0.06	231.62	375
33372	J-469	193.55	5	0.06	231.62	373
33377	J-470	190.35	5	0.06	231.62	404
33381	J-471	189.65	5	0.06	231.62	411
33385	J-472	193.05	5	2.99	231.62	378

Pressures (kPa)	
MAX	411
MIN	350

2018 MDD						
ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	5.16	230.44	338
33342	J-455	189.7	5	0.11	230.42	399
33344	J-456	189.9	5	0	230.42	397
33346	J-457	190.1	5	0.11	230.42	395
33348	J-458	190.4	5	0.11	230.42	392
33350	J-459	190.05	5	0.11	230.42	395
33352	J-460	189.85	5	0	230.42	397
33358	J-462	193.6	5	0.11	230.42	360
33360	J-463	193.72	5	0.11	230.42	359
33362	J-464	193.4	5	0.11	230.42	362
33364	J-465	193.45	5	0.11	230.42	362
33366	J-466	193.2	5	0.11	230.42	364
33368	J-467	193.3	5	0.11	230.42	363
33370	J-468	193.35	5	0.11	230.42	363
33372	J-469	193.55	5	0.11	230.42	361
33377	J-470	190.35	5	0.11	230.42	392
33381	J-471	189.65	5	0.11	230.42	399
33385	J-472	193.05	5	5.68	230.42	366

Pressures (kPa)	
MAX	399
MIN	338

2018 PHD						
ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	8.14	228.3	318
33342	J-455	189.7	5	0.17	228.26	377
33344	J-456	189.9	5	0	228.26	375
33346	J-457	190.1	5	0.17	228.26	373
33348	J-458	190.4	5	0.17	228.26	370
33350	J-459	190.05	5	0.17	228.26	374
33352	J-460	189.85	5	0	228.26	376
33358	J-462	193.6	5	0.17	228.26	339
33360	J-463	193.72	5	0.17	228.26	338
33362	J-464	193.4	5	0.17	228.26	341
33364	J-465	193.45	5	0.17	228.26	341
33366	J-466	193.2	5	0.17	228.26	343
33368	J-467	193.3	5	0.17	228.26	342
33370	J-468	193.35	5	0.17	228.26	342
33372	J-469	193.55	5	0.17	228.26	340
33377	J-470	190.35	5	0.17	228.26	371
33381	J-471	189.65	5	0.17	228.26	378
33385	J-472	193.05	5	8.98	228.26	345

Pressures (kPa)	
MAX	378
MIN	318

2031 ADD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	2.71	232.92	363
33342	J-455	189.7	5	0.06	232.91	423
33344	J-456	189.9	5	0	232.91	421
33346	J-457	190.1	5	0.06	232.91	419
33348	J-458	190.4	5	0.06	232.91	416
33350	J-459	190.05	5	0.06	232.91	419
33352	J-460	189.85	5	0	232.91	421
33358	J-462	193.6	5	0.06	232.91	385
33360	J-463	193.72	5	0.06	232.91	384
33362	J-464	193.4	5	0.06	232.91	387
33364	J-465	193.45	5	0.06	232.91	386
33366	J-466	193.2	5	0.06	232.91	389
33368	J-467	193.3	5	0.06	232.91	388
33370	J-468	193.35	5	0.06	232.91	387
33372	J-469	193.55	5	0.06	232.91	385
33377	J-470	190.35	5	0.06	232.91	417
33381	J-471	189.65	5	0.06	232.91	423
33385	J-472	193.05	5	2.99	232.91	390

Pressures (kPa)	
MAX	423
MIN	363

2031 MDD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	5.16	230.38	338
33342	J-455	189.7	5	0.11	230.35	398
33344	J-456	189.9	5	0	230.35	396
33346	J-457	190.1	5	0.11	230.35	394
33348	J-458	190.4	5	0.11	230.35	391
33350	J-459	190.05	5	0.11	230.35	394
33352	J-460	189.85	5	0	230.35	396
33358	J-462	193.6	5	0.11	230.35	360
33360	J-463	193.72	5	0.11	230.35	359
33362	J-464	193.4	5	0.11	230.35	362
33364	J-465	193.45	5	0.11	230.35	361
33366	J-466	193.2	5	0.11	230.35	364
33368	J-467	193.3	5	0.11	230.35	363
33370	J-468	193.35	5	0.11	230.35	362
33372	J-469	193.55	5	0.11	230.35	360
33377	J-470	190.35	5	0.11	230.35	392
33381	J-471	189.65	5	0.11	230.35	398
33385	J-472	193.05	5	5.79	230.35	365

Pressures (kPa)	
MAX	398
MIN	338

2031 PHD

ID	Label	Elevation (m)	Zone	Demand (L/s)	Hydraulic Grade (m)	Pressure (kPa)
33335	J-452	195.85	5	8.14	225.72	292
33342	J-455	189.7	5	0.17	225.67	352
33344	J-456	189.9	5	0	225.67	350
33346	J-457	190.1	5	0.17	225.67	348
33348	J-458	190.4	5	0.17	225.67	345
33350	J-459	190.05	5	0.17	225.67	349
33352	J-460	189.85	5	0	225.67	351
33358	J-462	193.6	5	0.17	225.67	314
33360	J-463	193.72	5	0.17	225.67	313
33362	J-464	193.4	5	0.17	225.67	316
33364	J-465	193.45	5	0.17	225.67	315
33366	J-466	193.2	5	0.17	225.67	318
33368	J-467	193.3	5	0.17	225.67	317
33370	J-468	193.35	5	0.17	225.67	316
33372	J-469	193.55	5	0.17	225.67	314
33377	J-470	190.35	5	0.17	225.67	346
33381	J-471	189.65	5	0.17	225.67	353
33385	J-472	193.05	5	8.98	225.67	319

Pressures (kPa)	
MAX	353
MIN	292

APPENDIX E:

Fire Flow Results

Alternative 1, 2031 MDD + FF Results

Label	Zone	Fire Flow Iterations	Satisfies Fire Flow Constraints?	Fire Flow (Needed) (L/s)	Fire Flow (Available) (L/s)	Flow (Total Needed) (L/s)	Flow (Total Available) (L/s)	Pressure (Residual Lower Limit) (kPa)	Pressure (Calculated Residual) (kPa)	Pressure (Zone Lower Limit) (kPa)	Pressure (Calculated Zone Lower Limit) (kPa)	Junction w/ Minimum Pressure (Zone)	Pressure (Calculated System Lower Limit) (kPa)	Junction w/ Minimum Pressure (System)	Is Fire Flow Run Balanced?
J-452	5	3	TRUE	151.29	155.73	156.45	160.89	138	138	138	162	J-463	-12	SA03E016	TRUE
J-455	5	7	TRUE	150.11	157.9	150.22	158.01	138	177	138	138	J-463	-12	SA03E016	TRUE
J-456	5	13	FALSE	250	157.2	250	157.2	138	175	138	138	J-463	-12	SA03E016	TRUE
J-457	5	7	TRUE	150.11	157.28	150.22	157.39	138	171	138	138	J-463	-12	SA03E016	TRUE
J-458	5	6	TRUE	150.11	157.17	150.22	157.28	138	163	138	138	J-469	-12	SA03E016	TRUE
J-459	5	7	TRUE	150.11	154.54	150.22	154.65	138	170	138	138	J-469	-12	SA03E016	TRUE
J-460	5	14	FALSE	250	153.95	250	153.95	138	173	138	138	J-469	-12	SA03E016	TRUE
J-462	5	5	TRUE	150.11	155.32	150.22	155.43	138	139	138	138	J-463	-12	SA03E016	TRUE
J-463	5	3	TRUE	150.11	153.71	150.22	153.82	138	138	138	142	J-464	-12	SA03E016	TRUE
J-464	5	3	TRUE	150.11	152.63	150.22	152.74	138	138	138	139	J-465	-12	SA03E016	TRUE
J-465	5	3	TRUE	150.11	151.02	150.22	151.13	138	138	138	143	J-464	-12	SA03E016	TRUE
J-466	5	3	TRUE	150.11	151.22	150.22	151.33	138	138	138	141	J-467	-12	SA03E016	TRUE
J-467	5	3	TRUE	150.11	150.95	150.22	151.06	138	138	138	142	J-468	-12	SA03E016	TRUE
J-468	5	3	TRUE	150.11	151.53	150.22	151.64	138	138	138	141	J-467	-12	SA03E016	TRUE
J-469	5	3	TRUE	150.11	152.08	150.22	152.19	138	138	138	142	J-468	-12	SA03E016	TRUE
J-470	5	6	TRUE	150.11	155.91	150.22	156.02	138	164	138	138	J-469	-12	SA03E016	TRUE
J-471	5	7	TRUE	150.11	154.8	150.22	154.91	138	170	138	138	J-469	-12	SA03E016	TRUE
J-472	5	5	TRUE	155.68	157.2	161.47	162.99	138	141	138	138	J-463	-12	SA03E016	TRUE

APPENDIX F:

Flushing Results

Flushing Results (2018 ADD)

ID	Label	Length (m)	Diameter (mm)	Flushing Event	Velocity (Maximum Flushing) (m/s)	Satisfies Flushing Target Velocity?	Shear Stress (Maximum Flushing) (kg/m²)	Satisfies Flushing Target Shear Stress?	Satisfies Flushing Target?
33336	P-284	18	300	Event [J-452]	0.92	TRUE	0.25	TRUE	TRUE
33345	P-287	13	300	Event [J-456]	0.82	TRUE	0.21	TRUE	TRUE
33347	P-288	21	300	Event [J-457]	0.79	FALSE	0.19	TRUE	FALSE
33349	P-289	38	250	Event [J-458]	0.9	TRUE	0.25	TRUE	TRUE
33353	P-291	14	250	Event [J-459]	0.76	FALSE	0.18	TRUE	FALSE
33359	P-293	33	300	Event [J-462]	0.79	FALSE	0.19	TRUE	FALSE
33361	P-294	35	250	Event [J-463]	0.97	TRUE	0.29	TRUE	TRUE
33363	P-295	38	250	Event [J-464]	0.87	TRUE	0.23	TRUE	TRUE
33365	P-296	33	250	Event [J-465]	0.79	FALSE	0.19	TRUE	FALSE
33367	P-297	59	250	Event [J-466]	0.67	FALSE	0.14	TRUE	FALSE
33369	P-298	37	250	Event [J-466]	0.68	FALSE	0.15	TRUE	FALSE
33371	P-299	37	250	Event [J-467]	0.76	FALSE	0.18	TRUE	FALSE
33373	P-300	36	250	Event [J-468]	0.83	TRUE	0.22	TRUE	TRUE
33374	P-301	23	250	Event [J-469]	0.92	TRUE	0.26	TRUE	TRUE
33378	P-290(1)	37	250	Event [J-470]	0.77	FALSE	0.18	TRUE	FALSE
33379	P-290(2)	38	250	Event [J-459]	0.65	FALSE	0.14	TRUE	FALSE
33382	P-303(1)	26	250	Event [J-471]	0.95	TRUE	0.28	TRUE	TRUE
33383	P-303(2)	62	250	Event [J-471]	0.46	FALSE	0.07	TRUE	FALSE
33386	P-304	20	300	Event [J-472]	0.95	TRUE	0.27	TRUE	TRUE
33395	P-305	108	300	Event [J-455]	0.84	TRUE	0.21	TRUE	TRUE